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Structural Standard for Antenna Supporting Structures and Antennas and Small Wind Turbine Support Structures

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This revision of the Standard began in 2013 and incorporates information as described in the Standard.

This Standard was prepared through the consensus standards process by balloting in compliance with the procedures of Telecommunications Industry Association (TIA) and American National Standard Institute (ANSI).

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STRUCTURAL STANDARD FOR ANTENNA SUPPORTING STRUCTURES, ANTENNAS AND SMALL WIND TURBINE SUPPORT STRUCTURES

Notes:

1. Normative annexes are integral parts of the Standard which for reasons of convenience are placed after other requirements of the Standard.
2. Informative annexes contain additional information that are not considered part of the Standard.

STRUCTURAL STANDARD FOR ANTENNA SUPPORTING STRUCTURES, ANTENNAS AND SMALL WIND TURBINE SUPPORT STRUCTURES

OBJECTIVE

The objective of this Standard is to provide recognized literature for antenna supporting structures, antennas and small wind turbine support structures pertaining to: (a) minimum load requirements as derived from ASCE 7-16, "Minimum Design Loads for Buildings and Other Structures", and (b) design criteria as derived from AISC-360-16, "2016 AISC Specification for Structural Steel Buildings" and ACI 318-14, "Building Code Requirements for Structural Concrete". The information contained in this Standard was obtained from available sources and represents, in the judgment of the subcommittee, the accepted industry minimum structural standards for the design of antenna supporting structures, antennas and small wind turbine support structures. While it is believed to be accurate, this information should not be relied upon for a specific application without competent professional examination and verification of its accuracy, suitability, and applicability by a licensed professional engineer. This Standard utilizes loading criteria based on an annual probability and is not intended to cover all environmental conditions which could exist at a particular location.

When this Standard is adapted for international use, it is necessary to determine the appropriate basic wind speeds (3-second gust), wind on ice loads, and earthquake design parameters at the site location based on local data for the mean recurrence interval associated with the risk category of the structure.

Equivalent International System of Units (SI) are given in square brackets [ ] throughout this Standard. SI conversion factors have been provided in Annex T.

Annex A provides procurement and user guidelines to assist in specifying the requirements for a specific structure. The user is cautioned that site-specific loading requirements, if known, take precedence over the minimum requirements of this Standard. Site-specific data and requirements differing from those contained in this Standard are to be included in the procurement specifications for the structure.

This Standard is intended to cover the requirements for most antenna supporting structures, antennas and small wind turbine support structures but recognizes that structures that are unusual for their height or shape, or for the shape and size of individual members, or located at sites having unusual geological or climatic conditions may require additional considerations. In these cases, a rational design based on theory, analysis, knowledge of local conditions and sound engineering practice, shall be used. The design shall be carried out by a licensed professional engineer qualified in the specific design methods and materials to be used, and shall provide a level of safety and performance equal to or better than that implicit in this Standard.

SCOPE

This Standard provides the requirements for the structural design and fabrication of new and the modification of existing antenna supporting structures, antennas, small wind turbine supporting structures, appurtenance mounting systems, structural components, guy assemblies, insulators and foundations.

This Standard is based on limit states design. It is applicable mainly to steel structures but may also be applied to other materials, when required, so as to provide an equivalent level of reliability.

The appropriate standards should be referenced for structures that support antennas but that are primarily intended for other applications, such as water towers, electrical transmission and distribution structures, sign support structures, lighting support structures, buildings, bridges, etc. This Standard, however, does apply to the calculation of effective projected areas of appurtenances (antennas, mounts, lines, etc.) and to the serviceability limit states appropriate for structures that support antennas.

Appropriate analysis and design criteria for other structural materials are outlined in Section 6.0 of the Standard. When a structure with a lower reliability is utilized as part of a communication system, the structure may require modification in order to meet the reliability requirements of this Standard. When the primary use of the structure is other than for communications, a higher reliability may be required in accordance with the applicable standard governing the primary use of the structure.

Structural requirements during construction and construction means and methods provisions are not within the scope of this Standard. For construction related loading, analysis, and design requirements during construction, installation, alteration, and maintenance, refer to the ANSI/TIA-322 Standard, "Loading, Analysis, and Design Criteria Related to the Installation, Alteration and Maintenance of Communication Structures". For applicable construction means and methods provisions, refer to the ANSI/ASSE A10.48 Standard, "Criteria for Safety Practices with the Construction, Demolition, Modification and Maintenance of Communication Structures".

1.0 GENERAL

1.1 Strength Limit States

A structure designed to this Standard shall have sufficient strength and stability such that the design strength, ϕR_n, defined in Section 4.0 equals or exceeds the required strength, $\sum \alpha_i Q_i$, defined in Section 2.0 as expressed by the following relationship:

$$\phi R_n \geq \sum \alpha_i Q_i$$

1.2 Serviceability Limit States

A structure designed to this Standard shall have sufficient rigidity such that the limit state deformations defined in 2.8.2 are not exceeded under the service loads defined in 2.8.3.

1.3 Analysis

Load effects on individual structural members shall be determined by methods of structural analysis that take into account equilibrium, general stability, geometric compatibility and material properties in accordance with Section 3.0.

1.4 Definitions

Antenna supporting structure: a structure, including guy assemblies, guy anchorages and substructures that support antennas or antenna arrays.

Design strength, ϕR_n: the product of nominal strength and a resistance factor.

Factored load: the product of the nominal load and a load factor.

Limit state: a condition beyond which a structure or member becomes unfit for service and is judged to be no longer useful for its intended function or unsafe.

Load effects: force and deformation responses produced in structures and their members by applied factored loads.

Load factor, α_i: a factor that accounts for deviations of the actual load from the nominal load, for uncertainties in the analysis that transforms the load into load effects, and for the probability that more than one extreme load will occur simultaneously.

Nominal loads: the magnitudes of the loads specified in this Standard for dead, wind, ice, wind on ice, earthquake, and working and climbing facilities.

Nominal strength, R_n: the capacity of a structure or member to resist the effects of loads.

Required strength, $\sum \alpha_i Q_i$: the sum of the load effects due to applied factored loads and load combinations.

Resistance factor, ϕ: a factor that accounts for the manner and consequences of failure and for unavoidable deviations from a calculated nominal strength.

Strength design: method of proportioning structural members such that the computed forces produced in the members by the factored loads do not exceed the design strengths of the members.

Structural antenna: a structure for radiating or receiving electromagnetic waves including reflectors, directors and screens.

1.5 Symbols and Notations

ϕR_n = design strength (nominal strength multiplied by a resistance factor);

$\Sigma \alpha_i Q_i$ = required strength (load effects due to factored loads and load combinations).

2.0 LOADS

2.1 Scope

This Section provides minimum load requirements for structures addressed in the scope of this Standard.

2.2 Risk Categorization of Structures

2.2.1 Definitions

Facility: all or any portion of buildings, structures, site improvements, elements and pedestrian or vehicular routes located on a site.

Hardened network: a network intended to provide the level of reliability and resiliency of an emergency communication system during and immediately following a natural or manmade disaster in accordance with a service level agreement with the operator of the emergency communication system that specifies requirements including, but not limited to, the following: protection from storm damage, immediate and long-term clean power backup systems, surge protection for electronics, monitoring, security and maintenance schedules.

Site: a parcel of land bounded by a lot line or a designated portion of public right-of-way.

2.2.2 General

Structures shall be categorized according to Table 2-1 for the purposes of determining wind, ice and earthquake loads based on the risk to human life, damage to a facility in the event of failure, and the primary use and intended purpose of the structure.

When surrounding facilities have a higher risk category than the structure, categorization of the structure shall use the risk category of surrounding facilities if the facility could be damaged or physically impacted in the event of failure of the structure.

When surrounding facilities have a higher risk category than the structure but the facility would not be damaged or physically impacted in the event of failure of the structure, the risk category of the structure shall not be based on the risk category of the surrounding facilities.

2.2.2.1 Multiple Services

It shall be permissible to categorize structures used primarily for commercial services as Risk Category II when the structure is also used to support antennas, radios, equipment or other appurtenances for fire, police or other emergency communication services and the site is not required to be part of a hardened network by the owner and service provider.

2.3 Combination of Loads

2.3.1 Symbols and Notations

D = dead load of structure and appurtenances, excluding guy assemblies;

D_g = dead load of guy assemblies;

D_i = weight of ice;

E_h = horizontal seismic load effect;
E_v = vertical seismic load effect;
T_i = load effects due to temperature;
W_i = concurrent wind load with ice;
W_o = wind load without ice.

2.3.2 Strength Limit State Load Combinations

Structures and foundations shall be designed such that their design strength equals or exceeds the load effects of the factored loads in each of the following limit state combinations:

1. $1.2\ D + 1.0\ D_g + 1.0\ W_o$
2. $0.9\ D + 1.0\ D_g + 1.0\ W_o$
3. $1.2\ D + 1.0\ D_g + 1.0\ D_i + 1.0\ W_i + 1.0\ T_i$
4. $1.2\ D + 1.0\ D_g + 1.0\ E_v + 1.0\ E_h$
5. $0.9\ D + 1.0\ D_g - 1.0\ E_v + 1.0\ E_h$

Exceptions:

1. Temperature effects need not be considered for self-supporting structures.
2. Ice and earthquake loading need not be considered for Risk Category I structures.
3. No load factor shall be applied to the initial tension of guys.
4. Load combinations 2 and 5 apply to self-supporting structures only.

Notes:

1. Unfactored dead loads shall be used to determine seismic load effects in loading combinations 4 and 5.
2. For foundation designs, the weight of soil directly supported by the foundation and the weight of the foundation shall be considered as a dead load in all loading combinations. For guy anchor foundations, the weight of soil directly above the foundation and the weight of the foundation shall utilize a 0.9 dead load factor. For all foundation types, when determining a soil nominal resistance that is a function of soil weight (e.g. gross soil bearing resistance, skin friction, lateral soil resistance, etc.), a factor of 1.0 shall be applied to the weight of soil and the resulting soil nominal strength shall be multiplied by the appropriate resistance factor defined in 9.4 to determine the soil design strength. (Refer to sections 9.4 and 9.7).

2.4 Temperature Effects

The design tension of guys shall be based on an initial temperature of 60 degrees Fahrenheit [16 degrees C]. In the absence of more accurate site data, a 50 degrees Fahrenheit [28 degrees C] reduction in temperature shall be considered to occur with loading combinations that include ice.

2.5 Dead Loads

2.5.1 Definitions

Dead load, D: the weight of the structure, and appurtenances, excluding guy assemblies, and for foundation design, the weight of soil and substructure.

Guy assembly dead load, D_g: the weight of all guy assemblies, including guys, end fittings, and insulators.

2.6 Wind and Ice Loads

2.6.1 Definitions

Appurtenances: items attached to the structure such as antennas, antenna mounts, transmission lines, conduits, lighting equipment, climbing devices, platforms, signs, anti-climbing devices, etc.

Basic wind speed, V: 3-second gust wind speed at 33 ft. [10 m] above the ground in Exposure C as defined in 2.6.5.1.2 for a mean recurrence interval associated with the risk category of the structure.

Basic wind speed with ice, V_i: 3-second gust wind speed concurrent with the design ice thickness at 33 ft. [10 m] above the ground in Exposure C as defined in 2.6.5.1.2.

Design ice thickness, t_i: the uniform radial thickness of glaze ice at 33 ft. [10 m] above the ground in Exposure C as defined in 2.6.5.1.2 for a 500-year mean recurrence interval.

Design wind load, F_W: equivalent static force to be used in the determination of wind loads.

Discrete appurtenance: an appurtenance modeled as a concentrated load.

Effective projected area, EPA: projected area of an object multiplied by a force coefficient (also called a drag factor) used in the determination of wind loads.

Escarpment: a steep slope generally separating two levels or gently sloping areas.

Flat topped hill: a land surface characterized by a strong relief in all horizontal directions with a flat top.

Flat topped ridge: an elongated crest characterized by a strong relief in one horizontal direction.

Glaze ice: ice accretion assumed to have a unit weight of 56 lb/ft^3 [8.8 kN/m^3].

Height of structure, h: the height of a structure, including latticed or tubular poles mounted on the structure, but excluding lightning rods and similar appurtenances.

Hill: a land surface characterized by a strong relief in all horizontal directions.

Linear appurtenance: an appurtenance modeled as a distributed load.

Ridge: an elongated crest characterized by strong relief in two horizontal directions.

Symmetrical appurtenance: an appurtenance for which the effective projected area (EPA) is considered constant for all wind directions.

Velocity pressure, q_z: equivalent static pressure used in the determination of wind loads.

Weight of ice, D_i: the weight of factored ice accumulated on the structure, guys, and appurtenances.

2.6.2 Symbols and Notations

α' = 3-second gust wind speed power law exponent;

β = topographic feature factor modifier for slope;

ε = solidity ratio of one face of a latticed structure or mounting frame without appurtenances;

θ = relative angle between the azimuth of an appurtenance and the wind direction;

θ_g = angle of wind incidence to a guy chord;

A_a = projected area of an appurtenance;

A_f = projected area of flat structural components;

A_{fs} = projected area of flat components supporting a mounting frame;

A_g = gross area of one face of a latticed structure or mounting frame;

A_{iz} = cross-sectional area of ice at height z;

A_p = projected area of a tubular structure;

A_r = projected area of round structural components and ice thickness on round or flat structural components;

A_{rs} = projected area of round components supporting a mounting frame;

C = velocity coefficient for round and polygonal tubular members;

C_a = force coefficient for a linear or discrete appurtenance;

C_{as} = force coefficient for a mounting frame;

C_d = force coefficient for a guy;

C_f = force coefficient (drag factor);

C_{fm} = force coefficient for a polygonal shape without a reduction based on corner radius;

C_{fr} = force coefficient for a round shape with an outside diameter equal to the outside flat-to-flat width of a polygonal shape;

D = diameter of a round shape, outside corner-to-corner width of a polygonal shape;

D_c = largest out-to-out dimension of a member;

D_f = wind direction factor for flat structural components;

D_i = weight of ice;

D_r = wind direction factor for round structural components;

Ds = smallest projected dimension of a component;

d = guy diameter;

$(EPA)_A$ = effective projected area of an appurtenance;

$(EPA)_{FN}$ = normal effective projected area of members supporting a mounting frame;

$(EPA)_{FT}$ = transverse effective projected area of members supporting a mounting frame;

$(EPA)_{MN}$ = normal effective projected area of a mounting frame;

$(EPA)_{MT}$ = transverse effective projected area of a mounting frame;

$(EPA)_N$ = effective projected area associated with the windward face normal to the azimuth of an appurtenance;

$(EPA)_S$ = effective projected area of a structure;

$(EPA)_T$ = effective projected area associated with the windward side face of an appurtenance;

e = natural logarithmic base, 2.718;

F_A = design wind force on appurtenances;

F_G = design wind force on guys;

F_{ST} = design wind force on a structure;

F_W = design wind load;

f = height attenuation factor;

G_h = gust effect factor;

H = height of crest above surrounding terrain;

H_1 = height above a rooftop below which no wind speed-up applies;

H_2 = height above a rooftop above which wind speed-up applies;

H_s = height of windward face of a building;

h = height of structure;

I = importance factor;

K_1 = topographic feature factor adjusted for slope;

K_1' = topographic feature factor;

K_2 = horizontal distance factor;

K_3 = vertical distance factor;

K_a = shielding factor for appurtenances (or wake interference factor);

K_c = terrain constant;

K_d = wind direction probability factor;

K_e = ground elevation factor;

K_h = height reduction factor;

K_{iz} = height escalation factor for ice thickness;

K_s = rooftop wind speed-up factor;

K_t = topographic constant;

K_z = velocity pressure coefficient;

K_{zmin} = minimum value for K_z;

K_{zt} = topographic factor;

L = length of an isolated topographic feature;

L_g = length of guy;

q_z = velocity pressure;

R_a = ratio of projected area of attachments to the projected area of a round structural member or appurtenance perpendicular to the wind direction;

R_L = maximum ratio of the projected area of round or flat attachments that project beyond the width of a tubular structure in the direction of wind to the projected area of the tubular structure;

R_r = reduction factor for a round element in a latticed tower face;

R_{rf} = reduction factor for a round element in a mounting frame;

R_W = ratio of the projected area of flat attachments on the windward face of a tubular structure to the projected area of the tubular structure;

r_c = ratio of outside corner radius to half outside flat-to-flat width of a polygonal shape;

r_m = value of r_c below which there is no reduction in C_f for a polygonal shape due to corner radius;

r_r = value of r_c above which a polygonal shape is considered round;

r_s = ratio of outside corner radius to outside width normal to the wind direction for a square or rectangular HSS member;

t_i = design ice thickness;

t_{iz} = thickness of radial glaze ice at height z;

V = basic wind speed without ice;

V_i = basic wind speed with ice;

W_s = width of windward face of a building;

X_b = horizontal distance from the windward face of a building to the centerline of a roof mounted structure;

x = horizontal distance between a structure and the crest of a topographic feature;

z = height above ground;

z_g = nominal height of atmospheric boundary layer;

z_r = height above roof;

z_s = mean elevation of base of structure above sea level.

2.6.3 General

Structures addressed in the scope of this Standard have unusual shapes and response characteristics due to wind load. The provisions of this Standard take into consideration the load magnification effects caused by wind gusts in resonance with along-wind vibrations of self-supporting, bracketed and guyed structures.

1. The basic wind speed without ice, V, the basic wind speed with ice, V_i, and the design ice thickness t_i shall be determined from 2.6.4.
2. A wind direction probability factor, K_d, shall be determined from Table 2-2.
3. An exposure category and velocity pressure coefficient, K_z, shall be determined for the site location in accordance with 2.6.5.
4. A topographic factor, K_{zt}, shall be determined in accordance with 2.6.6.
5. A rooftop wind speed-up factor, K_s, shall be determined in accordance with 2.6.7.
6. A ground elevation factor, K_e, shall be determined in accordance with 2.6.8.
7. A gust effect factor, G_h, shall be determined in accordance with 2.6.9.
8. The design ice thickness shall be escalated with height in accordance with 2.6.10.
9. The design wind force shall be determined in accordance with 2.6.11.

2.6.4 Basic Wind Speed and Design Ice Thickness

The basic wind speed without ice, the basic wind speed with ice and the design ice thickness shall be as given in Annex B except as provided in 2.6.4.1. It shall be permissible to determine site-specific basic wind speeds and design ice thicknesses from the ASCE 7 online Hazard Tool based on ASCE 7-16.

It shall be permissible to ignore ice for structures located in regions where the design ice thickness is less than or equal to a thickness of 0.50 in. [13 mm].

2.6.4.1 Estimation of Basic Wind Speeds and Design Ice Thickness from Regional Climatic Data

For regions not included in Annex B, for the special wind or ice regions indicated in Annex B, and for sites where records indicate that in-cloud icing produces significant loads, extreme-value statistical-analysis procedures shall be used to establish design values consistent with this Standard from available climatic data accounting for the length of record, sampling error, averaging time, anemometer height, data quality and terrain exposure.

2.6.5 Exposure Category and Velocity Pressure Coefficient

2.6.5.1 General

An exposure category shall be determined for a site based on the ground surface roughness that is determined from natural topography, vegetation and constructed facilities. The surface roughness categories defined in 2.6.5.1.1 shall be used for determining the exposure category.

2.6.5.1.1 Surface Roughness Categories

Surface Roughness B: urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.

Surface Roughness C: open terrain with scattered obstructions having heights generally less than 30 ft. [9.1 m]. This category includes flat open country, grasslands and athletic fields.

Surface Roughness D: flat, unobstructed areas, shorelines and water surfaces. This category includes smooth mud flats, salt flats and unbroken ice.

2.6.5.1.2 Exposure Categories

The Exposure Category for a structure shall be assessed as being one of the following:

Exposure B: Exposure Category B shall be limited to structures where Surface Roughness B surrounds the structure in all directions to satisfy the following minimum distances:

a) 2,600 ft. [790 m]

b) Twenty times the height of the structure, but need not exceed 24,000 ft. [7,300 m]

Exposure C: Exposure Category C applies to structures where Exposure Categories B or D does not apply.

Exposure D: Exposure Category D shall be used for structures exposed to wind flowing over Surface Roughness D for a distance greater than the following distances:

a) 5,000 ft. [1,500 m]

b) The lesser of twenty times the height of the structure or 14,000 ft. [4,300 m]

Exposure Category D shall be used for structures located beyond the Surface Roughness D area within a distance equal to the following:

a) For Surface Roughness C boundary areas, a distance equal to the following:

    i. 600 ft. [180 m]

    ii. Twenty times the height of the structure, but not greater than 18,000 ft. [5,500 m]

b) For Surface Roughness B boundary areas, a distance equal to 600 ft. [180 m] or twenty times the height of the structure, whichever is greater. Alternatively, Exposure Category C shall be permitted to be used for structures of any height that are located further than 10,000 ft. [3,000 m] from the Surface Roughness D area.

Structure height, for structures mounted on buildings or other structures, shall be based on the height of the structure above ground elevation.

For a site located where the exposure may be different when considered from two or more directions, the category resulting in the largest wind forces shall be used for all load combinations.

Site-Specific Exposure: It shall be permissible to apply a Site-Specific Exposure to structures when a site-specific investigation is performed using a directional procedure in accordance with ASCE 7-16 or other similar recognized literature to account for varying or intermediate ground surface roughness surrounding the structure. For Site-Specific Exposures, K_{zmin} shall be equal to 0.70.

2.6.5.2 Velocity Pressure Coefficient

Based on the exposure category determined in 2.6.5.1.2, a velocity pressure coefficient, K_z, shall be determined as follows:

$$K_z = 2.01(z/z_g)^{2/\alpha'}$$

$$K_{zmin} \le K_z \le 2.01$$

where:

z = height above ground level at the base of the structure

z_g, α' and K_{zmin} are tabulated in Table 2-4

2.6.6 Topographic Effects

2.6.6.1 Wind Speed-Up Over Hills, Ridges and Escarpments

Wind speed-up effects at isolated hills, ridges and escarpments constituting abrupt changes in the general topography, located in any exposure category, shall be included in the calculation of design wind loads under the following conditions:

1. The hill, ridge or escarpment is isolated and unobstructed by other similar topographic features of comparable height for a radius of 2 miles [3.22 km] or 100 times the height of the feature, whichever is less, measured horizontally from the crest, and
2. The hill, ridge or escarpment protrudes by a factor of two or more above the average height of the surrounding upwind terrain features within a 2 mile [3.22 km] radius. The slope (vertical to horizontal ratio) of the topographic feature exceeds 0.10, and
3. The height of the topographic feature is greater than or equal to 15 ft. [4.57 m] for Exposure C, D, or Site-Specific Exposures; and 60 ft. [18 m] for Exposure B, and
4. The structure is located on the upper half of the hill, ridge or escarpment.

2.6.6.2 Topographic Factor Procedures

When required from 2.6.6.1, the topographic factor, K_{zt}, shall be determined from 2.6.6.2.1 (Method 1), from 2.6.6.2.2 (Method 2) or from 2.6.6.2.3 (Method 3). For the Site-Specific Exposure categories, Method 3 shall be used. In all other cases the value of K_{zt} shall be equal to 1.0.

2.6.6.2.1 Simplified Topographic Factor Procedure (Method 1)

The topographic category for a structure shall be assessed as being one of the following:

1. **Category 1:** No abrupt changes in general topography, e.g. flat or rolling terrain, no wind speed-up consideration shall be required ($K_{zt} = 1.0$).
2. **Category 2:** Structures located at or near the crest of an escarpment. Wind speed-up shall be considered to occur in all directions. Structures located horizontally beyond 16 times the height of the escarpment from its crest, shall be permitted to be considered as Topographic Category 1.
3. **Category 3:** Structures located in the upper half of a hill. Wind speed-up shall be considered to occur in all directions.
4. **Category 4:** Structures located in the upper half of a ridge. Wind speed-up shall be considered to occur in all directions.

The factor K_{zt} shall be calculated by:

$$K_{zt} = [1 + \frac{K_c K_t}{K_h}]^2$$

where:

K_c = terrain constant given in Table 2-4

K_t = topographic constant given in Table 2-5

K_h = height reduction factor given by the following equation:

$$= e^{(\frac{f \cdot z}{H})}$$

e = natural logarithmic base = 2.718

f = height attenuation factor given in Table 2-5

z = height above ground level at the base of the structure

H = height of crest above surrounding terrain

2.6.6.2.2 Rigorous Topographic Factor Procedure (Method 2)

The feature under consideration shall be classified as a ridge, flat-topped ridge, escarpment, hill or flat-topped hill as illustrated in Figure 2-1. K_{zt} shall be calculated by:

$$K_{zt} = [1 + K_1 K_2 K_3]^2$$

where:

K_1 = topographic feature factor adjusted for slope from Figure 2-1

$= \beta K_1'$

β = slope modifier from Figure 2-1

K_1' = topographic feature factor from Figure 2-1

K_2 = horizontal distance factor from Figure 2-1

K_3 = vertical distance factor from Figure 2-1

2.6.6.2.3 Site-Specific Topographic Procedure (Method 3)

Wind speed-up criteria shall be based on a site-specific investigation and K_{zt} shall be based on recognized published literature or research findings.

2.6.7 Rooftop Wind Speed-Up Factor

Wind speed-up effects for the calculation of design wind loads shall be considered for structures or appurtenances supported on enclosed buildings under either of the following conditions:

1. The building is greater than 50 ft. [15 m] in height and is isolated and unobstructed for a continuous 90 degree quadrant by other buildings of comparable height for a distance from the windward wall equal to 2,600 ft. [792 m] or twenty times the building height, whichever is less.
2. The building protrudes 50 ft. [15 m] above the average height of immediately adjacent buildings in a continuous 90 degree quadrant.

Unless a different method for determining wind speed-up effects is justified by wind tunnel tests or other site-specific information, the wind speed-up factor, K_s, shall be determined as follows:

K_s = 1.0 for $z_r < H_1$

$K_s = 1 + 0.3\ (W_s/H_s \le 1)$ but not less than 1.10 for $H_1 \le z_r \le H_2$

K_s = 1.0 for $z_r > H_2$

where:

z_r = height above roof

H_1 = parapet height + $(X_b / 5)$

X_b = horizontal distance from windward face to center of structure

W_s = width of the windward face of the building

H_s = height of the windward face of the building

H_2 = parapet height + lesser of H_s or W_s

Refer to Figure 2-2.

Notes:

1. It shall be permissible to calculate the value of K_s based on the largest value of W_s for the building for all wind directions and to use the maximum value of K_s over the full height of the structure.
2. The height above ground level for calculating K_z shall be referenced to the ground elevation of the building.

2.6.8 Ground Elevation Factor

The ground elevation factor to adjust for air density, K_e, shall be determined in accordance with the following equations for all elevations:

$$K_e = e^{-0.0000362\, z_s} \quad [z_s \text{ in ft.}]$$

$$K_e = e^{-0.000119\, z_s} \quad [z_s \text{ in m}]$$

where:

z_s = mean elevation of base of structure above sea level

Alternatively, it shall be permitted to determine K_e from Table 2-6.

A value of K_e equal to 1.0 shall be permitted for any ground elevation.

2.6.9 Gust Effect Factor

2.6.9.1 Self-Supporting or Bracketed Latticed Structures

For self-supporting or bracketed latticed structures, the gust effect factor shall be 1.00 for structures 600 ft. [183 m] or greater in height. For structures 450 ft. [137 m] or less in height, the gust effect factor shall be 0.85. The gust effect factor shall be linearly interpolated for structure heights between 450 ft. [137 m] and 600 ft. [183 m].

These conditions are expressed by the following equations:

$$G_h = 0.85 + 0.15\left[\frac{h}{150} - 3.0\right] \quad h, \text{ in feet}$$

$$G_h = 0.85 + 0.15\left[\frac{h}{45.7} - 3.0\right] \quad h, \text{ in meters}$$

$$0.85 \le G_h \le 1.00$$

where:

h = height of structure

Note: For structures supported on buildings or other structures, the height of structure, h, shall not include the height of the supporting structure.

2.6.9.2 Guyed Masts

For guyed masts the gust effect factor shall be 0.85.

2.6.9.3 Pole Structures

For pole structures the gust effect factor shall be 1.10.

2.6.9.4 Spines and Pole Structures Supported on Flexible Structures

For the strength and connection design of cantilevered tubular or latticed spines, poles or similar structures (fundamental frequency less than 1.1 Hertz) mounted on guyed masts, latticed self-supporting structures or flexible buildings (height to width ratio greater than 5), the gust effect factor shall be 1.10 for latticed spines and 1.35 for tubular spines.

Gust effect factors for supporting guyed masts and latticed self-supporting structures shall be in accordance with 2.6.9.1 or 2.6.9.2 using the loads from the cantilever based on the gust effect factor for the supporting structure.

Notes:

1. Connection design shall include the design of all members and components transferring the spine loads to the main members of the supporting structure or candelabra.
2. For structures supporting spines, the height of the structure shall include the height of the spine.

2.6.10 Design Ice Thickness

The design ice thickness, t_i, shall be escalated with height when calculating ice weight and wind on ice loads in accordance with the following equations:

$$t_{iz} = t_i I K_{iz} (K_{zt})^{0.35}$$

$$K_{iz} = \left[\frac{z}{33}\right]^{0.10} \leq 1.4 \quad \text{z, in feet}$$

$$K_{iz} = \left[\frac{z}{10}\right]^{0.10} \leq 1.4 \quad \text{z, in meters}$$

where:

t_{iz} = thickness of radial glaze ice at height z

t_i = design ice thickness (500-year mean recurrence interval)

I = importance factor for structure from Table 2-3

K_{iz} = height escalation factor for ice thickness

K_{zt} = topographic factor from 2.6.6.2

z = height above ground level at base of structure

For purposes of calculating the additional projected area of ice, ice thickness shall be considered to accumulate with a uniform thickness around the exposed surfaces of the structure, guys and appurtenances (refer to Figure 2-3). The additional projected area of ice may be considered round when calculating wind on ice loads even though the bare projected area is flat.

For purposes of calculating the weight of ice, the cross-sectional area of ice shall be determined by:

$$A_{iz} = \pi(t_{iz})(D_c + t_{iz})$$

where:

A_{iz} = cross-sectional area of ice at height z

t_{iz} = thickness of radial ice at height z

D_c = largest out-to-out dimension of a member (refer to Figure 2-4)

z = height above ground level at base of structure

The weight of ice shall be based on a unit weight of 56 lb/ft^3 [8.8 kN/m^3].

Note: The rooftop wind speed-up factor (K_s) does not apply to the escalation of the design ice thickness (t_{iz}).

2.6.11 Design Wind Load

The design wind load shall include the sum of the horizontal design wind forces applied to the structure in the direction of the wind and the design wind forces on guys and appurtenances. All appurtenances, including antennas, mounts and lines, shall be assumed to remain intact and attached to the structure.

Strength design shall be based on the wind directions resulting in the maximum responses. For latticed structures, each of the wind directions indicated in Table 2-7 shall be considered for each face.

The horizontal design wind force for the strength design of appurtenances and their connections to supporting structures shall be determined using a gust effect factor of 1.0 and a wind direction probability factor determined from Table 2-2. No shielding from the structure shall be considered.

The horizontal design wind force for the strength design of a cantilevered tubular or latticed spine, pole or similar structure mounted on a guyed mast, latticed self-supporting structure, or flexible building shall be determined using a gust effect factor from 2.6.9.4 and a wind direction probability factor determined from Table 2-2 for the cantilevered structure.

Note: The wind direction probability factor for determining the design wind load for the total structure, including the cantilever, shall be determined from Table 2-2 based on the type of supporting structure.

The design wind load, F_W, shall be determined in accordance with the following:

$$F_W = F_{ST} + F_A + F_G$$

where:

F_{ST} = design wind force on the structure from 2.6.11.1

F_A = design wind force on appurtenances from 2.6.11.2

F_G = design wind force on guys from 2.6.11.3

The design wind forces, F_{ST} + F_A for a latticed structure, need not exceed the wind force calculated for a structure using a solidity ratio of 1.0 (solid-faced) plus the wind load on externally mounted appurtenances that are outside the normal projected area of the structure in the direction of the wind. For a structure with a partial or full-height shroud, the total design wind load, F_W, for the shrouded section shall be determined based on the shape and aspect ratio of the shroud using the force coefficient for appurtenances from Table 2-9.

2.6.11.1 Design Wind Force on Structure

The design wind force, F_{ST}, applied to each section of a structure shall be determined in accordance with the following:

$$F_{ST} = q_z G_h (EPA)_S$$

where:

F_{ST} = horizontal design wind force on the structure in the direction of the wind

q_z = velocity pressure from 2.6.11.6

G_h = gust effect factor from 2.6.9

$(EPA)_S$ = effective projected area of the structure from 2.6.11.1.1 or 2.6.11.1.2

2.6.11.1.1 Effective Projected Area of Latticed Structures

The effective projected area of structural components for a section, $(EPA)_S$, shall be determined from the equation:

$$(EPA)_S = C_f [D_f \Sigma A_f + D_r \Sigma (A_r R_r)]$$

where:

C_f = $4.0\varepsilon^2 - 5.9\varepsilon + 4.0$ (square cross sections)

C_f = $3.4\varepsilon^2 - 4.7\varepsilon + 3.4$ (triangular cross sections)

ε = solidity ratio = $(A_f + A_r)/A_g$

A_f = projected area of flat structural components in one face of the section

A_r = projected area of round structural components in one face of the section including the projected area of ice on flat and round structural components in one face for loading combinations that include ice

A_g = gross area of one face as if the face were solid

D_f = wind direction factor for flat structural components determined from Table 2-7

D_r = wind direction factor for round structural components determined from Table 2-7

R_r = reduction factor for a round element

$= 0.57 - 0.14\varepsilon + 0.86\varepsilon^2 - 0.24\varepsilon^3 \leq 1.0$ when $C < 39$ [5.3] and for all iced conditions (subcritical flow)

$= 0.36 + 0.26\varepsilon + 0.97\varepsilon^2 - 0.63\varepsilon^3$ when $C > 78$ [10.6] for no-ice conditions (supercritical flow)

where:

$C = [K_z K_{zt} K_e]^{1/2} V D$

K_z = velocity pressure coefficient from 2.6.5.2

K_{zt} = topographic factor from 2.6.6.2

K_e = ground elevation factor from 2.6.8

V = the basic wind speed for the loading condition under investigation, mph [m/s]

D = outside diameter of the structural component without ice, ft. [m]

Notes:

1. The projected area of structural components shall include the projected area of connection plates in the face of a section.
2. In order for a structural component to be considered as a round structural component, the component must have a round profile on the windward and leeward sides of the component. (Formed U-shaped angle or channel members shall be considered as flat structural components.)
3. Bracing members in adjacent faces and internal plan and hip bracing need not be included in the projected area of structural components.
4. For no-ice conditions, linear interpolation may be used when $39\ [5.3] \leq C \leq 78\ [10.6]$ to determine R_r. For iced conditions, R_r shall be based on subcritical flow for all values of C.
5. When attachments such as step bolts, or similar regularly spaced irregularities, or linear appurtenances (such as waveguides or feed lines) are attached to a round structural member, the reduction factor for the round elements, R_r shall be calculated as follows:

a) When $R_a \leq 0.1$, it shall be permissible to ignore the projected areas of the attachments.

b) When $0.1 < R_a \leq 0.2$, the value for R_r shall be multiplied by $1.0 + 3(R_a - 0.1)$, and it shall be permissible to ignore the projected areas of the attachments.

c) When $R_a > 0.20$, or alternatively for any value of R_a, the value of R_r for subcritical flow shall be used. The projected areas of attachments shall be considered separately in addition to the structural member using appropriate force coefficients for appurtenances.

Where R_a is the ratio of the sum of the projected areas of the attachments on both sides of the structural member (perpendicular to the wind direction) to the projected area of the structural member without the attachments for the portion being considered. For iced conditions, the ice thickness need not be included in the determination of R_a.

6. When attachments such as step bolts, or similar regularly spaced irregularities, or linear appurtenances (such as waveguides or feed lines) are attached to a flat structural member, the projected areas of the attachments shall be considered separately in addition to the structural member using appropriate force coefficients except when R_a is less than or equal to 0.1, the projected areas of the attachments may be ignored.

2.6.11.1.1.1 Effective Projected Area of Latticed Leg Structures

Latticed legs shall be considered as equivalent round members for the purpose of determining the effective projected area, $(EPA)_S$, of structures with latticed legs.

The effective projected area of an individual latticed leg shall be determined in accordance with 2.6.11.1.1 with R_r based on subcritical flow and the direction factors, D_f and D_r, equal to 1.0. The diameter of the equivalent round member shall be determined by dividing the $(EPA)_S$ of the individual latticed leg by the quantity of 1.2 times the length of the latticed leg. Gross area, A_g, of the structure shall be based on the full width of the structure including the width of the latticed leg and ice when applicable.

The reduction factor, R_r, for the equivalent round member shall be based on subcritical flow.

For loading conditions that include ice, the factored thickness of ice, t_{iz}, shall be considered uniformly distributed around each member of the latticed leg for determining effective projected areas (ice thickness need not be added to the equivalent round member). The weight of ice shall be determined by considering each member of the latticed leg in accordance with 2.6.10.

2.6.11.1.2 Effective Projected Area of Tubular Structures

The effective projected area of a tubular section, $(EPA)_S$, shall be determined from the equation:

$$(EPA)_S = C_f A_P$$

where:

C_f = force coefficient for a tubular structure from Table 2-8a, 2-8b and 2-8c

A_P = actual projected area based on the outside diameter (for rounds), the outside corner-to-corner width (for polygons), including ice thickness for load combinations that include ice

Note: In the absence of a detailed feed line layout including the bend radius of each line, the minimum diameter of a tubular structure shall not be less than the diameter which results in 45% utilization of the cross-section for the placement of internal feed lines.

2.6.11.1.3 Uniform Wind and Ice Applied to Structure

The design wind force and ice thickness applied to a section of a structure shall be permitted to be based on the velocity pressure and ice thickness at the mid-height of the section. The section length considered to have uniform velocity pressure and ice thickness shall not exceed the following:

1. 60 ft. [18 m] for latticed structures
2. 20 ft. [6 m] for pole structures

2.6.11.2 Design Wind Force on Appurtenances

The design wind force on appurtenances (either discrete or linear but excluding microwave antennas), F_A, shall be determined from the equation:

$$F_A = q_z G_h (EPA)_A$$

where:

q_z = velocity pressure at the centerline height of the appurtenance from 2.6.11.6

G_h = gust effect factor from 2.6.9

(Note: See 2.6.11 for G_h for the strength design of appurtenances.)

$(EPA)_A$ = effective projected area of the appurtenance including ice for loading combinations that include ice

The design wind force, F_A, shall be applied at the centroid of the effective projected area of the appurtenance in the direction of the wind. For a linear appurtenance, the length considered to have uniform velocity pressure and ice thickness shall not exceed the section length specified in 2.6.11.1.3.

In the absence of more accurate data, the design wind force on microwave antennas shall be determined using Annex C.

In the absence of more accurate data specifying effective projected area values for each critical wind direction, the effective projected area, $(EPA)_A$, of an appurtenance shall be determined from the equation:

$$(EPA)_A = K_a[(EPA)_N \cos^2(\theta) + (EPA)_T \sin^2(\theta)]$$

where:

K_a = 1.0 for round appurtenances, regardless of location, when transitional or supercritical force coefficients are considered

= $(1-\varepsilon)$ for appurtenances when subcritical force coefficients are considered, located entirely inside the cross section of a latticed structure or outside the cross section entirely within a face zone as defined in Figure 2-5, where ε is the minimum solidity ratio of the structure considering each face for the section containing the appurtenance. K_a need not exceed 0.6

= 0.8 for antennas and antenna mounting configurations that do not project above the top of the structure (when subcritical force coefficients are considered only) such as side arms, T-arms, stand-offs, etc. when 3 or more mounts are located at the same relative elevation. Shielding from the mounting configuration and shielding of mounting members from antennas is excluded except as provided in 2.6.11.2.1.

= 1.0 for other appurtenances unless otherwise specified in this section

Notes:

1. K_a = 1.0 may be conservatively used for any appurtenance.
2. The value of K_a is constant for all wind directions.
3. The K_a values for determining effective projected areas for the analysis of supporting structures for mounting frames, frame/truss platforms, low profile mounts and circular ring platforms are specified in 2.6.11.2.2 through 2.6.11.2.5. (Note: Refer to Section 16.0 for K_a values for determining effective projected areas for the design and analysis of appurtenance mounting systems.)
4. Subcritical force coefficients may conservatively be used for any value of C (refer to Table 2-9, Note 5) in combination with a value of $K_a < 1.0$, in accordance with the criteria specified above.

θ = relative angle between the azimuth associated with the normal face of the appurtenance and the wind direction (refer to Figure 2-6)

$(EPA)_N$ = effective projected area associated with the windward face normal to the azimuth of the appurtenance

$(EPA)_T$ = effective projected area associated with the windward side face of the appurtenance

It shall be permissible to use the larger value of $(EPA)_N$ or $(EPA)_T$ for $(EPA)_A$ for all wind directions.

In the absence of more accurate data, an appurtenance shall be considered as consisting of flat and round components in accordance with the following:

$$(EPA)_N = \Sigma(C_a A_a)_N$$

$(EPA)_T = \Sigma(C_a A_a)_T$

C_a = force coefficient from Table 2-9

A_a = projected area of a component of the appurtenance. The additional projected area of ice shall be considered as a round component for loading combinations that include ice

Equivalent flat plate areas based on Revision C of this Standard shall be multiplied by a force coefficient, C_a, equal to 2.0 except when the appurtenance is made up of round members only, a force coefficient of 1.8 may be applied.

2.6.11.2.1 Antenna Mounting Pipes

The effective projected area of a mounting pipe above and below the shielded portion of the mounting pipe in accordance with 2.6.11.4 shall be included in the term $\Sigma(C_a A_a)_N$ with C_a equal to 1.0. The effective projected area of the entire mounting pipe shall be included in the term $\Sigma(C_a A_a)_T$ with C_a determined from Table 2-9.

2.6.11.2.2 Effective Projected Area for Mounting Frames (Figure 2-7)

The effective projected area associated with the windward face normal to the azimuth of a mounting frame, $(EPA)_N$, shall be determined from the equation:

$(EPA)_N = (EPA)_{MN} + (EPA)_{FN}$

where:

$(EPA)_{MN}$ = effective projected area of the frame = $C_{as}(A_f + R_{rf} A_r)$

$C_{as} = 1.58 + 1.05\,(0.6 - \varepsilon)^{1.8}$ for $\varepsilon \leq 0.6$

$C_{as} = 1.58 + 2.63\,(\varepsilon - 0.6)^{2.0}$ for $\varepsilon > 0.6$

A_f = projected area of flat components of the mounting frame

$R_{rf} = 0.6 + 0.4\,\varepsilon^2$

ε = solidity ratio of mounting frame without antennas and mounting pipes

= $(A_f + A_r)/A_g$

A_r = projected area of round components of the mounting frame

A_g = gross area of the frame as if it were solid defined by the largest outside dimensions of the elements included in A_f and A_r

Note: For square or triangular truss mounting frames (refer to Figure 2-7), C_{as} shall be equal to C_f in accordance with 2.6.11.1.1.

$(EPA)_{FN}$ = the effective projected area in a plane parallel to the face of the mounting frame of all members supporting the mounting frame

$= 0.5\,[\,2.0(\Sigma A_{fs}) + 1.2(\Sigma A_{rs})]$

A_{fs} = projected area of flat components supporting the mounting frame without regard to shielding or overlapping members

A_{rs} = projected area of round components supporting the mounting frame without regard to shielding or overlapping members

The effective projected area associated with the windward side of a mounting frame, $(EPA)_T$, shall be determined from the equation:

$$(EPA)_T = (EPA)_{FT} + 0.5\Sigma\,(EPA)_{FTi} + 0.5\,\Sigma\,(EPA)_{MT}$$

where:

$(EPA)_{FT}$ = the effective projected area in a plane transverse to the face of the mounting frame of a frame/truss supporting the mounting frame (the larger frame/truss when more than one is present)

$(EPA)_{FTi}$ = the effective projected area in a plane transverse to the face of the mounting frame of any additional frames/trusses supporting the mounting frame

Note: The effective projected area of frame/truss supporting members shall be determined in accordance with the equation for $(EPA)_{MN}$. Alternatively, a drag factor of 2.0 may be applied to flat members and a drag factor of 1.2 applied to round members without regard to shielding or overlapping members.

$(EPA)_{MT}$ = the effective projected area, in a plane transverse to the face of the mounting frame, of all mounting frame members and all other miscellaneous support members (i.e. tiebacks) without regard to shielding or overlapping members determined using a drag factor of 2.0 applied to flat members and a drag factor of 1.2 applied to round members

The shielding factor, K_a, shall be equal to 1.0 except when three or more mounting frames are mounted at the same relative elevation, a 0.80 shielding factor, K_a, may be applied to the mounting frame $(EPA)_N$ and $(EPA)_T$. When the three or more mounting frames are mounted in an arrangement that results in shielding to the structure and to the other mounting frames (refer to Figure 2-8), the shielding factor, K_a, may be reduced to a value equal to 0.75. No shielding shall be considered for the supporting structure.

Antennas and mounting pipes supported on mounting frames shall be considered as generic appurtenances using a value of K_a equal to 0.9 except a value of K_a equal to 0.8 may be used when 3 or more mounting frames are mounted at the same relative elevation.

2.6.11.2.3 Effective Projected Area for Symmetrical Frame/Truss Platforms

The effective projected area, $(EPA)_A$, of frame/truss triangle or square symmetrical platforms (refer to Figure 2-9) that are continuous around the perimeter of a structure (or with a horizontal gap between the corners of adjacent faces less than or equal to 10% of the width of the platform) shall be determined as if the platform were a section of a latticed structure in accordance with 2.6.11.1 using directionality factors D_f and D_r = 1.0. The projected area of all supporting members for the entire platform shall be projected onto a plane parallel to a face without regard to shielding or overlapping members of the platform or the supporting structure. A drag factor of 2.0 for flat members, a drag factor of 1.2 for round members and a drag factor of $2.0 - 6.0(r_s) \geq 1.25$ for square and rectangular HSS members shall be applied to the projected areas of the supporting members where r_s is the ratio of the outside corner radius to the outside HSS projected width. When the outside corner radius of an HSS section is not known, r_s shall be determined based on an outside corner radius equal to 2.25 times the nominal HSS wall thickness. Fifty percent of the total effective projected area of the supporting members shall be added to the projected area of the platform. The resulting total effective projected area shall be used for all wind directions with a shielding factor, K_a, equal to 1.0. No shielding shall be considered for the supporting structure. Antennas and mounting pipes supported on the platform shall be considered as generic appurtenances with a value of K_a equal to 0.75.

2.6.11.2.4 Effective Projected Area for Low Profile Platforms

An effective projected area, $(EPA)_A$, of low profile symmetrical platforms (refer to Figure 2-10) that are continuous around the perimeter of a structure (or with a horizontal gap between the corners of adjacent faces less than or equal to 10% of the width of the platform) shall be determined by summing the projected areas of all members of the platform onto a plane parallel to a face of the platform without regard to shielding or overlapping members of the platform or the supporting structure. A drag factor of 2.0 for flat members, a drag factor of 1.2 for round members and a drag factor of $2.0 - 6.0(r_s) \geq 1.25$ for square and rectangular HSS members shall be applied to the projected areas of the supporting members where r_s is the ratio of the outside corner radius to the outside HSS projected width. When the outside corner radius of an HSS section is not known, r_s shall be determined based on an outside corner radius equal to 2.25 times the nominal HSS wall thickness. The total effective projected area shall be multiplied by a shielding factor, K_a, equal to 0.75 for square platforms and 0.67 for triangular platforms. The resulting effective projected area shall be used for all wind directions. No shielding shall be considered for the supporting structure. Antennas and mounting pipes supported on the platform shall be considered as generic appurtenances using a value of K_a equal to 0.8.

2.6.11.2.5 Effective Projected Area for Symmetrical Circular Ring Platforms

The effective projected area, $(EPA)_A$, of symmetrical circular ring platforms (refer to Figure 2-11) that are continuous around the perimeter of a structure shall be determined by considering the supporting members of the platform and the ring members as individual members. The projected area of each ring member shall be equal to the product of the diameter of the ring and the projected vertical dimension of the ring member exposed to the wind. The projected area of all supporting members for the entire platform shall be determined by projecting all supporting members onto a vertical plane without regard to shielding or overlapping members of the platform or the supporting structure. A drag factor of 2.0 for flat members, a drag factor of 1.2 for round members and a drag factor of $2.0 - 6.0(r_s) \geq 1.25$ for square and rectangular HSS members shall be applied to the projected areas of the supporting members and the ring members where r_s is the ratio of the outside corner radius to the outside HSS projected width. When the outside corner radius of an HSS section is not known, r_s shall be determined based on an outside corner radius equal to 2.25 times the nominal HSS wall thickness. A 0.50 factor shall be applied to the total effective projected area of the supporting members and a 1.75 factor shall be applied to the total projected area of the ring members. The resulting total effective projected area shall be used for all wind directions with a shielding factor, K_a, equal to 1.0. No shielding shall be considered for the supporting structure. Antennas and mounting pipes supported on the platform shall be considered as generic appurtenances using a value of K_a equal to 0.8.

Notes for all mounting frame/platform types:

1. K_a shall equal 1.0 for antennas and antenna mounting pipes under transitional or supercritical flow conditions.
2. Grating and other horizontal working surfaces need not be included in the projected area.

2.6.11.3 Design Wind Force on Guys

The design wind force on guys, F_G, shall be determined in accordance with the following equation:

$$F_G = C_d\, d\, L_G\, G_h\, q_z \sin^2(\theta_g)$$

where:

F_G = force applied normal to the chord of the guy in the plane containing the guy chord and the wind (refer to Figure 2-12)

C_d = 1.2, force coefficient for guy

d = guy diameter including ice for loading combinations that include ice

L_G = length of guy

G_h = gust effect factor from 2.6.9.2

q_z = velocity pressure at mid-height of guy from 2.6.11.6

θ_g = true angle of wind incidence to the guy chord

Note: A higher force coefficient, C_d, or an increased effective guy diameter may be required when attachments such as spoilers, insulators, markers, etc. are attached to a guy.

The design wind force and ice thickness may be assumed to be uniform based on the velocity pressure and ice thickness at the mid-height of each guy or guy segment. The length of each guy or guy segment may be assumed to equal the chord length. The design wind force shall be considered as a distributed force normal to the guy chord.

For ground-supported structures, mid-height shall be referenced to the ground elevation at the base of the structure. For structures supported on buildings or other supporting structures, the mid-height of a guy shall be measured from the mid-height elevation of the guy to the ground level of the building or other supporting structure. The height above ground, z, for a guy segment shall not be less than zero.

2.6.11.4 Shielding

Shielding, except as noted herein, may be considered for intersecting or parallel elements. The unshielded element shall be considered as flat unless both elements are round. Full shielding may be considered when the clear distance between the elements in the direction under consideration for determining effective projected areas (EPA) is less than or equal to 2.0 times the smallest projected dimension of the element in the direction under consideration. No shielding shall be considered for clear distance ratios greater than 4.0. Linear interpolation shall be allowed for ratios between 2.0 and 4.0 (refer to Figure 2-13).

Shielding of the members of the supporting structure from an appurtenance shall not be considered when a value of K_a less than 1.0 per 2.6.11.2 is used to determine the design wind force on the appurtenance. Refer to 2.6.11.2.1 for shielding of antenna mounting pipes.

Note: Shielding considerations will vary with wind direction.

2.6.11.5 Round or Elliptical Transmission Lines Mounted in Clusters or Blocks

The projected area of each line in a cluster or block, independent of their spacing or location within the group, (i.e. no shielding of lines and no reduction of ice thickness) shall be included in the calculation of wind loads using a force coefficient, C_a, equal to 1.2 (based on round/elliptical lines), except that the group of lines need not be considered larger than an equivalent appurtenance with a width equal to the maximum out-to-out dimension of the group for both the normal and transverse sides with a force coefficient, C_a, equal to 1.5 for square or rectangular clusters and 1.2 for round clusters (refer to Figure 2-14). For loading conditions that include ice, a force coefficient, C_a, equal to 1.5 shall apply for both round, square and rectangular clusters.

Note: The width of the equivalent appurtenance may be used for determining shielding in accordance with 2.6.11.4.

Appurtenances may be considered as part of a cluster or block when the center-to-center spacing between the adjacent appurtenances does not exceed 3 times the larger width of the appurtenance or the adjacent appurtenances within the cluster or block.

For purpose of calculating the weight of ice, the radial thickness of ice shall be considered on each individual line except that the total cross section of ice need not exceed the area of a cluster as indicated in Figure 2-14.

2.6.11.6 Velocity Pressure

The velocity pressure, q_z, evaluated at height z shall be calculated by the following equation:

$$q_z = 0.00256\, K_z\, K_{zt}\, K_s\, K_e\, K_d\, V^2 \ (\text{lb/ft}^2)$$

$$= 0.613\, K_z\, K_{zt}\, K_s\, K_e\, K_d\, V^2 \ [\text{N/m}^2]$$

where:

K_z = velocity pressure coefficient from 2.6.5.2

K_{zt} = topographic factor from 2.6.6.2

K_s = rooftop wind speed-up factor from 2.6.7

K_e = ground elevation factor from 2.6.8

K_d = wind direction probability factor from Table 2-2

V = the basic wind speed for the loading condition under investigation, mph [m/s]

2.7 Seismic Load Effects

2.7.1 Definitions

Design Earthquake: the earthquake effects that are two-thirds of the corresponding maximum considered earthquake.

Overstrength Factor: a factor applied to seismic load effects to determine design strengths for anchorages to provide sufficient reserve strength to remain elastic during potential inelastic behavior of the structure.

Redundancy Factor: a factor applied to horizontal seismic load effects based on the extent of the structural redundancy associated with the value of response modification coefficient, R, used to determine seismic load effects.

Site Class: a classification assigned to a site based on the types of soil present.

2.7.2 Symbols and Notations

δ_i = lateral deflection at level i;

ϕ_{im} = displacement amplitude at level i when vibrating in the m^{th} mode;

ϕ_{zm} = displacement amplitude at level z when vibrating in the m^{th} mode;

Ω = overstrength factor;

ρ = redundancy factor;

A_{gi} = area of an individual guy at level i;

A_s = earthquake amplification factor;

C_g = natural frequency conversion factor for guyed masts;

C_s = seismic response coefficient;

C_{zm} = seismic force distribution factor for the m^{th} mode;

D = dead load of structure and appurtenances, excluding guy assemblies;

D_g = dead load of guy assemblies;

E = modulus of elasticity, 29,000 ksi [200,000 MPa];

E_h = horizontal seismic load effects;

E_v = vertical seismic load effects;

F_a = short-period site coefficient;

F_{SZ} = lateral seismic force at level z;

F_v = long-period site coefficient;

F_y = minimum design yield strength;

F_{zm} = seismic forces at level z for the m^{th} mode;

f_1 = fundamental frequency of structure;

f_m = frequency of structure for the mode under consideration;

G_{ri} = average guy radius for guys at level i;

g = acceleration due to gravity, 32.2 ft/sec^2 [9.81 m/sec^2];

H_{gi} = height above base to guy level i;

h = height of structure;
h_i = height from the base of structure to level i;
h_z = height from the base of structure to level z;
I = importance factor;
i = number designating a level of a structure;
I_{avg} = average moment of inertia of structure;
I_{bot} = moment of inertia at base of structure;
I_{top} = moment of inertia at top of structure;
K_f = coefficient used to determine fundamental frequencies of a structure;
K_g = equivalent stiffness of guys;
K_m = simplified natural frequency conversion factor for guyed masts;
k_e = seismic force distribution exponent;
L_{gi} = average chord length of guys at level i;
L_p = height of a pole structure;
m = subscript denoting quantities in the m^{th} mode;
N = standard penetration resistance of a soil;
N_i = number of guys at guy level i;
n = number of levels comprising a structural model or number of guy levels for a guyed mast;
PI = plastic index of a soil;
Q_E = effects of horizontal seismic (earthquake-induced) forces;
R = response modification coefficient;
R_y = ratio of expected yield stress to the minimum specified yield stress;
S_1 = spectral response acceleration parameter at a period of 1 second;
S_{am} = design spectral response acceleration parameter at period T_m;
S_{D1} = design spectral response acceleration parameter at a period of 1 second;
S_{DS} = design spectral response acceleration parameter at short periods;
S_s = spectral response acceleration parameter at short periods;
S_U = undrained shear strength of a soil;
T = fundamental period of structure ($1/f_1$);
T_L = long-period transition period;
T_m = period for the m^{th} mode;
T_O, T_S = periods used to define a design response spectrum;
V_s = total seismic shear force;
V_{sm} = portion of the base shear contributed by the m^{th} mode;
W = weight of structure above ground including appurtenances and upper half of guy assemblies;
W_1 = weight used to determine the fundamental frequency of a structure;
W_2 = weight of structure and appurtenances within top 5% of structure height;

W_L = weight of structure excluding appurtenances;

W_m = effective modal gravity load;

W_t = total weight of structure including appurtenances and guy assemblies;

W_u = weight of discrete appurtenances in the top third of structure;

w_a = average face width of a latticed structure;

w_i = portion of total gravity load assigned to level i;

w_o = face width at base of a latticed structure;

w_z = portion of total gravity load assigned to level z;

z = number designating the level under consideration.

2.7.3 General

Structures addressed in the scope of this Standard require special considerations of their response characteristics in regions of seismicity. The provisions of this Standard provide design criteria to ensure sufficient strength, ductility, stability and post-elastic energy dissipation to resist the effects of seismic ground motions. Special detailing requirements for steel structures are not required due to the magnitude of the response modification coefficients specified in this Standard. All foundation and anchorage requirements to provide the level of ductility and post-elastic energy dissipation of the structure assumed for earthquake design are specified in 2.7.9 and Section 9.0.

Risk Category IV structures shall not be located where a known potential exists for an active fault to cause rupture of the ground surface.

Drift limitations do not apply to structures addressed in the scope of this Standard.

Foundations shall be considered as non-elastic for the purposes of determining seismic load effects. Reduction of foundation reactions due to seismic load effects shall not be permitted.

Seismic load effects shall be evaluated in accordance with the seismic analysis procedures specified in 2.7.6 and 2.7.7.

1. An importance factor, I, shall be determined from Table 2-3 based on the structure risk category listed in Table 2-1.
2. Determine the spectral response acceleration parameters (expressed as a ratio to the acceleration due to gravity) at short periods, S_s, and at a period of 1 second, S_1, and the long-period transition period, T_L, from 2.7.4.
3. Determine the site class based on the soil properties at the site in accordance with Table 2-10.
4. Determine the short-period and long-period site coefficients, F_a and F_v, based on the site class from Tables 2-11 and 2-12 respectively.
5. The design spectral response acceleration parameters at short periods, S_{DS}, and at a period of 1 second, S_{D1}, shall be determined in accordance with 2.7.5.

2.7.4 Seismic Design Parameters

The spectral response acceleration parameter at short periods, S_s, the spectral response acceleration parameter at a period of 1 second, S_1, and the long-period transition period, T_L, shall be as given in Annex B except as provided in 2.7.4.1. It shall be permissible to determine seismic design parameters from the ASCE 7 online Hazard Tool based on ASCE 7-16.

The value of T_L shall be 12 seconds for Guam, the Northern Mariana Islands and American Samoa.

2.7.4.1 Site-Specific Procedures for Determining Seismic Design Parameters

For regions not included in Annex B, the seismic design parameters S_s, S_1 and T_L shall be based on regional seismicity and geology. The maximum considered earthquake ground motion shall be based on a risk-targeted maximum considered earthquake ground motion response acceleration in accordance with ASCE 7-16 represented by a 5% damped acceleration response spectrum expected to achieve a 1% probability of collapse within a 50-year period.

A site-specific geotechnical investigation and a dynamic site response analysis shall be used to determine S_s, S_1 and T_L for structures in Site Class E locations where $S_s \geq 1.0$ and for all Site Class F locations (refer to Tables 2-11 and 2-12).

2.7.5 Design Spectral Response Acceleration Parameters

The design spectral response acceleration parameter at short periods, S_{DS}, and at a period of 1 second, S_{D1}, shall be determined from the following equations:

$$S_{DS} = 2/3\ F_a\ S_s, \text{ but not less than } S_{D1}$$

$$S_{D1} = 2/3\ F_v\ S_1$$

where:

F_a = short-period site coefficient based on site class and spectral response acceleration parameter at short periods from Table 2-11

F_v = long-period site coefficient based on site class and spectral response acceleration parameter at a period of 1 second from Table 2-12

Note: When S_s and S_1 are based on site-specific response analysis procedures, F_a and F_v shall be equal to 1.0.

2.7.6 Vertical Seismic Load Effect

The vertical seismic load effects, E_v, shall be determined in accordance with the following equation:

$$E_v = 0.2\ S_{DS}\ (D + D_g)$$

where:

S_{DS} = design spectral response acceleration parameter at short periods from 2.7.5

D = dead load of structure and appurtenances, excluding guy assemblies

D_g = dead load of guy assemblies

2.7.7 Horizontal Seismic Load Effect

The horizontal seismic load effects, E_h, shall be determined in accordance with the following equation:

$$E_h = \rho Q_E$$

where:

ρ = redundancy factor equal to 1.0

Q_E = effects of horizontal seismic forces determined in accordance with 2.7.7.1 (Equivalent Lateral Force Procedure) or alternatively 2.7.7.2 (Modal Analysis Procedure) for self-supporting structures

Torsional moments shall be considered in the analysis for high mass appurtenances located outside the cross-section of the structure.

The horizontal seismic load effect for appurtenances and structures supported on buildings or other supporting structures shall be determined in accordance with 2.7.8.

2.7.7.1 Equivalent Lateral Force Procedure

1. Determine the total weight, W, of the structure above ground including appurtenances. For guyed masts, W shall also include the weight of the upper half of the guy assemblies attached to the structure.
2. Calculate the seismic response coefficient, C_s, in accordance with 2.7.7.1.1.
3. Calculate the total seismic shear force, V_s, in accordance with 2.7.7.1.1.
4. Distribute the total seismic shear force in accordance with 2.7.7.1.2.
5. Analyze the structure statically in accordance with Section 3.0 using the seismic forces as external loads considered to occur in the same directions considered for wind loading without interaction effects from multiple directions.

2.7.7.1.1 Total Seismic Shear Force

The total seismic shear force, V_s, in a given direction shall be determined in accordance with the following equation:

$$V_s = C_s W$$

The seismic response coefficient, C_s, shall be determined as follows:

$$C_s = \frac{S_{DS} I}{R}$$

The value of C_s need not exceed the following:

$$C_s = \frac{S_{D1}\, I}{T R} \text{ when } T \leq T_L$$

$$C_s = \frac{S_{D1}\, T_L I}{T^2 R} \text{ when } T > T_L$$

The value of C_s shall not be less than the following:

$$C_s = 0.044\, S_{DS}\, I$$

or

$$C_s = 0.03$$

In addition, for sites where $S_1 \geq 0.6$, C_s shall not be less than:

$$C_s = \frac{0.8\, S_1 I}{R}$$

where:

S_{DS} = design spectral response acceleration parameter at short periods from 2.7.5

S_{D1} = design spectral response acceleration parameter at a period of 1 second from 2.7.5

S_1 = spectral response acceleration parameter at a period of 1 second from 2.7.4

T = fundamental period of the structure in accordance with 2.7.7.1.3

T_L = long-period transition period from 2.7.4

W = total weight of the structure above ground including appurtenances, for guyed masts, W also includes one-half the weight of guy assemblies

I = importance factor from Table 2-3

R = response modification coefficient equal to 3.0 for latticed self-supporting or bracketed structures and guyed masts and 1.5 for self-supporting pole structures

2.7.7.1.2 Vertical Distribution of Seismic Forces

The lateral seismic force, F_{sz}, induced at any level, z, shall be determined from the following equation:

$$F_{sz} = \frac{w_z h_z^{k_e}}{\sum_{i=1}^{n} w_i h_i^{k_e}} Vs$$

where:

V_s = total seismic shear force from 2.7.7.1.1

n = number of levels comprising the structural mode

i = number designating the level of the structure starting from the base to the uppermost level

z = number designating the level under consideration

w_z = portion of total gravity load, W, assigned to level under consideration

h_z = height from the base of structure to level z

w_i = portion of total gravity load, W, assigned to level i

h_i = height from the base of structure to level i

k_e = seismic force distribution exponent, equal to 1 for structures having a fundamental period of 0.5 sec. or less, equal to 2 for structures having a fundamental period of 2.5 sec. or more. For structures having a fundamental period between 0.5 sec. and 2.5 sec., k_e shall be 2 or shall be determined by linear interpolation between 1.0 and 2.0.

Note: For guyed masts, one-half of the weight of guys shall be assigned to the corresponding guy attachment points on the mast.

2.7.7.1.3 Fundamental Period of the Structure

The fundamental period, T, of the structure in the direction under consideration shall be determined using the structural properties and deformational characteristics of the resisting elements. Alternatively, the fundamental period, T, shall be permitted to be determined from 2.7.7.1.3.1 through 2.7.7.1.3.4.

Note: For guyed masts, the initial tension and weight of guy assemblies must be included in the analysis to properly model the stiffness of the structure.

2.7.7.1.3.1 Self-Supporting or Bracketed Structures

1. Construct a mathematical model that represents the spatial distribution of mass and stiffness throughout the structure.
2. Complete a linear elastic analysis using a lateral force distribution equal to the distribution of weight for the structural model.
3. Calculate the fundamental period according the to the following equation:

$$T = 2\pi\sqrt{\frac{\sum_{i=1}^{n} w_i\delta_i^2}{g\sum_{i=1}^{n} w_i\delta_i}} \quad \text{Seconds}$$

where:

n = number of levels comprising the structural model

i = number designating the level of the structure starting from the base to the uppermost level

w_i = lateral force at level i (equivalent to the vertical weight at level i)

δ_i = lateral defection at level i

g = acceleration due to gravity

2.7.7.1.3.2 Self-Supporting Latticed Structures

$T = 1/f_1$ Seconds

$$f_1 = \frac{K_f(w_a)}{h^2}\sqrt{\frac{W_1}{W_1 + W_2}} \quad \text{Hertz}$$

$$W_1 = W\left[\left(\frac{w_a}{w_o}\right)^2 + 0.15\right]$$

where:

K_f = 4,540 for h and w_a in feet and 1,500 for h and w_a in meters

W = total weight of the structure including appurtenances

W_2 = weight of structure and appurtenances within top 5% of the structure height

w_a = average face width of structure, ft. [m]

w_o = face width at base of structure, ft. [m]

h = height of structure, ft. [m]

2.7.7.1.3.3 Self-Supporting Pole Structures

T = $1/f_1$ Seconds

$$f_1 = \frac{1}{2\pi}\sqrt{\frac{3EI_{avg}g}{L_p^{\,3}(W_u + 0.236W_L)}} \text{ Hertz}$$

where:

E = modulus of elasticity

I_{avg} = $(I_{top} + I_{bot})/2$

I_{top} = moment of inertia at top of structure

I_{bot} = moment of inertia at base of structure

g = acceleration due to gravity

L_p = height of the structure

W_u = weight of discrete appurtenances in the top third of structure

W_L = $W_t - W_u$

W_t = total weight of the structure including appurtenances

2.7.7.1.3.4 Guyed Masts

T = $1/f_1$ Seconds

$$f_1 = C_g\sqrt{\frac{K_g}{W_t}} \text{ Hertz}$$

$$K_g = \sum_{i=1}^{n}\left[\frac{N_i(A_{gi})(G_{ri})(H_{gi})}{h(L_{gi})^2}\right]$$

where:

C_g = 176.5 [8.70]

W_t = total weight of structure including appurtenances and the total weight of all guy assemblies, kips [kN]

n = number of guy levels

i = number designating guy level starting from the base to the uppermost guy level

N_i = number of guys at guy level i

A_{gi} = area of an individual guy at level i, in.^{2} [mm^2]

G_{ri} = average guy radius for guys at level i, ft. [m]

H_{gi} = height above base to guy level i, ft. [m]

h = height of structure, ft. [m]

L_{gi} = average chord length of guys at level i, ft. [m]

Alternatively, the following simplified equation may be used to determine f_1:

$$f_1 = K_m\sqrt{\frac{1}{h^{1.5}}} \text{ Hertz}$$

where:

K_m = 122 [50]

h = height of structure, ft. [m]

2.7.7.2 Modal Analysis Procedure

The modal analysis procedure shall be limited to pole structures and triangular or square cross section latticed self-supporting or bracketed towers. Latticed towers shall be limited to structures with symmetrical identical members in each panel of the structure (i.e. same main bracing pattern with same size bracing members and same size leg members in a given panel).

1. Construct a mathematical model of the structure that represents the spatial distribution of mass and stiffness throughout the structure.
2. Conduct an analysis for two horizontal directions corresponding to each principle orthogonal axis of the cross section (i.e. global horizontal X and Y axes) to determine for each direction the natural modes of vibration for the structure including the period of each mode, the modal shape vectors, and the effective modal gravity load. For each horizontal direction, the analysis shall include a sufficient number of modes to obtain a combined effective modal gravity load, $\sum W_m$, of at least 90% of the total gravity load, W, for pole structures and 85% of the total gravity load, W, for self-supporting or bracketed latticed structures.
3. Establish the design response spectrum from 2.7.7.2.1.
4. For each horizontal direction, calculate the effective modal gravity load, W_m, and the base shear, V_{sm}, contributed by each mode from 2.7.7.2.2.

5. For each horizontal direction, scale the modal base shears, V_{sm}, when required by comparing the combined modal base shear for all modes, $\sum V_{sm}$, for a given direction to the total seismic shear force, V_s, from 2.7.7.1.1. When $\sum V_{sm}$ is less than 0.85 V_s, scale V_{sm} for each mode for a given direction by multiplying by the ratio of 0.85 $V_s / \sum V_{sm}$. The combined modal base shear for a given direction shall be determined by calculating the square root of the sum of squares of the base shears for each mode.
6. For each horizontal direction, determine the seismic forces for each level of the structure for each mode in accordance with 2.7.7.2.3 using the scaled magnitude of V_{sm} as required.
7. For each horizontal direction, analyze the structure statically using the seismic forces as external loads for each mode without dead loads.
8. For each horizontal direction, combine the load effects of each mode by calculating the square root of the sum of the squares of the load effects for each mode. For a square cross section latticed tower, the combined leg load effects shall be multiplied by 1.41 to account for a diagonal direction.
9. The highest combined load effects considering both horizontal directions shall be considered as the horizontal seismic load effect Q_E. The seismic load effect for each member shall be considered as both additive and subtractive to the other load effects from the loading combinations in 2.3.2.

2.7.7.2.1 Design Response Spectrum

The design response spectrum shall be determined in accordance with the following equations as represented in Figure 2-15:

S_{am} = spectral response acceleration parameter at period T_m for the mode under investigation

T_m = $1/f_m$, sec

f_m = frequency of structure for the mode under consideration, Hertz

$S_{am} = S_{DS} (0.4 + 0.6\, T_m / T_o)$ when $T_m \le T_o$

$S_{am} = S_{DS}$ when $T_o < T_m \le T_s$

$S_{am} = S_{D1} / T_m$ when $T_s < T_m \le T_L$

$S_{am} = \dfrac{S_{D1}\, T_L}{T_m^{\ 2}}$ when $T_m > T_L$

where:

S_{DS} = design spectral response acceleration parameter at short periods from 2.7.5

S_{D1} = design spectral response acceleration parameter at a period of 1 second from 2.7.5

T_o = $0.2\, S_{D1}/S_{DS}$

T_s = S_{D1}/S_{DS}

T_L = long-period transition period from 2.7.4

2.7.7.2.2 Base Shear Contributed by Each Mode

The base shear, V_{sm}, contributed by each mode shall be determined from the following equations:

$$V_{sm} = \frac{S_{am} W_m I}{R}$$

$$W_m = \frac{\left(\sum_{i=1}^{n} w_i \phi_{im} \right)^2}{\sum_{i=1}^{n} w_i \phi_{im}^2}$$

where:

S_{am} = spectral response acceleration parameter at period T_m from 2.7.7.2.1

W_m = effective modal gravity load

I = importance factor from Table 2-3

R = response modification coefficient equal to 3.0 for latticed self-supporting or bracketed structures and 1.5 for self-supporting pole structures

n = number of levels comprising the structural model

i = number designating the level of the structure starting from the base to the uppermost level

m = subscript denoting quantities in the m^{th} mode

w_i = portion of total gravity load, W, assigned to level i

ϕ_{im} = displacement amplitude at the i^{th} level of the structure when vibrating in its m^{th} mode

Note: Sign of ϕ_{im} shall be maintained for determining W_m.

2.7.7.2.3 Seismic Forces Contributed by Each Mode

The seismic forces, F_{zm}, at each level of the structure, z, for each mode, m, shall be determined in accordance with the following equation:

$$F_{zm} = C_{zm} V_{sm}$$

where:

$$C_{zm} = \frac{w_z \phi_{zm}}{\sum_{i=1}^{n} w_i \phi_{im}}$$

z = number designating the level under consideration

n = number designating the uppermost level of the structure

i = number designating the level of the structure starting from the base to the uppermost level

C_{zm} = seismic force distribution factor for the m^{th} mode

V_{sm} = portion of the base shear contributed by the m^{th} mode from 2.7.7.2.2

w_z = portion of total gravity load, W, assigned to level z

w_i = portion of total gravity load, W, assigned to level i

ϕ_{zm} = displacement amplitude at level z when vibrating in the m^{th} mode

ϕ_{im} = displacement amplitude at level i when vibrating in the m^{th} mode

Note: Sign of ϕ_{zm} and ϕ_{im} shall be maintained for determining C_{zm}.

2.7.8 Structures Supported on Buildings or Other Supporting Structures

Seismic load effects for appurtenances and for structures less than or equal to 100 ft. in height may be evaluated in accordance with 2.7.6 and 2.7.7.

Interaction effects between the structure and the supporting structure shall be considered for structures over 100 ft. [30 m] in height. Rational methods that account for the dynamic characteristics of the structures shall be used for these structures and may be used for any supported structure to determine seismic load effects. However, seismic load effects shall not be less than 80% of the seismic load effects determined in accordance with 2.7.6 and 2.7.7. Alternatively, it shall be permitted to apply the amplification factor from 2.7.8.1 to the seismic load effects determined in accordance with 2.7.6 and 2.7.7.

2.7.8.1 Amplification Factor

Seismic load effects shall be multiplied by an amplification factor, A_s, as follows:

1. For self-supporting or guyed structures, the amplification factor shall be equal to 3.0.
2. For structures bracketed to the supporting structure at the mid-height of the structure or above, the amplification factor shall be equal to 1.0.
3. For structures bracketed to the supporting structure below the mid-height of the structure, the amplification factor shall be equal to 3.0 or may be linearly interpolated between 3.0 and 1.0 based on the elevation of the bracket with respect to the mid-height of the structure.

2.7.9 Anchorage Design Strengths

An overstrength factor, Ω, equal to 1.5 shall be applied to seismic load effects for the purpose of determining the required design strength of guy anchor shafts and anchor rods in accordance with Section 4.0.

For Risk Category II, III and IV pole structures, when the actual material specification used to fabricate a pole is known, anchorage design strengths need not exceed the expected strength equal to the nominal flexural strength of the pole structure (i.e. without a resistance factor) in accordance with Section 4.0 multiplied by the ratio of the expected yield stress to the minimum specified yield stress, R_y, from Table 2-13.

2.8 Serviceability Requirements

2.8.1 Definitions

Displacement: the horizontal displacement under service loads of a point from the unfactored no-wind load position.

Service loads: the loading combination used to calculate serviceability limit state deformations.

Sway: the angular rotation under service loads of an antenna beam path in the local vertical plane of the antenna from the unfactored no-wind load position.

Twist: the angular rotation under service loads of an antenna beam path in the local horizontal plane of the antenna from the unfactored no-wind load position.

2.8.2 Limit State Deformations

The deformations under service loads at any point on a structure, unless otherwise required, shall not exceed the following:

1. A rotation of 4 degrees about the vertical axis (twist) or any horizontal axis (sway) of the structure.
2. A horizontal displacement of 3% of the height above the base of the structure.
3. For cantilevered tubular or latticed spines, poles or similar structures mounted on latticed structures, a relative horizontal displacement of 1.5% of the cantilever height measured between the tip of the cantilever and its base.

In the absence of more accurate data, the rotation (twist and sway) for microwave antennas shall be determined using Annex D based on an overall allowable 10 dB degradation in radio frequency signal level.

2.8.3 Service Loads

Service loads shall be defined by the following loading combination for a 60 mph [27 m/s] basic wind speed:

$$1.0\ D + 1.0\ D_g + 1.0\ W_o \quad \text{(refer to section 2.3.1)}$$

The horizontal wind forces for determining service loads shall be based on a wind direction probability factor, K_d, of 0.85 for all structures. The velocity pressure, q_z, shall be determined in accordance with 2.6.11.6 for a basic wind speed, V, equal to 60 mph [27 m/s].

Table 2-1: Risk Categorization of Structures

Use or Description of Structure	Risk Category
Structures that due to use or location represent a low risk to human life and/or damage to surrounding facilities in the event of failure. Structures in this category are used for services that are optional and/or where an extensive delay in returning the services would be acceptable such as: redundant wireless antennas; low-power radio access nodes (small cell); single-appurtenance supporting structures that allow for rapid repair or replacement, residential wireless and conventional 2-way radio communications; television, radio and scanner reception; wireless cable; amateur and CB radio communications.	I
Structures that due to use or location represent a moderate risk to human life and/or damage to surrounding facilities in the event of failure. Structures in this category are used primarily for redundant services (i.e. services that may be provided by other means) such as: commercial wireless communications (cellular, PCS, 3G, LTE, 4G, 5G, etc.); television and radio broadcasting; community access television (CATV); microwave communications; non-hardened sites that support antennas or equipment that may be used for redundant communications by police and fire departments, first responders, etc. during emergencies and small wind turbines. This category applies to all structures except those identified in Risk Categories I, III, and IV.	II
Structures that due to use or location represent a substantial risk to human life and/or damage to surrounding facilities in the event of failure. Structures with the potential to cause mass disruption (loss of power, transportation, water, etc.) of day-to-day civilian life in the event of failure. Structures in this category are used for communications across non-redundant and hardened networks such as: civil or national defense; rescue or disaster operations; military and navigation facilities.	III
Structures that due to use or location represent a substantial hazard to the community in the event of failure. Structures in this category are those that in the event of failure would threaten the functionality or integrity of facilities that are designated as Risk Category IV facilities.	IV

Table 2-2: Wind Direction Probability Factor

Structure Type	Wind Direction Probability Factor, K_d
Latticed structures with triangular, square or rectangular cross sections	0.85
All other latticed structures with other than triangular, square or rectangular cross sections	0.95
Tubular pole structures supporting exposed appurtenances and for the strength design of appurtenances	0.95
Tubular pole structures supporting antennas enclosed within a cylindrical shroud, with or without a flag or tubular structures which do not support appurtenances	1.00

Table 2-3: Importance Factors

Risk Category	Ice Thickness	Earthquake
I	N/A	N/A
II	1.00	1.00
III	1.15	1.25
IV	1.25	1.50
Note: Ice and earthquake loads do not apply to Risk Category I structures.		

Table 2-4: Exposure Category Coefficients

Exposure Category	Z_g	α'	K_{zmin}	K_e
B	1200 ft. [366 m]	7.0	0.70	0.90
C	900 ft. [274 m]	9.5	0.85	1.00
D	700 ft. [213 m]	11.5	1.03	1.10

Table 2-5: Topographic Category Coefficients

Topographic Category	K_t	f
2	0.43	1.25
3	0.53	2.00
4	0.72	1.50

Table 2-6: Ground Elevation Factor

Ground Elevation, z_s		Ground Elevation Factor K_e
Feet	Meters	
< 0	< 0	1.00
0	0	1.00
1000	305	0.96
2000	610	0.93
3000	914	0.90
4000	1219	0.86
5000	1524	0.83
6000	1829	0.80
> 6000	> 1829	See Note 2

Notes:

1. z_s = mean elevation of base of structure above sea level (negative below sea level).
2. The factor K_e shall be determined from the table using interpolation.
3. K_e is permitted to be equal to 1.0 for any ground elevation.

Table 2-7: Wind Direction Factors

Tower Cross Section	Square		Triangular		
Wind Direction	Normal	45°	Normal	60°	±90°
D_f	1.0	1 + .75ε (1.2 max)	1.0	0.80	0.85
D_r	1.0	1 + .75ε (1.2 max)	1.0	1.0	1.0
Wind directions measured from a line normal to the face of the structure					

Table 2-8a: Force Coefficients, C_f, for Tubular Structures with Linear Attachments

C mph-ft [m/s-m]	Round	18-Sided	16-Sided	12-Sided	8-Sided
< 39 [5.3] (Subcritical)	1.2	1.2	1.2	1.2	1.2
39 to 78	$46.8/(C)^{1.0}$	$16.6/(C)^{0.717}$	$14.4/(C)^{0.678}$	$4.12/(C)^{0.337}$	1.2
[5.3 to 10.6] (Transitional)	$[6.36/(C)^{1.0}]$	$[3.97/(C)^{0.717}]$	$[3.72/(C)^{0.678}]$	$[2.10/(C)^{0.337}]$	[1.2]
> 78 [10.6] (Supercritical)	0.60	0.73	0.75	0.95	1.2

$C = (K_{zt} K_z K_e)^{0.5} (V)(D)$ for D in ft. [m], V in mph [m/s]

V is the basic wind speed for the loading condition under investigation.

D is the pole outside diameter for rounds or the outside corner-to-corner width for polygons.

Notes:

The tabulated force coefficients include the effect of step bolts. When linear appurtenances such as ladders, waveguides, coax, brackets, or other similar projections are attached on the outside of the pole shaft, their projected areas shall be considered as follows (refer to Figure 2-5):

1. Attachments located entirely within the Windward or Leeward Zones do not have to be considered separately in addition to the structure unless otherwise noted. Attachments or portions of attachments located outside the Windward, Leeward or Lateral Zones shall be considered separately with appropriate force coefficients for appurtenances and the force coefficient for the pole shall be determined in accordance with Note 3.
2. For attachments within the Lateral Zones:
   a) When $R_L \le 0.2$, it shall be permissible to ignore the projected areas of the attachments.
   b) When $R_L > 0.2$, the projected areas of the attachments shall be considered separately in addition to the structure using appropriate force coefficients for appurtenances.
3. The C_f coefficients listed in this table shall be modified as follows:
   a) When $R_W \le 0.2$ and:
      i. $R_L \le 0.1$, modification of C_f is not required.
      ii. $0.1 < R_L \le 0.2$, C_f shall be multiplied by $[1.0 + 3(R_L - 0.1)]$.
      iii. $R_L > 0.2$, or alternatively for any value of R_L, C_f for subcritical flow shall be used.
   b) When $R_W > 0.2$, C_f for subcritical flow shall be used. In addition, C_f shall be multiplied by $[1.0 + 0.3125(R_W - 0.2)]$.
4. For iced conditions, C_f shall be based on subcritical flow for all values of C.
5. Linear interpolation, based on the inscribed angle of each side, between the values shown, shall be used for other cross-sections. The inscribed angle for a round cross section shall be considered to be 0 degrees.
6. Subcritical force coefficients shall be used for investigating fatigue strength requirements for small wind turbine support structures in accordance with Section 17.0.

R_W is the ratio of the sum of the projected areas of flat attachments in the Windward Zone to the projected area of the structure without attachments for the portion being considered. R_L is the ratio of the sum of the projected areas of round and flat attachments in both Lateral Zones to the projected area of the structure without attachments for the portion being considered. The projected area of the structure shall be determined using the average pole outside diameter for rounds and the outside corner-to-corner width for polygons for the section under consideration. For iced conditions, ice thickness need not be included in the determination of R_W and R_L.

Table 2-8b: Force Coefficients, C_f, for Tubular Structures without Linear Attachments

C mph-ft [m/s-m]	Round	18-Sided	16-Sided	12-Sided	8-Sided
< 39 [5.3] (Subcritical)	1.1	1.1	1.1	1.2	1.2
39 to 78	$124/(C)^{1.29}$	$20.9/(C)^{0.804}$	$17.7/(C)^{0.759}$	$7.43/(C)^{0.498}$	1.2
[5.3 to 10.6] (Transitional)	[$9.45/(C)^{1.29}$]	[$4.20/(C)^{0.804}$]	[$3.90/(C)^{0.759}$]	[$2.75/(C)^{0.498}$]	[1.2]
> 78 [10.6]	0.45	0.63	0.65	0.85	1.2

$C = (K_{zt} K_z K_e)^{0.5} (V)(D)$ for D in ft. [m], V in mph [m/s]
V is the basic wind speed for the loading condition under investigation.
D is the pole outside diameter for rounds or the outside corner-to-corner width for polygons.

Notes:

1. For iced conditions, C_f shall be based on subcritical flow for all values of C.
2. Linear interpolation, based on the inscribed angle of each side, between the values shown, may be used for other cross-sections. The inscribed angle for a round cross section shall be considered to be 0 degrees.
3. Subcritical force coefficients shall be used for investigating fatigue strength.
4. The tabulated force coefficients shall be permitted for pole structures with step bolts and a safety cable when no other linear attachments are present. The step bolts and safety cable shall be considered separately in addition to the structure using appropriate force coefficients for appurtenances.

Table 2-8c: Force Coefficient Reduction Based on Corner Radius

When the outside corner radius of a polygonal shape is known, the force coefficient, C_f, shall be permitted to be determined in accordance with the following equations:

$r_c \leq r_m,\ C_f = C_{fm}$

$r_m < r_c < r_r,\ C_f = C_{fr} + (C_{fm} - C_{fr})(r_r - r_c) / (r_r - r_m)$

$r_c \geq r_r,\ C_f = C_{fr}$

where:

r_c = ratio of outside corner radius to half outside flat-to-flat width of the polygonal shape

r_m = value of r_c below which there is no reduction in C_f due to corner radius from table below

r_r = value of r_c above which the shape is considered round

C_{fm} = force coefficient for the polygonal shape from Table 2-8a or 2-8b

C_{fr} = force coefficient for a round shape with an outside diameter equal to the outside flat-to-flat width of the polygonal shape from Table 2-8a or 2-8b

Corner Radius Ratios for Reduced Drag Factors

Shape	r_m	r_r
18-Sided	0.217	0.570
16-Sided	0.260	0.625
12-Sided	0.500	0.750
8-Sided	0.750	1.00

Note: No reduction in C_f shall be allowed when the corner radius is not known.

Table 2-9: Force Coefficients, C_a, For Appurtenances

Member Type		Aspect Ratio ≤ 2.5 C_a	Aspect Ratio = 7 C_a	Aspect Ratio ≥ 25 C_a
Flat		1.2	1.4	2.0
Square & Rectangular HSS		$1.2 - 2.8(r_s) \geq 0.85$	$1.4 - 4.0(r_s) \geq 0.90$	$2.0 - 6.0(r_s) \geq 1.25$
Round	C < 39 [5.3] (Subcritical)	0.70	0.80	1.2
	39 ≤ C ≤ 78 [5.3 ≤ C ≤ 10.6] (Transitional)	$4.14/(C)^{0.485}$ [$1.57/(C)^{0.485}$]	$3.66/(C)^{0.415}$ [$1.60/(C)^{0.415}$]	$46.8/(C)^{1.0}$ [$6.36/(C)^{1.0}$]
	C > 78 [10.6] (Supercritical)	0.50	0.60	0.60

where:

r_s = ratio of outside corner radius to outside width normal to the wind direction

$C = (K_{zt} K_z K_e)^{0.5} (V)(D)$ for D in ft. [m], V in mph [m/s]

V is the basic wind speed for the loading condition under investigation.

D is the pole outside diameter for rounds or the outside corner-to-corner width for polygons.

When the outside corner radius is not known, r_s shall be determined based on an outside corner radius equal to 2.25 times the nominal wall thickness of the HSS member.

Aspect ratio is the overall length/width ratio in the plane normal to the wind direction. (Aspect ratio is independent of the spacing between support points of a linear appurtenance, and the section length considered to have uniform wind load.)

Notes:

1. For cylindrical appurtenances, when irregularities such as flanges, hangers, etc., are present, effective projected areas shall be calculated as follows:
   a) When $R_a \leq 0.1$, it shall be permissible to ignore the projected areas of the irregularities.
   b) When $0.1 < R_a \leq 0.2$, the value for C_a shall be multiplied by $[1.0 + 3(R_a - 0.1)]$, and it shall be permissible to ignore the projected areas of the irregularities.
   c) When $R_a > 0.2$, or alternatively for any value of R_a, the value of C_a for subcritical flow shall be used. The projected areas of irregularities shall be considered separately in addition to the appurtenance using the appropriate force coefficients.

   Where R_a is the ratio of the sum of the projected areas of the irregularities on both sides of the appurtenance (perpendicular to the wind direction) to the projected area of the appurtenance without the irregularities for the portion being considered. For iced conditions, the ice thickness need not be considered in the determination of R_a.
2. For flat appurtenances, when irregularities such as flanges, hangers, etc., are present, the projected areas of the irregularities shall be considered separately in addition to the appurtenance using appropriate force coefficients except when R_a is less than or equal to 0.1, the projected areas of the irregularities may be ignored.
3. For iced conditions, C_a shall be based on subcritical flow for all values of C.
4. Linear interpolation may be used for aspect ratios other than those shown.
5. Subcritical force coefficients may conservatively be used for any value of C.

Table 2-10: Site Class Definitions

Site Class	Description of Soil for the Site Location to a Depth of 100 ft [30.5 m]	Standard Penetration Resistance, N Cohesionless Soils PI ≤ 20	Undrained Shear Strength, S_u Cohesive Soils PI > 20
A	Hard rock with 10 ft. [3 m] or less of soil between the rock surface and the bottom of the foundation.	N/A	N/A
B	Competent rock with moderate fracturing and weathering with 10 ft. [3 m] or less of soil between the rock surface and the bottom of the foundation.	N/A	N/A
C	Very dense soil, soft rock or highly fractured and weathered rock.	> 50	> 2 ksf [100 kPa]
D	Stiff soil.	15 to 50	1.0 to 2.0 ksf [50 to 100 kPa]
E	Soft clay (excluding site class F).	< 15	< 1.0 ksf [50 kPa]
		Soil profiles over 10 ft. [3 m] thick of soft clay (PI > 20, moisture content ≥ 40%, S_u < 0.5 ksf [25 kPa])	
F	Soils vulnerable to potential failure or collapse under seismic loading.	Soil profiles containing any of the following: peat and/or highly organic clays over 10 ft. [3 m] thick, very high plasticity clays (PI > 75) over 25 ft. [7.6 m] thick, soft/medium clays (S_u < 1.0 ksf [50 kPa]) over 120 ft. [36.6 m] thick, liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.	

Table 2-11: Short-Period Site Coefficient, F_a

Site Class	Spectral Response Acceleration Parameter at Short Periods, (S_s)					
	$S_s \le 0.25$	$S_s = 0.5$	$S_s = 0.75$	$S_s = 1.0$	$S_s = 1.25$	$S_s \ge 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D*	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4	1.7	1.3	Note 1	Note 1	Note 1
F	Note 1	Note 1	Note 1	Note 1	Note 1	Note 1

Linear interpolation is allowed between values shown.

Note 1: Site-specific procedures required in accordance with 2.7.4.1.

*The minimum value of F_a shall be equal to 1.2 when Site Class D is considered as a default Site Class, and is not based on geotechnical data from the site.

Table 2-12: Long-Period Site Coefficient, F_v

Site Class	Spectral Response Acceleration Parameter at a period of One Second, (S_1)					
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \ge 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2	2.0	1.9	1.8	1.7
E	4.2	3.3	2.8	2.4	2.2	2.0
F	Note 1	Note 1	Note 1	Note 1	Note 1	Note 1

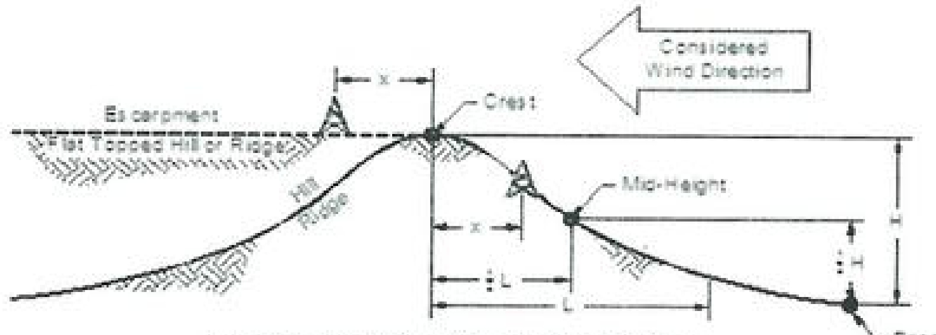
Linear interpolation is allowed between values shown.

Note 1: Site-specific procedures required in accordance with 2.7.4.1.

Table 2-13: R_y Values for Steel Materials

Minimum Design Yield Strength Fy, ksi [MPa]	Shapes	Plates/Coils
Fy < 36 [248]	1.5	1.3
36 [248] ≤ Fy ≤ 42 [290]	1.4	1.3
Fy > 42 [290]	1.3	1.1

Figure 2-1: Isolated Topographic Feature



Topographic Feature Factor, $K_1 = \beta K_1'$

K_1'					
Exposure Category	**Ridge**	**Flat Topped Ridge**	**Hill**	**Flat Topped Hill**	**Escarpment**
B	0.65	0.58	0.48	0.43	0.38
C	0.73	0.63	0.53	0.48	0.43
D	0.78	0.68	0.58	0.53	0.48
Slope Modifier, β					
$1/10 \leq H/L < 1/4$			$4(H/L)$		
$H/L \geq 1/4$			1.0		

Horizontal Distance Factor, K_2

Slope	Upwind: Hill, Ridge, Escarpment Downwind: Hill, Ridge	Downwind: Escarpment
$1/10 \leq H/L < 1/4$	$1 - (^{4x}/_{3L}) \geq 0$	$1 - (^{x}/_{2L}) \geq 0$
$H/L \geq 1/4$	$1 - (^{x}/_{3H}) \geq 0$	$1 - (^{x}/_{8H}) \geq 0$

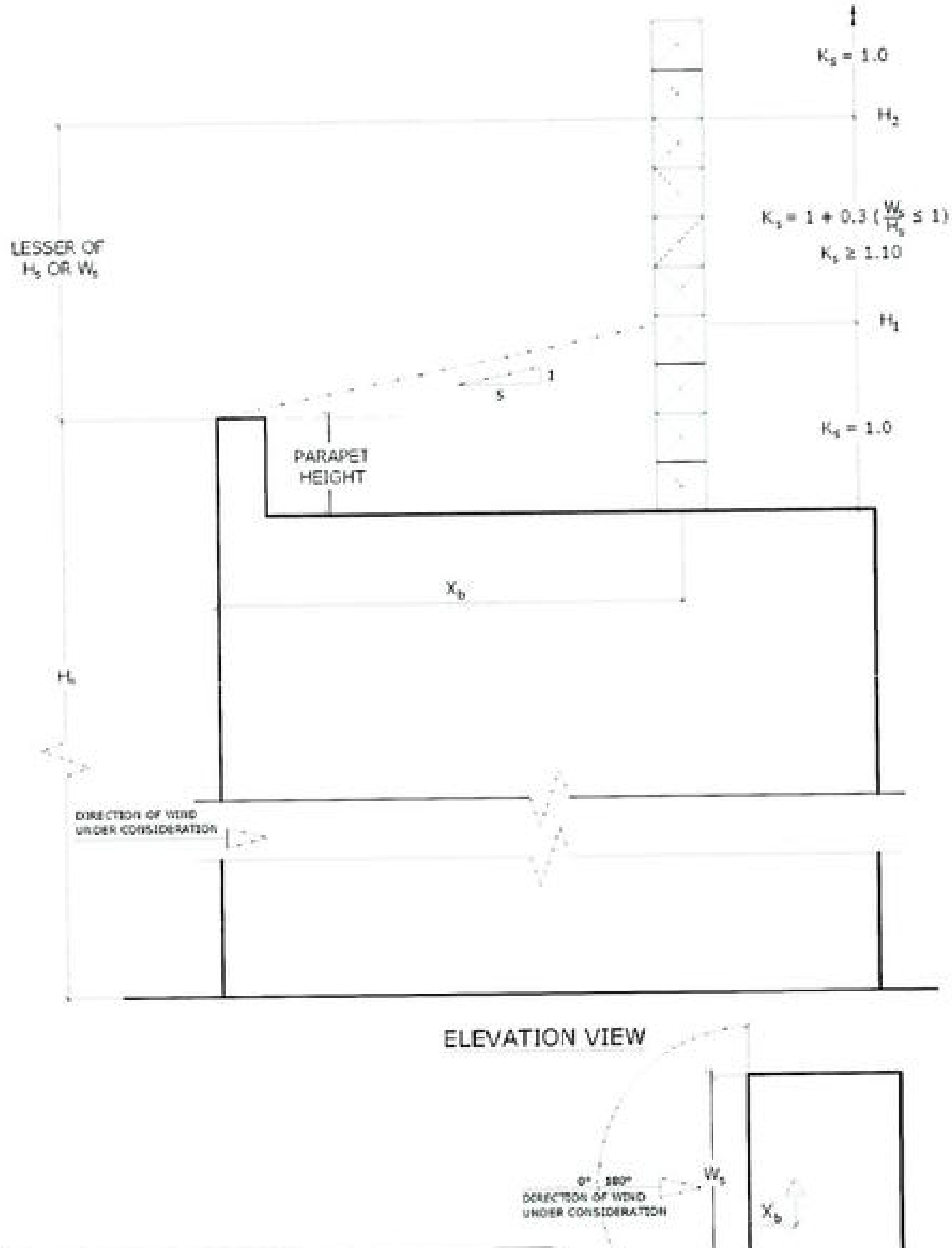
Vertical Distance Factor, K_3

Slope	Ridge	Hill	Escarpment
$1/10 \leq H/L < 1/4$	$e^{-6z/L}$	$e^{-8z/L}$	$e^{-5z/L}$
$H/L \geq 1/4$	$e^{-1.5z/H}$	$e^{-2z/H}$	$e^{-1.25z/H}$

z = height above ground level at base of structure

K_3 = 0 for $z \geq L$ when $H/L < 1/4$ and for $z \geq 4H$ when $H/L \geq 1/4$

Figure 2-2: Rooftop Wind Speed-Up Factor, K_s



K_s = Rooftop wind speed-up factor
X_b = Horizontal distance from windward face to center of structure
W_s = Width of the windward face of the building
H_s = Height of the windward face of the building

Figure 2-3: Projected Area of Ice

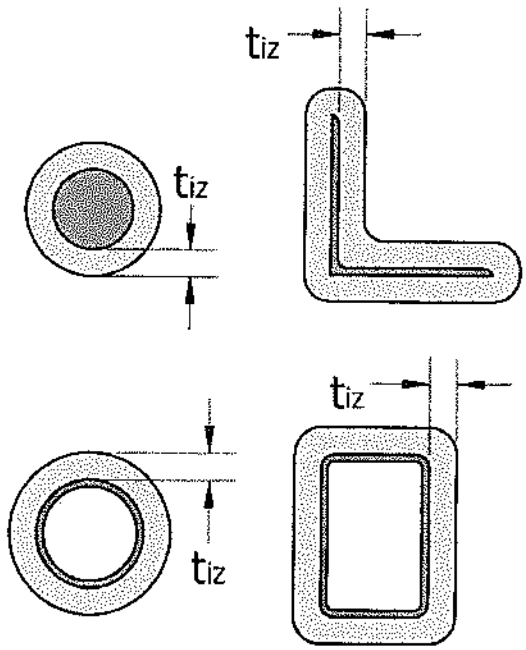
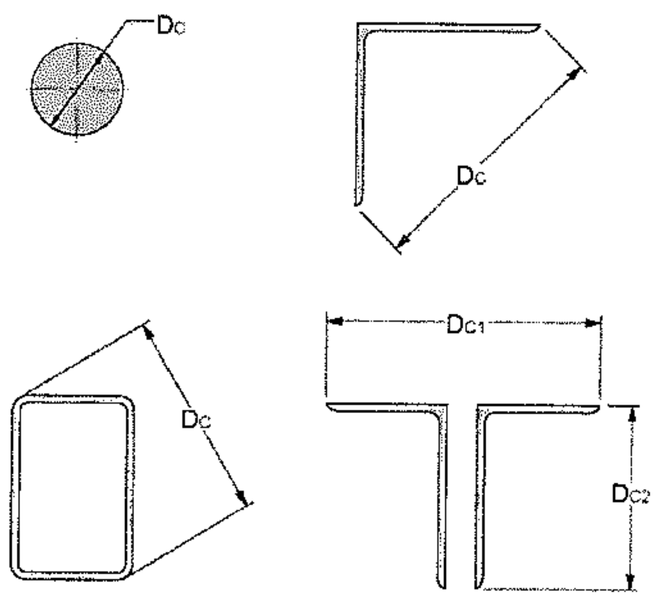
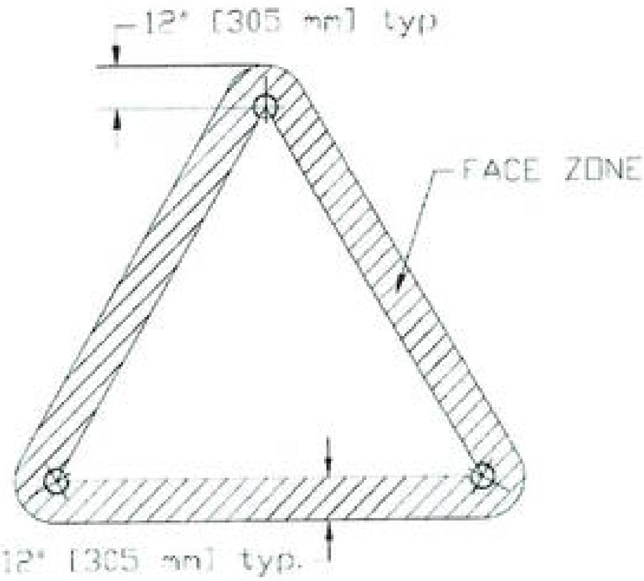


Figure 2-4: Out-to-Out dimensions for Calculating Ice Weight

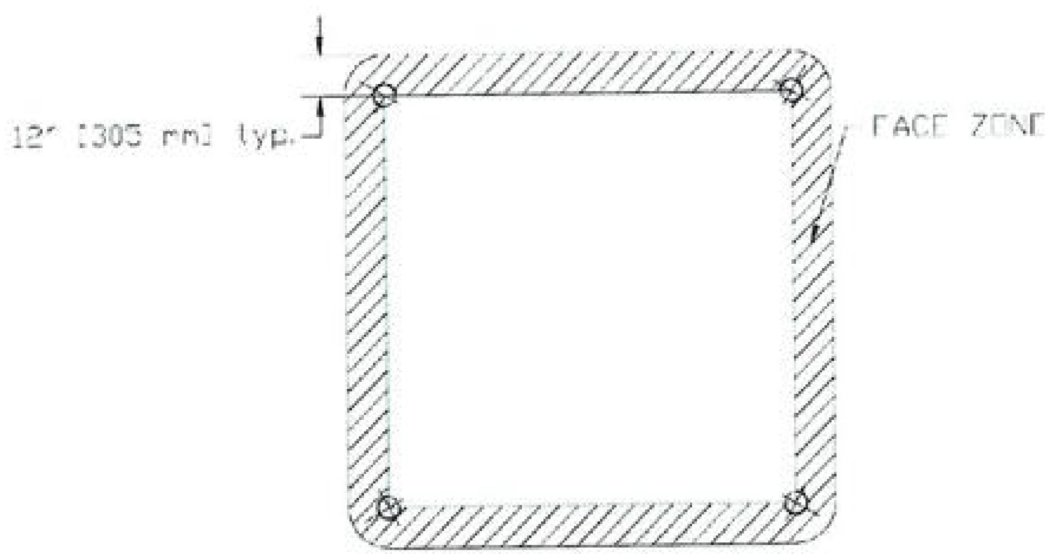


Note: D_c is the larger of D_{c1} and D_{c2}

Figure 2-5: Shielding Zones for Appurtenances

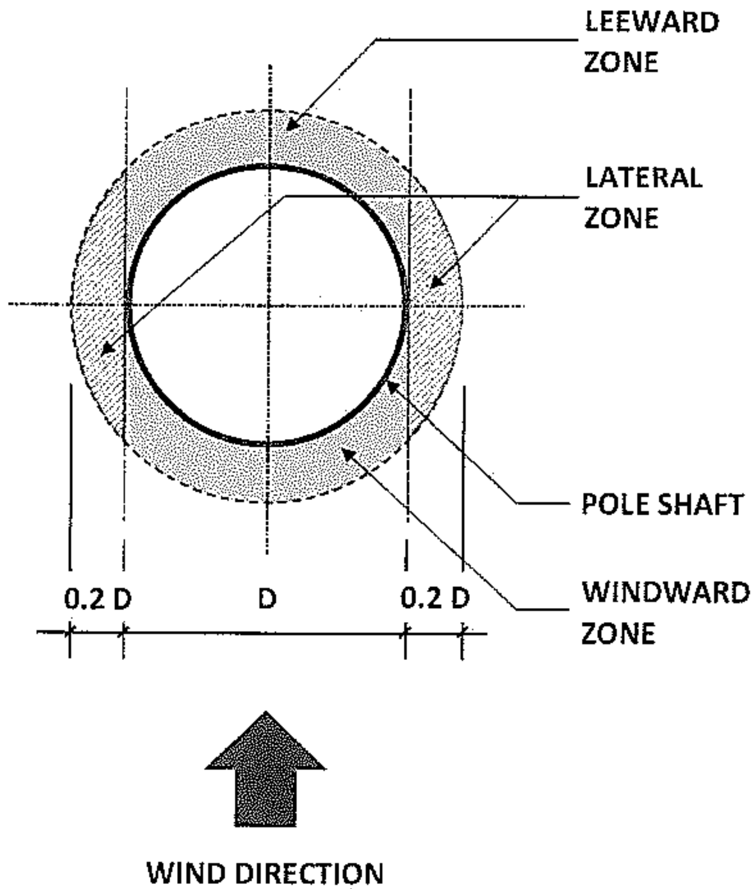


Latticed Structure Face Zone - Triangular Cross-Section



Latticed Structure Face Zone - Square Cross-Section

Figure 2-5: Shielding Zones for Appurtenances (continued)

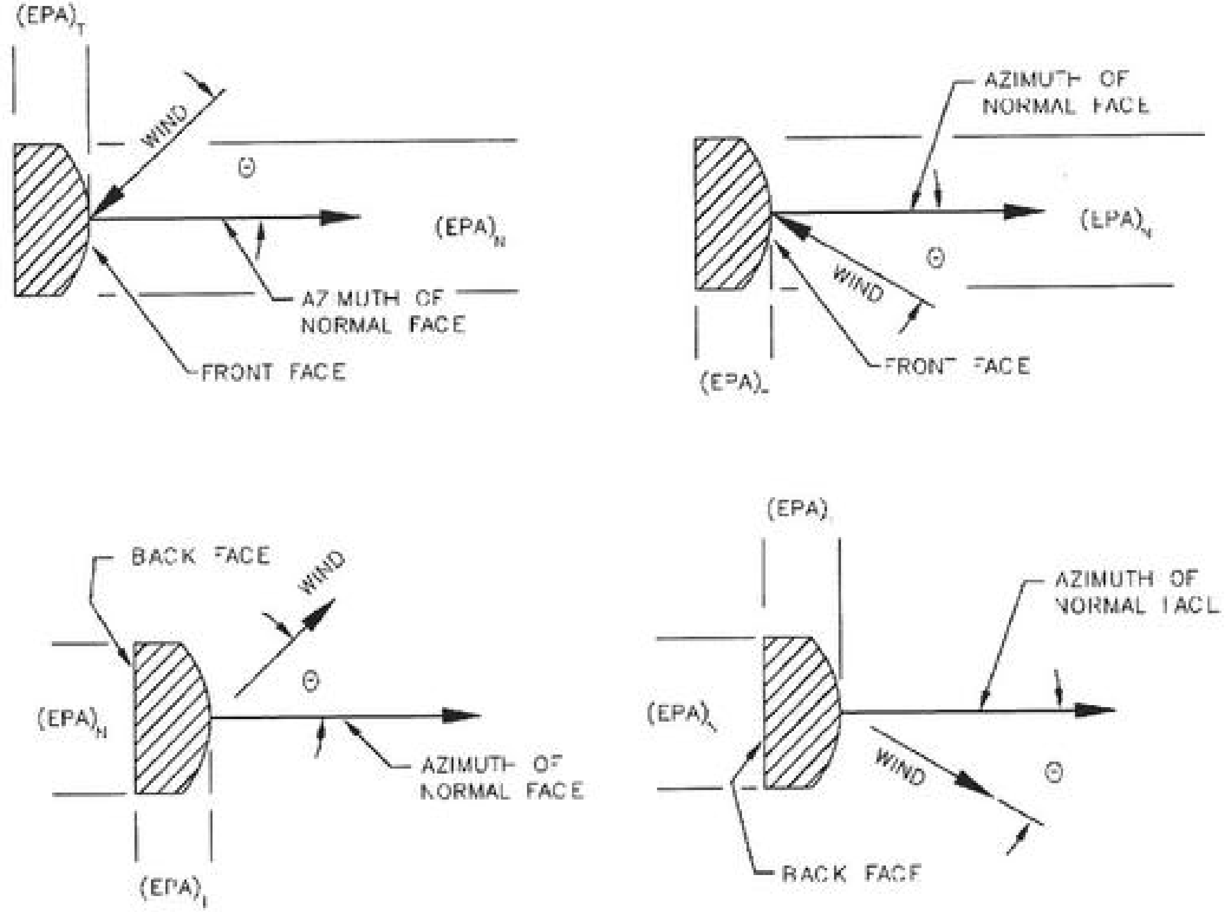


D is the outside diameter for rounds or outside corner-to-corner width for polygons.

Note: Locations of the Windward, Lateral, and Leeward Zones depend on the wind direction under consideration.

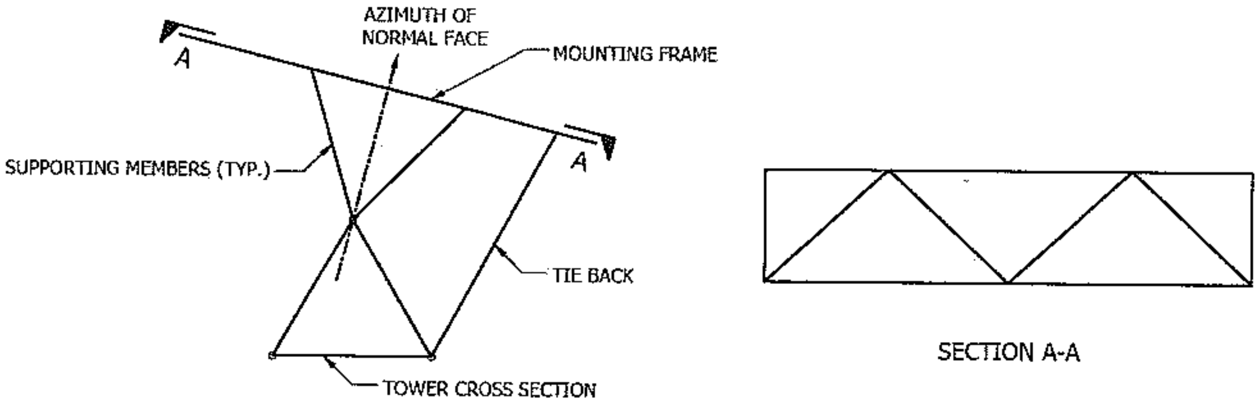
Tubular Structures

Figure 2-6: Wind Force on Appurtenances

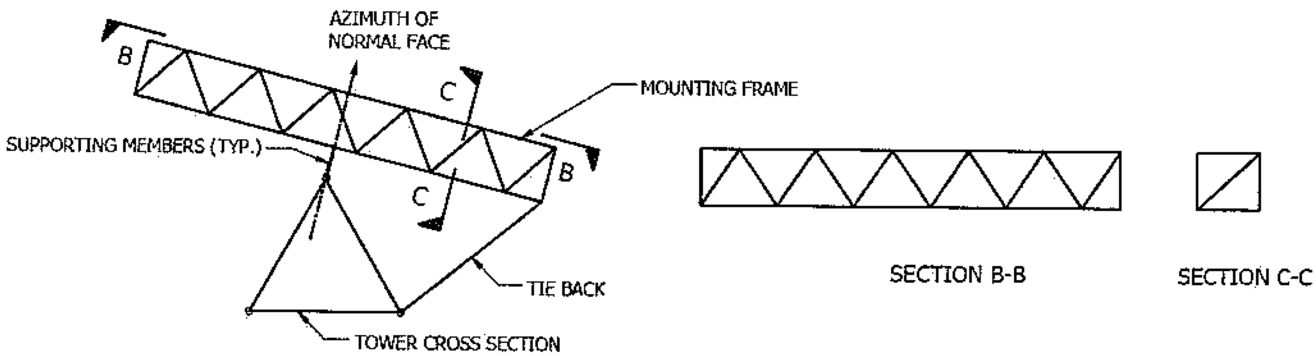


Note: $(EPA)_N$ and $(EPA)_T$ represent the effective projected areas of the appurtenance for the windward normal and transverse faces of the appurtenance.

Figure 2-7: Mounting Frames

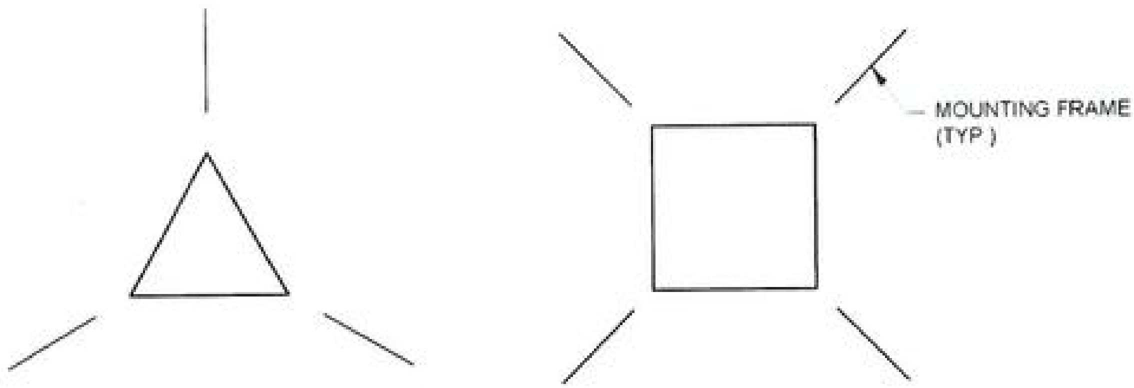


Plane Frame Type Mounting Frames

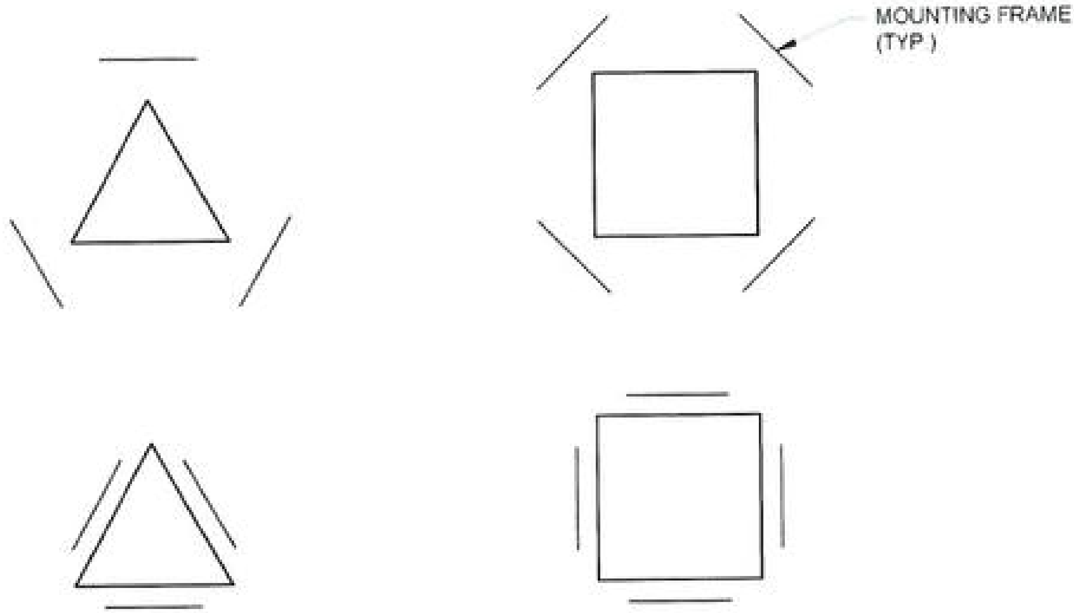


Truss Type Mounting Frames

Figure 2-8: Multiple Mounting Frames

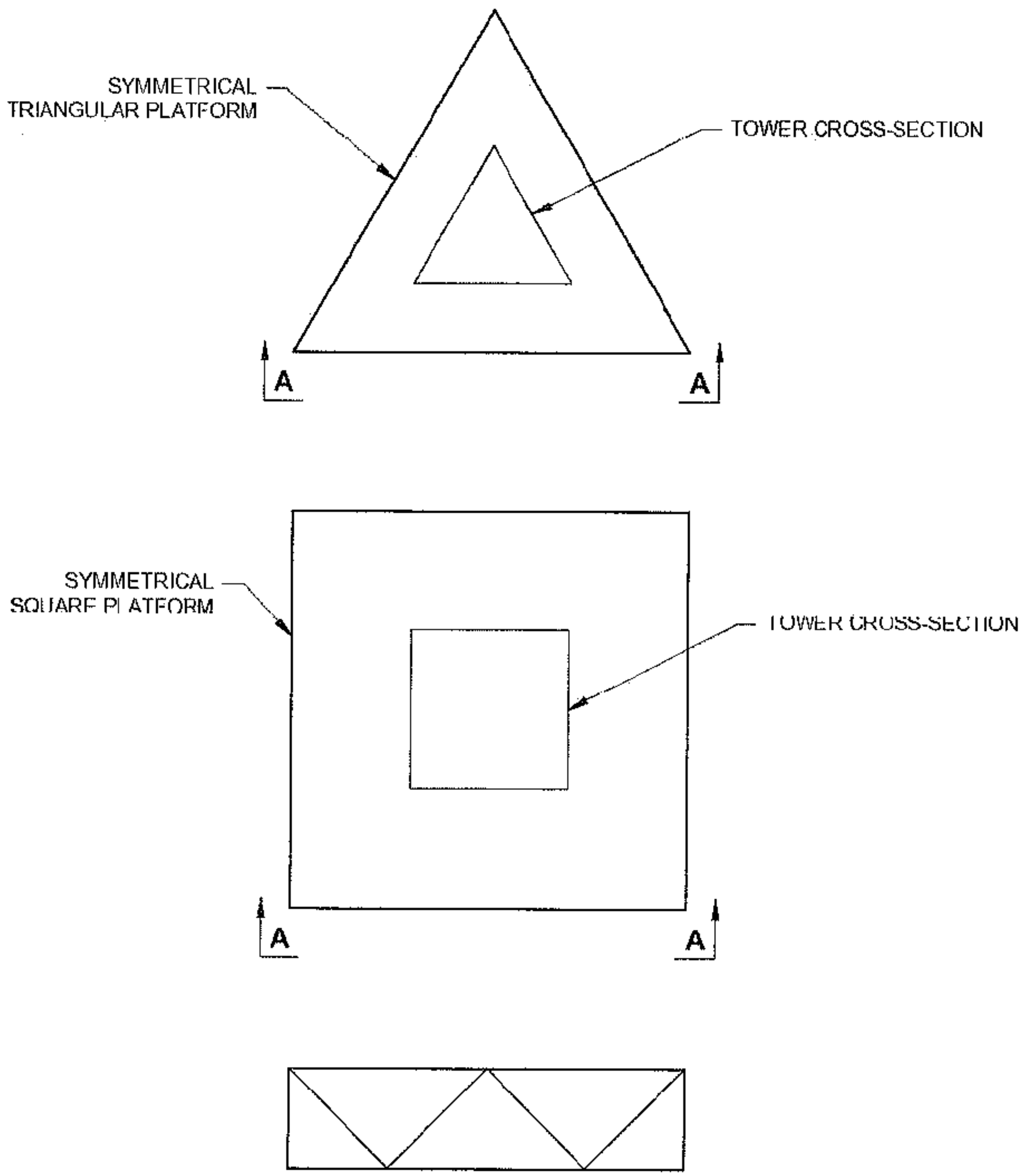


**0.80 Shielding Factor, K_a, Applies
(Minimum of 3 Mounting Frames Required)**



**0.75 Shielding Factor, K_a, Applies
(Minimum of 3 Mounting Frames Required)**

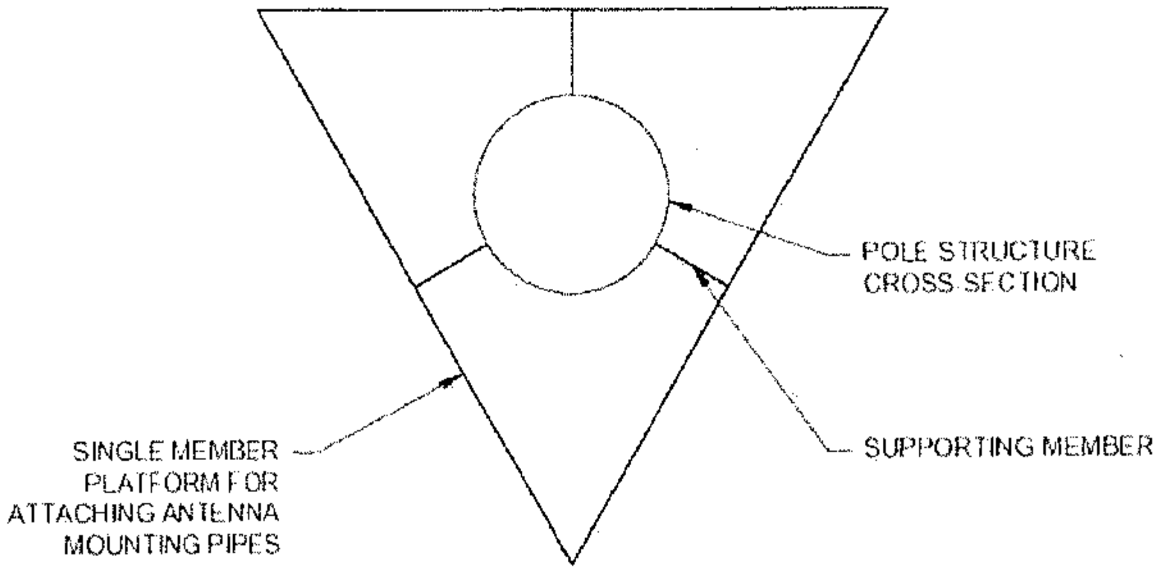
Figure 2-9: Symmetrical Frame/Truss Platforms



SECTION A-A

(Truss Type)

Figure 2-10: Low Profile Platforms



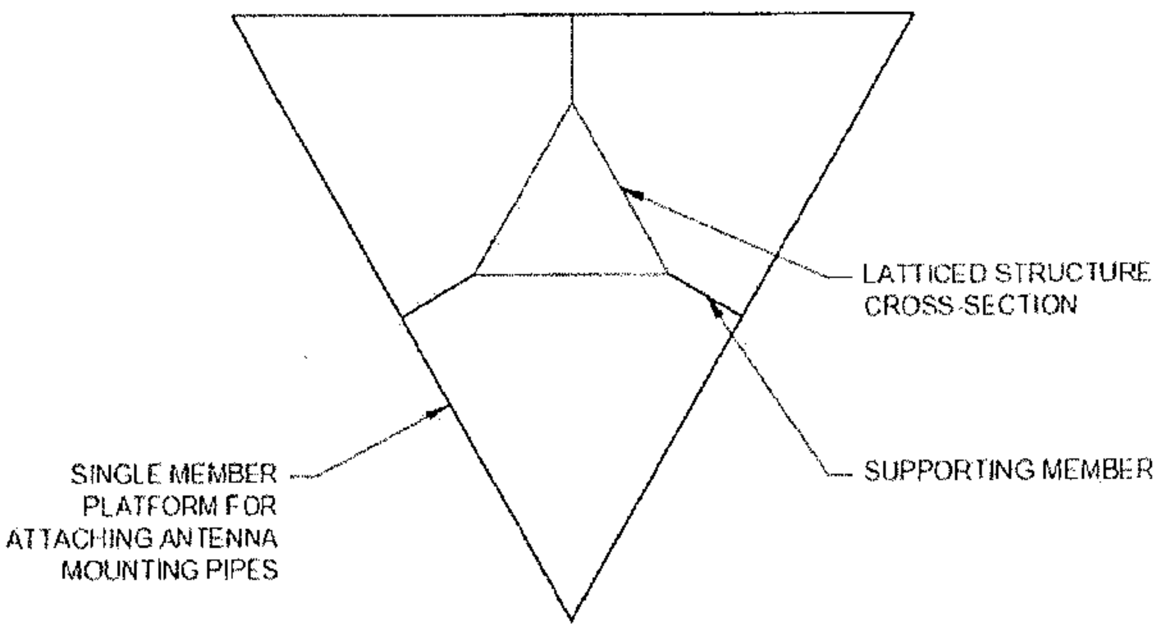
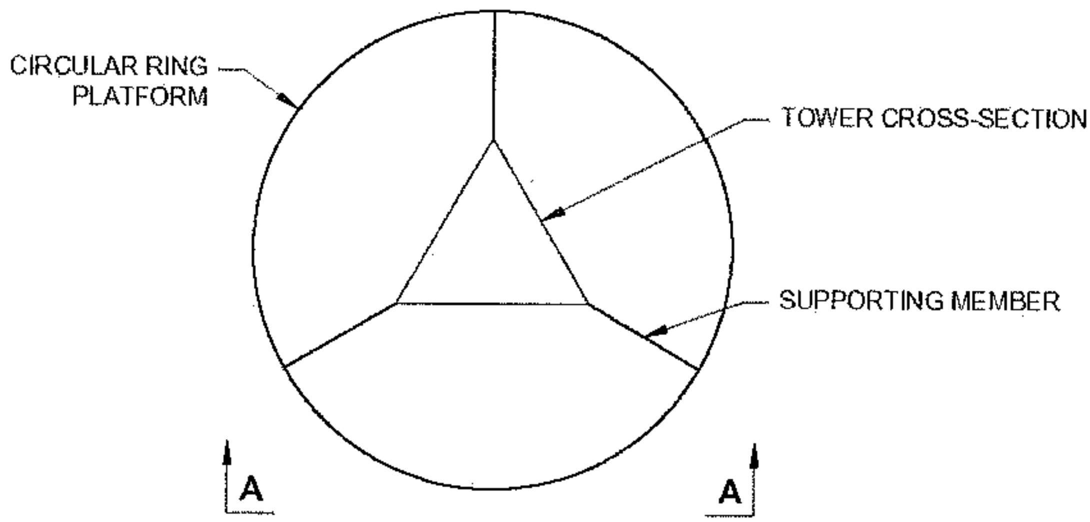
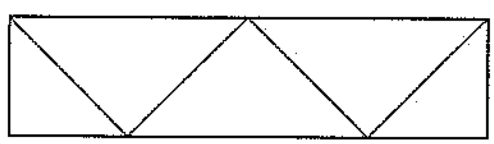


Figure 2-11: Circular Ring Platforms





SECTION A-A

Figure 2-12: Wind Force on Guys

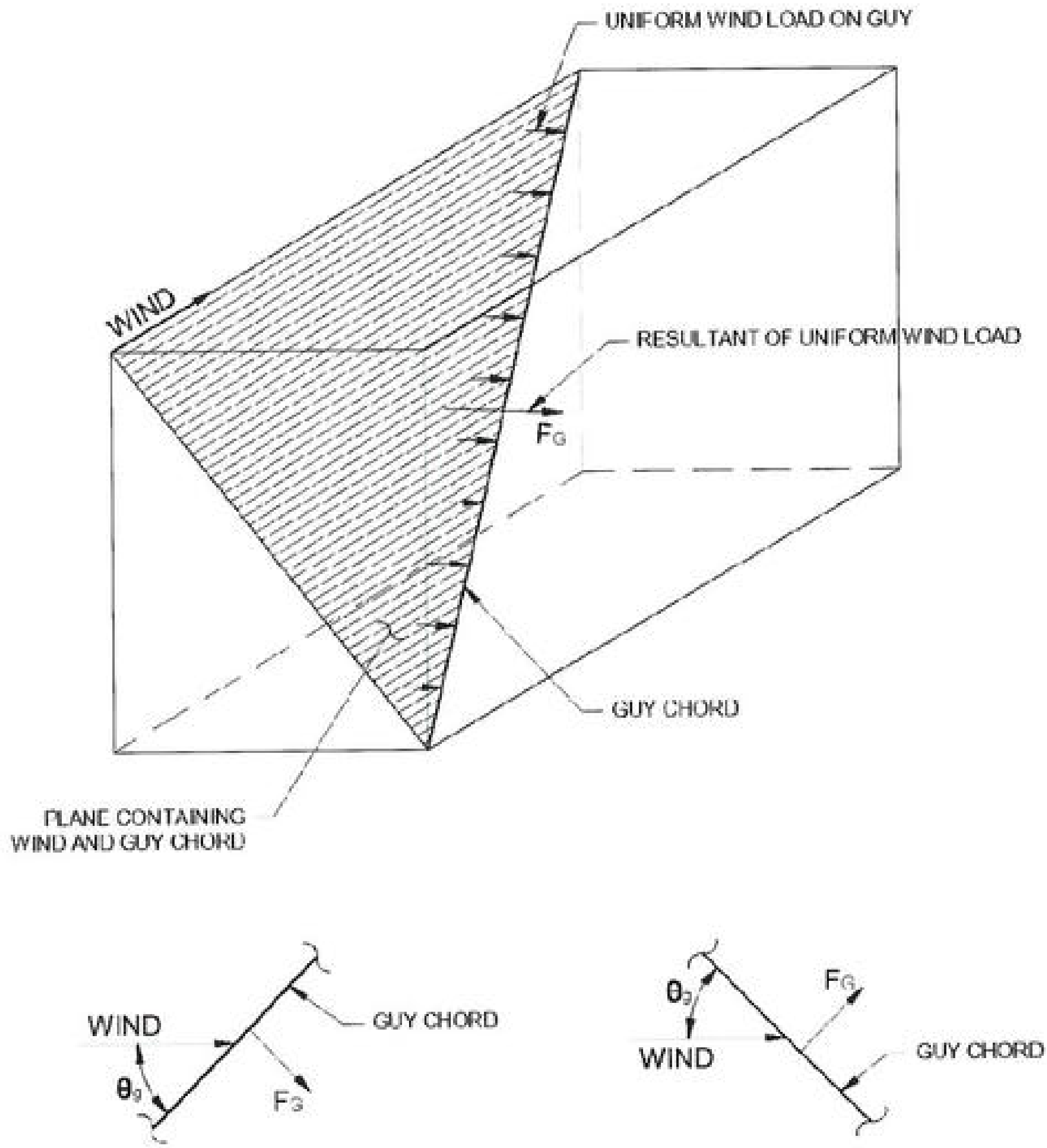


Figure 2-13: Shielding Limitations

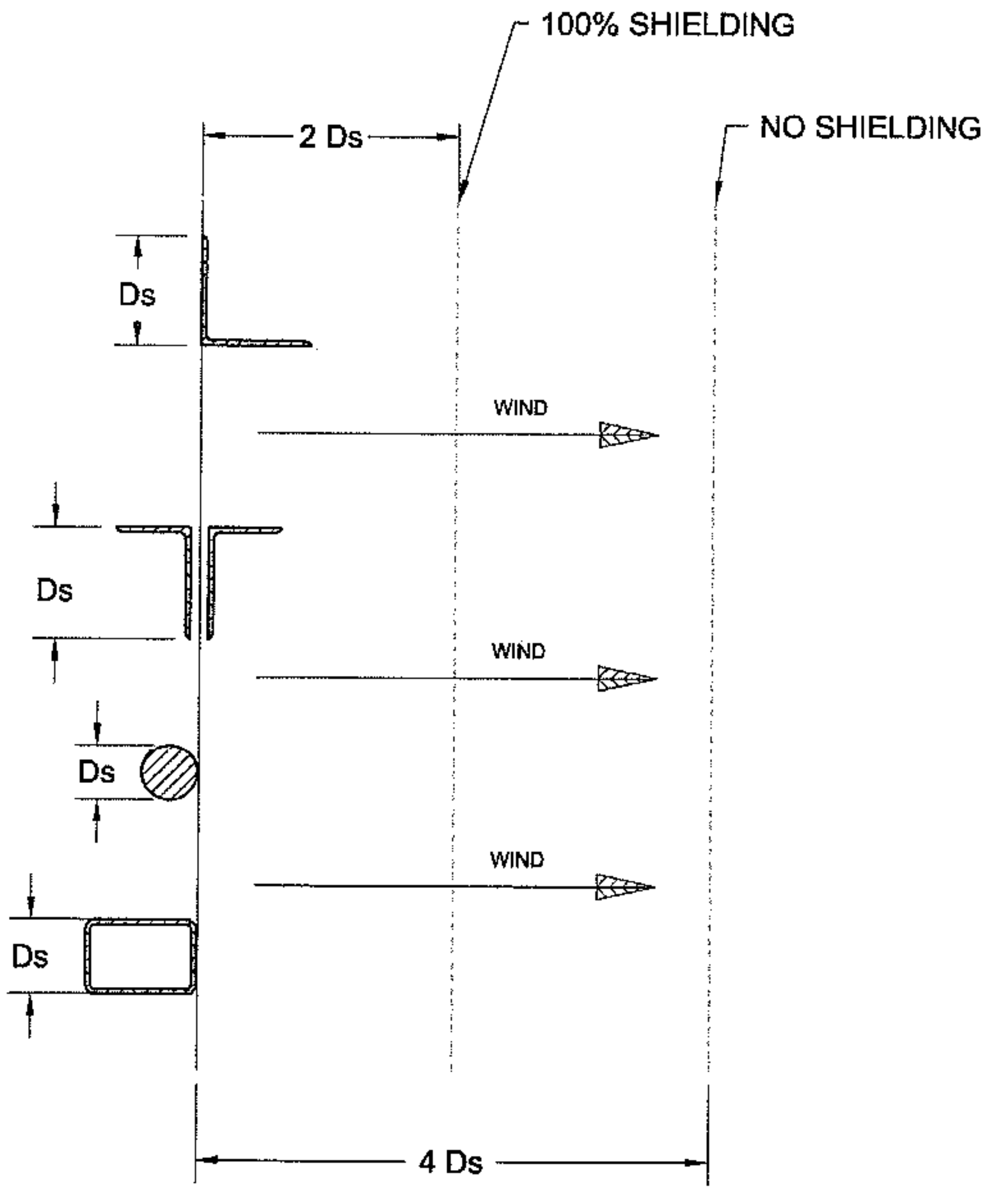
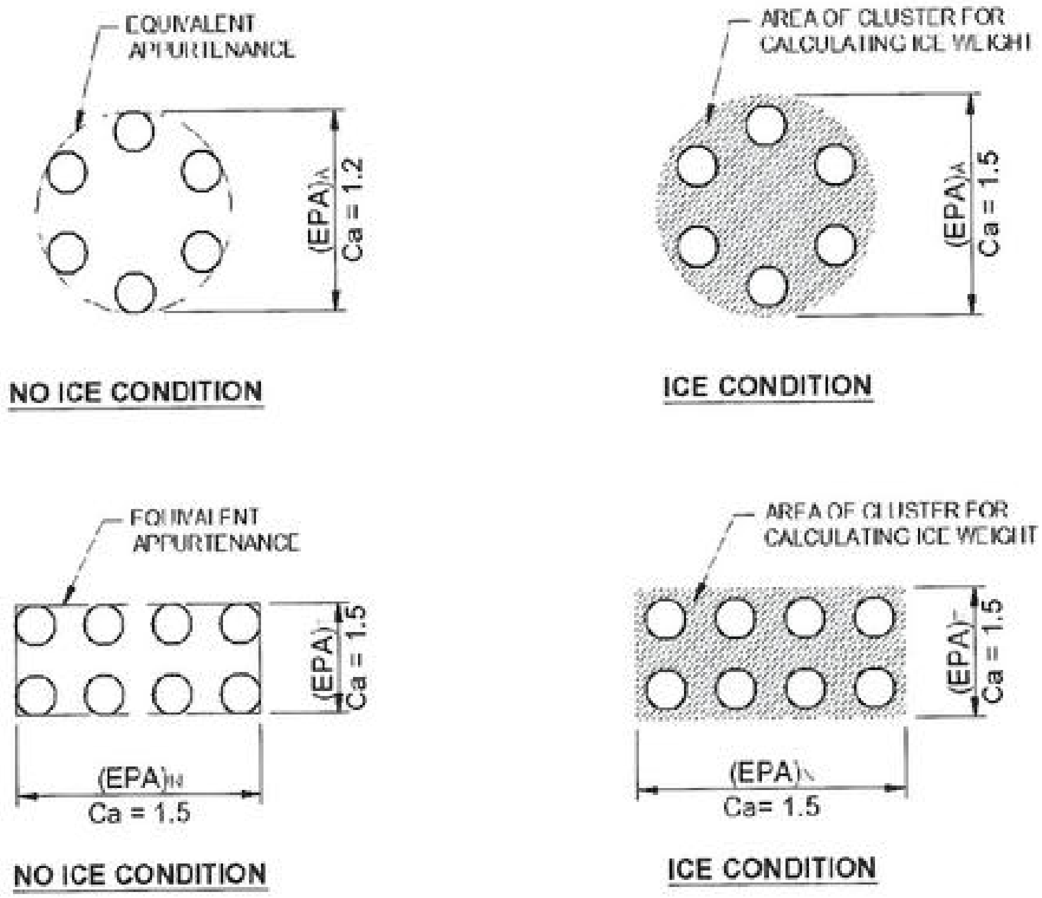
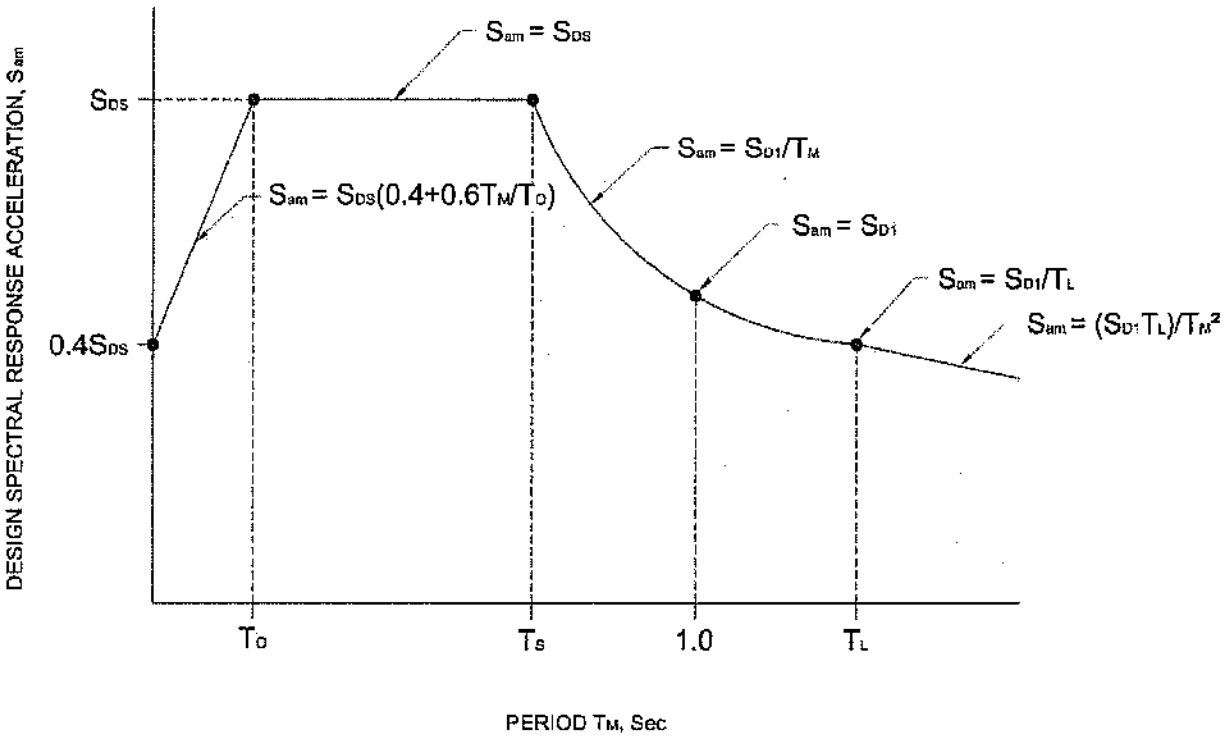


Figure 2-14: Equivalent EPA of Transmission Line Clusters



Note: $(EPA)_A$, $(EPA)_N$, and $(EPA)_T$ represent the effective projected areas of the equivalent appurtenance based on the appropriate out-to-out dimension of the cluster (including ice for loading combinations that include ice).

Figure 2-15: Design Response Spectrum



3.0 ANALYSIS

3.1 Scope

This Section defines: (i) the minimum acceptable analysis models and techniques, and (ii) the requirements to account for the dynamic effects of wind gusts.

3.2 Definitions

Cable element: a tension-only structural element with forces applied to the deflected shape with axial deformations and catenary effects considered.

Guyed mast: a latticed or pole structure with supporting guys.

Mast span: the distance between the base and the first guy level, the distance between two successive guy levels, or the distance above the top guy level to the top of the structure (cantilever span).

Mean wind conversion factor, m: a factor used to determine the mean hourly wind pressure.

3.3 Symbols and Notations

F_A = horizontal design wind force for appurtenances;

F_{ST} = horizontal design wind force on the structure;

f_{wi} = width of segment of structure;

h = height of a guyed mast;

h_i = height of segment of structure;

m = mean wind conversion factor;

P-Δ = effects of displacement on member forces;

q_z = velocity pressure.

3.4 Analysis Models

It shall be permissible to model members using gross cross-sectional properties.

The minimum acceptable models of analysis are as follows:

3.4.1 Self-Supporting Latticed Towers

1. An elastic three-dimensional truss model made up of straight members pin connected at joints producing only axial forces in the members.
2. An elastic three-dimensional frame-truss model where continuous members (legs, K-type bracing horizontals without plan bracing) are modeled as 3-D beam elements producing both moments and axial forces in the members while the remaining members which are subjected primarily to axial loads may be modeled as 3-D truss elements producing only axial forces in the members.

3.4.2 Self-Supporting Pole Structures

1. An elastic three-dimensional beam-column model producing moments, shears and axial forces in the pole structure. Unless the analysis model considers second order effects within each element, the minimum number of beam elements shall be equal to five per pole section and the maximum beam element length shall not exceed 6 ft. [1.8 m].
2. It shall be permissible to model flange and base plates as nodes without additional flexibility when the plates are designed based on rigid plate behavior (refer to Annex Q).

Note: Due to modeling complexity (e.g. meshing, element interconnection, etc.) of plate or shell models, the stresses obtained from such models shall not be less than the stresses obtained from the beam-column model defined above.

3.4.3 Guyed Masts

1. An elastic three-dimensional beam-column where the mast is modeled as equivalent three-dimensional beam-column members. This analysis produces moments, shear and axial forces in the mast, which results in individual member forces. For finite element beam-column models, unless the analysis model considers second-order effects within each element, a minimum of five elements shall be used in any span or cantilever.
2. An elastic three-dimensional truss model where individual members of the mast are modeled as straight members connected at joints producing only axial forces in the members.
3. An elastic three-dimensional frame-truss model where continuous members (legs) of the mast are modeled as 3-D beam elements producing both moments and axial forces in the members while other members may be modeled as 3-D truss members.

Guys shall be represented as cable elements or as non-linear elastic supports with axial deformations and catenary effects taken into account.

3.4.4 Application of Wind Forces to Structural Models

The horizontal design wind force on the structure, F_{ST}, shall be equally distributed to each leg joint of the cross-section at the panel points for three-dimensional truss or frame-truss models. For three-dimensional beam-column models, wind forces shall be applied as either uniform loads or as concentrated loads distributed to the nodes on the beam-column.

The horizontal design wind force, F_A, for appurtenances shall be distributed to the appropriate nodes in the model according to the location of the appurtenance (i.e. lateral load and torsion considered). For three-dimensional beam-columns, this will require applying torsional moments at the appropriate nodes.

Local bending shall be considered for structural components supporting appurtenances that are supported in the middle half of the component. For main bracing members, under this condition, local bending shall be considered for the condition of wind normal to the plane of the bracing members with no axial member load considered.

Note: Weight and earthquake forces shall be distributed and considered in a similar manner.

3.5 Displacement Effects

The analysis of all structures, except as provided herein, shall take into account the effects of displacements on member forces (P-Δ effects). The P-Δ effects shall be established using elastic analysis methods consistent with the analysis models adopted in accordance with section 3.4.

For guyed structures the effects of displacements of the guy points as well as the effects of displacements between guy points shall be considered.

For finite element beam-column models, unless the analysis model considers second order effects within each element, the minimum number of beam elements between guy levels shall be equal to 5.

P-Δ effects need not be considered for self-supporting latticed towers with heights less than 450 ft. [137 m] provided that the height-to-face width ratios, h_i/f_{wi}, are less than 10 as shown in Figure 3-1.

3.6 Global Stability Considerations

The P-Δ analysis shall include the effects of geometric imperfections. It shall be permissible to include the imperfections directly in the model. Alternatively, this requirement shall be considered satisfied for any load combination that includes horizontal seismic load effects or wind loads at a basic wind speed of not less than 30 mph [13.4 m/s].

3.7 Wind Loading Patterns

To account for the dynamic effects of wind gusts, the following wind loading patterns shall be considered for the strength limit state condition (refer to Figures 3-2 and 3-3):

3.7.1 Latticed Self-Supporting Towers

When the apex defined by the projection of the inclined legs of a latticed self-supporting tower lies within the height of the tower (refer to Figure 3-2), the following wind loading patterns shall be investigated for load combination 1 as specified in 2.3.2 by varying the velocity pressure as follows:

1. Full velocity pressure over the entire height of the structure.
2. Full velocity pressure below the apex point and mean velocity pressure above the apex point.
3. Full velocity pressure above the apex point and mean velocity pressure below the apex point.

The mean velocity pressure shall be determined by multiplying the velocity pressure, (q_z per 2.6.11.6) by the mean wind conversion factor, m, from Table 3-1.

The above loading patterns shall apply for each apex point in towers with multiple legs slopes that differ by more than 1 degree in adjacent sections. All combinations of wind loading patterns shall be considered when determining maximum load effects.

3.7.2 Guyed Masts

For guyed masts with three or more spans and with at least one mast span greater than 80 ft. [24 m] within the top one-third of the height of the structure, the following wind loading patterns (refer to Figure 3-3) shall be investigated for load combination 1 as specified in 2.3.2 by varying the velocity pressure as follows:

1. Full velocity pressure over the entire height of the structure. For masts greater than 450 ft. [137 m] in height, full wind pressure over the entire structure need not be considered when pattern loading is investigated.
2. Mean velocity pressure on the top mast span and full velocity pressure on the remaining spans.
3. Mean velocity pressure on the second mast span from the top and full velocity pressure on the remaining spans.
4. Mean velocity pressure on the third mast span from the top and full velocity pressure on the remaining spans.

The mean velocity pressure shall be determined by multiplying the velocity pressure, (q_z per 2.6.11.6) by the mean wind conversion factor, m, from Table 3-1. Full wind pressure shall be applied to guys for all pattern loadings.

Notes:

1. For masts with cantilevers (e.g. broadcast antenna structures, spines, or the mast itself), the cantilevers shall be considered as the top span.
2. For masts where the total length of the top three mast spans is less than one-third the height of the structure, the above wind loading patterns shall be continued for each subsequent span until the total length of the considered spans is greater than one-third the height of the structure (refer to Figure 3-3).
3. When the distance between two guy elevations is less than 3 times the larger face width between the guy elevations, the wind pressure patterns shall extend to the mid-point of the two guy elevations. The short span shall not be considered as an independent span for purposes of this Section.

3.8 Mast Shear and Torsion Responses for Guyed Masts

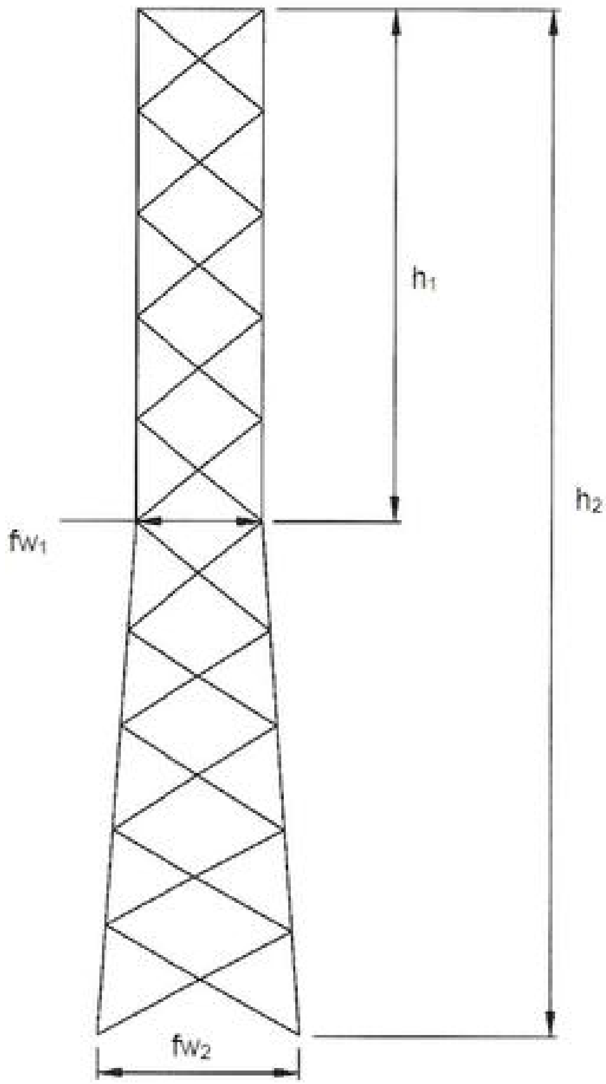
For all latticed masts, the mast face shear in a non-cantilever span due to mast shear and torsion shall not be less than 40% of the maximum absolute value of the face shear in the span. For all tubular masts, the mast shear in a non-cantilever span shall not be less than 40% of the maximum absolute value of the shear in the span.

Table 3-1: Mean Wind Conversion Factor

Exposure Category	Mean Wind Conversion Factor (m)
B	0.55
C	0.60
D	0.65

Note: The mean wind conversion factor corresponding to the predominant surface roughness for the site shall be used for a Site-Specific Exposure.

Figure 3-1: Height-to-Width Ratios



Note: Max $\frac{\text{height}}{\text{face width}}$ ratio = max of $\frac{h_1}{f_{w1}}$ or $\frac{h_2}{f_{w2}}$

Figure 3-2: Pattern Loading for Self-Supporting Towers

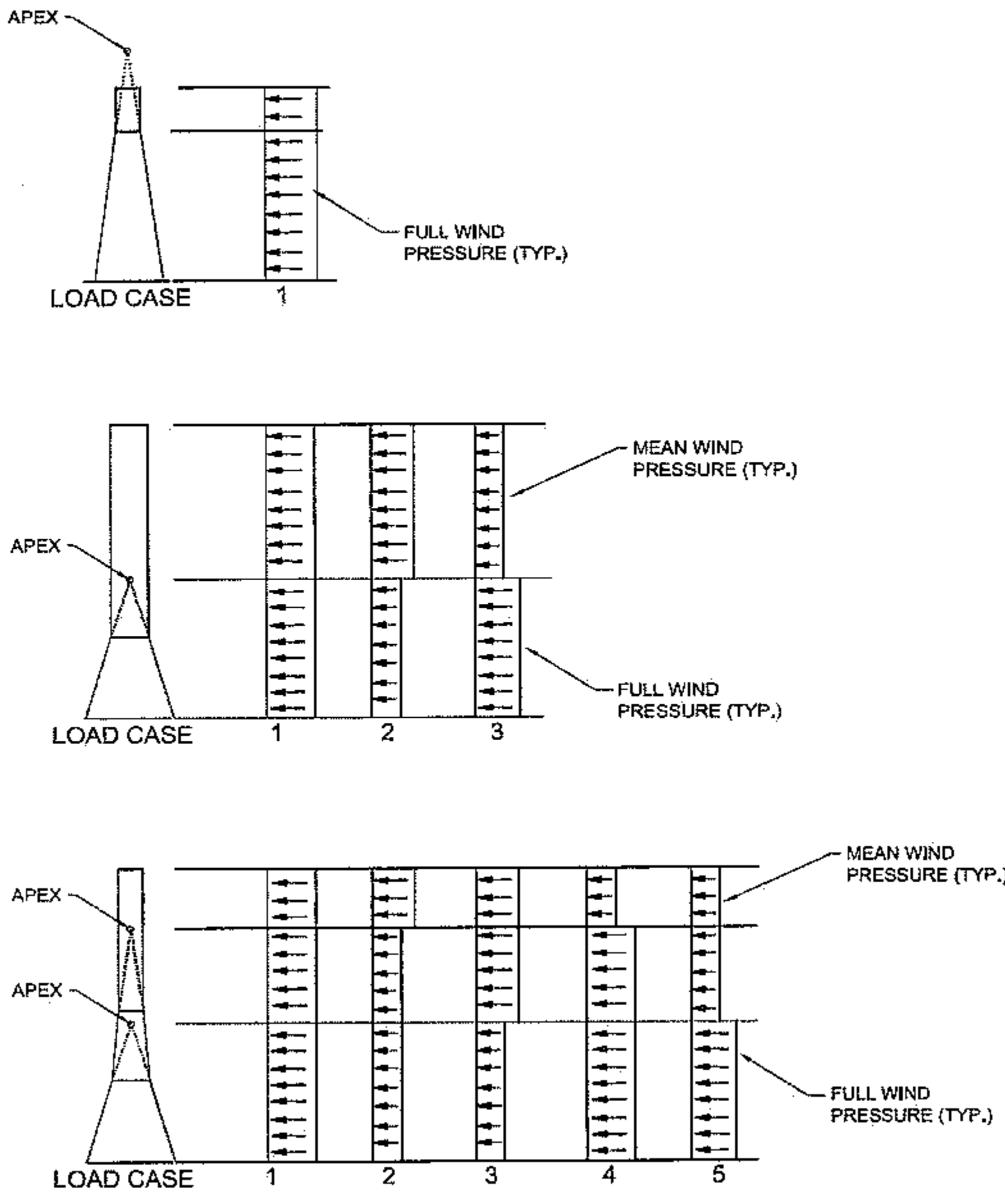
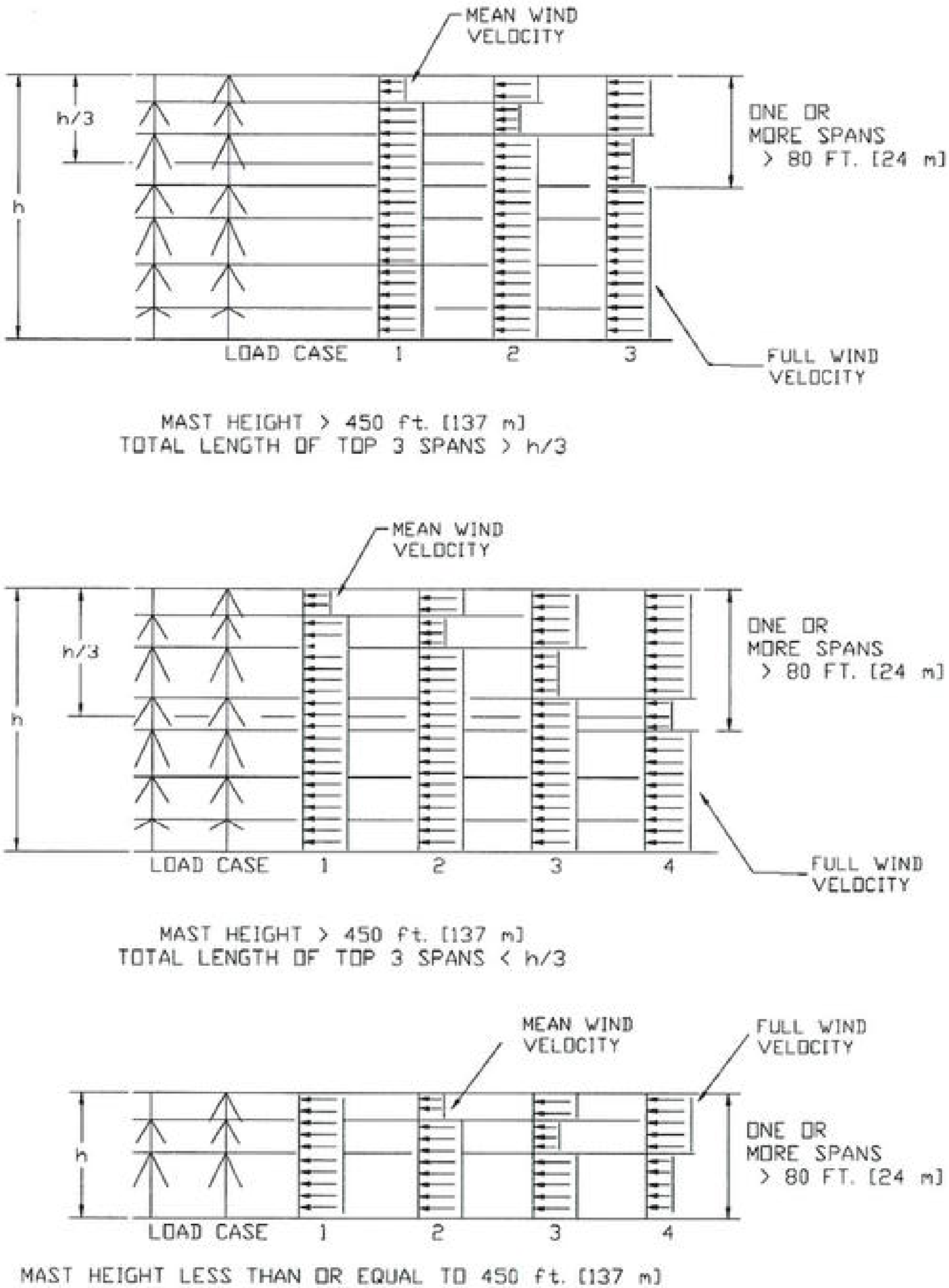


Figure 3-3: Pattern Loading for Guyed Masts



4.0 DESIGN STRENGTH OF STRUCTURAL STEEL

4.1 Scope

This Section relates to the strength design of structural steel plates, angles, solid rounds and tubular members used in latticed towers, poles and guyed masts. The following clauses are based on the LRFD provisions of the AISC 360-16 Standard, "Specification for Structural Steel Buildings", (AISC 360). When the requirements in AISC 360 differ from this Standard, this Standard shall govern. If other shapes or types of structures are utilized, the requirements of AISC 360 shall be used.

Cold formed light gauge steel structural members not covered by this Standard shall conform to the requirements of the AISI S100-16 Standard, "North American Specification for the Design of Cold-formed Steel Structural Members."

4.2 Definitions

Effective length factor: a factor, K, to modify the unbraced length, L, to take into account the structural configuration and end conditions.

Effective slenderness ratio: the slenderness ratio modified to take into account the structural configuration and end conditions for the purpose of calculating the design compression strength.

Guy assembly link plate: a plate connecting two guy assemblies or a guy assembly to an anchorage or attachment point.

Line of action: a line that is parallel to the longitudinal axis of a member that passes through the centroid of the bolt group connecting the member to another member.

In-plane buckling: direction of buckling considered in plane defined by the face of a latticed structure.

Out-of-plane buckling: direction of buckling considered normal to the face of a latticed structure.

Panel points: the centerline location of bracing member connections to a leg.

Radius of gyration: the square root of the moment of inertia about the axis of buckling under consideration divided by the area of a member.

Secondary members: members used primarily to reduce the unbraced length of a supported member.

Slenderness ratio: the ratio of the unbraced length, L, to the corresponding radius of gyration, r.

Slip-critical connection: a bolted joint governed by shear using oversized or slotted holes parallel to the line of force.

Unbraced length: the length between panel points or nodes providing restraint, which may vary for different planes of buckling depending on the bracing pattern. For leg members, L shall not be less than the centerline distance between panel points. For bracing members, L shall not be less than the length between the centers of connecting bolt or weld patterns.

Weak-axis buckling: direction of buckling considered about the weak principal axis of a member.

4.3 Symbols and Notations

ϕ = resistance factor for connections;

ϕ_a = resistance factor for normal stress;

ϕ_c = resistance factor for compression;

ϕ_f = resistance factor for flexure;

ϕ_k = resistance factor for guy assembly link plates;

ϕ_p = resistance factor for connecting elements;

ϕ_T = resistance factor for torsion;

ϕ_t = resistance factor for tension;

ϕ_u = resistance factor for U-bolts;

ϕ_v = resistance factor for shear;

θ = angle between the longitudinal axis of two structural members of a latticed structure;

A = area for determining effective net area;

A_b = nominal unthreaded body area of a bolt or anchor rod;

A_{en} = effective net area;

A_g = gross area;

A_{gt} = gross area subject to tension;

A_{gv} = gross area subject to shear;

A_n = net area of a member, bolt or anchor rod;

A_{nt} = net area subject to tension;

A_{nv} = net area subject to shear;

A_p = cross sectional area removed for an opening in a tubular section;

A_r = cross sectional area of reinforcing for an opening in a tubular section;

A_{sf} = effective shear area of a guy assembly link plate;

a = end distance of a guy assembly link plate;

a_i = spacing of intermediate connectors for a built-up member;

B_1 = moment amplification factor;

b = width of an angle leg;

b_{eff} = effective edge distance of a guy assembly link plate;

C_t = torsional constant;

D = outer diameter of a round member or outside flat-to-flat width of a polygonal member;

d = nominal diameter of a bolt, pin or anchor rod;

d_n = tensile root diameter of an anchor rod;

E = modulus of elasticity, 29,000 ksi [200,000 MPa];

EI = flexural stiffness;

F_{cr} = critical compression stress;

F_e = elastic buckling stress;

F_{ey} = elastic buckling stress about the principle axis of a double angle member;

F_{ez} = torsional elastic buckling stress;

F_{nt} = critical shear stress for torsion;
F_{nv} = critical shear stress for direct shear;
F_s = axial design compressive force in a supported member;
F_u = specified minimum tensile strength;
F_{ub} = specified minimum tensile strength of a bolt or anchor rod;
F_y = specified minimum yield strength;
F_{yr} = specified minimum yield strength of reinforcement for an opening in a tubular section;
F'_y = effective yield stress;
G = shear modulus of elasticity, 11,200 ksi [77,200 MPa];
g = transverse center-to-center spacing between fastener gage lines;
g_c = distance from the heel of an angle to the end connection centroid;
H = flexural constant;
H1 = unbraced length for investigating buckling for horizontal bracing members;
l_{ar} = anchor rod projection;
I_x, I_y = moment of inertias about the principle axes;
J = St. Venant torsional constant;
K = effective length factor;
K_i = effective length factor for the individual members of a built-up member;
KL/r = effective slenderness ratio;
$\left(\frac{KL}{r}\right)_m$ = modified effective slenderness ratio of a built-up member acting as a unit;
$\left(\frac{KL}{r}\right)_o$ = effective slenderness ratio of a built-up member acting as a unit;
L = laterally unbraced length of a member;
L1, L2, L3 = unbraced lengths for investigating buckling for diagonal bracing members;
L_b = connection length for a multiple bolt connection;
L_c = clear distance for design bearing strength;
L_p = length of a tubular structure considered for shear strength;
L/r = slenderness ratio of a member;
M_n = nominal flexural strength;
M_u = flexural moment due to factored loads;
m = torsional constant shape coefficient for a member resisting torsion;
N = staggered bracing panel spacing ratio;
n = number of threads per inch;
P_e = elastic critical buckling load;
P_n = nominal axial strength;
P_r = resistance required to provide lateral support at a panel point within a face of a latticed structure;
P_s = minimum bracing resistance normal to a supported member required to provide lateral support;
P_u = axial force due to factored loads;

P_{uc} = anchor rod axial compression force due to factored loads;

P_{ut} = anchor rod axial tension force due to factored loads;

p = pitch of threads;

R_b = connection length reduction factor;

R_n = nominal bearing strength at a bolt or attachment hole;

R_{nb} = nominal buckling strength of an anchor rod acting as a column;

R_{nc} = nominal compression yield strength of an anchor rod;

R_{nk} = nominal strength of a guy assembly link plate;

R_{np} = nominal strength of a connecting element;

R_{nr} = nominal torsional strength (twisting) of a U-bolt connection;

R_{ns} = nominal sliding strength of a U-bolt connection;

R_{nt} = nominal tensile strength of a bolt or anchor rod;

R_{nv} = nominal shear rupture strength of a bolt or anchor rod;

R_{nvc} = nominal shear yield strength of an anchor rod;

r = radius of gyration;

r_i = minimum radius of gyration of an individual component of a built-up member;

r_{ib} = radius of gyration of an individual component about its centroidal axis in the direction of buckling under consideration for a built-up member;

r_{min} = minimum radius of gyration for a member;

$\bar{r}_o$ = polar radius of gyration about the shear center;

r_{out} = radius of gyration associated with out-of-plane buckling with respect to the face of a latticed structure;

rx = radius of gyration about the x-axis of buckling;

ry = radius of gyration about the y-axis of buckling;

rz = radius of gyration about the z-axis of buckling;

S = elastic section modulus;

s = longitudinal center-to-center spacing (pitch) of any two consecutive holes;

T_n = nominal torsional strength;

T_p = installed pretension in each leg of a U-bolt connection;

T_u = torsional moment due to factored loads;

T_{ub} = bolt tensile force due to factored loads;

T_{ur} = torsional moment (twisting) applied to a member from a U-bolt connection due to factored loads;

T_{ut} = tension force applied to a U-bolt assembly due to factored loads;

t = thickness of a member or connected part;

U = reduction factor for effective net area calculation;

U_{bs} = reduction coefficient for block shear rupture;

V_n = nominal shear strength;

V_u = transverse shear force due to factored loads;

V_{ub} = bolt shear force due to factored loads;

V_{us} = sliding force applied to a member from a U-bolt connection due to factored loads;

W_n = net width for net area calculation;
w = width of a flat element of a member;
X_r = location of reinforcement for an opening in a tubular section from the section centerline;
y_o = coordinates of shear center with respect to the centroid of a member;
Z = plastic section modulus.

4.4 General

4.4.1 Minimum Bracing Resistance

In order to consider a reduction in the unbraced length of a supported member at a node or panel point, bracing and secondary members shall provide a minimum resistance, P_s, normal to the supported member (in both directions) in the plane of buckling under consideration. P_s shall be determined in accordance with the following:

$$P_s = [1.5 + \frac{(\frac{KL}{r} - 60)}{60}]\frac{F_s}{100}$$

$$1.5\frac{F_s}{100} \le P_s \le 2.5\frac{F_s}{100}$$

where:

KL/r = effective slenderness ratio of the supported member in the plane of buckling under consideration

F_s = axial design compressive force in the supported member

The resistance required for leg members at a panel point within a face of a tower, P_r, shall be determined from Table 4-1.

The minimum required design strengths of multiple members connecting at a panel point within a face shall be determined from Table 4-2.

A secondary diagonal member that is connected to either end of a horizontal secondary member shall have a minimum design strength equal to one-half of the required design strength of the horizontal divided by the cosine of the angle between the members (refer to Table 4-2) unless a more rigorous analysis is performed.

Note: The minimum resistance, P_s shall be determined by considering each supported member independently. The highest resistance required considering all supported members shall be used as the minimum required design strength. The minimum resistance need not be considered in conjunction with any loading combination.

4.4.2 Slenderness Ratios

The effective slenderness ratio, KL/r, shall preferably not exceed:

1. 150 for leg members,
2. 200 for main compression members other than leg members,

3. 250 for secondary members, and
4. 300 for tension members, except for tension rod bracing and cables.

4.4.3 Design Values

For design purposes, the minimum nominal values for the yield strength and ultimate tensile strength for the type and grade of steel specified shall be used.

The design wall thickness shall be taken equal to 0.93 times the nominal wall thickness for electric-resistance-welded (ERW) HSS and pipe shapes.

Nominal thicknesses shall be used for all other shapes.

4.4.4 Normal Framing Eccentricities

4.4.4.1 Leg Members

Eccentricities shall be considered unless the following conditions are met:

1. For solid round and tubular leg members, the lines of action of the bracing members meet at a point within the diameter of the leg.
2. For angle leg members, the lines of action of the bracing members meet at a point within a distance equal to 75% of the angle width b, on either side of the centroid of the angle.
3. For built-up leg members, the lines of action of the bracing members meet at a point within the cross-section of the built-up leg.

4.4.4.2 Bracing Members

The effective slenderness ratio formulas specified in 4.5.2 account for the effects of eccentric axial loading for angles connected by one leg with normal framing eccentricities. Normal framing eccentricity shall be defined as the condition where the centroid of the bolt or weld group is located between the heel of the angle and the centerline of the connected leg of the member. When a joint eccentricity exceeds this condition, the bracing member resistance shall be multiplied by the factor $b/2g_c$, where "b" is the width of the connected leg and "g_c" is the distance from the heel of the angle to the centroid of the connection. If the width of the connected leg is 3 in. [76 mm] or less or the slenderness ratio, L/r, is greater than 120, the reduction factor need not be applied.

For tubular and other shapes, which have eccentric connections, the effective slenderness ratio formulas given in 4.5.2 shall be used.

4.4.5 Member Continuity

The consideration of the combined effects of forces and bending moments in the design of a member shall be consistent with its assumed continuity attributes in the analysis model (refer to Section 3.0).

4.5 Compression Members

4.5.1 Leg Members

The effective slenderness ratios, KL/r, for leg members shall be as provided in Table 4-3. The minimum effective length factor, K, shall be equal to 1.0 for latticed structures.

4.5.2 Bracing Members

The effective slenderness ratios, KL/r, for bracing members shall be determined taking into account the loading condition, bracing pattern, member end restraints and framing eccentricities. Effective slenderness ratios, (KL/r), shall be determined from Table 4-4, except for round members welded directly to leg members where the effective length factors, K, shall be taken from Table 4-5. Unbraced lengths and slenderness ratios for commonly used bracing patterns are shown in Tables 4-6 and 4-7. The unbraced length, L, shall be the distance between the centroids of the intermediate or end connections.

A single bolt shall not be considered as providing partial restraint against rotation. It is permissible to consider a multiple bolt or welded connection to provide partial restraint if the connection is to a member capable of resisting rotation of the joint.

A multiple bolt or welded connection made only to a gusset plate without also being connected directly to the member providing restraint (i.e. leg member) shall not be considered to provide partial restraint in the out-of-plane direction.

4.5.2.1 Cross Bracing

The crossover point when connected shall be considered to provide support resisting out-of-plane buckling under any one of the following conditions:

1. One of the diagonal members is continuous and one of the diagonal members is subjected to tension. For built-up diagonal members, one individual member of each diagonal (i.e. one angle of each double angle diagonal) need to be continuous.
2. Triangulated horizontal plan bracing (refer to Figure 4-2) is provided at the intersection point with sufficient resistance as defined in 4.4.1.
3. A continuous horizontal member meeting the following criteria is connected at the crossover point:
   a) The continuous horizontal has sufficient strength to provide resistance to the leg as defined in 4.4.1.
   b) The strength of the continuous horizontal is determined ignoring the out-of-plane buckling resistance of the diagonals.

Otherwise, the crossover point shall not be considered as providing support resisting out-of-plane buckling (refer to Table 4-6).

When there are no diagonal members continuous through the crossover point, either of the following conditions shall be satisfied:

1. Triangulated horizontal plan bracing with sufficient resistance as defined in 4.4.1 is provided at the crossover point.
2. A continuous horizontal with sufficient strength as defined in 3a) and 3b) above is provided through the crossover point.

4.5.2.2 K-Type or Portal Bracing

Triangulated plan bracing shall be provided at the bracing apex point with sufficient resistance as defined in 4.4.1 when the horizontal member is not a continuous member.

When triangulated plan bracing is not provided with a continuous horizontal, the out-of-plane unbraced length of the horizontal shall be considered to be 0.75 times the total length of the horizontal. The horizontal member shall have sufficient strength to provide strength to the legs as defined in 4.4.1 determined using the full length of the horizontal (refer to Table 4-7).

4.5.2.3 Cranked K-Type or Portal Bracing

Triangulated internal hip bracing, with sufficient resistance as defined in 4.4.1, shall be provided at the main diagonal bend (refer to Figure 4-1).

4.5.2.4 Tension-Only Bracing

When a redundant horizontal member is utilized to reduce the unbraced length of a leg member in tension-only bracing systems, the design tensile strength of each diagonal shall not be less than $P_r / (2 \cos \theta)$ calculated in accordance with 4.4.1 where θ is the maximum angle between the diagonals and the redundant horizontal member. In addition, the redundant horizontal member shall be connected to each diagonal or to a gusset plate connected to each diagonal to ensure proper load transfer to the diagonals. The unbraced length of the redundant horizontal member shall be considered to equal the horizontal distance between the legs when both diagonals and horizontals are not continuous at the crossover point.

4.5.3 Built-Up Members

Individual components of built-up members composed of two or more shapes shall be connected to one another at intervals, a_i, such that the maximum slenderness ratio, a_i/r_i, of each of the component shapes between the connectors does not exceed 100% of the governing effective slenderness ratio of the built-up member (except for built-up compression members intended to act compositely in resisting buckling as specified in paragraphs 1. & 2. below).

A minimum of two bolts or equivalent in welding shall be provided in line across the connected width (i.e. connected leg width of a double angle) at each intermittent connector when the connected width is greater than 5 in. [127 mm].

For buckling modes that involve relative deformations that produce shear forces in the connectors (e.g. buckling about the axis parallel to the back-to-back connected legs for double angles), the effective slenderness ratio shall be modified by the following equations:

1. When either end or intermediate bolted connectors are snug-tight bearing connections and a minimum of 2 intermediate connectors are used over the length considered for out-of-plane buckling such that a_i/r_i is not greater than $0.75(KL/r)_o$:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a_i}{r_i}\right)^2} \leq \frac{KL}{r_{ib}}$$

2. When both end and intermediate connectors are welded or bolted using high-strength bolts tensioned to 70% of the published ultimate tensile bolt strength and the spacing of intermediate connectors result in a_i/r_i not greater than $0.75(KL/r)_o$:

$$\frac{a_i}{r_i} \leq 40 \qquad \left(\frac{KL}{r}\right)_m = \left(\frac{KL}{r}\right)_o \leq \frac{KL}{r_{ib}}$$

$$\frac{a_i}{r_i} > 40 \qquad \left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{K_i a_i}{r_i}\right)^2} \leq \frac{KL}{r_{ib}}$$

3. For all other conditions:

$$\left(\frac{KL}{r}\right)_m = \frac{KL}{r_{ib}}$$

where:

$\left(\frac{KL}{r}\right)_m$ = modified effective slenderness ratio of built-up member

$\left(\frac{KL}{r}\right)_o$ = effective slenderness ratio of built-up member acting as a unit

$\frac{KL}{r_{ib}}$ =

r_{ib} = radius of gyration of individual component about its centroidal axis for the direction of buckling under consideration for the built-up member

$\frac{a_i}{r_i}$ = largest slenderness ratio of individual components

a_i = largest distance between connectors

r_i = minimum radius of gyration of individual component

K_i = 0.50 for angles back-to-back

= 0.75 for channels back-to-back

= 0.86 for all other cases

When lacing is used to connect built-up compression members consisting of two or more components, the lacing shall be triangulated and extend the full length of the member. Built-up compression members without triangulated lacing shall be modeled as a Vierendeel truss considering combined bending and axial forces in accordance with section 4.8. In addition, the design strength of the bracing system shall be capable of providing the resistance, P_s, as required in 4.4.1.

4.5.4 Design Compression Strength

The design compression strength shall be equal to the lower strength considering flexural buckling in accordance with 4.5.4.2 and flexural-torsional buckling in accordance with 4.5.4.3.

4.5.4.1 Effective Yield Stress

For 60 degree and 90 degree single and double angle members and for formed 60 degree U-shaped members, the effective yield stress for axial compression, F'_y, shall be determined as follows:

$w/t \le 0.47\sqrt{\frac{E}{F_y}}$ $\qquad$ $F'_y = F_y$

$0.47\sqrt{\frac{E}{F_y}} < w/t \le 0.85\sqrt{\frac{E}{F_y}}$ $\qquad$ $F'_y = \left[1.677 - 0.677\left(\frac{w/t}{0.47\sqrt{E/F_y}}\right)\right] F_y$

$0.85\sqrt{\frac{E}{F_y}} < w/t \le 25$ $\qquad$ $F'_y = [0.0332\,\pi^2\,E/(w/t)^2]$

The width to thickness ratio, w/t, shall not exceed 25 (refer to Figure 4-3).

For solid round members, the effective yield stress, F'_y, shall be equal to F_y.

For tubular round members, the diameter to thickness ratio, D/t, shall not exceed 300. The effective yield stress, F'_y, shall be determined as follows:

$D/t \le 0.114\,E/F_y$ $\qquad$ $F'_y = F_y$

$0.114\,E/F_y < D/t \le 0.448\,E/F_y$ $\qquad$ $F'_y = \left(\frac{0.0379\,E}{(D/t)\,F_y} + \frac{2}{3}\right) F_y$

$0.448\,E/F_y < D/t \le 300$ $\qquad$ $F'_y = \frac{0.337E}{(D/t)}$

where:

w/t = width to thickness ratio

D/t = diameter to thickness ratio

E = modulus of elasticity

F_y = specified minimum yield strength

For polygonal tubular steel members, the maximum width to thickness ratio (w/t) and effective yield stress, F'_y, shall be determined from Table 4-8.

4.5.4.2 Flexural Buckling Compression Strength

The design flexural buckling strength shall be taken as $\phi_c P_n$:

For 60 degree and 90 degree single and double angle members and for formed 60 degree U-shaped members:

$\phi_c = 1.0$

$$P_n = F_{cr} A_g$$

$$\frac{KL}{r} \le 4.44\sqrt{\frac{E}{F'_y}} \qquad F_{cr} = \left[1 - 0.25\frac{F'_y}{F_e}\right]F'_y$$

$$\frac{KL}{r} > 4.44\sqrt{\frac{E}{F'_y}} \qquad F_{cr} = F_e$$

where:

F_{cr} = critical compression stress

A_g = gross area of member(s)

KL/r = effective slenderness ratio for the direction of buckling under consideration

E = modulus of elasticity

F'_y = effective yield stress

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

For solid and tubular round members:

ϕc = 0.97 for ERW HSS and pipe shapes with $F_y \le 52$ ksi [360 MPa]

ϕc = 0.90 for ASTM A1085 HSS shapes based on nominal thickness and other members

$$P_n = F_{cr} A_g$$

$$\frac{KL}{r} \le 4.71\sqrt{\frac{E}{F'_y}} \qquad F_{cr} = \left[0.658^{\frac{F'_y}{F_e}}\right]F'_y$$

$$\frac{KL}{r} > 4.71\sqrt{\frac{E}{F'_y}} \qquad F_{cr} = 0.877 F_e$$

where:

F_{cr} = critical compression stress

A_g = gross area of member

KL/r = effective slenderness ratio for the direction of buckling under consideration

E = modulus of elasticity

F'_y = effective yield stress

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

For other compression members, the design flexural buckling strength shall be determined in accordance with AISC 360 Chapter E.

4.5.4.3 Flexural-Torsional Buckling Compression Strength

For 60 degree and 90 degree single angle members and for formed 60 degree U-shaped members, the design flexural-torsional buckling strength shall be equal to the flexural buckling strength in accordance with 4.5.4.2.

For double angle members, the design flexural-torsional buckling strength shall be equal to the flexural buckling strength based on $KL/r = (KL/r)_m$ in accordance with 4.5.4.2 substituting a value of F_e determined in accordance with the following:

$$F_e = \frac{F_{ey} + F_{ez}}{2H}\left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2}}\right]$$

where:

$$F_{ey} = \frac{\pi^2 E}{\left(\left(\frac{KL}{r}\right)_m\right)^2}$$

$$F_{ez} = \frac{2GJ}{A_g \bar{r}_o^2}$$

H = $1 - y_o^2/\bar{r}_o^2$

$(KL/r)_m$ = modified effective slenderness ratio from 4.5.3

G = shear modulus of elasticity

J = St. Venant torsional constant of single angle

A_g = gross area of double angles

$\bar{r}_o^2$ = $y_o^2 + (I_x + I_y)/A_g$

y_o = distance along the principal axis parallel to the back-to-back legs from the centroid of the double angles to the center thickness of the outstanding legs

I_x, I_y = moment of inertia of the double angles about the principle axes

Flexural-torsional buckling need not be considered for solid round or tubular members. For other compression members, the design flexural-torsional buckling strength shall be determined in accordance with AISC 360 Chapter E.

4.6 Tension Members

4.6.1 Built-up Members

The longitudinal spacing of connectors between components of built-up members composed of two or more shapes, shall preferably limit the effective slenderness ratio in any component between the connectors to 300.

4.6.2 Tension-Only Bracing Members

Welded end tabs for tension-only bracing members shall be detailed to develop the design strength of the member based on yielding of the gross section of the member. The member length between connections shall be detailed short such that the member is in tension when installed.

4.6.3 Design Tensile Strength

The design axial tensile strength, $\phi_t P_n$, of a member shall be taken as the lesser of yielding in the gross section, rupture in the net effective section, or block shear rupture.

For tension yielding in the gross section:

ϕ_t = 0.80 for guy anchor shafts

ϕ_t = 0.97 for ERW HSS and pipe shapes with $F_y \leq 52$ ksi [360 MPa]

ϕ_t = 0.90 for ASTM 1085 HSS shapes based on nominal thickness and other members

$P_n = F_y A_g$

For tension rupture in the effective net section:

ϕ_t = 0.65 for guy anchor shafts

ϕ_t = 0.81 for ERW HSS and pipe shapes with $F_y \leq 52$ ksi [360 MPa]

ϕ_t = 0.75 for for ASTM 1085 HSS shapes based on nominal thickness and other members

$P_n = F_u A_{en}$

For block shear rupture:

ϕ_t = 0.65 for guy anchor shafts

ϕ_t = 0.81 for ERW HSS and pipe shapes with $F_y \leq 52$ ksi [360 MPa]

ϕ_t = 0.75 for ASTM 1085 HSS shapes based on nominal thickness and other members

P_n = $0.6\ F_u\ A_{nv} + U_{bs}\ F_u\ A_{nt}$ [$0.6\ F_y\ A_{gv} + U_{bs}\ F_u\ A_{nt}$

U_{bs} = 1.0 for latticed tower connections

U_{bs} = 0.5 for coped beam shear end connections made with multiple rows of bolts

where:

A_g = gross area

A_{en} = effective net area

A_{nv} = net area subject to shear

A_{gv} = gross area subject to shear

A_{nt} = net area subject to tension

F_y = specified minimum yield strength of the critical connected part

F_u = specified minimum tensile strength of the critical connected part

4.6.3.1 Net Area

The net area of a member, A_n, shall be taken as the sum of the products of the thickness and the net width of each element computed as follows:

In computing the net area of the section, the width of the bolt hole shall be taken as 1/16 in. [2 mm] greater than the nominal dimension of the hole.

For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part, W_n, shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions of all holes in the chain, and adding, for each gage space in the chain, the quantity $s^2 / 4g$. Each possible chain of holes shall be considered. The minimum net area considering each chain of holes and the net area through each individual hole shall be used to determine P_n. The net area shall be determined as follows:

$$A_n = W_n\, t + (s^2 t) / (4g)$$

where:

W_n = net width

s = the longitudinal center-to-center spacing (pitch) of any two consecutive holes

t = member thickness

g = the transverse center-to-center spacing (gage) between fastener gage lines

4.6.3.2 Effective Net Area

When a tension force is transmitted directly to each of the member's cross-sectional elements by fasteners or welds, the effective net area, A_{en}, is equal to the net area A_n.

When a tension force is transmitted by fasteners or welds through some, but not all of the cross-section elements of the member, the effective net area, including shear lag effects shall be taken as:

$$A_{en} = A\,U$$

where:

A = A_n for bolted members, and A_g for welded members

U = a reduction factor in accordance with the following:

1. For single and multiple bolted angle members: U = 0.75. Alternatively, when the outstanding leg (unconnected leg) is ignored in calculating A_n, U = 1.0
2. For HSS and pipe bracing members with single concentric gusset plates: U = 0.75
3. For solid round bracing members with single concentric gusset plates: U = 1.0
4. For bracing members with single eccentric gusset plates, flattened ends or other eccentric connections: U = 0.70
5. For leg members eccentrically connected: U = 0.80
6. For other members: U shall be in accordance with AISC 360 Chapter D. It shall also be permissible for all connections to determine U in accordance with AISC 360 Chapter D.

Note:

1. The effective net area reduction accounts for the effects of eccentric axial forces on the design tensile strength of a member.

4.7 Flexural Members

The design flexural strength shall be taken as, $\phi_f M_n$:

ϕ_f = 0.97 for ERW HSS and pipe shapes with $F_y \leq 52$ ksi [360 MPa]

ϕ_f = 0.90 for ASTM 1085 HSS shapes based on nominal thickness and for other members

M_n = nominal flexural strength

Note: Bracing members connected with normal framing eccentricities as defined in 4.4.4.2 need not be considered as flexural members.

4.7.1 Solid Round Members

For solid round members, M_n shall be determined as follows:

$$M_n = F_y Z$$

where:

F_y = specified minimum yield strength

Z = plastic section modulus

4.7.2 Tubular Round Members

For tubular round members, the diameter to thickness ratio (D/t) shall not exceed 300. M_n shall be determined as follows:

$$\frac{D}{t} \le 0.0714\frac{E}{F_Y} \qquad M_n = F_y Z$$

$$0.0714\frac{E}{F_Y} < \frac{D}{t} \le 0.309\frac{E}{F_Y} \qquad M_n = \left(\frac{0.0207\ E}{(D/t)\ F_y} + 1\right)F_y S$$

$$0.309\frac{E}{F_Y} < \frac{D}{t} \le 300 \qquad M_n = \left(\frac{0.330\ E}{(D/t)}\right)S$$

where:

D/t = diameter to thickness ratio

E = modulus of elasticity

F_y = specified minimum yield strength

Z = plastic section modulus

S = elastic section modulus

4.7.3 Polygonal Tubular Members

For polygonal tubular members, M_n shall be determined as follows:

M_n = $F'_y S$

where:

F'_y = effective yield stress as determined from 4.5.4.1

S = minimum elastic section modulus

4.7.4 Other Members

For other shapes, the design flexural design strength shall be determined in accordance with AISC 360 Chapter F.

4.8 Combined Bending and Axial Forces

4.8.1 Latticed Structures

Factored moments, M_u, shall be multiplied by an amplification factor, B_1, in accordance with the following, to account for secondary moments in individual members:

B_1 = 1.0 when member displacement effects (P-delta) are considered between supports or for tension members

B_1 = $1.00 / (1-P_u/P_e)$ for all other members

where:

P_u = Axial compressive force due to factored load

P_e = $\pi^2 EI / (KL)^2$

EI = flexural stiffness in plane of buckling under consideration

KL = laterally unbraced effective length of member in the direction of bending under consideration

4.8.1.1 Leg Members

Axial forces and secondary moments induced in solid and tubular round leg members due to member continuity assumed for analysis per 4.4.5 shall satisfy the following interaction equations when leg members are modeled as 3-D beam elements:

$$\left|\frac{P_u}{\phi_a P_n}\right| \geq 0.2 \qquad \frac{8}{9}\left(\left|\frac{P_u}{\phi_a P_n}\right| + \left|\frac{B_1 M_u}{\phi_f M_n}\right|\right) \leq 1.0 \quad \text{and} \quad \left|\frac{P_u}{\phi_a P_n}\right| \leq 1.0$$

$$\left|\frac{P_u}{\phi_a P_n}\right| < 0.2 \qquad \left|\frac{P_u}{2\phi_a P_n}\right| + \left|\frac{B_1 M_u}{\phi_f M_n}\right| \leq 1.0$$

Moments induced in solid and tubular round leg members for other conditions combined with axial forces shall satisfy the following interaction equations:

$$\left|\frac{P_u}{\phi_a P_n}\right| \geq 0.2 \qquad \left|\frac{P_u}{\phi_a P_n}\right| + \frac{8}{9}\left|\frac{B_1 M_u}{\phi_f M_n}\right| \leq 1.0$$

$$\left|\frac{P_u}{\phi_a P_n}\right| < 0.2 \qquad \left|\frac{P_u}{2\phi_a P_n}\right| + \left|\frac{B_1 M_u}{\phi_f M_n}\right| \leq 1.0$$

where:

P_u = axial force due to factored loads

$\phi_a P_n$ = design axial strength from 4.5.4 (compression) or 4.6.3 (tension)

$B_1 M_u$ = amplified moment due to factored loads

$\phi_f M_n$ = design flexural strength

Combined bending and axial forces for other shapes shall be investigated in accordance with AISC 360 Chapter H.

When investigating joint eccentricities that exceed normal framing eccentricities (i.e. eccentric diagonal to leg connections) as defined in 4.4.4 or for eccentric leg splices, the following interaction equation shall be satisfied:

$$\left|\frac{P_u}{\phi_a P_n}\right| + \left|\frac{M_u}{\phi_f M_n}\right| \leq 1.0$$

where:

P_u = axial force due to factored loads

$\phi_a P_n$ = design axial strength = smaller of 0.90 $F_y A_g$ and 0.75 $F_u A_{en}$

F_y = specified minimum yield strength

A_g = gross area of leg member

F_u = specified minimum ultimate tensile strength

A_{en} = effective net area of leg member

M_u = moment due to factored loads based on the eccentricity that exceeds normal framing eccentricities or moment from eccentric leg splices

$\phi_f M_n$ = design flexural strength

4.8.2 Tubular Structures

For tubular structures, the following interaction equation shall be satisfied:

$$\left|\frac{P_u}{\phi_c P_n}\right| + \left|\frac{M_u}{\phi_f M_n}\right| + \left[\left|\frac{V_u}{\phi_v V_n}\right| + \left|\frac{T_u}{\phi_T T_n}\right|\right]^2 \leq 1.0$$

where:

P_u = axial compressive force due to factored loads

P_n = nominal axial compressive strength = $F'_y A_g$, where $F'_y \leq Fy$

F'_y = effective yield stress determined from 4.5.4.1

A_g = gross area of cross section

F_y = specified minimum yield strength

M_u = flexural moment due to factored loads

M_n = nominal flexural strength from section 4.7

V_u = transverse shear force due to factored loads

V_n = nominal shear strength from 4.8.2.1 for round sections and 4.8.2.2 for polygonal sections

T_u = torsional moments due to factored loads

T_n = nominal torsional strength from 4.8.2.1 for round sections and 4.8.2.2 for polygonal sections

ϕ_c = 0.90 = resistance factor for axial compression

ϕ_f = 0.90 = resistance factor for flexure

ϕ_v = 0.90 = resistance factor for shear

ϕ_T = 0.95 = resistance factor for torsion

Entry/exit ports, hand holes and other similar openings in tubular sections shall be reinforced to provide a reinforced section strength equal to or greater than the strength of the tubular section without an opening. This condition shall be considered to be satisfied when the area of the reinforcing satisfies the following criteria:

$$A_r F_{yr} \geq A_p F_y$$

$$A_r F_{yr} X_r \geq A_p F_y \, 0.5(D\text{-}t)$$

where:

A_r = cross sectional area of reinforcing at the centerline of opening

F_{yr} = specified minimum yield strength of the reinforcing material but not greater than F_y

A_p = cross sectional area of the tubular section removed for the opening at the centerline of the opening

F_y = specified minimum yield strength of the tubular section

X_r = distance from the centerline of the section to the centroid of the reinforcement material at the centerline of the opening

D = outside diameter for round sections and outside flat-to-flat width for polygonal sections

t = thickness of tubular section

4.8.2.1 Round Tubular Sections

The nominal shear strength, V_n, and nominal torsional strength, T_n, for round pole sections shall be determined as follows:

$$V_n = 0.5 \, F_{nv} A_g$$

$T_n = F_{nt}\, C_t$

where:

F_{nv} = greater of following but shall not exceed 0.6 F_y:

$$\frac{1.60E}{\sqrt{\frac{L_p}{D}\left(\frac{D}{t}\right)^{\frac{5}{4}}}} \quad \text{and} \quad \frac{0.78E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}}$$

F_{nt} = greater of following but shall not exceed 0.6 F_y:

$$\frac{1.23E}{\sqrt{\frac{L_p}{D}\left(\frac{D}{t}\right)^{\frac{5}{4}}}} \quad \text{and} \quad \frac{0.60E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}}$$

where:

A_g = gross area of cross section

C_t = torsional constant

F_y = specified minimum yield strength

E = modulus of elasticity

L_p = height of a pole structure or distance between guy/base elevations or cantilevered length for a guyed mast

D = outside diameter

t = wall thickness

Note: C_t may be conservatively determined from Table 4-9.

4.8.2.2 Polygonal Tubular Sections

For polygonal pole sections, V_n and T_n shall be determined as follows:

$V_n = 0.6\, F_y A_g/2$

$T_n = 0.6\, F_y C_t$

where:

F_y = specified minimum yield strength

A_g = gross area of cross section

C_t = torsional constant

Note: C_t may be conservatively determined from Table 4-9.

4.9 Connections

4.9.1 Bolts

Pretensioned bolts shall not be reused once they are placed in service and tensioned beyond 40% of their ultimate capacity.

Hot-dip (or mechanically) galvanized A490 bolts, hot-dip (or mechanically) galvanized A354 Grade BD anchor rods and other threaded fasteners / anchor rods with a specified minimum tensile strength greater than 125 ksi [862 MPa] shall not be used.

Note: F_{ub} shall be equal to the minimum tensile strength based on bolt nominal diameter for calculating design tensile and shear strengths.

4.9.2 Nut-Locking Devices

Bolts used to connect load-carrying members shall be provided with a nut-locking device or mechanism such as, but not limited to, lock nuts, lock washers, or palnuts, to prevent loosening, except as provided in 4.9.3. The use of lock washers shall be limited to structures 1,200 ft. [366 m] or less in height.

4.9.3 Pretensioned Bolts

Slip-critical connections and connections subjected to tension where the applications of externally applied load results in prying action produced by deformation of the connected parts shall be made with high-strength bolts tightened to 70% of the ultimate tensile strength of the bolt. Nut-locking devices are not required for pretensioned bolts.

Exception: For tension connections, where it can be shown that the stiffness of the connected parts is sufficient to reduce prying forces to insignificance, tension connections may be made with high-strength bolts tightened to a snug-tight condition. Snug tight is defined as the tightness that exists as a result of a few impacts of an impact wrench or the full effort of a person using an ordinary spud wrench.

Note: Contact surfaces for slip-critical connections shall not be oiled or painted. .

4.9.4 Edge Distances

Table J3.4 of the AISC 360 specification shall apply except at sheared edges where the minimum edge distance shall be 1.5 times the bolt diameter and except for flange and base plates where edge distances shall be adequate to prevent the nuts of the connection bolts or anchor rods from extending over the edge of the flange or base plate.

The minimum spacing of bolts in a line is preferably 3 bolt diameters but shall not be less than 2-2/3 diameters.

4.9.5 Bearing Type Connections

Bolts (including high strength bolts) tightened to a snug-tight condition as defined in 4.9.3 are permissible for use in bearing-type connections. Bearing-type connections shall not be used with oversize or slotted holes parallel to the line of force for bolts loaded primarily in shear.

4.9.6 Connection Resistance

4.9.6.1 Design Tensile Strength

The design tensile strength of a single bolt or threaded part shall be taken as ϕR_{nt}:

ϕ = 0.75

$R_{nt} = F_{ub} A_n$

The net area, A_n, through the threaded portion of the bolt is given by:

For ANSI inch series thread geometry:

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

For ISO metric series thread geometry:

$$A_n = \frac{\pi}{4}(d - 0.9382\,p)^2 \ [\text{mm}^2]$$

where:

F_{ub} = specified minimum tensile strength
d = nominal diameter of bolt, in. [mm]

n = number of threads per inch

p = pitch of threads, [mm]

4.9.6.2 Design Bearing Strength

The design bearing strength at bolt or attachment holes, ϕR_n, shall be taken as:

ϕ = 0.80

$R_n = 1.2\,(L_c + d/4)\,t\,F_u \leq 2.4\,d\,t\,F_u$

When slotted holes are used perpendicular to the line of force, ϕR_n shall be taken as:

$R_n = 1.0\,L_c\,t\,F_u \leq 2.0\,d\,t\,F_u$

where:

L_c = clear distance, in the direction of the force, between the edge of the hole and the edge of an adjacent hole or edge of the material, in. [mm]

t = thickness of the critical connected part

d = nominal bolt diameter

F_u = specified minimum tensile strength of the critical connected part

For multiple bolt connections, the bearing resistance shall be taken as the sum of the bearing resistances of the individual bolts.

4.9.6.3 Design Shear Strength

The design shear strength of a bolt, ϕR_{nv}, shall be taken as:

ϕ = 0.75

When threads are excluded from the shear plane:

$R_{nv} = 0.625\ R_b\ F_{ub}\ A_b$

When threads are included in the shear plane:

$R_{nv} = 0.625\ R_b\ F_{ub}\ 0.8\ A_b$

where:

R_b = connection length reduction factor

= 1.00 where $L_b \leq$ 16 in. [406 mm] or for a single bolt connection

= 0.90 where 16 in. [406 mm] < $L_b \leq$ 38 in. [965 mm]

= 0.75 where L_b > 38 in. [965 mm]

L_b = connection length equal to the distance along the longitudinal axis of a member between the first and last bolt in a multiple bolt connection

F_{ub} = specified minimum tensile strength of bolt

A_b = nominal unthreaded area of bolt or threaded part

Slotted holes must be perpendicular to the line of force unless slip-critical connections in accordance with AISC 360 Chapter J are utilized. Hardened steel washers conforming to ASTM F436 shall be used over slotted holes in an outer ply.

4.9.6.4 Combined Shear and Tension

For bolts subjected to combined shear and tension, the following relationship shall be satisfied:

$$\left(\frac{V_{ub}}{\phi R_{nv}}\right)^2 + \left(\frac{T_{ub}}{\phi R_{nt}}\right)^2 \leq 1$$

where:

V_{ub} = bolt shear force due to factored loads

ϕR_{nv} = design shear strength of bolt

T_{ub} = bolt tensile force due to factored loads

ϕR_{nt} = design tensile strength of bolt

4.9.6.5 Connecting Elements

The design strength of welded and bolted connecting elements, $\phi_p R_{np}$, shall be the lower value obtained according to limit state of yielding, rupture and block shear.

ϕ_p = 0.90 for tension yielding

ϕ_p = 1.00 for shear yielding

ϕ_p = 0.75 for tension and shear rupture

ϕ_p = 0.75 for block shear

For tension yielding:
$R_{np} = F_y A_{gt}$

For tension rupture:
$R_{np} = F_u A_{nt}$

For shear yielding:
$R_{np} = 0.6 F_y A_{gv}$

For shear rupture:
$R_{np} = 0.6 F_u A_{nv}$

For block shear:
Refer to 4.6.3.

where:

A_{gt} = gross area subject to tension

A_{nt} = net area subject to tension

A_{gv} = gross area subject to shear

A_{nv} = net area subject to shear

F_y = specified minimum yield strength

F_u = specified minimum tensile strength

Notes:

1. Refer to 4.6.3.1 for the determination of net area.
2. The net area of a bolted connection plate shall not be considered larger than 85% of the gross area.
3. Angles and other shapes used as connecting elements shall also satisfy the strength limits from 4.6.

4.9.7 Splices

Splices shall be designed to resist the maximum tensile, compressive and shear forces occurring at the splice.

For leg members of guyed masts, unless the additional guy rupture loading requirements of Annex E are satisfied for each guy, the leg splices shall develop a minimum design tensile strength equal to the lower of 33% of the design compression force at the splice or 500 kips [2,200 kN].

When eccentricity of a joint exists, the additional forces introduced into the connection shall be considered.

For pole structures, base and flange plate connections shall resist a minimum of 50% of the design flexural strength, $\phi_f M_n$, of the lowest strength connected member.

4.9.7.1 Tubular Pole Structures

The nominal design length of a slip splice shall be determined considering manufacturing tolerances to result in a minimum installed slip splice length equal to 1.5 times the inside width of the base of the upper section. The inside width shall be measured between flats for polygonal cross sections.

The height of a tubular pole structure determined using the individual section lengths of the structure shall be based on nominal design slip splice lengths equal to 1.6 times the inside width of the base of the upper section unless otherwise documented.

Provisions for applying jacking forces to a slip splice shall be provided for the pole sections at a slip splice (refer to 13.3.5).

4.9.8 Guy Assembly Link Plates

The design strength of a link plate, $\phi_k R_{nk}$, shall be taken as the lowest value of:

Tension on the effective area:
$\phi_k = 0.75$ $R_{nk} = 2\, t\, b_{eff}\, F_u$

Shear on the effective area:
$\phi_k = 0.75$ $R_{nk} = 0.6\, A_{sf}\, F_u$

Bearing on the projected area at the pin:
$\phi_k = 0.90$ $R_{nk} = 1.8\, d\, t\, F_y$

Yielding on the gross area:
$\phi_k = 0.90$ $R_{nk} = A_g\, F_y$

where:

t = thickness of link plate

b_{eff} = 2 t + 0.625 in. [2 t + 16 mm], but not more than the actual distance from the edge of the pin hole to the edge of the part measured in a direction normal to the applied force

A_{sf} = 2 t (a + d/2)

a = shortest distance from the edge of the pin hole to the edge of the member measured parallel to the direction of the force

d = pin diameter

A_g = gross area of link plate

F_u = specified minimum tensile strength

F_y = specified minimum yield strength

The pin hole shall be located midway between the edges of the member in the direction normal to the applied force. The width of the plate beyond the pin hole shall not be less than 2 b_{eff} + d and the minimum extension, a, beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than 1.33 b_{eff}.

The corners beyond the pin hole are permitted to be cut at 45 degrees to the axis of the member, provided the net area beyond the pin hole, on a plane perpendicular to the cut, is not less than that required beyond the pin hole parallel to the axis of the member.

Note: The 1/32 in. [1 mm] oversize pinhole diameter specified in AISC 360 does not apply.

4.9.9 Anchor Rods

Leveling nuts shall be provided with anchor rods unless otherwise specified for site-specific applications. Cementitious grout, if utilized below a base plate in combination with leveling nuts, shall not be considered as load bearing for determining anchor rod compression and tension axial forces.

All anchor rod nuts (top and leveling nuts) shall be tightened to a snug tight condition. Top nuts shall be rotated, with the leveling nut secured an additional 1/3 third turn for anchor rods 1.5 in. [38 mm] or less in diameter and an additional 1/6 turn for anchor rod diameters greater than 1.5 in. [38 mm]. Appropriate washers shall be used above and below oversized flange plate holes. Nut locking devices are not required for properly tightened anchor rod nuts.

The following interaction equations shall be satisfied for anchor rod forces:

$l_{ar} \leq 1(d)$

$$\left[\frac{P_{ut}}{\phi_t R_{nt}}\right]^2 + \left[\frac{V_u}{\phi_v R_{nv}}\right]^2 \leq 1.0$$

$$\left|\frac{P_{uc}}{\phi_c R_{nc}}\right| + \left[\frac{V_u}{\phi_c R_{nvc}}\right]^2 \leq 1.0$$

$1(d) < l_{ar} \leq 4(d)$

$$\left[\left|\frac{P_{ut}}{\phi_t R_{nt}}\right| + \left|\frac{M_u}{\phi_f M_n}\right|\right]^2 + \left[\frac{V_u}{\phi_v R_{nv}}\right]^2 \leq 1.0$$

$$\left|\frac{P_{uc}}{\phi_c R_{nc}}\right| + \left|\frac{M_u}{\phi_f M_n}\right| + \left[\frac{V_u}{\phi_c R_{nvc}}\right]^2 \le 1.0$$

$l_{ar} > 4(d)$

$$\left[\left|\frac{P_{ut}}{\phi_t R_{nt}}\right| + \left|\frac{M_u}{\phi_f M_n}\right|\right]^2 + \left[\frac{V_u}{\phi_v R_{nv}}\right]^2 \le 1.0$$

$$\left|\frac{P_{uc}}{\phi_c R_{nb}}\right| + \left|\frac{M_u}{\phi_f M_n}\right| + \left[\frac{V_u}{\phi_c R_{nvc}}\right]^2 \le 1.0$$

where:

l_{ar} = anchor rod projection from supporting surface to bottom of leveling nut

d = nominal anchor rod diameter, in. [mm]

P_{ut} = anchor rod axial tension force from uplift and overturning factored reactions of structure

P_{uc} = anchor rod axial compression force from download and overturning factored reactions of structure

V_u = anchor rod shear force occurring with axial force under consideration (from shear and torsion reactions of structure)

M_u = moment applied to anchor rod from factored shear force on anchor rod equal to $0.65\ l_{ar}\ V_u$

R_{nt} = nominal tensile strength = $F_u\ A_n$

R_{nc} = nominal compression yield strength = $F_y\ A_n$

R_{nv} = nominal shear rupture strength = $0.5\ F_u\ A_g$

R_{nvc} = nominal shear yield strength = $0.6\ F_y\ A_n/2$

M_n = nominal flexural strength = $F_y\ Z$

R_{nb} = nominal buckling strength of anchor rod acting as a column = $F_{cr}\ A_n$

F_u = specified minimum tensile strength of anchor rod

F_y = specified minimum yield strength of anchor rod

F_{cr} = critical compression stress from 4.5.4.2 for solid round members based on an effective length equal to $1.2\ l_{ar}$ and a radius of gyration based on the tensile root diameter (d_n)

A_n = net area of anchor rod from 4.9.6.1

A_g = gross area of anchor rod

Z = plastic section modulus of anchor rod based on the tensile root diameter (d_n)

d_n = tensile root diameter

= $d - 0.9743/n$ for ANSI inch series thread geometry

= $d - 0.9382(p)$ for ISO metric series thread geometry

n = number of threads per inch

p = pitch of threads, [mm]

ϕ_t = 0.75

ϕ_v = 0.75

ϕ_c = 1.00

ϕ_f = 0.90

Note: When the anchor rod projection, l_{ar}, for an installation exceeds 1(d) but is not more than 3 in. [75 mm], it shall be permitted to consider l_{ar} less than or equal to 1(d) when 5,000 psi [35 MPa] minimum 7-day strength non-shrink, non-metallic grout is properly installed between the supporting surface and a base plate with properly installed leveling nuts. Drainage is required for all grouted base plates for base plates supporting tubular sections.

4.9.10 Welded Connections

Welded connections shall conform to AISC 360 Chapter J.

4.9.10.1 Tubular Pole Structures

Longitudinal seam welds for tubular pole sections shall have 60% minimum penetration, except in the following areas where the longitudinal seam welds shall be complete penetration or full fusion through the full cross-section:

1. Longitudinal seam welds within 6 in. [150 mm] of circumferential welds or flange or base plates.
2. Outside (female) section longitudinal seam welds in the slip splice area for a minimum distance equal to the maximum lap dimension plus 6 in. [150 mm].

Transverse seam welds shall be complete penetration or full fusion through the full cross-section.

Base plate to pole shaft welds shall be complete penetration welds. Alternatively, for pole diameters 24 in. [610 mm] or less, pole shafts may be inserted into exterior flanges or base plates and connected with inner and outer fillet welds.

4.9.11 U-Bolt Connections

Nut locking devices shall be provided for all U-bolt connections.

Nuts and U-bolts shall be provided from the same source to insure compatibility of the nuts with the threads of the U-bolts after galvanizing.

Pretensioned U-bolts shall not be re-used once they have been placed in service.

Heat treatment for U-bolts after forming shall be in accordance with the SAE AMS2759 Standard, "Heat Treatment of Steel Parts, General Requirements".

4.9.11.1 Round U-Bolts

Round U-bolts shall conform to one of the following pre-qualified material specifications:
ASTM A36, A529, A572, A449, A193 Gr B7, A354 Gr BC, F1554, SAE J429 Gr 2 or 5. It shall be permissible to use other steel materials suitable for the application and site.

Round U-bolts shall be stress relieved after forming for a minimum of 1 hour or until the entire part reaches the stress relieving temperature. The stress relieving temperature shall be between 1,000 degrees Fahrenheit [538 degrees C] and 1,200 degrees Fahrenheit [649 degrees C] but no more than 50 degrees Fahrenheit [10 degrees C] below the temperating temperature for heat treated U-bolts. U-bolts shall be air cooled in still air.

4.9.11.2 Square U-Bolts

Square U-bolts shall conform to one of the following pre-qualified material specifications:

ASTM A36, A529, A572 Gr 42, 50 or 55, F1554 Gr 36 or 55, SAE J429 Gr 2. It shall be permissible to use other steel materials suitable for the application and site.

Square U-bolts shall be normalized after forming and shall meet the mechanical properties including the minimum specified yield and tensile strengths of the material specification after normalizing.

The inside nominal width of a square U-bolt shall be detailed to not exceed 1/8 in. [3 mm] greater than the nominal width of the attached member. The inside corner radius of a square U-bolt shall not be less than the outside corner radius of an HSS member or the diameter of the U-bolt. In lieu of more accurate information, the outside corner radius of an HSS member shall be assumed to equal 2.25 times the nominal wall thickness of the HSS.

Note: The limitation of material grades for square U-bolts is intended to provide sufficient ductility for a square U-bolt to conform to the shape of an HSS member upon tightening.

4.9.11.3 U-Bolts Strength

U-bolt connections shall not be utilized to transfer torsion to a round supporting member (i.e. moment about a member's longitudinal axis) required to maintain strength and stability of a structure. The limitation under this loading condition shall not apply to connections used for appurtenances.

Shear and axial forces on each leg of a U-bolt shall meet the strength requirements for bolted bearing connections from 4.9.6. In addition, the design axial tensile strength, ϕR_{nt}, of each leg of the U-bolt shall not exceed 0.85 $F_y A_g$ where F_y is the U-bolt minimum specified yield strength and A_g is the gross area of one leg of the U-bolt.

The strength of a U-bolt connection in transferring forces parallel to the longitudinal axis of a supporting member (sliding) and moments about the longitudinal axis of a supporting member (torsion, applicable to appurtenance connections only) shall meet the following interaction equation:

$$\left(\frac{V_{us}}{\phi_u R_{ns}}\right)^2 + \left(\frac{T_{ur}}{\phi_u R_{nr}}\right)^2 \leq 1.0$$

where:

V_{us} = shear force applied parallel to the supporting member (sliding) from factored loads

T_{ur} = torsional moment applied about the longitudinal axis of the supporting member (twisting) from factored loads

R_{ns} = nominal sliding strength in accordance with the following:

$0.30(2T_p - T_{ut}) \geq 0$

R_{nr} = nominal torsional strength = $0.5\, D\, R_{ns}$

T_p = installed pretension in each leg of U-bolt

T_{ut} = exterior tension force applied to the U-bolt assembly due to factored loads

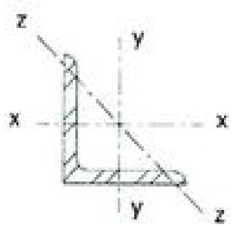
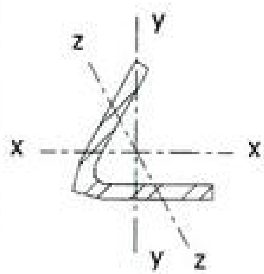
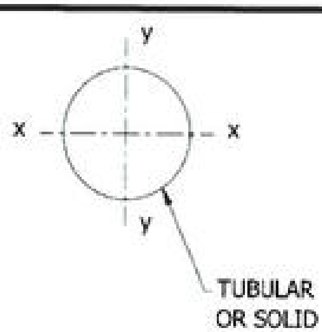
D = diameter of supporting member

ϕ_u = 1.00

The installed pretension in each leg of a U-bolt shall be considered to equal 20 ksi [140 MPa] times the gross area of a U-bolt leg based on its nominal diameter but not greater than the maximum force that a U-bolt leg can be tightened based on the U-bolt bracket strength.

Note: The results of documented tests for specific U-bolt connections shall be permitted to be substituted for the strength equations specified in this Section.

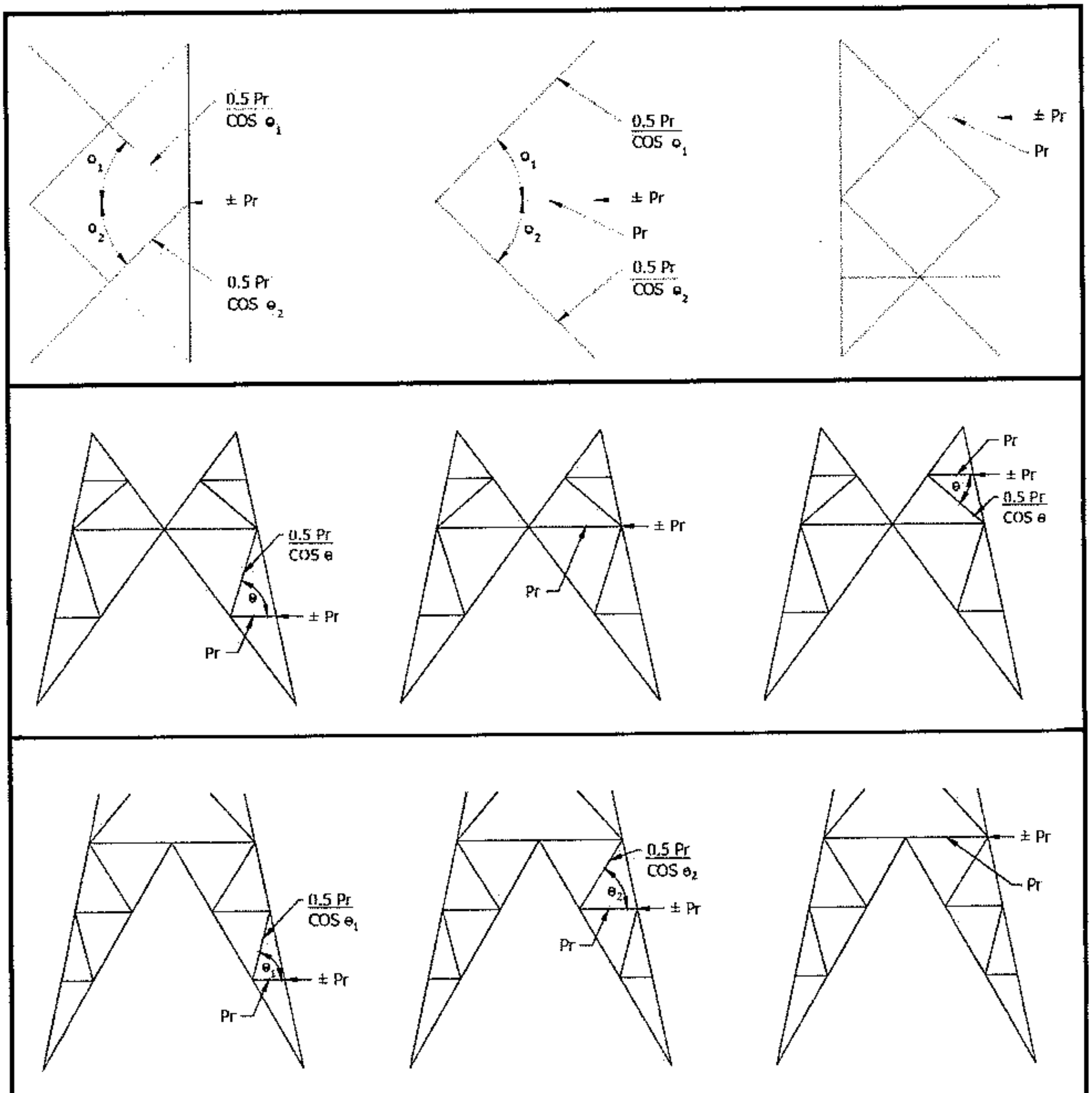
Table 4-1: Bracing Resistance Required for Leg Members

Leg Shape	Tower Cross Section	Bracing Resistance, P_r, Required in a Face at a Panel Point
	Square	When weak-axis buckling (KL/r_z) governs: $P_r = P_s / (2 \times 0.707) = 0.707\ P_s$ When in-plane buckling (KL/r_x or KL/r_y) governs: $P_r = P_s$
 Note: $r_x > r_y > r_z$	Triangular	When weak-axis buckling (KL/r_z) governs: $P_r = P_s / (2 \times 0.866) = 0.577\ P_s$ When out-of-plane buckling (KL/r_x) governs: $P_r = P_s / (0.866) = 1.15\ P_s$
	Square	In-plane buckling (KL/r_x or KL/r_y) governs $P_r = P_s$
	Triangular	Out-of-plane buckling (KL/r_x) governs $P_r = P_s / (0.866) = 1.15\ P_s$

Notes:

1. Alternatively, P_r may be determined using the worse case effective slenderness ratio (highest ratio) to determine P_s and multiplying the result by 1.15 for triangular cross sections or by 1.00 for square cross sections.
2. One value of P_r applies for both faces when investigating a segment of a leg. The larger value shall be used considering the leg segment above and below a panel point.
3. Weak-axis buckling governs angle legs for symmetrical bracing patterns. In-plane, out-of-plane or weak-axis buckling may govern staggered bracing patterns.

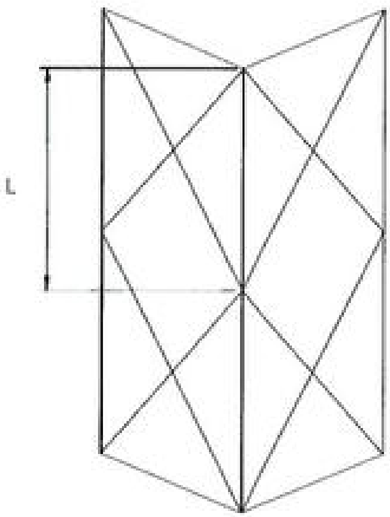
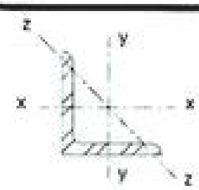
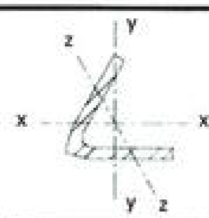
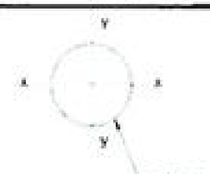
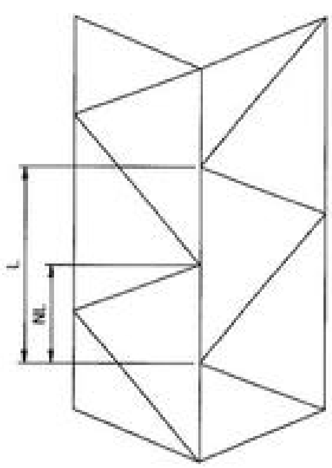
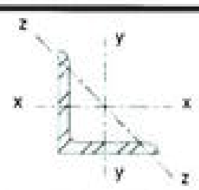
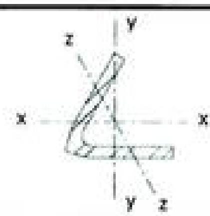
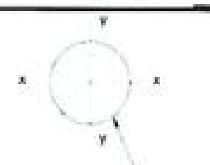
Table 4-2: Minimum Required Resistance at Panel Points



Notes:

1. For leg slopes greater than 15 degrees from vertical, P_r shall be divided by the cosine of the leg slope.

Table 4-3: Effective Slenderness Ratios for Leg Members

Symmetrical Bracing Patterns		
	**Leg Shapes**	**Effective Slenderness Ratios K = 1.0**
		$\frac{KL}{rz}$
		$\frac{KL}{rz}$
	 TUBULAR OR SOLID	$\frac{KL}{rx}$
Staggered Bracing Patterns		
 N ≥ 0.5	**Leg Shapes**	**Effective Slenderness Ratios K = 1.0**
		$\frac{KL}{rx}, \frac{KL}{ry}, \left(\frac{(1+2N)}{3}\right)\frac{KL}{rz}$
		$\frac{KL}{rx}, \left(\frac{(1+2N)}{3}\right)\frac{KL}{rz}$
	 TUBULAR OR SOLID	$\frac{KL}{rx}$

Notes:

1. L shall equal the panel spacing measured along the axis of the leg.
2. The maximum effective slenderness ratio shall be used to determine the design compression strength and the bracing resistance required to provide lateral support.

Table 4-4: Effective Slenderness Ratios for Bracing Members

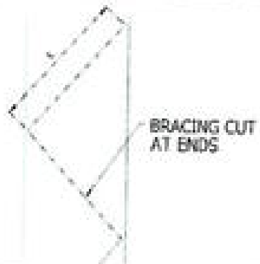
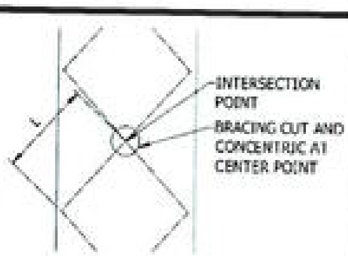
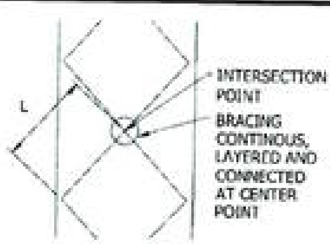
Slenderness Ratios < 120, Eccentricity Governs		
Formula Number	**Equation**	**Conditions at Ends of the Buckling Length Under Consideration**
1	$KL / r = L / r$	Concentric at both ends
2	$KL / r = 30 + 0.75\ L / r$	Concentric at one end and normal framing eccentricity at the other end
3	$KL / r = 60 + 0.50\ L / r$	Normal framing eccentricity at both ends
Concentric Conditions	Double angles or channels Round members with concentric end plate	
Normal Framing Eccentricity Conditions	Single angles or channels Round members with eccentric end plate Round members with flattened ends	

Slenderness Ratios ≥ 120, Restraint Governs		
Formula Number	**Equation**	**Conditions at Ends of the Buckling Length Under Consideration**
4	$KL / r = L / r$	Unrestrained against rotation at both ends
5	$KL / r = 28.6 + 0.762\ L / r$	Partially restrained at one end and unrestrained at other end
6	$KL / r = 46.2 + 0.615\ L / r$	Partially restrained against rotation at both ends
Unrestrained Conditions	Single bolt connections	
Partially Restrained Conditions	Multiple bolt or welded connections to a stiffer member or group of members (refer to 4.5.2)	

Notes:

1. Formula 2 applies to single angles in cross bracing patterns when $L / r < 120$ and the angles are connected back-to-back at the crossover point.
2. Different equations may apply for each direction of buckling under consideration. The maximum effective slenderness ratio shall be used to determine the compressive strength.

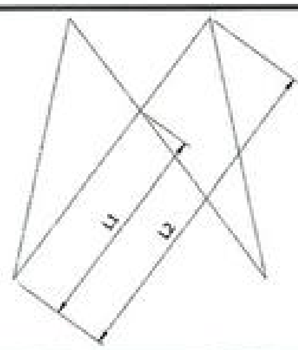
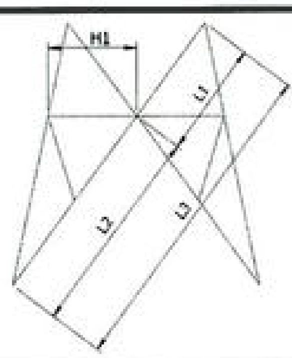
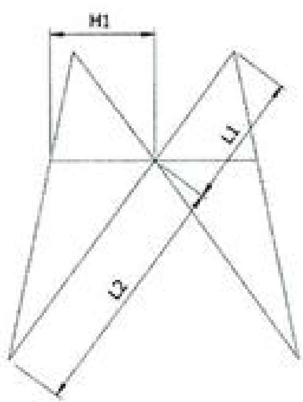
Table 4-5: Effective Length Factors for Round Bracing Members Directly Welded to Legs

Bracing Pattern	Slenderness Ratio of Brace Member		
	$L / r < 80$	$80 \leq L / r \leq 120$	$L / r > 120$
	K = 1.0	K = 0.70 + 0.30 (120 – L / r) / 40	K = 0.70
	K = 1.1	K = 0.70 + 0.40 (120 – L / r) / 40	K = 0.70
 (See Note 2)	K = 1.0	K = 0.75 + 0.25 (120 – L / r) / 40	K = 0.75
 (See Note 2)	K = 1.1	K = 0.90 + 0.20 (120 – L / r) / 40	K = 0.90

Notes:

1. L shall be determined using the panel spacing and the clear distance between the legs.
2. For loading combinations that result in a compression force in both diagonals of a double bracing pattern, the values of KL shall be based on a single bracing condition.

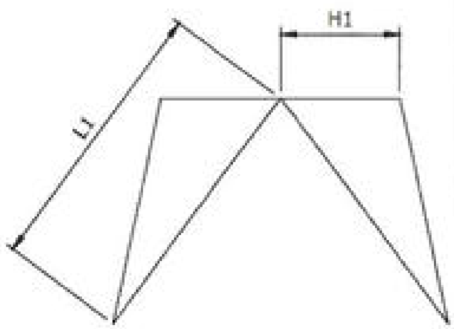
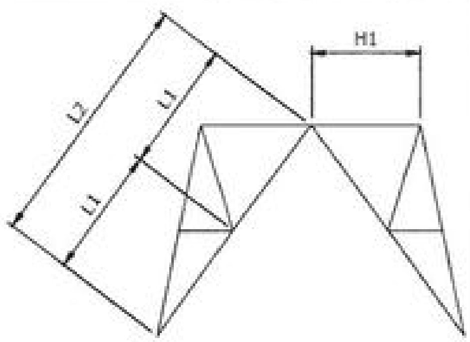
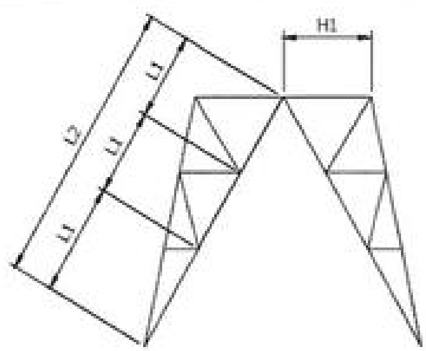
Table 4-6: Buckling Length Considerations for Cross Bracing

	**Crossover Point Providing Support**	**Crossover Point Connected but No Support Provided**
	$L1/r_{min}$	$L1/r_{min}$ $L2/r_{out}$
	**Crossover Point Providing Support**	**Crossover Point Connected but No Support Provided**
	$L1/r_{min}$ $L2/r_{out}$ $H1/r_{min}$	$L1/r_{min}$ $L3/r_{out}$ $H1/r_{min}$ & $2H1/r_{out}$
	**Diagonal Members Not Continuous[5]**	
	Plan Bracing at Crossover Point	**No Plan Bracing at Crossover Point**
	$L1/r_{min}$ $L2/r_{min}$ $H1/r_{min}$	$L1/r_{min}$ $L2/r_{min}$ $H1/r_{min}$ & $2H1/r_{out}$

Notes:

1. L shall be determined using the length between the centers of connecting bolt or weld patterns.
2. "r_{min}" refers to the minimum radius of gyration for a member (i.e. the z-z axis for a single angle member, r for a round member).
3. "r_{out}" refers to the radius of gyration associated with out-of-plane buckling. Refer to 4.5.2.1 for criteria to determine support at the crossover point.
4. Plan bracing must be triangulated and meet the strength requirement of 4.4.1.
5. Horizontals must be continuous and meet the strength requirement of 4.5.2.1 when diagonals are not continuous through the crossover point without triangulated horizontal plan bracing.

Table 4-7: Buckling Length Considerations for K-Type or Portal Bracing

	Horizontal Member Continuous	
	Plan Bracing Support at Apex[5]	No Plan Bracing
	$L1/r_{min}$ $H1/r_{min}$	$L1/r_{min}$ $H1/r_{min}$ & $1.5\ H1/r_{out}$ (see note 4)
	Horizontal Member Continuous	
	Plan Bracing Support at Apex[5]	**No Plan Bracing**
	$L1/r_{min}$ $L2/r_{out}$ $H1/r_{min}$	$L1/r_{min}$ $L2/r_{out}$ $H1/r_{min}$ & $1.5\ H1/r_{out}$ (see note 4)
	Horizontal Member Not Continuous Plan Bracing Required at Apex[5]	
	Hip Bracing Provided[5]	**Hip Bracing Not Provided**
	$L1/r_{min}$ $H1/r_{min}$	$L1/r_{min}$ $L2/r_{out}$ $H1/r_{min}$

Notes:

1. L shall be determined using the length between the centers of connecting bolt or weld patterns.
2. "r_{min}" refers to the minimum radius of gyration for a member (i.e. the z-z axis for a single angle member, r for a round member).
3. "r_{out}" refers to the radius of gyration associated with out-of-plane buckling.
4. $2.0H1/r_{out}$ shall be considered to determine strength for providing support to a leg as defined in 4.4.1.
5. Plan and Hip bracing must be triangulated (refer to Figure 4-2) and meet the strength requirement of 4.4.1.

Table 4-8: Effective Yield Stress for Polygonal Tubular Pole Members

Shape	(w/t) Ratios	Effective Yield Stress
18-Sided	$(F_Y/E)^{1/2}(w/t) < 0.759$	$F'_Y = 1.27\ F_Y$
	$0.759 \leq (F_Y/E)^{1/2}(w/t) \leq 2.14$	$F'_Y = 1.560\ F_Y[1.0 - 0.245\ (F_Y/E)^{1/2}(w/t)]$
16-Sided	$(F_Y/E)^{1/2}(w/t) < 0.836$	$F'_Y = 1.27\ F_Y$
	$0.836 \leq (F_Y/E)^{1/2}(w/t) \leq 2.14$	$F'_Y = 1.578\ F_Y[1.0 - 0.233\ (F_Y/E)^{1/2}(w/t)]$
12-Sided	$(F_Y/E)^{1/2}(w/t) < 0.992$	$F'_Y = 1.26\ F_Y$
	$0.992 \leq (F_Y/E)^{1/2}(w/t) \leq 2.14$	$F'_Y = 1.611\ F_Y[1.0 - 0.220\ (F_Y/E)^{1/2}(w/t)]$
8-Sided	$(F_Y/E)^{1/2}(w/t) < 1.10$	$F'_Y = 1.24\ F_Y$
	$1.10 \leq (F_Y/E)^{1/2}(w/t) \leq 2.14$	$F'_Y = 1.578\ F_Y[1.0 - 0.194\ (F_Y/E)^{1/2}(w/t)]$

where:

F_Y = specified minimum steel yield strength

t = wall thickness

w = flat side dimension calculated using an inside bend radius equal to 1.5t unless the inside bend radius is known but in no case greater than 4t.

E = modulus of elasticity

Notes:

1. For polygonal members, w/t shall not exceed $2.14\ (E/F_Y)^{1/2}$
2. Polygonal members with more than 18 sides, shall be considered as round members for strength investigation purposes using a diameter equal to distance across flats.

Table 4-9: Torsional Constant, C_t, for Tubular Pole Members

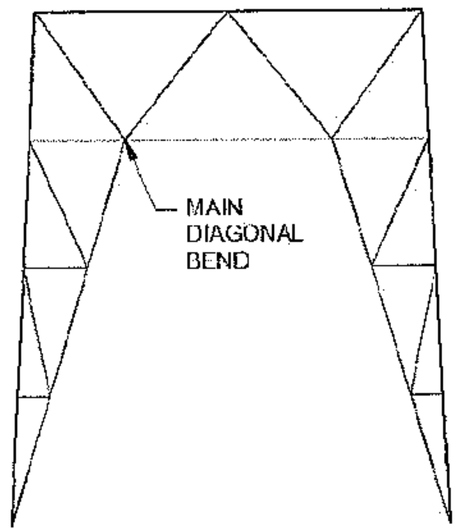
$C_t = m(t)(D - t)^2$	
Shape	**m**
Round	1.57
18-Sided	1.58
16-Sided	1.59
12-Sided	1.61
8-Sided	1.66

where:

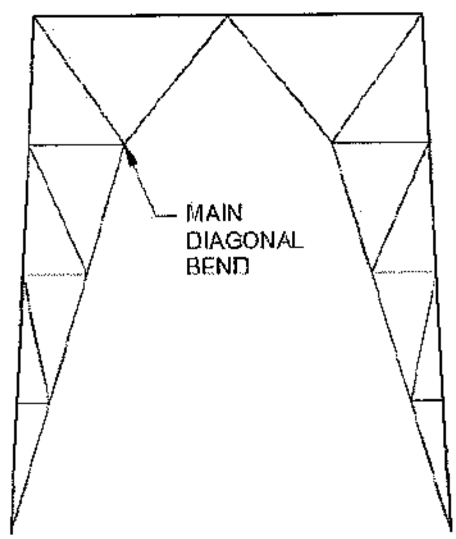
t = wall thickness

D = outside diameter for round poles or outside flat-to-flat width for polygonal poles

Figure 4-1: Cranked K-Type or Portal Bracing



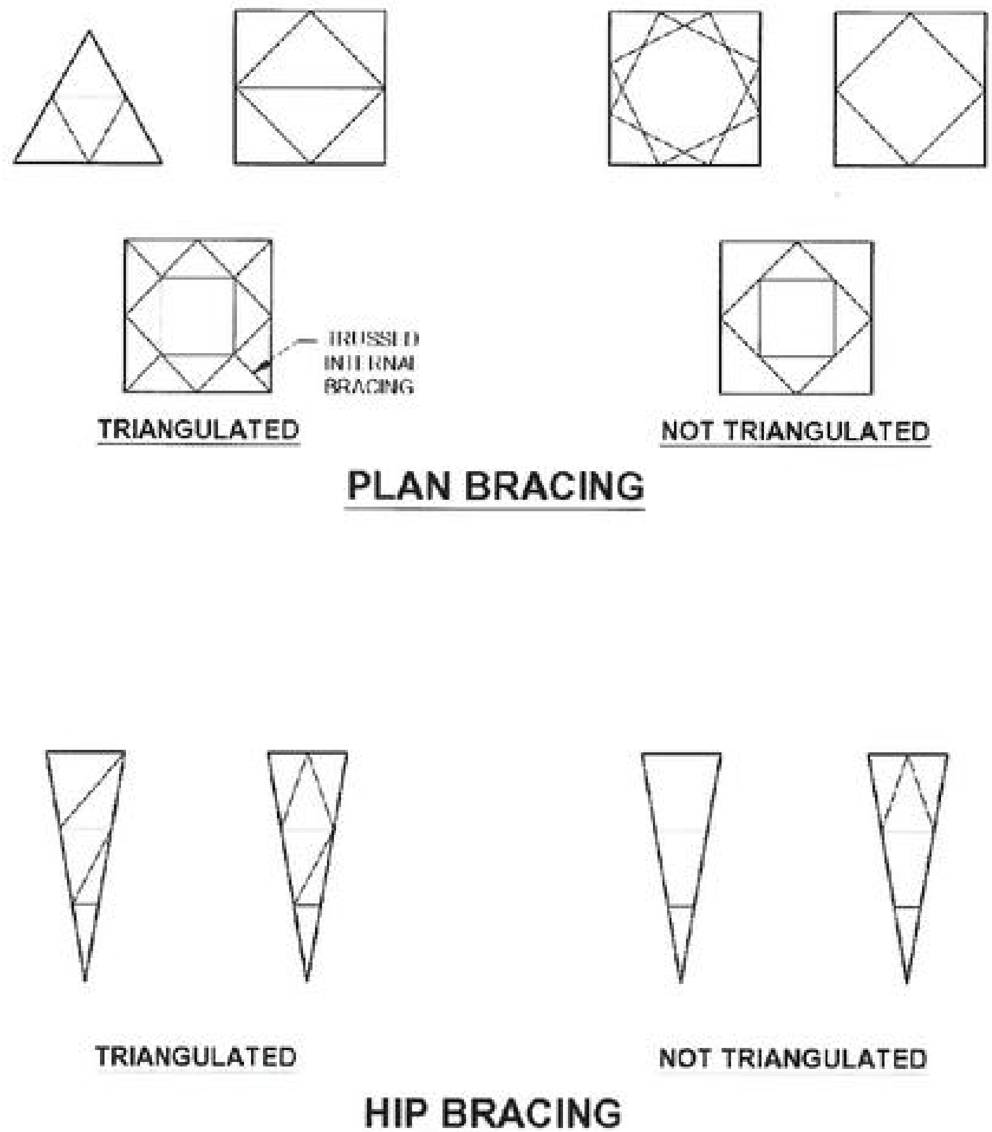
PORTAL BRACE



CRANKED K-BRACE

Note: Triangulated hip bracing (refer to Figure 4-2) required at the main diagonal bend.

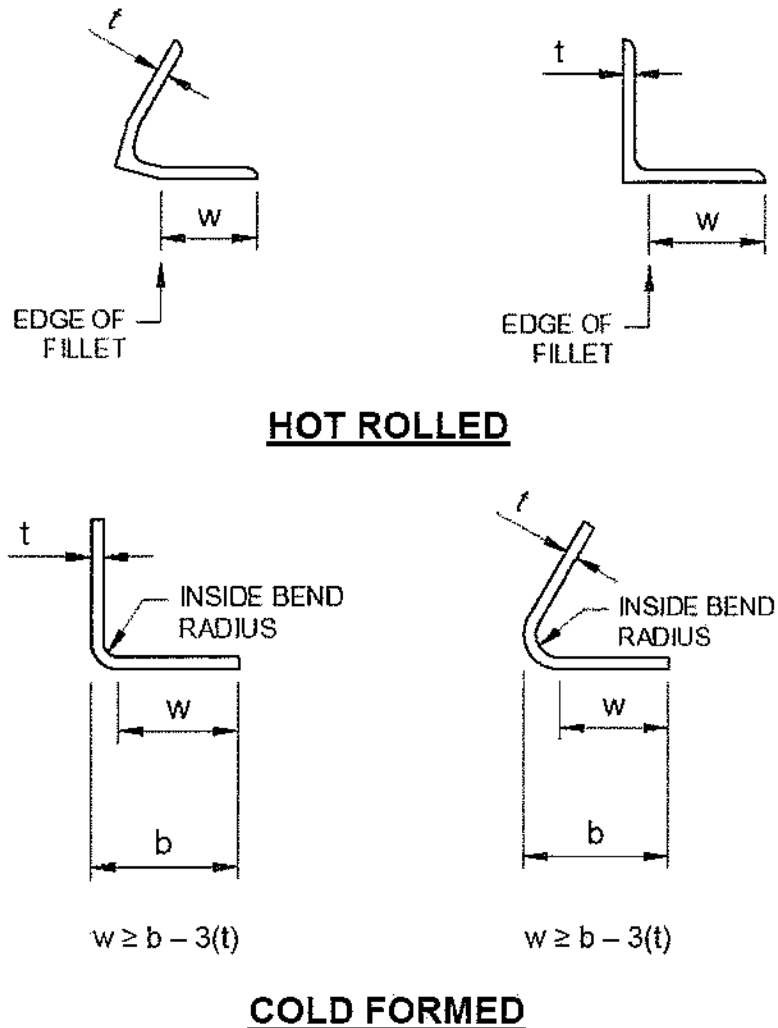
Figure 4-2: Plan and Hip Bracing



Notes:

1. Plan or hip bracing must be triangulated and meet the requirements of 4.4.1 to provide lateral support to a member.
2. Effective slenderness ratios for main diagonals with staggered hip/face bracing shall be determined in accordance with Table 4-3 using the equations provided for leg members with staggered bracing patterns.
3. The minimum required resistance of hip bracing shall be determined in accordance with 4.4.1 with $P_r = 1.15\ P_s$ where P_s is based on the governing effective slenderness ratio of the main diagonal.

Figure 4-3: Width-to-Thickness Ratios (w/t) for Angle Members



Note: w/t shall not exceed 25.

5.0 MANUFACTURING

5.1 Scope

This Section outlines the structural steel material requirements, fabrication tolerances, and corrosion control applicable for the structures designed and manufactured to this Standard.

5.2 Definitions

Structural steel: steel material used for structural members and components excluding guys, fasteners and hardware.

5.3 Symbols and Notations

CVN = Charpy V-notch value;

F_y = minimum specified yield stress;

pH = hydrogen ion concentration of soil (acidity/alkalinity index);

t = thickness (diameter) of material;

c = coefficient for CVN.

5.4 Material, Structural Steel

5.4.1 General

Structural steels used for structures designed to this Standard shall conform to one of the pre-qualified steel material standards listed in Table 5-1. Other steel materials suitable for the application and site and conforming to 5.4.2 shall be considered acceptable.

Structural steel for polygonal tubular poles and butt-welded flange plates for polygonal tubular poles shall have longitudinal Charpy V-notch values not less than 15.0 ft-lbs [20 J] at -20 degrees F [-29 degrees C].

The tensile strength for material used for polygonal tubular poles intended to be hot-dip galvanized shall not exceed 100 ksi (690 MPa).

All Charpy V-notch (CVN) tests shall be in accordance with American Society for Testing and Materials ASTM A370.

5.4.2 Non Pre-Qualified Steel

The Carbon Equivalent for structural steel shall not exceed 0.65 for non-welded applications and 0.50 for welded applications, as calculated by the following formula:

$$C + \frac{Mn + Si}{6} + \frac{Cr + Mo + V}{5} + \frac{Ni + Cu}{15}$$

Note: Special welding procedures, including but not limited to minimum pre-heat temperatures, may be required in accordance with AWS D1.1-15, "Structural Welding Code-Steel".

The elongation shall not be less than 14%.

For solid round shapes, the design values for the yield strength and ultimate tensile strength shall be based on a longitudinal sample located at the half-radius point.

For members and components with thickness greater than 5 in. [127 mm] and a specified minimum yield stress of 50 ksi [350 MPa] or greater, the required Charpy V-Notch (CVN) value shall not be less than 15 ft-lbs [20 J] at 0 degrees Fahrenheit [-17 degrees C]. Alternatively, the CVN at the lowest monthly mean temperature for the site shall not be less than the following:

$$CVN = \frac{F_y\ t}{5.54\ c}\ \text{ft-lbs} \quad \text{or} \quad CVN = \frac{F_y\ t}{710\ c}\ \text{Joules}$$

where:

F_y = the minimum specified yield stress of the type of steel being used, ksi [MPa]

t = the thickness (diameter) of the material, in. [mm]

c = 2 for drilled, reamed holes and non-welded components and for all members subjected to a design tensile stress less than 15 ksi [100 MPa]

= 1 for punched and welded components subjected to a design tensile stress greater than or equal to 15 ksi [100 MPa]

For solid round shapes, the CVN values shall be based on a longitudinal sample located at 1 in. [25 mm] below the surface.

5.4.3 Test Reports

Certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6 or A568, as applicable, shall constitute sufficient evidence of conformity to the requirements of 5.4.1 and 5.4.2.

5.4.4 Tolerances

Acceptable dimensional tolerances shall be determined from ASTM A6.

5.5 Fabrication

Fabrication shall be in accordance with AISC 360 section M2.

Fabrication of Risk Category II, III & IV self-supporting latticed structures, guyed masts and tubular pole structures shall be performed by a fabricator certified in accordance with the American Institute of Steel Construction meeting the requirements of AISC 201-06, "Standard for Steel Building Structures - 2006".

Unless otherwise specified, structural members shall be fabricated to the tolerances given in ASTM A6 for the type of material being utilized. Completed members shall be free from twists and bends.

Fabricated compression members shall not deviate from straightness by more than one in five hundred (1/500), but not more stringent than 1/16 in. [1.5 mm], of the length between points that are to be laterally supported.

Splices in compression members that are designed for direct bearing shall have at least 75% of the nominal area in contact.

Welding shall be in accordance with AWS D1.1.

For polygonal tubular poles with butt-welded flange plates, the weld metal and the heat affected zone area shall exhibit longitudinal Charpy V-notch values not less than 15.0 ft-lbs [20 J] at -20 degrees F [-29 degrees C] as determined in the qualifying welding procedure specification prepared in accordance with AWS D1.1 for the butt-welded flange connections.

Tubular pole complete penetration base and flange plate welds and complete penetration splices of tubular pole sections shall be 100% non-destructive tested after hot-dip galvanizing.

5.6 Corrosion Control

5.6.1 General

All structural steel members and components shall have a zinc coating. Hot-dip galvanizing is the preferred process. Other methods that provide equivalent corrosion control are acceptable.

5.6.2 Structural Steel

Structural steel members shall be hot-dip galvanized in accordance with ASTM Standard A123. Tubular steel poles shall be considered as a shape/plate (Class 100). Alternative methods shall provide minimum corrosion control equivalent to ASTM Standard A123.

5.6.3 Fasteners and Hardware

Fasteners and hardware shall be galvanized in accordance with one of the following: ASTM Standard A153 (hot-dip zinc coating), ASTM Standard F2329 (hot-dip zinc coating), ASTM Standard B695 Class 55 (mechanically deposited zinc coating), ASTM F1136 (zinc/aluminum dispersion) Grade 3 (bolts and washers) and Grade 5 (nuts). Alternative methods shall provide minimum corrosion control equivalent to ASTM Standards A153 and F2329.

5.6.4 Repair

Repairs shall be in accordance with ASTM Standard A780 or as required by the providers of alternate corrosion control processes.

5.6.5 Guy Strand

Zinc coated guy strand shall be galvanized in accordance with ASTM A475 or ASTM A586 as applicable. A minimum Class A coating shall be supplied.

5.6.6 Guy Anchorages

Steel guy anchorages in direct contact with the soil shall as a minimum, have corrosion control in accordance with 5.6.2. When the measured soil electrical resistivity is less than 50 ohm-m and/or the measured soil pH values are below 3 or greater than 9, additional corrosion control shall be required for Risk Categories II, III and IV structures. Refer to Annex H for acceptable corrosion control methods.

5.6.7 Ground Embedded Poles

Steel poles in direct contact with the soil shall as a minimum, have corrosion control in accordance with 9.4.3.2.

Table 5-1: Pre-Qualified Structural Steels

API 5L	API Specification (Spec. 5L)
ASTM A36	Structural Steel
ASTM A53	Pipe, Steel, Black and Hot-Dipped, Zinc-Coated Welded and Seamless
ASTM A106	Seamless Carbon Steel Pipe
ASTM A500	Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Round and Shapes
ASTM A501	Hot-Formed Welded and Seamless Carbon Steel Structural Tubing
ASTM A514	High-Yield Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding
ASTM A529	High-Strength Carbon-Manganese Steel of Structural Quality
ASTM A572	High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality
ASTM A618	Hot-Formed Welded and Seamless High-strength Low-Alloy Structural Tubing
ASTM A633	Normalized High-Strength Low-Alloy Structural Steel Plates
ASTM A913	High-Strength Low-Alloy Steel Shapes of Structural Quality
ASTM A992	Steel for Structural Shapes for Use in Building Framing
ASTM A1011	Steel, Sheet and Strip, Carbon, Hot-rolled Structural Quality
ASTM A1018	Steel, Sheet and Strip, Carbon, Heavy-Thickness, Hot-rolled Structural Quality
ASTM A1085	Cold-Formed Welded Carbon Steel Hollow Structural Sections

6.0 OTHER STRUCTURAL MATERIALS

6.1 Scope

This Section provides criteria for the design of structures addressed in the scope of this Standard using materials other than steel.

6.2 General

Other structural materials may be used for the supply of structures in accordance with the requirements of this Standard. Conventional materials such as: concrete, aluminum and wood, shall conform to current limit states design standards for these materials. For other materials, which do not have established limit states design standards, factored resistances shall be established to ensure that the level of reliability implied by this Standard is achieved.

6.3 Loads

Strength and serviceability limit states load combinations shall be in accordance with Section 2.0.

6.4 Analysis

Analysis models and procedures shall be in accordance with Section 3.0.

6.5 Design Strength

Appropriate resistance factors shall be established such that the level of reliability implied by this Standard is achieved.

7.0 GUY ASSEMBLIES

7.1 Scope

This Section provides the minimum requirements for the design and supply of the guy cables, end fittings, and components used in guy assemblies for guyed masts supplied in accordance with this Standard. This Section does not apply to safety climb system cable assemblies.

7.2 Definitions

Cable: a flexible tension member consisting of strand or rope.

Damper: a device, attached to the cable that modifies the structural response to dynamic loads.

Fitting: any accessory used as an attachment to, or support for, the cable or its component.

Guy assemblies: a cable used to support a structure including accessories used as an attachment for the ends and for adjusting the tension in the assembly, insulators and nonmetallic material where applicable.

Manufacturer's rated breaking strength: ultimate guaranteed tensile breaking strength of a cable also known as minimum breaking force.

Pre-stretching: the removal of inherent constructional stretch of a cable under a sustained tensile load.

Proof loading: the assurance by load test of the mechanical strength of factory assembled end connections.

Rope: a plurality of strands twisted about an axis, or about a core, which may be a strand or another wire rope.

Strand: a plurality of wires either parallel or helically twisted about an axis, usually about a central wire.

Wire: a single continuous length of steel with a circular cross-section cold-drawn from rod.

7.3 Cables

7.3.1 Guy Strand

Galvanized steel guy strand shall conform to the minimum requirements of ASTM Standard A475 Extra High Strength (EHS) or equivalent recognized standard. Aluminum clad steel guy strand shall conform to ASTM B416 or equivalent recognized standard.

7.3.2 Structural Strand

Structural Strand shall conform to the minimum requirements of ASTM Standard A586 or equivalent recognized standard.

7.3.3 Wire Rope

Wire rope shall not be used for guy assemblies unless higher flexibility is required for special applications. When utilized, wire rope shall conform to ASTM A603 or equivalent recognized standard. Aluminum clad steel guy wire shall conform to ASTM B415 or equivalent recognized standard.

7.4 End Attachments

7.4.1 Thimbles

An adequate bend radius of a thimble at a cable termination attachment shall have a bend radius per the end termination manufacturer's recommendation and adequate strength to maintain the bend radius under the design load of the assembly.

7.4.2 Formed Guy Grips

Formed guy grips shall be designed specifically for the length, size and type of cable being used. This shall include the size, number, and lay of the wires and the electrochemical compatibility of the material. The guy-grip manufacturer shall perform tests to demonstrate the capacity and efficiency of the product for different size guys.

Formed guy grips shall not be reused after being in service and removed.

7.4.3 Clips

U-bolt or twin base clips that are used to secure looped ends shall match the size of the cable within a tolerance of 1/16 in. [1.6 mm].

7.4.4 Sockets

Sockets shall be of the open or closed type. They shall be manufactured in accordance with ASTM Standards A27 and A148.

Sockets manufactured for use only with wire rope shall not be used for strand, nor shall sockets made for galvanized strand be used for aluminum-coated strand.

Sockets for use with other types of cables may be made from other materials providing they conform to recognized standards and demonstrate the same performance characteristics as implied by this Standard.

7.4.4.1 Zinc-Poured Attachments

Zinc for zinc-poured attachments shall conform to a prime western, high grade or higher purity zinc as defined in ASTM B6.

7.4.4.2 Resin-Poured Attachments

Resin-poured attachments shall be acceptable when installed per the resin manufacturer's recommendations.

7.4.5 Shackles

Shackles used to connect guy assemblies shall be forged from AISI grade 1030, 1035 or 1045 steel or equivalent, and suitably heat-treated (quenched and tempered, normalized or annealed).

7.4.6 Take-up Devices

A take-up device shall be supplied at the anchor end of the guy assembly for adjusting the guy tension.

7.4.6.1 Turnbuckles

Turnbuckles used to connect guy assemblies, shall be forged from AISI grade 1030, 1035 or 1045 steel or equivalent, and suitably heat-treated (quenched and tempered, normalized or annealed).

7.4.6.2 Bridge Sockets

Take-up devices used in conjunction with bridge sockets or similar devices shall be suitably heat-treated (normalized or annealed).

7.5 Guy Dampers

High frequency low amplitude (Aeolian) and low frequency high amplitude (galloping) vibrations are difficult to predict prior to the installation of a structure. Dampers can be retrofitted when necessary. For guyed masts with structure heights above 1,200 ft. [366 m], high frequency dampers shall be provided for cables with rigid end connections such as bridge sockets or similar devices unless otherwise determined by a site-specific analysis.

The size, number and position of dampers shall be in accordance with the recommendations of the damper manufacturer.

7.6 Design

7.6.1 Initial Tension

The initial tension in guys, for design purposes, at an ambient temperature of 60 degrees F [16 degrees C] shall be within upper and lower limits of 15% and 7%, respectively, of the manufacturer's rated breaking strength of the strand. Values of initial tension beyond these limits may be used provided consideration is given to the sensitivity of the structure to variations in initial tension.

The design ambient temperature may be adjusted based on site-specific data.

Notes:

1. Initial tension is defined as the guy tension at an anchor point corresponding to the unfactored dead load condition at the design ambient temperature.
2. When using initial tension values above 15%, consideration shall be given to the possible effects of Aeolian vibration. Likewise, when using initial tension values less than 7%, consideration shall be given to the effects of galloping and slack-taut pounding.

7.6.2 Design Strength

The design strength of guy assemblies shall be taken as $\phi_g \phi_e T_g$:

where:

ϕ_g = 0.6 for metallic guy cables

= 0.5 for non-metallic cables

ϕ_e = end fitting strength efficiency factor from 7.6.2.2

T_g = the ultimate breaking strength of the guy assembly from 7.6.2.1

7.6.2.1 Ultimate Breaking Strength

The ultimate breaking strength of a guy assembly shall be the lesser of: (a) the manufacturer's rated breaking strength of the guy times the end fitting efficiency factor from 7.6.2.2, or (b) the rated breaking strength of the end fittings or take-up device.

7.6.2.2 End Fittings Strength Efficiency Factor

An end fittings strength efficiency factor shall be applied to the ultimate breaking strength of a guy assembly to account for the possible reduction in strength due to slippage or strand deformation. The end fitting efficiency factor shall be in accordance with the end fitting manufacturer's recommendation. A steel strand cable shall not be utilized as a wrap-around end termination.

7.6.3 Modulus of Elasticity

In the absence of specific cable manufacturer's data, the modulus of elasticity of a steel cable used for analysis shall be 23,000 ksi [159 GPa] except for pre-stretched cables 2-9/16 in. [65 mm] diameter and smaller, a modulus of elasticity of 24,000 ksi [166 GPa] shall be used.

7.6.4 Articulation

Articulation at both ends of guy assemblies shall be provided for assemblies consisting of non-metallic guys with rigid end connections such as end sockets or similar devices that do not include low frequency dampers. Articulation shall provide a minimum 10 degrees rotation in both the vertical and the horizontal directions.

7.7 Manufacture

Manufacturers of non-metallic components of guy assemblies shall provide the expected life of the component.

7.7.1 Proof Loading of Assemblies

Factory installed end sockets shall be proof loaded to 55% of the manufacturer's rated breaking strength of the cable and held for a minimum of three cycles with a minimum duration of five minutes for each cycle.

7.7.2 Pre-stretching

Pre-stretching shall be required for guy assemblies (excluding insulators) with factory installed end fittings at both ends. The pre-stretching force for a cable shall be equal to 45% of the manufacturer's rated breaking strength of the cable.

7.7.3 Length Measurements

Length measurements for guy assemblies with factory installed end fittings at both ends shall be made under the design initial tension of the cable. Measurements shall be taken after pre-stretching. Design initial tension and temperature shall be specified.

7.7.4 Striping

When pre-stretching is specified, a longitudinal paint stripe shall be applied to the cable while it is subjected to the tension specified for length measurements.

7.8 Installation

Cable or other devices shall be installed on turnbuckles to prevent disengagement under wind loading.

Striped cables shall be erected such that the paint stripe applied during measuring is straight after erection.

Initial tensions shall be measured by either direct or indirect methods (refer to Annex K).

8.0 INSULATORS

8.1 Scope

This Section provides the minimum requirements for the design of base and guy insulators for structures supplied in accordance with this Standard.

8.2 Design

For the design of base insulators, tension and compression forces, horizontal shear and moments shall be taken into account.

Where steel end fittings are used, they shall be forged from AISI grade 1030, 1035 or 1045 steel or equivalent or cast from steel according to the requirements of ASTM Standards A27 or A148, and suitably heat-treated (quenched and tempered, normalized or annealed).

The design strength of base and guy insulators shall be taken as $\phi_i R_i$:

where:

ϕ_i = 0.5 for non-metallic fail-safe insulators (i.e. insulators capable of transferring load after a failure of the non-metallic portion of the insulator)

= 0.4 for other non-metallic insulators

R_i = the ultimate strength of the insulator

8.3 Manufacture

Insulator assemblies shall be proof loaded to 45% of the manufacturer's rated ultimate strength.

Insulator manufacturers shall provide the expected life of base and guy insulators.

Note: For strain insulators, the manufacturer shall define shipping, handling, and inspection procedures to insure the integrity of the product.

9.0 FOUNDATIONS AND ANCHORAGES

9.1 Scope

This Section defines criteria for foundations and anchorages for structures designed in accordance with this Standard.

Supplemental requirements for high seismic locations are defined in section 9.8.

Note: The supplementary requirements for high seismic locations provide for the desired ductility and energy absorbing properties assumed for the earthquake loading design criteria of this Standard based on the unique characteristics and responses of structures addresses in the scope of this Standard.

9.2 Definitions

Anchorage: an embedment developed into a foundation, soil or rock for supporting a structure.

Corrosive soil: soil identified in a geotechnical report as being corrosive to concrete or steel.

Direct embed foundation: a pole or precast concrete pole section extended into soil or rock acting as a foundation.

Foundation: a substructure or extension of the superstructure designed to transmit reactions to soil or rock.

Headed anchor rod: a deformed reinforcing bar or a smooth round bar with an attached end plate or nut(s).

High seismic location: a location where the spectral response acceleration parameter at short periods from Section 2.0 exceeds 1.0.

Rock: a natural aggregate of mineral grains connected by strong and permanent cohesive forces.

Soil: a natural aggregate of mineral grains, with or without organic constituents that can be separated by gentle mechanical means, such as agitation in water.

9.3 Geotechnical Investigation

A site-specific geotechnical investigation shall be required for Risk Category III or IV structures, and is preferred for Risk Category I and II structures. Recommendations for information to be reported in site-specific geotechnical investigations are contained in Annex G.

Reduction in the weight of material due to buoyancy and the effect on soil properties shall be considered when submerged conditions are a design consideration. Direct embed tubular pole foundations shall be designed to prevent upheaving based on the design water table elevation.

The minimum foundation base depth shall be at or below the frost depth provided in Annex B, the frost depth based on regional climatic data and knowledge of local conditions or the frost depth recommended in a geotechnical report for foundations supported on soil which may display significant ice lens development during freezing.

Foundations shall extend below the depth of seasonal moisture variation in expansive soils as indicated in a geotechnical report. The depth of the foundation shall be adequate to prevent uplift of the foundation. The tension capacity of the foundation must be adequate to resist the net uplift tension forces from the expansive soil as specified in the geotechnical report. Net uplift tension forces from expansive soil shall be based on a load factor of 1.2 applied to the expansive soil tension force and a load factor of 0.9 applied to dead loads. The tension forces from expansive soil need not be considered to occur simultaneously with the loading combinations defined in 2.3.2 specified in Section 2.0.

In the absence of a site-specific geotechnical report for Risk Category I and II structures, presumptive soil parameters are provided in Annex F. Presumptive soil parameters and other assumptions required for foundation design shall be validated for a specific site prior to installation.

9.4 Foundation Design

The design strength of foundations, anchorages and supporting soil or rock shall equal or exceed the factored reactions of a structure for all of the loading combinations defined in 2.3.2.

The weight of the foundation and the weight of soil or other material directly above a foundation shall be considered as dead load when determining foundation, soil or rock strengths requirements. A 1.2 load factor shall be applied when dead loads act in combination with factored reactions to increase soil or rock strength requirements and a 0.9 dead load factor shall be applied when dead loads act in combination with factored reactions to decrease soil or rock strength requirements. (Refer to Note 2 from 2.3.2.) The weight of soil or other material outside the perimeter of a foundation if considered to resist overturning or uplift reactions shall be considered as a nominal soil strength and shall be multiplied by a 0.75 resistance factor.

Foundation and anchorage displacements need not be considered for the strength and serviceability limit states analysis of structures, except for structures solely supported by a single caisson foundation or for other foundation types for site-specific conditions identified as having critical displacement sensitive soils where displacements may only be ignored when the lateral displacement at grade level is less than or equal to 0.75 in. [20 mm] for the serviceability limit state condition specified in section 2.8.

Concrete mixtures shall meet the durability requirements of Chapter 19 of the American Concrete Institute Standard, ACI 318-14, "Building Code Requirements for Structural Concrete" (ACI 318).

9.4.1 Design Strength

Unless otherwise specified in this Standard, the design strength of concrete (e.g. spread footings, piers (pedestals), caissons, drilled shafts, deadman guy anchors) and steel foundations (e.g. grillages, H-piles, pipe-piles) shall be in accordance with ACI 318 and AISC 360, respectively.

The distribution of soil bearing stress on the base of a spread foundation subjected to a factored overturning moment may be considered as either triangular or square. The maximum eccentricity of loading (i.e. gross overturning moment divided by gross axial load) shall not exceed 45% of the foundation base dimension in the direction of loading. Overturning shall be considered to occur from both parallel and diagonal axes of the foundation. For latticed structures, slab moments and shears shall be determined both interior and exterior to the footprint of the structure.

Note: It shall be permissible to consider higher eccentricities of loading when a settlement analysis is performed and the associated P-Delta effects are considered for the strength and serviceability limit-states analysis of the structure.

For structures supported solely by a single caisson or drilled shaft foundation, an analysis based on rigid analysis methods shall be limited to aspect ratios (depth/diameter) not exceeding 6. Flexible analysis methods shall be used for aspect ratios greater than 6.

The minimum longitudinal reinforcement ratio for piers, caissons and drilled shafts shall be 0.005. The longitudinal reinforcement in piers supported on spread foundations or supporting spread foundations shall be fully developed in tension into the foundation with or without hooks. In addition, when compression in longitudinal bars are included in the nominal strength determination for piers, the longitudinal reinforcement shall also be developed in compression for the maximum longitudinal reinforcement compressive force considered for the determination of pier nominal strength. Hooks shall be ignored in the development of longitudinal reinforcement for compression. The embedment depth for a hooked bar in compression shall be considered equal to the depth to the outer surface of the hook.

Closed stirrups used in cast-in-place foundations enclosing anchor rods or used as transverse reinforcement enclosing longitudinal reinforcement in piers, caissons and drilled shafts providing lateral support of longitudinal bars under compression or provided for shear strength shall have staggered lap lengths equal to 1.3 times the development length based on the nominal stirrup diameter or with staggered hooks in accordance with ACI 318 Chapter 25.

9.4.2 Transfer of Pier Forces

Factored overturning moments in piers supported on spread foundations or supporting pier caps, shall be transferred to the spread foundation by a combination of flexure and eccentricity of shear (punching shear).

For a round or square pier, 60% of the factored overturning moment shall be considered to be transferred by flexure and 40% transferred by punching shear. For rectangular piers, the transfer by flexure shall be determined in accordance with ACI 318 Equation 8.4.2.3.2 with the remaining overturning moment transferred by punching shear.

The effective slab width for resisting overturning moment transferred by flexure shall not exceed the width of the pier plus 1.5 times the thickness of the slab on either side of the pier. The factored shear stress resulting from the overturning moment transferred by punching shear shall be considered to vary linearly about the centroid of the critical section combined with the factored shear stress from axial loads transferred from the pier.

For latticed structures, punching shear strength shall be investigated as interior, edge and corner conditions in accordance with ACI 318 Chapter 22 with reduced strengths due to edge conditions.

For downward pier loads, punching shear shall be based on the dimensions of the pier. For uplift loads, punching shear shall be based on the dimensions of the pier reinforcement extended into the foundation or the dimensions of the anchor bolt arrangement in slabs without pier reinforcement.

9.4.3 Direct Embed Foundations

The strength of a pole or precast foundation below grade shall be investigated for the forces and moments in the embedded sections based on the soil strength distribution along the length of the embedment required to resist the factored reactions from all of the loading combinations defined in 2.3.2.

The strength of all backfill materials, including concrete, specialty backfill materials, soil, gravel, etc. shall be ignored for the strength investigation of embedded sections.

Precast foundations with a slip joint connection with the pole shall be designed for the shear forces from the internal couple resisting the factored shear and overturning moment reactions from all of the loading combinations defined in 2.3.2 and additionally in high seismic locations, the expected moment capacity of the pole per 6.14.1. The slip length shall be designed to prevent a crushing failure at the surface of the concrete at the top of the foundation and at the termination of the pole section below the top of the foundation. Longitudinal reinforcement shall be fully developed in tension within the design minimum slip length. The default minimum slip length specified in 4.9.7.1 shall not apply to the slip length for precast foundations.

9.4.3.1 Effective Foundation Diameter

For concrete backfill, it shall be permissible to consider a constant effective foundation diameter over the embedment depth equal to the outer diameter of the concrete annulus surrounding the pole.

For gravel backfill, it shall be permissible to consider a constant effective foundation diameter over the embedment depth equal to the average of the mid-depth diameter of the pole and the outer diameter of the gravel annulus, not to exceed the pole base diameter plus 9 in. [229 mm].

For soil or other backfill materials, it shall be permissible to consider a constant effective foundation diameter over the embedment depth equal to the mid-depth diameter of the pole.

9.4.3.2 Corrosion Control

A protective coating shall be provided around the perimeter of direct embed foundations in accordance with this Section.

Unless otherwise specified, the protective coating shall extend 12 in. [305 mm] minimum above grade. Protective coatings shall have a feathered edge at the termination of the coating.

A surface coating providing UV protection shall be applied over the coated area extending above grade under either of the following conditions:

1. The protective coating is not intended for exposure to UV (the physical properties of the coating required for corrosion protection significantly degrade under UV exposure).

2. When the color of the protective coating is selected for landscaping or other aesthetic purposes and the base protective coating color is not intended to be stable under exposure to UV.

9.4.3.2.1 Direct Embed Steel Sections

A protective coating shall be provided for direct embed steel sections under the following conditions:

1. Soil backfill is utilized.
2. Gravel or concrete backfill is utilized with less than 6 in. [150 mm] cover.
3. Gravel backfill is utilized in corrosive soil conditions. (Refer to Annex H).
4. Concrete backfill does not meet the durability requirements of ACI 318 Chapter 19.

Note: When concrete or gravel backfill is not extended to grade and soil is used for the upper portion of the embedment (i.e. for landscaping purposes), a protective coating is required for the embedment depth exposed to soil plus an additional 18 in. [460 mm] of embedment length.

When concrete is extended above grade, temperature steel shall be provided for the portion above grade.

Ground sleeves shall be considered as supplementary corrosion protection and shall not eliminate the requirements of protective coatings.

Note: Ground sleeves shall be continuously welded to the embedded section at the top and bottom of the ground sleeve. Seam welds for ground sleeves shall meet the requirements for the embedded pole section. Provisions shall be provided for venting of the air gap between the pole section and the ground sleeve during galvanizing operations.

It shall be permissible to eliminate protective coatings for direct embed steel sections when cathodic corrosion protection methods are installed utilizing sacrificial anodes or impressed currents.

9.4.3.2.2 Direct Embed Precast Concrete Sections

Precast concrete foundations shall meet the durability requirements of ACI 318 Chapter 19 for high sulfate conditions and other soil and groundwater conditions indicated in a geotechnical report to be corrosive to concrete unless a protective coating is provided over the embedment depth.

9.5 Foundation Installation

The area around all foundations shall be free from vegetation and shall be sloped to drain away from the foundation.

For all concrete foundations and direct embed foundations with concrete backfill, the top of concrete shall be sloped to drain away from the embedded components.

9.6 Anchorages

Anchorages shall meet the strength requirements of Section 4.0.

Anchor rods shall be developed into a foundation to support the anchor rod forces due to the factored reactions from the loading combinations specified in Section 2.0.

Anchor rod forces shall be determined by elastic analysis. Alternatively, plastic analysis shall be permitted when the design strength of a group of anchors is governed by the steel strength of the individual anchor rods.

Deformed reinforcing bars used as anchor rods shall conform to ASTM A615, "Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Placement".

Smooth anchor rods shall conform to one of the following pre-qualified material specifications:

ASTM A354 Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs and Other Externally Threaded Fasteners (Grade BC)

ASTM A449 Standard Specification for Hex Cap Screws, Bolts and Studs, Steel, Heat Treated, 120/105/90 ksi Minimum Tensile Strength, General Use

ASTM F1554 Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength

It shall be permissible to use other steel materials suitable for the application and site.

Anchor rods shall be hot-dip galvanized in accordance with ASTM F2329. The minimum length of galvanizing shall be as follows:

Upper 12 in. [305 mm] for nominal diameters ≤ 1.25 in. [32 mm]

Upper 16 in. [405 mm] for nominal diameters > 1.25 in. [32 mm] but ≤ 1.5 in. [38 mm]

Upper 20 in. [510 mm] for nominal diameters > 1.5 in. [38 mm]

Alternatively, it shall be permitted to galvanize the entire anchor rod length.

Anchor rod material with specified minimum yield strengths greater than 55 ksi [379 MPa] shall have a minimum Charpy V-Notch impact strength in the longitudinal direction of 15 ft-lbs [20 J] absorbed energy at a test temperature of -20 degrees F [-29 degrees C]. Tests shall be in accordance with ASTM A370. Testing frequency shall be in accordance with ASTM A673/A673M. When heat treatment is performed after threading or bending, impact tests shall be performed at Test Frequency P (Piece Testing) on the finished anchor rods. For other anchor rods, impact tests shall be performed at either Test Frequency H (Heat Lot Testing) on the bar stock used for the anchor rods or at Test Frequency P (Piece Testing) on the finished anchor rods.

The center-to–center spacing of anchor rods shall not be less than four times the nominal diameter of the anchor rod for diameters up to 1.5 in. [38 mm] and 6 in. [150 mm] for larger diameters. In addition, anchor rods shall satisfy the minimum cover, spacing and edge distances for reinforcing bars in accordance with ACI 318. (Note: minimum spacing requirements apply to both cast-in-place and post-installed anchorages.)

Embedded plates connecting a group of anchor rods shall have center holes as required to facilitate the placement of concrete.

Anchor rods in piles, piers or caissons shall be enclosed in the top 5 in. [130 mm] of the foundation with a minimum of two #3 [#10M] transverse ties around the longitudinal reinforcement anchored at each end with staggered hooks in accordance with ACI 318 Chapter 25.

Longitudinal foundation reinforcing bars in piers or caissons used to develop anchor rods in tension shall be fully developed in accordance with ACI 318 Chapter 25 on both sides of an assumed potential concrete tension breakout surface originating at the end of a deformed reinforcing bar anchor rod or at the bearing surface of a headed anchor rod and projecting outward and upward from the centerline of the anchor rod at a 35 degree angle with the horizontal. The center-to-center diameter of the longitudinal reinforcing bar arrangement shall not exceed the anchor rod bolt circle by more than the embedment depth of the anchor rods.

For piers, caissons and drilled shafts, horizontal reinforcement for shrinkage and temperature stresses shall be provided within the top 6 in. [150 mm] of the exposed horizontal surface when the distance between the anchor rod bolt circle and the perimeter of the foundation exceeds 30 in. [760 mm]. The total area of reinforcement provided shall be at least 0.125 in.^{2} [80 mm^2] per foot [305 mm] in each direction with a center-to-center spacing not to exceed 9 in. [230 mm].

9.6.1 Development of Anchor Rods

Deformed reinforcing bar anchor rods shall be developed in accordance with 9.6.2 or 9.6.3.

Smooth anchor rods shall be developed in accordance with 9.6.3.

For anchor rods utilizing leveling nuts, the design axial load shall be the larger of the compression and tension anchor rod forces.

9.6.2 Deformed Anchor Rods

Deformations of deformed anchor rods shall meet or exceed the deformation requirements of ASTM A615 Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Placement.

The minimum embedment depth for deformed anchor rods shall be calculated in accordance with ACI 318 Chapter 25 except for straight anchor rods under tension which shall be determined in accordance with the following equations:

$$L_d = l_d \alpha \beta$$

where:

L_d = minimum embedment depth of anchor rod

l_d = basic development length of anchor rod

The basic development length of an anchor rod shall be as follows:

$$l_d = \text{larger of } \frac{1.27(A_g)(F_y)}{\sqrt{f'_c}} \text{ and } 0.400(d)(F_y) \text{ for bars up to and including \#11}$$

$$l_d = \frac{2.69(F_y)}{\sqrt{f'_c}} \text{ for \#14 and \#14J bars}$$

$$l_d = \frac{3.52(F_y)}{\sqrt{f'_c}} \text{ for \#18 and \#18J bars}$$

For SI units:

$$l_d = \text{larger of } \frac{0.0191(A_g)(F_y)}{\sqrt{f'_c}} \text{ and } 0.058(d)\,(F_y) \text{ for bars up to and including \#35M}$$

$$l_d = \frac{26.0(F_y)}{\sqrt{f'_c}} \text{ for \#45M bars}$$

$$l_d = \frac{34.0(F_y)}{\sqrt{f'_c}} \text{ for \#55M bars}$$

where:

A_g = gross area of the anchor rod, in.^{2} [mm^2]

F_y = specified minimum yield stress of anchor rod, ksi [MPa]

f'_c = specified compressive strength of concrete, ksi [MPa]

d = anchor rod diameter, in. [mm]

α = 1.0 for $F_y \leq 60$ ksi [415 MPa] and 1.2 for $F_y = 75$ ksi [517 MPa]

β = 1.0 for center-center spacing less than 6 in. [150 mm] and 0.8 for greater spacing

Note: Threads shall not be considered as deformations for the purpose of determining embedment depths.

9.6.3 Headed Anchor Rods

The development of headed anchor bolts under tension, compression and shear shall satisfy the provisions of ACI 318 Chapter 17 for concrete strength except only the provisions for side-face blowout and pullout strength shall apply to a group of cast-in headed anchor rods connected to an embedded plate developed with foundation reinforcement or placed to result in adequate punching shear strength (two-way shear) in accordance with ACI 318 Chapter 22. For smooth headed anchor rods with the headed end near the bottom surface of a slab, additional nut(s) or plates(s) on the upper end of the anchor rods shall be provided when required to provide additional strength due to the limited punching shear capacity of the anchor rod group.

When deformed reinforcing bars are utilized as headed anchor rods, the contribution from bar deformations shall be ignored except it shall be permitted to develop deformed reinforcement anchor rods in compression in accordance with the provisions of ACI 318 Chapter 25 for compression reinforcement.

Concrete pryout need not be considered as a potential failure mode for anchor rods with an embedment depth greater than or equal to 25 times the nominal anchor rod diameter.

9.7 Design Strength of Soil or Rock

The design strength of soil or rock shall be determined by multiplying the nominal soil or rock strength by the appropriate resistance factor specified in 9.4.1. The nominal soil or strength shall be determined in accordance with geotechnical recommendations from a site geotechnical investigation or based on the principles of soil or rock mechanics.

The nominal strength of soil or rock determined from geotechnical recommendations that are based on allowable strengths shall be determined by multiplying the allowable strength by the corresponding safety factor reported in the geotechnical recommendations. When specific geotechnical parameters or the factor of safety used to determine allowable strengths are not reported in the geotechnical recommendations, a safety factor equal to 2.0 shall be used to determine nominal strengths.

The design strength of soil or rock shall be equal to $\phi_s R_s$ where:

ϕ_s = 0.60 for rock or soil nominal strengths for bases of guyed masts including spread footings, driven piles, drilled caissons, steel grillages

ϕ_s = 0.75 for bearing on rock or soil for bases of self-supporting structures including spread footings, mats, driven piles, drilled caissons, steel grillages

ϕ_s = 0.75 for pull-out or uplift in rock or soil for foundations and anchorages including spread footings, deadman anchors, drilled caissons, steel grillages and battered piles

ϕ_s = 0.50 for pull-out or uplift in rock or soil for foundations and anchorages which utilize one rock/soil bolt, dowel or anchoring device

ϕ_s = 0.40 for pull-out or uplift in rock or soil for foundations and anchorages which utilize non-battered piles with a tapered cross-section

ϕ_s = 0.75 for friction or lateral resistance of soil or rock for all types of foundations excluding bases of guyed masts

R_s = nominal soil resistance

Notes:

1. For foundation analyses which model the lateral stiffness of the soil, factored reactions for the analysis shall be divided by ϕ_s. The soil parameters related to the lateral stiffness

of the soil shall not be reduced using a resistance factor. The foundation internal forces and moments from the foundation analysis shall be multiplied by ϕ_s for the strength design of the foundation. Unfactored reactions shall not be modified by ϕ_s when investigating displacements for serviceability limit states conditions.

2. Dead loads that include concrete, soil or other material directly above the foundation that resist uplift and overturning forces shall be factored by the appropriate dead load factor with no resistance factors applied.
3. Nominal soil strength or stiffness that are determined using the overburden weight of soil (e.g. gross soil bearing resistance, skin friction, lateral soil resistance, etc.) shall be determined using the unfactored unit weight of soil. The design soil strength shall be equal to the calculated nominal soil strength multiplied by the appropriate resistance factor.

For soil nominal strengths that are a function of the soil overburden weight, the unfactored weight of the soil shall be used to determine nominal soil strengths. The design strength shall be determined by multiplying the nominal strength times the appropriate resistance factor from 9.4.1. (Refer to Note 2 from 2.3.2.)

9.8 Seismic Considerations

The seismic considerations specified in this section apply to foundations for Risk Category II, III and IV structures located in regions where the earthquake spectral response acceleration at short period, S_s, from 2.7.4 is greater than 1.0.

9.8.1 Independent Foundations

When a self-supporting latticed structure is supported by independent foundations, the foundations shall be connected together at the base by a grade beam or similar device. The grade beam or similar device shall resist 2/3 of the total horizontal seismic shear as calculated in 2.7.7 in compression and in tension.

Exception: Other approved methods may be used where it can be demonstrated that equivalent restraint can be provided. Equivalent restraint can be provided by horizontal structural members above the base of the structure, reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils or very dense granular soils.

9.8.2 Longitudinal Reinforcement

Longitudinal reinforcement in piers supported on an isolated spread footing or in piles, piers or caissons supporting a pile cap or mat shall be continuous and extend into the footing, pile cap or mat and be fully developed in tension at the interface. The free ends of hooks utilized for longitudinal reinforcement shall be permitted to be oriented either inward or outward from the center of the longitudinal bar arrangement. When grouted reinforcing bars are used at the interface, the grouting system shall be demonstrated by testing to develop at least 125% of the minimum specified yield strength of the reinforcing bars.

9.8.3 Transverse Reinforcement

Stirrup splices in piles, piers or caisson shall be staggered with a nominal 180 degree separation. Transverse reinforcement at the top of piles, piers or caissons supporting a pile cap or mat shall be in accordance with ACI 318 Section 18.13.4.3 and shall be anchored in accordance with ACI 318 Sections 25.7.2, 25.7.3 or 25.7.4.

9.8.4 Batter Piles

Pile caps and mats supporting batter piles shall be designed to resist the full compressive strength of the batter piles acting as short columns.

9.8.5 Precast Concrete Foundations

Precast foundations shall have a design moment strength equal to or greater than the expected moment capacity of the pole determined in accordance with 2.7.9.

10.0 PROTECTIVE GROUNDING

10.1 Scope

This Section provides standard protective grounding requirements for structures addressed in the scope of this Standard. The protective grounding requirements are intended to provide a safe discharge of electrical energy to earth. A maximum ground resistance to earth is specified together with provisions to reduce the grounding impedance to earth for power surges and lightning protection.

Site-specific grounding systems based on site-specific conditions shall be permitted to be substituted for the standard protective grounding system specified in section 10.4. Site-specific grounding systems are required for AM tower installations and for any site where the standard protective grounding system specified in this Section cannot be installed due to site conditions.

Grounding requirements for appurtenances supported by the structure or for surrounding structures, conductive objects and equipment are not within the scope of this Standard.

Lightning rods are not required as part of the standard protective grounding system specified in this Section. Provisions for lightning rods required for the site-specific protection of a structure, equipment, obstruction lighting systems, etc. are not within the scope of this Standard.

10.2 Definitions

Ground resistance: a measured value from direct current or low frequency testing.

Grounding impedance: the restriction of the flow of energy from surges and lightning strikes that typically include alternating currents of many frequencies.

Grounding system: a system of grounding materials with electrical connections to a structure to direct energy to the earth resulting from lightning, high voltage and static discharges.

10.3 General

Unless otherwise specified for a site-specific grounding system considering available property, onsite soil characteristics and lightning protection requirements, the measured ground resistance of a grounding system to earth shall not exceed 10 ohms.

10.4 Standard Protective Grounding System

Additional or alternate grounding materials from the standard protective grounding system specified in this Section may be provided based on site-specific conditions.

10.4.1 Materials

Grounding rod-type electrodes (also referred to as ground rods) shall be 5/8 in. [16 mm] minimum diameter, 10 ft. [3 m] minimum length, stainless steel, copper, copper clad steel, or zinc coated steel.

Ground wires shall be #2 AWG or larger solid bare tinned copper wire.

Connections used for the grounding system shall be made with components that are compatible with the connected materials with respect to galvanic corrosion.

10.4.2 Grounding System Configuration

The grounding system shall consist of rod-type electrodes and ground wires (including rings and radials specified in 10.4.2.1 and 10.4.2.2) installed below the frost depth but not less than 30 in [760 mm] below grade.

Rod-type electrodes shall be installed vertically below grade. The number of ground rods installed shall be in accordance with 10.4.2.1. When site conditions do not allow for a vertical installation, battering the electrodes away from the base of the structure as close as possible to vertical shall be considered acceptable when the minimum depth below grade is maintained.

Ground wires connected to the structure shall continuously flow down or away from the structure with bends not less than 90 degrees. The minimum bend radius shall be 12 in. [300 mm].

No portion of the grounding system shall pass through or be connected to a concrete foundation.

10.4.2.1 Ground Rods and Ground Wire Radials

Ground rods shall be installed with a 20 ft. [6 m] minimum separation.

Each ground rod shall be connected to a ground wire radial installed around the base of the structure. The ground wire radials shall project outward from the base of the structure for a minimum length of 25 ft [7.6 m] per radial. Each ground wire radial shall be connected to the base of the structure and be directed away from the base of the structure. Ground radials are not required for ground rods installed at guy anchors for guyed masts.

When site conditions do not allow for a straight line radial, the ground wire for any radial shall be permitted to deviate from a straight line using one or more 120 degree minimum bends with a 12 in. [300 mm] minimum bend radius.

One ground rod and one ground wire radial shall be installed for each leg of self-supporting and guyed latticed structures with a face width (center-to-center of legs) exceeding 4 ft. [1.2 m].

For all tubular pole structures and latticed structures with a face width 4 ft. [1.2 m] or less, one ground rod and one ground wire radial shall be installed on both sides of the structure base foundation, with a separation of 180 degrees (+/- 30 degrees).

One ground rod shall be installed at each guy anchor for guyed masts and connected to the guy anchor with a ground wire.

10.4.2.2 Ground Wire Ring

A ground wire ring shall be installed around the perimeter of self-supporting and guyed latticed structures with a face width (center-to-center of legs) exceeding 4 ft [1.2 m]. The ground wire ring shall be continuous around the perimeter and shall not be closer than 2 ft. [0.7 m] to the structure or a foundation. Each ground wire radial shall be connected to the ground wire ring. The diameter of the ground wire ring shall be adequate to accommodate a 20 ft. [6 m] minimum separation between connections of the radials to the ground wire ring.

11.0 OBSTRUCTION MARKING

Structures shall be marked in accordance with Federal Communications Commission (FCC), Federal Aviation Authority (FAA), and/or the local aviation authority requirements.

For structures requiring painting for obstruction marking purposes, the termination of a color band shall be permitted to coincide with a panel point on the structure provided that the panel point is within 10 ft. [3 m] of the calculated termination point based on equal color bands.

12.0 CLIMBING FACILITIES

12.1 Scope

This Section provides minimum requirements for the design and construction of rung and rail climbing facilities, steps, step bolts, safety climb systems and climber platforms used for climbing or working on structures addressed in the scope of this Standard.

When fall arrest, restraint or rescue system anchorages are specified in a procurement specification, minimum strength requirements for fall protection anchorages are provided in this Section.

The climbing facility requirements in this Section are based on use during erection, modification or demolition in accordance with the American Society of Safety Engineers Standard, ANSI/ASSE 10.48, "Criteria for Safety Practices with the Construction, Demolition, Modification and Maintenance of Communication Structures", and the proper use and inspection of personal protection equipment in accordance with Industry Standards.

12.2 Definitions

Authorized climber: an individual, supervised by a competent climber, with the physical capabilities to climb, who may or may not have previous climbing experience and has documented training in the applicable fall protection equipment and the regulations pertaining to their proper use and inspection.

Climbing facilities: a climbing facility intended to provide access by a climber to a location on a structure such as:

1. steps, rung and rail or step bolts which are attached to the structure;
2. members which form an integral part of the structure;
3. fall protection anchorages;
4. climber platforms and support rails.

Climber rest platform: a permanent or temporary horizontal landing surface for rest purposes that is used by a climber.

Climber working platform: a permanent or temporary horizontal landing surface that is used by a climber as a working or standing location.

Competent climber: an individual with the physical capabilities to climb, has applicable climbing experience, has documented training in the applicable fall protection equipment and the regulations pertaining to their proper use and inspection, is capable of identifying existing and potential fall hazards, supervises authorized climbers and has the employer's authority to take prompt corrective action to eliminate or control those hazards.

Factored load: a nominal load multiplied by a load factor intended to represent an ultimate load.

Fall protection anchorage: an anchorage specifically designed for fall arrest, restraint or rescue systems.

Rung and rail climbing facility: a climbing facility used by a climber for access consisting of rungs and side rails joined at regular intervals.

Safety climb system: a set of components that working together form a system that will minimize or limit the fall distance from a climbing facility.

Safety sleeve: the part of a safety climb system consisting of the moving component with a locking mechanism attached to the full body harness.

Step bolt: a round or flat member used by a climber for access affixed to the structure on one end with the other end having a means to prevent the foot from sliding off.

Support rail: a horizontal member around the sides or ends of a platform.

12.3 General

Structures exceeding 10 ft. [3 m] in height intended for climbing shall be equipped with a minimum of one climbing facility equipped with a safety climb system, unless otherwise required.

To ensure compatibility with a climber's cable safety sleeve, cable safety climb systems shall have a stamped or engraved metal identification tag affixed at the base of the structure indicating the size and type of the safety climb system cable.

For cable safety climb systems, a 3/8 in. diameter wire rope shall be considered as standard in order to minimize safety sleeve size requirements. Safety sleeves shall be clearly marked to identify compatible cable sizes and types and any special requirements from the safety sleeve manufacturer to ensure proper performance when using the safety climb system as a part of a fall protection plan.

Cable safety climb systems shall be installed with the initial cable tension specified by the safety climb system manufacturer.

Steps, rung and rails, step bolts, climber platforms and support rails designed in accordance with this Section are not intended as fall protection anchorages.

Cages or barriers used with rung and rail climbing facilities (also referred to as cage guards, hoops or basket guards) are not recommended due to the need to service the structure at various locations and shall not be considered as a substitute for a safety climb system.

The requirements for cages or barriers are not within the scope of this Standard.

Toe boards are not required for climber platforms for structures addressed in the scope of this Standard.

Inspection and assessment of climbing and working facilities shall occur after initial installation, before each use and after each modification to a structure.

Notes:

1. When a climbing facility or safety climb system is not provided over the entire height or when an obstruction is required due to the primary use, function or performance of the structure, the structure shall be equipped with a warning sign or fall protection anchorages shall be available at a maximum spacing of 4 ft. [1.2 m] over the height not equipped with a safety climb system or over the length of the obstruction to the climbing facility.
2. An obstruction to a climbing facility or safety climb system shall only be permitted when required due to the primary use, function or performance of the structure, an antenna or other appurtenance.
3. Structures not intended for climbing and not equipped with a climbing facility that are maintained by other access means need not have warning signs or fall protection anchorages.
4. A safety climb system is not required for each climbing facility when multiple climbing facilities are provided. The safety climb system shall be provided for the climbing facility that is continuous over the intended climbing height.
5. It shall be permissible to begin climbing facilities, including safety climb systems, at a height above grade when required for site security to limit unauthorized access to a structure.

12.4 Strength Requirements

All loads in this Section represent factored loads. A load factor of 1.2 shall be applied to dead loads for determining strength requirements.

The minimum factored load on individual rungs or steps where both ends are attached to side rails or supporting members shall be equal to a normal concentrated load of 500 lbs [2.22 kN] applied at the worst-case location.

Step bolts and their associated attachments shall be designed to support a minimum normal concentrated factored load equal to 600 lbs [2.67 kN] applied 2 in. [51 mm] from the inside face of the step bolt head. The moment for step bolt strength requirements shall be determined at the outside face of the step bolt retaining nut. Step bolt clip strength requirements shall be determined in accordance with Section 4.0 without accounting for the contact of the end of the step bolt with the supporting member.

The minimum factored load on rail and rung climbing facilities shall be 500 lbs [2.22 kN] vertical and 75 lbs [333 N] horizontal applied to each rail simultaneously at the worst-case location between consecutive attachment points to the structure.

The minimum factored load on climber rest platforms shall be equal to a normal concentrated load of 375 lbs [1.67 kN] applied at the worst-case location.

The minimum factored uniform load, in addition to dead loads, on climber working platforms shall be equal to 37 pounds per square foot [1.8 kPa] over the entire working area but not less than a total normal factored load of 750 lbs [3.34 kN] .

The minimum factored concentrated load on a support rail for a climber platform shall be equal to 225 lbs [1.00 kN] applied in the worst-case location and direction. The minimum factored uniform load on a support rail shall be equal to 60 pounds per foot [880 N/m] applied in the worst-case direction, (not simultaneous with concentrated load).

The top anchorage of cable safety climb systems shall be designed for a minimum factored vertical load of 4,000 lbs [17.8 kN]. For rail-type safety climb systems that are utilized on rung and rail type climbing facilities, the climbing facility supports shall be designed for a minimum factored vertical load of 2,100 lbs [9.34 kN] for each 20 ft. [6 m] length.

The minimum vertical factored load for the design of a fall protection anchorage shall be 3,600 lbs [16.0 kN].

Notes:

1. The strength requirements for climbing and working facilities need not be considered in conjunction with any other loading combination.
2. The flexural strength of a threaded step bolt shall be determined in accordance with 4.7.1 using a plastic section modulus, Z, based on the tensile root diameter of the step bolt (refer to 4.9.9).

12.5 Dimensional Requirements

Climbing and working facilities shall be classified in accordance with Table 12-1 (Class A or B) for the purpose of determining dimensional requirements. The following dimensional requirements apply to all class systems except as noted:

1. Center-to-center spacing between rungs, alternatively spaced step bolts or structural members used for climbing shall be 10 in. [250 mm] minimum and 16 in. [410 mm] maximum; for Class A systems, the spacing shall remain uniform over a continuous length of climb within a tolerance of ± 1 in. [25 mm].
2. Clear spacing between side rails shall not be less than 12 in. [300 mm]; the clear spacing shall be increased by the width of the safety rail when used.
3. All rungs, steps, step bolts, and rails shall be free from splinters, sharp edges, burrs, or projections which may pose a hazard;
4. Rungs shall not be less than 0.625 in. [16 mm] in width, round rungs shall not be greater than 1.5 in. [38 mm] in diameter and flat rungs shall not be greater than 2 in. [51 mm] in width.
5. For Class A systems, a minimum clear space shall be provided at rungs, steps, step bolts, or applicable tower members equal to 4 in. [100 mm] vertically, 4.5 in. [110 mm] horizontally and 7 in. [180 mm] deep.
6. For Class A systems, a minimum 24 in. [610 mm] clearance shall be provided from the center line of a climbing facility to any obstruction on the climbing side (see Figure 12-1).
7. The climbing side of a rung and rail or other climbing facility shall be between 90 degrees and 60 degrees to the horizontal sloping away from the climber.
8. For rung and rail type climbing facilities the size of rungs and side rails shall be uniform in the same continuous length of climb.

Note: Structures that do not meet the dimensional requirements of this section shall be equipped with warning signs.

12.5.1 Step Bolts

The following additional requirements apply to step bolts:

1. Step bolts shall not be less than 0.625 in. [16 mm] in nominal diameter or width.
2. Clear width between the exterior face of the outer retaining nut supporting the step bolt and the inside face of the step bolt head shall not be less than 4.5 in. [110 mm].
3. The step bolt head shall provide an area not less than the area provided by a nominal 2 in. [51 mm] diameter round step bolt head to ensure a climber's foot cannot slide off the end of a step bolt.
4. The outer edge or perimeter of the step bolt head shall have a flat or curved surface to avoid sharp edges hazardous to climbers.
5. The horizontal spread between the attachment points of step bolts shall not exceed 24 in. [610 mm].
6. Step bolt material shall meet the requirements of ASTM A449 and shall be tested as finished step bolts at Test Frequency P (Piece Testing) of ASTM A673 to meet a minimum absorbed energy requirement at -20 degrees F [-29 degrees C] of 15 ft-lbs [20 J] average for 3 specimens and a minimum of 12 ft-lbs [16 J] for any 1 specimen in accordance with ASTM A370.
7. Step bolt clip material shall meet the requirements of one of the following: ASTM A572 Grade 50 plate steel; ASTM A1011/A1018 HSLAS Grade 50, Class 1 or 2, hot rolled sheet or strip steel; ASTM A36 steel shapes; ASTM A529 Grade 50 or 55 steel shapes.

Step bolt clips intended for use as a fall protection anchorage shall also meet the strength requirements for a fall protection anchorage.

12.5.1.1 Latticed Structures

Step bolts shall be placed on a minimum of one leg for the entire height of a latticed structure when utilized as the primary climbing facility. Additionally, step bolts shall be placed on all legs in latticed sections with face widths greater than 15 ft. [4.6 m] and on all legs in latticed sections with face widths greater than 6 ft. [1.8 m] when the panel spacing along the legs exceeds 5.5 ft. [1.7 m].

12.5.1.2 Pole Structures

Step bolts shall be placed over the entire height of pole structures when utilized as the primary climbing facility. Two step bolts spaced horizontally to align with the staggered step bolt pattern along the length of a section shall be placed at the top and bottom of all pole sections. For sections with slip splices, the additional step bolts at the top of the lower section shall be below the maximum slip length.

Additionally, two horizontal rows of step bolts with a 12 in. [300 mm] to 18 in. [457 mm] vertical separation between rows and a 12 in. [300 mm] to 18 in. [460 mm] horizontal separation between step bolts shall be provided around the perimeter of a pole at the following locations:

1. Ports.
2. Jacking lugs below a slip splice.

Notes:

1. The requirement for additional rows of step bolts apply only to ports, jacking lugs and flanges that have centerline elevations greater than 10 ft. [3 m] above the base of the structure.
2. The first row of step bolts shall be placed between 24 in. [610 mm] to 48 in. [1200 mm] below the centerline elevation of the port, jacking lug or flange.
3. When the spacing requirements for step bolts overlap, the pattern of the step bolts along the length of the pole for climbing shall be maintained.
4. The requirement for additional step bolts at slip splices are intended to provide access to jacking lugs and to provide additional steps at the discontinuous step bolt spacing that occurs at a slip splice due to variable slip splice lengths.

12.6 Step Bolt Installation Requirements

Step bolts shall be installed using double nuts. A step bolt installed in a step bolt clip shall be turned with the outer nut loose until the end of the step bolt makes contact with the supporting member. The outer nut of all step bolt installations shall be tightened to a snug tight condition and pretensioned by rotating the outer nut an additional 1/3rd turn.

Step bolts shall not be reused after pretensioning unless a step bolt nut can be run up and down the entire length of the step bolt threads by hand and an inspection is performed by a competent climber.

Welding shall not be permitted on step bolts.

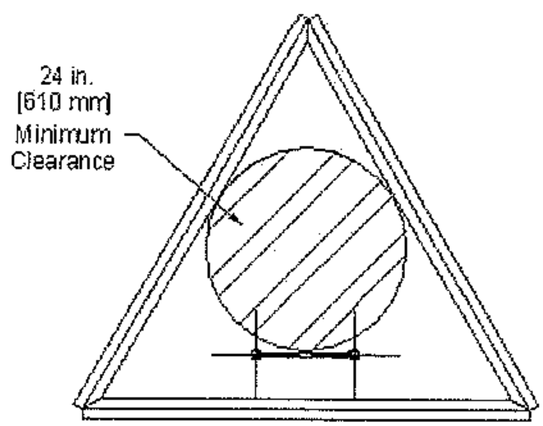
12.7 Climber Attachment Anchorages

Refer to Annex I for examples of climber attachment points.

Table 12-1: Classification of Climbing and Working Facilities

User	Class
Authorized or Competent Climbers	A
Competent Climbers only	B

Figure 12-1: Minimum Clearance for Class A Systems



13.0 PLANS, ASSEMBLY TOLERANCES AND MARKING

13.1 Scope

The purpose of this Section is to define the requirements for plans, assembly tolerances and marking appropriate for structures addressed in the scope of this Standard.

This Section does not address the safety and stability of the structure during assembly and erection, which are the responsibility of the erector, based on the means and methods chosen by the erector.

13.2 Plans

Complete plans, assembly drawings, or other documentation shall be supplied showing the necessary marking and details for the proper assembly and installation of the components, including the member sizes, design yield strength of the structural members, bend radius for tubular pole structures, the grade of structural bolts required and when pretensioning is required. Foundation reactions, when provided, shall be based on factored loads.

The tower plans shall detail each attachment height, antenna quantity, antenna model or type, mount quantity, mount type and line size that was included in the structural analysis. Alternatively, the total effective projected area representative of all of the antennas and mounts at each elevation may be provided along with the associated line sizes.

The plans for the structure shall detail the following data used in the structural analysis:

1. Risk Category (I, II, III or IV).
2. Basic wind speed (3-second gust) without ice.
3. Basic wind speed (3-second gust) with ice.
4. Design ice thickness (500-year return period).
5. Ground elevation.
6. Exposure Category (B, C, D or Site-Specific).
7. Topographic method and criteria as applicable:
    a) Category 1, 2, 3, or 4 and crest height (H) for Method 1.
    b) Feature type, slope and crest height, distance upwind or downwind from crest (x), feature base length from crest (L) for Method 2 or Method 3.
8. Rooftop wind speed-up parameters (H_S, W_S, x_b).
9. Seismic design parameters (S_s, S_1, T_L and Site Class).
10. Factored foundation reactions for the loading combinations considered.
11. Soil design parameters and geotechnical report reference when available.

13.3 Tolerances

The tolerances specified in this Section apply to the original erection of a structure or to the modification of a structure.

The tolerances specified in this Section do not apply to measurements made during condition assessments or other evaluations.

13.3.1 Overall Height

The overall height of an assembled structure excluding appurtenances shall be within +1% and -1/2% of the specified height, not to exceed +5 ft. [1.5 m] or -2 ft. [0.6 m].

13.3.2 Guy Tensions

The maximum deviation from the design initial tension shall be ± 10% for guys up to and including 1 in. [25 mm] diameter and ± 5% for guys greater than 1 in. [25 mm] diameter, of the specified design initial tension at an anchorage, corrected for the ambient temperature.

13.3.3 Plumb

The horizontal distance between the vertical centerlines at any two elevations throughout the height of the structure shall not exceed 0.25% of the vertical distance between the two elevations for latticed structures and 0.50% at top of tubular pole structures.

13.3.4 Twist

The twist between any two elevations throughout the height of the structure shall not exceed 0.5 degrees in 10 ft. [3 m]. The maximum twist over the structure height shall not exceed 5 degrees.

13.3.5 Slip Splice

The installed slip splice length shall not be less than 1.5 times the inside width of the base of the upper section. The inside width shall be measured between flats for polygonal cross sections. Splices shall be pulled together to ensure firm contact. When installed slip splices are in firm contact but do not satisfy the minimum installed splice length requirement, the installed joint shall be evaluated by multiplying the nominal strengths from 4.8.2 for each section at the splice by a factor that reduces linearly from 1.0 to 0.50 as the splice length reduces from 1.5 to 1.0 times the inside width of the base of the upper section. Installed slip splices with lengths less than 1.0 times the inside width of the base of the upper section shall require reinforcement.

Slip splices shall be jacked together to obtain a tight, even joint. The jacking force shall be applied regardless of the design length of the slip splice and the length obtained during initial fit up. The jacking force shall be increased until no additional movement of the joint occurs.

13.3.6 Straightness

The straightness of the individual members shall be within a tolerance of 1 in 500 but not more stringent than 1/16 in. [1.6 mm], of the length between points that are laterally supported.

13.3.7 Measurements

Measurements shall be taken at a time when the wind velocity is less than 15 mph [6.7 m/s] at ground level and with less than 1/8 in. [3 mm] radial ice on the structure or the guys at the base of the structure and with solar distortion effects considered.

13.3.8 Take-Up Devices

For initial installations, the minimum take-up adjustment available after the structure is plumb and the guy tensions are set shall be:

1. 6 in. [150 mm] for guys with nominal diameter of 0.5 in. [13 mm] or less.
2. 10 in. [250 mm] for guys with nominal diameter greater than 0.5 in. [13 mm].

13.4 Marking

All structural members or welded structural assemblies, except for hardware, shall have a part number. The part numbers shall correspond with the assembly drawings. The part number is to be permanently attached (stamped, welded lettering, stamped on a plate that is welded to the member, etc.) to the member before all protective coatings are applied. The part number shall have a minimum character height of 0.5 in. [13 mm].

14.0 MAINTENANCE AND CONDITION ASSESSMENT

14.1 Scope

This Section addresses the maintenance and condition assessment of structures.

Maintenance and condition assessment requirements for safety climb systems are not within the scope of this Standard.

Maintenance and condition assessment guidelines are provided in Annex J.

14.2 Requirements

Condition assessments require access to critical structural components.

Radomes enclosing main structural components (i.e. cylindrical radomes surrounding tubular or latticed support structures) shall be removed as required.

14.3 Intervals

Maintenance and condition assessment recommendations for communication structures are as follows:

1. Three-year intervals for guyed masts and five-year intervals for self-supporting structures.
2. After severe wind and /or ice storms, severe seismic events or other extreme conditions.
3. Shorter intervals may be required for Risk Category III or IV structures and structures in coastal regions, in corrosive environments, and in areas subject to frequent vandalism.

Note: Refer to Section 17.0 for intervals for small wind turbine support structures.

14.4 Guy Anchor Shafts

Guy anchor shafts with steel in direct contact with soil shall be assessed in accordance with a corrosion management plan based on site-specific corrosion conditions.

15.0 EXISTING STRUCTURES

15.1 Scope

This section addresses the evaluation and modification of existing structures.

It is not the intent of this Section to include as part of the evaluation of an existing structure the evaluation of strength or serviceability of appurtenances supported on an existing structure, including appurtenance mounting systems, climbing facilities, etc.

Proposed changed conditions that require an analysis to determine conformance with this Standard are specified in this Section. Requirements are also included for the modification of existing structures.

It is not the intent of this Section to require the analysis of existing structures and foundations to determine conformance to each revision of this Standard. In addition, it is not the intent of this Section to require conformance of existing structures and foundations to this Standard for proposed antenna and other appurtenance loading when the increase in demand-capacity ratios from an evaluation does not exceed the thresholds determined in accordance with section 15.5.

The evaluation criteria in this section assumes that an existing structure has been properly designed or structurally modified and maintained in accordance with this Standard or a previous edition of this Standard and that all items related to the integrity of the structure identified in a condition assessment (refer to Annex J) have been corrected unless otherwise addressed in a structural analysis report (refer to 15.6.3).

The loading, analysis and design criteria related to the installation, alteration and maintenance construction activities for existing structures is not within the scope of this Standard (refer to the ANSI/TIA-322 Standard).

The criteria for safety practices with the construction, demolition, modification and maintenance of existing structures is not within the scope of this Standard (refer to the ANSI/ASSE A10.48 Standard).

15.2 Definitions

Comprehensive structural analysis: a structural analysis of an existing structure to determine conformance to this Standard regarding the overall stability and the adequacy of structural members, connections, anchorages and foundations.

Demand-capacity ratio: the ratio of the required strength of a component as defined in Section 2.0 to the design strength of the component as defined in Section 4.0.

Design documents: documents indicating the proposed design, applicable standards, loading criteria, materials and the required field verifications for the modification of an existing structure to conform to the strength and serviceability requirements of this Standard.

Existing structure: an erected structure.

Feasibility study: a preliminary structural analysis of an existing structure to evaluate the feasibility of a proposed changed condition or to determine the increase in the demand-capacity ratio for a proposed changed condition, limited to the investigation of the overall stability of the structure and the strength requirements for the main load carrying members of a structure.

15.3 Changed Conditions Requiring an Evaluation

As a minimum, existing structures shall be evaluated in accordance with this Section, regardless of the standard used for the design of the original structure or the last modification, under any of the following changed conditions:

1. A change in type, size, or number of appurtenances such as antennas, radios, transmission lines, mounts, platforms, ladders, etc.
2. A change in the risk category of a structure to a higher risk category.
3. A change in serviceability requirements (i.e. more stringent twist or sway limitations due to a change involving microwave antennas).
4. A change to the geometry of the structure or to the strength of structural components (i.e. an extension in height of the structure, a change in the guying configuration of a guyed mast or a change to structural members resulting in lower strengths, etc.).

15.4 Risk Category

The risk category of an existing structure shall be determined in accordance with section 2.2 and Table 2-1 considering the proposed changed conditions and all existing antennas and other appurtenances to remain on the structure.

15.5 Evaluation of Changed Conditions

A proposed changed condition shall require the supporting structure and foundation to conform to this Standard when the increase in the demand-capacity ratio of any member of the supporting structure exceeds 5%.

Note: The increase in the demand-capacity ratio for Risk Category IV structures shall also not exceed the percentage allowed per the service level agreement for the hardened network.

The increase in demand-capacity ratio to determine if conformance of the supporting structure and foundation to this Standard is required shall be determined by comparing the demand-capacity ratios for supporting the proposed antennas and other appurtenances (together defined as the "Proposed Appurtenance Loading") to the demand-capacity ratios for supporting the antennas and other appurtenances used for the original design of the supporting structure (defined as the "Baseline Appurtenance Loading"). When a structure has been structurally modified since its original installation, the antennas and other appurtenances used for the design of the structural modification shall be considered as the Baseline Appurtenance Loading. The demand-capacity ratios for both the Proposed and Baseline Appurtenance Loading shall be determined in accordance with this Standard regardless of the design specification used for the original design or for the last structural modification of the supporting structure. A feasibility study or comprehensive structural analysis in accordance with this Standard shall be permitted for the purpose of determining the increase in demand-capacity ratios.

Note: The intent of this section is to consider the cumulative changed conditions which have occurred since the original construction or since the last structural modification of the supporting structure to be considered as the proposed appurtenance loading (all previous changed conditions combined with new changes to the antennas or other appurtenances to be supported by the structure). In no case shall the changed conditions that have occurred since the original construction or last structural modification of the supporting structure be considered as any portion of the baseline appurtenance loading for the purposes of determining the increase in demand-capacity ratios, regardless of the conformance of any previous changed condition to other revisions of this Standard or any other standard.

When documentation is not available for determining the Baseline Appurtenance Loading, a comprehensive structural analysis in accordance with this Standard shall be required for all Proposed Appurtenances Loadings or for other changed conditions.

When strengthening is required, modifications to the structure or foundation shall be in conformance with this Standard based on a comprehensive structural analysis to limit the maximum demand-capacity ratio for the changed condition to 1.05. Components of the supporting structure or foundation with lower demand-capacity ratios based on a comprehensive structural analysis need not be strengthened.

Alternatively, the evaluation of an existing antenna supporting structure or foundation shall be permitted to be performed in accordance with Annex S.

15.6 Structural Analysis

It shall be permitted to assume that existing structures and their foundations have been properly installed and maintained.

Assumptions regarding structural details that cannot be determined without dismantling the structure or extensive field non-destructive testing (such as inside weld sizes of socketed flanged leg connections of latticed structures) shall be included in the structural analysis report (refer to 15.6.3).

15.6.1 Source of Data

Sufficient up-to-date information shall be used in the evaluation to accurately represent the existing structure. The following sources may provide the information necessary for an evaluation:

1. Previous structural analysis.
2. Installation documents, material lists and fabrication drawings.
3. Geotechnical reports.
4. As-built drawings of the original installation and/or subsequent modifications.
5. Field mapping, measurements and/or material testing (refer to Annex J).
6. Existing and proposed appurtenances listing and locations.

Annex R provides recommended material grades to consider based on the age of a structure when structural material grades are unknown. Material grade assumptions shall be documented in the structural analysis report (refer to 15.6.3).

15.6.2 Foundation Analysis

A comprehensive structural analysis of a foundation requires site-specific geotechnical and as-built foundation data. It shall be permissible to supplement incomplete foundation or geotechnical data with assumptions noted in the structural analysis report.

When documentation is available for an existing foundation that documents the reactions utilized for the design of the existing foundation, it shall be permissible to determine the adequacy of an existing foundation by comparing the reactions from the analysis for a proposed changed condition to the documented foundation design reactions. When the documented foundation design reactions are based upon an Allowable Stress Design procedure, it shall be permissible to multiply the documented foundation design reactions by 1.35 for comparison to the reactions from the analysis for a proposed changed condition. When a comparison of reactions is made in lieu of a comprehensive structural analysis of a foundation, it shall be stated in the structural analysis report.

15.6.3 Structural Analysis Report

The structural analysis report shall specify the source of the data used for the analysis and the type of analysis performed (comprehensive structural analysis or feasibility study). In addition, when a baseline appurtenance loading is considered in an evaluation, the maximum increase in demand-capacity ratio for the proposed and baseline appurtenance loadings from section 15.5 shall be documented in the structural analysis report.

This Standard does not require the modification of an existing structure to be in conformance with this Standard when a proposed changed condition does not increase the demand-capacity ratios of structural components of the structure beyond the thresholds determined in accordance with section 15.5 unless otherwise required. Conformance to this Standard, however, requires that the demand-capacity ratios of the structure and foundation based on a comprehensive structural analysis do not exceed 1.05 determined in accordance with this Section or Annex S.

Assumptions made for the analysis shall be documented in the report including assumptions made for the analysis of foundations. Assumptions that require field verification prior to the implementation of a changed condition or modification shall be documented in the report.

Structural analysis reports based on a feasibility study shall state that a comprehensive structural analysis shall be required prior to the implementation of a changed condition.

15.7 Exemptions

Existing structures originally designed in accordance with a previous revision of this Standard are exempt from the provisions of this Standard.

When investigating changed conditions for existing structures and modifying existing structures, the provisions of this Standard pertaining to materials, manufacturing and installation do not apply. In addition, the following sections of this Standard do not apply or shall be replaced with the substitutions noted:

1. Section 3.8 Mast shear and torsion: minimum 40% requirement is exempted.
2. Section 4.4.1 Minimum bracing resistance, P_r = 1.5% F_s may be used.
3. Section 4.6.2 Tension-only bracing members.

4. Section 4.9.2 Nut-locking devices: lock washer restriction for structures > 1,200 ft. [366 m] is exempted.
5. Section 4.9.7 Splices: minimum leg tension splice capacity and minimum strength requirement for base and flange plate connections for pole structures are exempted.
6. Section 4.9.7.1 The slip splice length shall be considered equal to be 1.5 times the inside width of the base of the upper section unless otherwise documented.
7. Section 7.6.2.2 End fittings strength efficiency factor: An end termination strength efficiency factor equal to 50% shall be permitted for undamaged existing steel strand wrap-around end terminations for steel strand not exceeding a 1/2 inch nominal diameter utilizing a minimum of 3 properly installed twin base or U-bolt clips.
8. Section 7.6.4 Guy articulation.
9. Section 8.3 Insulators: proof loading of insulators is exempted.
10. Section 9.4.1 Foundations design strength: minimum longitudinal reinforcement ratio for piers, caissons and drilled shafts is exempted.
11. Section 9.4.3.2 Corrosion control: direct embed foundations exempted.
12. Section 9.6 Anchorages: minimum spacing of anchor rods, top shrinkage, temperature and transverse reinforcement for piers, caissons and drilled shafts are exempted.
13. Section 9.6.3 Headed anchor rods.
14. Section 9.8 Foundation seismic considerations: seismic reinforcing detailing requirements and verification of grout strength for grouted reinforcing bars are exempted.
15. Section 10.0 Protective grounding.
16. Section 12.0 Climbing facilities.

15.8 Modification of Existing Structures

15.8.1 Design

Modifications to existing structures shall be based on a comprehensive structural analysis.

Structural modifications shall result in the conformance of the structure and foundation with this Standard.

The demand-capacity ratio thresholds from section 15.5 shall not be utilized to determine the extent of the modifications required for conformance to this Standard. Structural modifications shall not result in a demand-capacity ratio greater than 1.05.

The structural analysis for design shall consider the structural load path based on composite action and strain compatibility between proposed and existing structural components.

Design documents shall indicate the proposed modifications to the structure and foundation required prior to the implementation of a changed condition, (i.e. reinforcement of existing members and/or connections, proposed additional members of the reinforcing system, foundation reinforcement. etc.).

The assumptions made for design that require field verification prior to the implementation of a changed condition shall be specified in the design documents.

15.8.2 Field Verification and Fabrication

The assumptions requiring verification documented in a structural analysis report or on a design document shall be validated prior to implementation of a changed condition or modification.

Assumptions found to be inaccurate and changes to a modification required due to field conditions shall be documented and investigated to determine conformance with this Standard.

Note: It is recommended that field verifications be performed prior to fabrication of components required for a modification.

15.8.3 Installation

Installation of the modification system shall be in accordance with design documents and the ANSI/TIA-322 and ANSI/ASSE A10.48 standards.

15.8.4 Verification of Installation

All modifications shall be inspected in accordance with the requirements specified on the design documents. Recommendations for the inspection of modifications to an existing structure are contained in Annex O.

16.0 APPURTENANCE MOUNTING SYSTEMS

16.1 Scope

Appurtenance mounting systems are considered as structures covered within the scope of this Standard. This Section addresses unique criteria for the design, evaluation and modification of appurtenance mounting systems. All other sections of this Standard pertaining to structures also apply to appurtenance mounting systems unless otherwise noted in this Section.

The loading, analysis and design criteria related to the installation, alteration and maintenance construction activities for appurtenance mounting systems is not within the scope of this Standard (refer to the ANSI/TIA-322 Standard).

The criteria for safety practices with the construction, demolition, modification and maintenance of appurtenance mounting systems is not within the scope of this Standard (refer to the ANSI/ASSE A10.48 Standard).

16.2 Definitions

Azimuth of mounting system: the radial direction in plan view that a mount projects outward from the supporting structure. For a sector mount, the direction normal to the plane formed by the mounting pipes. For a side arm mount, the direction parallel to the plane formed by the side arm. For multiple sector mounts or other similar mount arrangements intended to be assembled at a given elevation as an integral structural system, the direction normal to one sector.

Integral mounting system: a mounting system consisting of multiple sectors or other similar mount arrangements intended to be assembled at a given elevation as an integral structural system (i.e. symmetrical frame/truss platforms, low profile platforms, symmetrical circular ring platforms, inter-connected sector mounts, etc.).

Mounting pipe: a tubular member attached to a mount for supporting antennas or other associated appurtenances.

Mounting system: a combination of members and components designed to support antennas and associated appurtenances (also referred to as a mount).

Sector mount: a mount supporting multiple panel type directional antennas intended to provide coverage in a defined direction.

Side arm or Standoff: a mount with members in a vertical plane, with or without the use of tie-backs, supporting antenna(s).

Tie-back: a horizontal or near horizontal member of a mounting system used solely to prevent rotation (twist) of the mounting system which does not transfer vertical loads to the supporting structure (also referred to as a strut).

16.3 Symbols and Notations

L_M = maintenance load equal to a nominal 500 lbs [2.22 kN] vertical concentrated downward force applied at the mounting pipe locations, one location at a time. All mounting pipe locations are required to be considered to determine maximum member stresses;

L_V = maintenance load equal to a nominal 250 lbs [1.11 kN] vertical concentrated downward force, applied at one location at a time at the center of horizontal members supported at each end (other than tie-backs) and at the end of horizontal cantilevered members;

W_M = wind load calculated in accordance with Section 2.0 for a 30 mph [13.4 m/s] basic wind speed.

16.4 Strength Limit State Load Combinations

Mounting systems shall be analyzed for the loading combinations in this section in addition to the strength limit state load combinations 1, 3 and 4 from 2.3.2.

16.4.1 Sector Mounts and Integral Mounting Systems

1. 1.4D
2. $1.2D + 1.5L_M + 1.0\ W_M$
3. $1.2D + 1.5L_V$

16.4.2 Side Arms and Standoffs

1. 1.4D

16.5 Analysis Models

The minimum acceptable model for the analysis of a mounting system shall be as follows:

1. An elastic three-dimensional model where members subjected to bending from antenna or related appurtenance loading (i.e. low profile horizontal member supporting mounting pipes) are modeled as 3-D beam elements producing moments, shears and axial forces in the members. It shall be permissible to model other members subjected primarily to axial loads as either beam or truss elements.
2. Supports representing the supporting structure for the appurtenance mounting system shall be modeled with rotation and displacement releases corresponding to the type of connections used for the mounting system. Pinned connections shall be considered for all support points to the supporting structure, unless the connection is capable of transferring moment for specific directions and the strength of the members of the supporting structure at the support are adequate to resist the corresponding moment reactions from the appurtenance mounting system.

16.5.1 Application of Forces to Structural Models

Wind, ice, seismic and dead loads from appurtenances shall be applied at the centroid of the appurtenance and transferred to the supporting pipe at attachment locations, considering appropriate axial, shear, flexural and torsional demands. Wind, ice, seismic and dead loads from the members of the mounting system shall be applied at the nodes of the analysis model or alternately considered as uniform loads along the length of the members.

Local bending shall be considered for members supporting appurtenances between supports. Bending shall be considered in combination with axial loads in the members based on the structural analysis.

16.6 Wind and Ice Loads

Wind and ice loads shall be determined in accordance with Section 2.0.

The gust effect factor, G_h, shall be equal to 1.0 for all appurtenance mounting systems.

The directionality factor, K_d, shall be equal to 0.95 for all appurtenance mounting systems.

The shielding factor, K_a, shall be equal to 0.90 for the determination of wind forces for appurtenances and appurtenance mounting system members.

Wind shall be considered to occur from directions in 30 degree maximum increments around a mounting system and shall include directions normal and parallel to the face of mount.

16.6.1 Effective Projected Areas

16.6.1.1 Supported Appurtenances

The effected projected area of each appurtenance supported on a mounting system shall be determined in a direction normal and transverse to the azimuth of each appurtenance in accordance with section 2.6.11.2 with K_a equal to 0.90.

The effective projected area of mounting pipes shall be determined in accordance with 2.6.11.2.1.

16.6.1.2 Mounting Systems

The effected projected area of mounting system members shall be determined in accordance with section 2.6.11.2 with K_a equal to 0.90.

The normal and transverse effected projected areas, $(EPA)_N$, $(EPA)_T$, of mounting system members shall be determined using the projected area of each member onto a plane perpendicular to the direction under consideration (normal and transverse), without regard to shielding or overlapping members of the mounting system or from supported appurtenances, multiplied by the force coefficients from 2.6.11.2. No shielding from the supporting structure shall be considered.

For an integral mounting system, the normal and transverse effective projected areas shall be determined for each individual sector. One sector of the mounting system shall be selected as the reference axis for determining wind direction angles.

The effective projected area determinations for mounting systems of 2.6.11.2.2 through 2.6.11.2.5 represent the effective projected areas to be used in the design or analysis of a supporting structure. These EPA determinations include considerations for shielding and wake interference of the mounting system from the supporting structure. The EPA determinations from 2.6.11.2.2 through 2.6.11.2.5 do not apply to the design or analysis of mounting systems.

16.7 Seismic Load Effects

Seismic load effects shall be determined in accordance with section 2.7 using the equivalent lateral force procedure from 2.7.7.1 using the value of C_s applied to the weight of the appurtenances and the mounting system components. The maximum value of C_s shall not apply in determining seismic load effects for appurtenance supporting systems.

The response modification coefficient, R, shall be equal to 2.0 for mounting systems.

For mounting systems supported on latticed self-supporting, guyed masts or tubular pole structures, the amplification factor, A_s, shall be equal to the amplification factor for the structure determined in accordance with 2.7.8.1. For mounting systems directly supported on buildings or other supported structures (other than latticed self-supporting, guyed masts or tubular pole structures), the amplification factor, A_s, shall be equal to 3.0 (refer to 2.7.8).

Note: The vertical distribution of seismic forces from 2.7.7.1.2 does not apply to mounting systems (the value of N shall be permitted to equal 1).

16.8 Design Strength of Members

The design strength of members of an appurtenance mounting system shall be determined in accordance with Section 4.0.

Combined torsion, flexure and axial loads shall be considered in accordance with AISC 360 for open cross section members utilized for horizontal members supporting mounting pipes or appurtenances for low profile mounting systems (i.e. T-arms, low profile platforms, etc.).

16.9 Existing Appurtenance Mounting Systems

The changed conditions from 15.3 shall require an evaluation of an existing appurtenance mounting system in accordance with 15.5 with the exception for changed conditions that are within the documented capacity of an appurtenance mounting system.

When documentation is not available for determining the Baseline Appurtenance Loading for an appurtenance mounting system, a comprehensive structural analysis in accordance with this Standard shall be required for all Proposed Appurtenances Loadings or for other changed conditions.

It shall be permissible to use different values of L_M and L_V for the evaluation of existing appurtenance mounting systems. The value of L_M and L_V used for the analysis of an existing appurtenance mounting systems shall be reported in the structural analysis report.

The load modification factor for wind load from Annex S does not apply to W_M.

Modifications to existing appurtenance mounting systems shall be completed in accordance with section 15.8.

16.9.1 Source of Data

Documented material and capacity data are additional sources of data from 15.6.1 for existing appurtenance mounting systems.

16.10 Climbing Facilities

Climbing facilities used in conjunction with a mounting system shall meet the requirements of Section 12.0.

16.11 Assembly Documents

Documentation supplied with a mount requiring assembly or modifications shall include the following:

1. The necessary markings and details for the proper assembly of components.
2. Overall dimensions for determining effective projected areas of the mounting system.
3. The weight of the mounting system excluding mounting pipes.
4. Nominal structural dimensions and material grades for members of the mount.
5. Hardware sizes and material grades along with required bolt pretensions or nut locking devices.
6. Interface requirements with a supporting structure including limitations of leg type and size, etc.
7. Strut or tie-back requirements such as attachment point requirements on a supporting structure and minimum and maximum strut angle limitations to the supporting structure or to the main members of the mounting system.
8. Mounting pipe diameter limitations.

17.0 SMALL WIND TURBINE SUPPORT STRUCTURES

17.1 Objective

The objective of this Section is to provide recognized literature intended to be used for the design and analysis of structures supporting Small Wind Turbines (SWT's) defined as wind turbines with rotor swept areas smaller than or equal to 2,200 sq. ft. [200 sq. m].

All provisions of this Standard apply to SWT support structures. This Section defines how specific portions of this Standard apply to SWT supporting structures and provides supplementary requirements that pertain specifically to the unique characteristics of SWT supporting structures. Provisions of this Section have been derived from the IEC 61400-2 Standard, "Wind Turbines - Part 2: Design Requirements for Small Wind Turbines" (IEC) and other industry standards.

The provisions of this Standard are intended to represent the minimum structural standards for SWT support structures. Turbine specific requirements specified by the turbine manufacturer for supporting structures shall take precedence over the minimum requirements of this Standard.

Fatigue design procedures are provided in this Section. Fatigue thresholds are specified for structural members and components typically utilized for SWT support structures.

SWT support structures in conformance with this Section are intended to perform over their design life to an acceptable level of reliability without significant fatigue damage. Unpredictable environmental or turbine loading conditions may produce cyclic stresses exceeding the fatigue thresholds of structural members or components resulting in fatigue cracking during the design life of a SWT support structure. Other variables such as the installation of the structure and workmanship can also affect fatigue life. The maintenance and condition assessment recommendations of this Section are intended to identify and mitigate fatigue and other issues during the design life of a SWT supporting structure.

17.2 Scope

This Section is intended to apply to self-supporting or bracketed latticed towers, guyed masts and tubular pole structures that support small wind turbines that may also support antennas and other appurtenances.

The design and analysis of turbine components are not included within the scope of this Standard.

Strength requirements for erection and maintenance are not within the scope of this Standard. Refer to the ANSI/TIA-322 Standard "Loading, Analysis, and Design Criteria Related to the Installation, Alteration and Maintenance of Communication Structures" for construction considerations and strength requirements.

17.3 General

17.3.1 Design Criteria

SWT supporting structures including foundations shall be in conformance with the requirements of this Standard including the additional requirements of this Section.

Turbine operating loading conditions are not considered in this Section unless specific loading and associated wind conditions are specified by the turbine manufacturer. It is assumed that the requirements specified in this Section govern over the requirements for normal turbine operating conditions.

Conformance with this Section is not required for structures supporting wind turbines with rotor swept areas less than 22 sq. ft. [2 sq. m]. It shall be permissible to consider such wind turbines as appurtenances in accordance with Section 2.0 with the effective projected area of each turbine determined in accordance with section 17.5. The effective projected area shall be considered to be constant for all wind directions. The wind loads from the effective projected area of each turbine shall be considered as wind loads using a load factor equal to 1.0 and a wake interference factor, K_a, equal to 1.0.

Mast type supports for all small wind turbines, regardless of swept area, shall conform to this Section.

17.3.2 Turbine Model

A turbine shall be modeled as a mass and an effective projected area. For the fatigue loading condition specified in section 17.12, wind loads from the effective projected area of the turbine shall be replaced with the equivalent constant-range fatigue turbine loads specified in 17.12.2.

Unless otherwise specified, the center of mass and the centroid of the effective projected area shall be considered to be at the hub height for horizontal axis turbines and the mid-turbine height for vertical axis turbines and assumed to be distributed symmetrically about the vertical centerline of the turbine base connection to the supporting structure.

For investigating the extreme loading conditions specified in sections 17.6 through 17.9, when a horizontal offset of the turbine center of mass from the vertical centerline of the turbine base is specified by the turbine manufacturer, the additional overturning moment on the supporting structure due to turbine weight shall be considered to occur in the direction which adds to the overturning moment from the horizontal force from the turbine.

For the purpose of determining factored extreme loading conditions, turbine weight shall be considered as dead load and turbine forces and moments shall be considered as wind or earthquake loads.

17.3.3 Definitions

Equivalent constant-range fatigue load: a static load used for the analysis of a SWT supporting structure to determine nominal member stresses that represent an equivalent constant-range fatigue stress in the members of the structure.

Equivalent constant-range fatigue stress: a constant-amplitude stress range intended to represent the fatigue effects of actual variable amplitude loading events over the design life of a SWT supporting structure.

Fatigue failure: visible crack growth due to cyclic loading to an extent that a structure cannot be safely used in service.

Fatigue threshold (ΔF_{TH}): the equivalent constant-range fatigue stress below which a particular detail can withstand an infinite number of repetitions without fatigue failure.

Flange plate: a base, top or intermediate flange welded to a latticed tower leg or tubular pole structure.

Geometry coefficient: a factor based on the geometry of an external flange plate used to calculate a stress concentration factor.

Hub height above turbine base: the height of the center of a wind turbine rotor above a turbine base connection to a supporting structure (for horizontal axis turbines).

Initial tension condition: the equilibrium position of a guyed mast (with corresponding forces in the components of the mast) with guys at their specified installation tension.

Stress concentration factor: a factor applied to a calculated nominal stress to account for the local increase in stress.

Turbine base: the base of a turbine that interfaces with a supporting structure.

17.3.4 Abbreviations

AWEA	American Wind Energy Association Standard AWEA 9.1
EPA	Effective Projected Area (projected area times drag factor)
IEC	International Electrotechnical Commission Standard 61400-2
SWT	Small Wind Turbine

17.4 Turbine Manufacturer Data

The following turbine data shall be provided by the turbine manufacturer:

1. Type of turbine: horizontal or vertical axis.
2. Rotor diameter, ft. [m].
3. Rotational rotor speed at AWEA electrical power rating of turbine, RPM.
4. Hub height (horizontal axis turbines) or mid-turbine height (vertical axis turbines) above turbine base connection to supporting structure, ft. [m].
5. Maximum turbine horizontal thrust in parked stationary position (unfactored), lbs [N].
6. Wind speed at hub height associated with the specified maximum turbine horizontal thrust, mph [m/s].
7. Weight of turbine, lbs [N].
8. Horizontal offset of turbine center of mass from vertical centerline of turbine base, ft. [m].
9. Weight of rotor (blades and hub), lbs [N].
10. Horizontal distance from center of rotor mass to vertical centerline of turbine base, ft. [m].
11. Critical turbine operating moments (unfactored), ft-lbs [N-m].
12. Clearance requirements of turbine blades (considering deflected shape of blades and supporting structure).

13. Connection details for the turbine base to the supporting structure including required tolerances.
14. Natural frequencies and other limitations of the supporting structure to avoid damaging dynamic or resonant conditions, Hertz.

Notes:

1. The maximum turbine horizontal thrust required is the maximum thrust from the turbine in a parked stationary position considering all possible exposures with all components remaining intact and fully exposed to the wind assuming the failure of all pitch or furling mechanisms.
2. The wind speed required is the wind speed at hub height or mid-turbine height used to determine the maximum turbine horizontal thrust. This wind speed is used to determine the total effective projected area of the turbine in accordance with section 17.5. Alternately, the total effective projected area of the turbine may be provided by the turbine manufacturer for the turbine in the worst case orientation to the wind described in Note 1.
3. The strength requirements for the factored extreme wind condition for the turbine in the worst case orientation to the wind are assumed to govern over the strength requirements for turbine operating loading conditions occurring at lower wind speeds (such as IEC Load Case D).
4. Refer to Figure 17-1 for a coordinate system typically used by turbine manufacturers. Thrust is along the X-axis, side-force is along the Y-axis, weight is along the negative Z-axis, roll moment is about the X-axis, pitch moment is about the Y-axis, yaw moment is about the Z-axis.

17.5 Effective Projected Area

The effective projected area of a turbine shall be calculated in accordance with this Section unless the effective projected area is specified by the turbine manufacturer. The effective projected area of a turbine shall be considered to be constant for all wind directions with a wake interference factor, K_a, equal to 1.0.

The total effective projected area of the turbine, $(EPA)_W$, shall be calculated in accordance with the following equation:

$$(EPA)_W = \frac{F_{T\max}}{0.00256\,(V_{max})^2} \quad (\text{ft.}^2)$$

$$(EPA)_W = \frac{F_{T\max}}{0.613\,(V_{max})^2} \quad [\text{m}^2]$$

where:

F_{Tmax} = maximum unfactored turbine horizontal thrust for the parked stationary position specified by the turbine manufacturer, lbs [N]

V_{max} = wind speed specified by the turbine manufacturer at hub height associated with the specified maximum unfactored turbine horizontal thrust, mph [m/s]

17.6 Extreme Wind Condition

The basic wind speed (3-second gust at 33 ft. [10 m] height) used for investigating the extreme wind loading condition shall be the larger of the basic wind speed determined in accordance with Section 2.0 and 140 mph [63 m/s] regardless of the risk category of the structure.

Notes:

1. The design criteria for fatigue specified in section 17.12 is based on the supporting structure satisfying a minimum strength requirement for an extreme wind loading condition based on a 140 mph [63 m/s] basic wind speed, Exposure Category C, Topographic Category 1. Lower strength requirements would require extensive fatigue investigations of the supporting structure, the foundation and the soil supporting the foundation that are not within the scope of this Section as well as requiring investigation of possible governing turbine operating loading conditions.
2. When the effective projected area, $(EPA)_W$, of the turbine is less than 50% of the total effective projected area of all discrete appurtenances supported on the structure (turbine and discrete stationary appurtenances such as antennas), the minimum basic wind speed of 140 mph [63 m/s] may be linearly reduced from 140 mph [63 m/s] to 114 mph [51 m/s] as the ratio of the turbine $(EPA)_W$ to the total discrete appurtenance EPA varies from 0.50 (50%) to 0.10 (10%).
3. The basic wind speed at 33 ft. [10 m] above grade required for use with this Section is based on reliability requirements for the structure and is independent of the wind speed based on the IEC classification of a turbine.

17.7 Extreme Ice Condition

The design ice thickness and corresponding basic wind speed shall be determined from Section 2.0. Unless more accurate data are provided for the turbine, the weight of the turbine shall be increased by 25% and the calculated $(EPA)_W$ of the turbine shall be increased by 15% from the no-ice condition.

17.8 Extreme Earthquake Condition

The masses of the turbine, the structure and all appurtenances shall be included in the determination of seismic load effects in accordance with section 2.7.

Unless otherwise specified by the turbine manufacturer, normal turbine operating loads shall be considered insignificant compared to extreme earthquake loading and need not be considered to occur simultaneously with the extreme earthquake loading condition.

17.9 Critical Turbine Moments

Critical overturning or twisting moments from operating conditions including braking, electrical short circuits, shut down, maximum rotational speed condition, extreme yawing, etc., shall be specified by the turbine manufacturer when required for proper operation or support of the turbine.

When a critical turbine moment is specified by the turbine manufacturer, the moment shall be investigated in conjunction with a reduced extreme wind loading condition without ice. Unless otherwise specified by the turbine manufacturer, the specified moment shall be considered to occur at the turbine base connection to the supporting structure simultaneously with a reduced extreme wind loading condition based on a basic wind speed of 40 mph [18 m/s], regardless of risk category of the structure, using the calculated effective projected area of the turbine $(EPA)_W$.

Turbine moments shall be factored considering a 1.6 minimum load factor times the Critical Moment Importance Factor, I_{CM}, specified in Table 17-1 based on the risk category of the supporting structure. A specified turbine overturning moment shall be considered to occur in the same direction as the wind. A specified turbine twisting (yaw) moment shall be considered to act about the vertical centerline of the turbine base in a counterclockwise direction in the plan view.

17.10 Stiffness Requirements for Top Mounted Turbines

Stiffness of the supporting structure shall be investigated under the service load condition in accordance with section 2.8 (60 mph [27 m/s] basic wind speed without ice) with the calculated effective projected area of the turbine, $(EPA)_W$, supported at a horizontal distance equal to 1/12 the rotor diameter from the centerline vertical axis of the support structure. The stiffness of the supporting structure (assuming rigid foundations) shall result in a total top deflection under this loading no greater than 1% of the structure height and a torsional rotation no greater than 0.010 degrees/ft. [0.033 degrees/m] of height.

17.11 Dynamic Requirements

The natural frequency modes involving single, double and triple curvature of the supporting structure shall be determined for a no-ice condition. One of the elastic three-dimensional models specified in Section 3.0 shall be used to determine the fundamental frequency modes. The simplified fundamental frequency equations provided in section 2.7 shall not be used to investigate dynamic requirements for SWT supporting structures. The masses of the turbine, the structure and all appurtenances shall be included in the structural model at the proper locations.

Unless a detailed analysis is undertaken to determine an appropriate foundation spring constant to be used in the determination of natural frequencies, the calculated natural frequencies of the structure shall be adjusted +/- 0.10 Hertz for comparison to the turbine manufacturer's specified natural frequencies to be avoided. When frequency ranges or min/max frequencies are provided by the turbine manufacturer, no adjustments to the calculated natural frequencies of the supporting structure are required.

Notes:

1. Natural frequency modes involving torsion shall be investigated when specified by the turbine manufacturer. When torsional modes are required to be investigated, the eccentricity of turbine mass to the vertical centerline of the supporting structure shall be included in the frequency analysis.
2. Dynamic analysis of the turbine and structural support system incorporating damping considerations and wind velocity time histories are beyond the scope of this Standard. Natural frequency limitations or other structural requirements for the supporting structure to avoid damaging dynamic or resonant conditions shall be provided by the turbine manufacturer.

17.12 Design for Fatigue

An elastic analysis shall be performed in accordance with 17.12.3 using the equivalent constant-range fatigue wind loading from 17.12.1 and the equivalent constant-range fatigue turbine loads from 17.12.2. The resulting nominal member stresses shall be considered as equivalent constant-range fatigue stresses. Equivalent constant-range fatigue stresses shall not exceed the fatigue thresholds, ΔF_{TH}, for the critical members and components of the supporting structure specified in 17.12.4. Other conventional structural members and components that are not listed in 17.12.4

shall be considered to have adequate fatigue strength when the minimum strength requirements are satisfied for the extreme wind loading condition specified in section 17.6.

Notes:

1. The fatigue design provisions are based on limiting cyclic stresses in the members and components of the supporting structure to stress levels that would be expected to prevent the initiation of a fatigue crack from a defect, discontinuity or stress concentration. Unless otherwise specified, nominal member stresses from the fatigue analysis need not be amplified by stress concentration factors as the fatigue thresholds, ΔF_{TH}, specified in this Section correspond to defined details and components categorized according to their level of stress concentration, inherent weld defects and notch sensitivity.
2. The fatigue design provisions are based on limiting nominal member stress ranges to stress levels corresponding to the fatigue threshold of the respective details and connections to provide infinite life; therefore, the number of cycles based on the design life of the structure is not required for a fatigue analysis performed in accordance with this Section.
3. The fatigue strength requirements of this Section are based on the normal operating forces expected from a properly operating and maintained turbine. The turbine manufacturer shall provide more stringent fatigue strength criteria when the requirements of this Section are not adequate for a specific turbine.

17.12.1 Equivalent Constant-Range Fatigue Wind Loading on Supporting Structure

Fatigue wind loading on the supporting structure and supported appurtenances (excluding the turbine) shall be considered as an additional service load condition in accordance with section 2.8 (wind direction probability factor, K_d, equal to 0.85 for all support structure types) using a 30 mph [13 m/s] uniform wind speed (K_z, K_{zt}, K_s, K_e and G_h equal to 1.0) times the fatigue importance factor, I_f, specified in Table 17-1. Force coefficients for all structural members and appurtenances shall be based on subcritical conditions.

The fatigue loading on the supporting structure shall be considered to occur simultaneously with the equivalent constant-range fatigue turbine loads specified in 17.12.2.

Note: The 30 mph [13 m/s] wind speed represents the wind speed range to be considered for fatigue analysis and is not a wind speed for investigating serviceability or operational conditions.

17.12.2 Equivalent Constant-Range Fatigue Turbine Loads

Equivalent constant-range fatigue loads for horizontal axis turbines shall be calculated from the following equations:

F_{xt} = equivalent constant-range fatigue turbine horizontal force

$= 0.85(I_f)(C_{fxt})(D_r)^2$ lbs [N]

M_{ty} = equivalent constant-range fatigue turbine overturning moment (pitch moment)

$= 0.85[2(W_{tr})(L_{rc})] + 0.0833(D_r)(F_{xt})$ ft-lbs [N-m]

M_{tx} = equivalent constant-range fatigue turbine rotor shaft torsion (roll moment)

$= 0.85[(I_f)(C_{mtx})(D_r)^2 / N_r + 0.005(W_{tr})(D_r)]$ ft-lbs [N-m]

where:

0.85 = wind direction probability factor (applicable for all support structure types)

I_f = fatigue importance factor from Table 17-1

C_{fxt} = 1.0 [48]

D_r = rotor diameter, ft. [m]

W_{tr} = weight of rotor (hub and blades), lbs [N]

L_{rc} = horizontal distance from center of rotor mass to vertical centerline of the supporting structure, ft. [m]

C_{mtx} = 275 [4000]

N_r = rotational rotor speed at AWEA electrical power rating of turbine, RPM

Notes:

1. The 0.85 wind direction probability factor accounts for the probability of the load range occurring from a direction that creates a response in any one given support structure component.
2. C_{fxt} and C_{mtx} are conversion factors derived from the IEC normal operation load case for the measure of units indicated.
3. F_{xt} represents the turbine horizontal force and includes wind loading on the turbine; therefore, the effective area of the turbine, $(EPA)_W$, is not included in the horizontal force calculation.
4. M_{ty} represents the pitch moment and M_{tx} represents the roll moment with respect to the horizontal axis of the turbine. Yaw moment (about a vertical axis) is not considered for investigating fatigue strength unless otherwise specified by the turbine manufacturer.

The horizontal force, F_{xt}, shall be applied concentrically in the direction of the wind at the hub height of the turbine. The overturning moment, M_{ty}, shall be applied at the hub height in a vertical plane in the direction which adds to the overturning moment resulting from F_{xt}. The moment, M_{tx}, shall be applied at the hub height in a vertical plane normal to the wind direction (rotor shaft torsion).

The unit direction vector for M_{tx} shall be in the direction of the wind. Alternately, the moments M_{ty} and M_{tx} may be combined into a resultant overturning moment and applied in the direction which adds to the overturning moment resulting from F_{xt}.

The turbine weight shall be considered to be located at the centerline of the turbine base connection to the supporting structure.

Equivalent constant-range fatigue loads for vertical axis turbines shall be provided by the turbine manufacturer. The equivalent constant-range fatigue loads shall be based on the turbine cycling between 50% and 150% of the rated power at a 30 mph [13 m/s] constant wind speed at the mid-turbine height. Fatigue loads shall include the effects of eccentric wind loading on the turbine and the effects of eccentric rotor mass.

17.12.3 Fatigue Analysis

17.12.3.1 Self-Supporting or Bracketed Structures

Analysis of tubular pole and latticed self-supporting or bracketed structures shall be performed using a load factor of zero for dead load and a load factor of 1.0 for all other loads. The stress range in each component shall be considered to be equal to the absolute value of the stress in the component.

17.12.3.2 Cantilever Portions of Guyed Masts

The cantilever portion of a guyed mast shall be analyzed as a self-supporting structure in accordance with 17.12.3.1.

17.12.3.3 Guyed Masts below the Cantilever

17.12.3.3.1 Latticed Masts

The full height of the mast with the cantilever shall be analyzed using a load factor equal to 1.0 for all loads. The results of the initial tension condition and the results of the fatigue analysis shall be used to determine the stress ranges in the components of the mast.

Leg members below the cantilever that are subjected solely to axial compression from the fatigue analysis need not be investigated for fatigue.

The stress range in leg members below the cantilever subjected to axial tension from the fatigue analysis shall be considered equal to the sum of the axial tension stress in the leg from the fatigue analysis and the absolute value of the axial stress in the leg from the initial tension condition.

The stress range in bracing members shall be equal to the absolute value of the stress in the bracing members from the fatigue analysis.

17.12.3.3.2 Tubular Pole Masts

The full height of the mast with the cantilever shall be analyzed using a load factor equal to 1.0 for all loads. The results of the initial tension condition and the results of the fatigue analysis shall be used to determine the stress ranges in the components of the mast.

Tubular mast components below the cantilever with cross sections that are subjected solely to compression stresses (due to combined axial load and bending) from the fatigue analysis need not be investigated for fatigue.

The stress ranges in a tubular mast component below the cantilever subjected to tension stresses (due to combined axial load and bending) from the fatigue analysis shall be considered equal to the sum of the maximum tensile stress in the component from the fatigue analysis and the absolute value of the maximum stress in the component from the initial tension condition.

17.12.4 Fatigue Thresholds (ΔF_{TH})

The stresses calculated from the fatigue analysis shall be considered as equivalent constant-range fatigue stresses and shall not exceed the fatigue thresholds, ΔF_{TH}, specified in 17.12.4.1 through 17.12.4.7.

Cantilevered tubular type supports of SWT's attached to the supporting structure shall be considered as tubular pole structures for the purposes of determining fatigue thresholds, ΔF_{TH}.

The stress to be calculated from the fatigue analysis, unless otherwise indicated, is the nominal stress in the load carrying member or at the location of a detail defining the fatigue threshold (refer to Table 17-10).

It shall be permissible to determine the nominal stress at the nominal centerline elevation of a detail under investigation, including the investigation at the ends of welded attachments, the top and bottom of ground sleeves or collars, etc. The nominal stress for the investigation of a flange plate shall be permitted to be based on the nominal stress at the node on the structural model representing the nominal elevation of the flange plate, including the investigation at the termination of longitudinal stiffeners, annular ring plates, backing rings, etc.

Note: The criteria for determining fatigue thresholds are based on the AASHTO Standard, "LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals", First Edition, LRFDLTS-1, 2015, with 2017 interim revisions, American Association of State Highway and Transportation Officials (AASHTO LRFD) and other international standards.

17.12.4.1 Category 1 Components (ΔF_{TH} = 4.5 ksi [31 MPa]):

1. Tubular pole structures at tube-to-tube circumferential weld splices (full-penetration weld).
2. Tubular pole structures at ends of ground sleeves or collars (all-around fillet-weld).
3. Tubular pole structures or latticed tower legs at flanges where the termination of stiffeners are connected to a continuous annular ring plate.
4. Latticed tower legs at flanges.
5. Latticed tower legs at the ends of welded attachments exceeding a 2 in. [50 mm] length along the longitudinal axis of the leg.

17.12.4.2 Category 2 Components (ΔF_{TH} = 2.6 ksi [18 MPa]):

1. Tubular pole structures at internal flange plates.
2. Tubular pole structures at socketed external flange plates with longitudinal stiffeners.
3. Latticed tower legs at flanges with longitudinal stiffeners.
4. Latticed tower main load carrying bracing members with effective slenderness ratios (KL/r) less than 60 with welded end connections.
5. Latticed tower tension-only bracing members with welded end connections.

Note: For a bracing member, the fatigue threshold, ΔF_{TH}, applies to the nominal axial stress in the bracing member, not to the stress in the welded connection.

17.12.4.3 Welded Attachments to Tubular Pole Structures

The fatigue threshold, ΔF_{TH}, at the ends of attachments to tubular pole structures with an attachment length along the pole longitudinal axis 2 in. [50 mm] or greater shall be determined from Table 17-3.

Note: Refer to 17.12.4.6.1 for the fatigue threshold, ΔF_{TH}, for the nominal stress in the pole wall at the termination of longitudinal stiffeners used with flange plates.

17.12.4.4 Reinforced Hand-holes, Cutouts and Ports in Tubular Pole Structures

The fatigue threshold, ΔF_{TH}, for the pole cross section immediately above and below a reinforcing rim shall be equal to 7.0 ksi [48 MPa]. It shall be permissible to calculate the nominal stress in the pole wall using the moment and cross sectional pole properties without the cutout or reinforcing rim at the mid-height elevation of the opening. A stress concentration factor equal to 1.0 shall be applied to the nominal stress when the clear width between the rim reinforcing does not exceed 40% of the outside pole diameter or flat-to-flat width at the mid-height elevation of the opening and equal to 1.5 for larger clear widths.

The fatigue threshold, ΔF_{TH}, for stresses in the pole wall at the intersection with the reinforcing rim at the mid-height elevation of the opening shall be equal to 16 ksi [110 MPa]. The nominal stress shall be based on the moment and cross sectional pole properties with the cutout and reinforcing rim at the mid-height elevation of the opening. A stress concentration factor equal to 4.0 shall be applied to the nominal stress when the clear width between the rim reinforcing does not exceed 40% of the outside pole diameter or flat-to-flat width at the mid-height elevation of the opening and equal to 6.0 for larger clear widths.

The section modulus for the cross section with the cutout and reinforcing rim shall be based on the distance from the centroid of the cross section with the cutout and reinforcing rim to the extreme fiber of the pole cross section at the interface with the reinforcing rim. The section modulus shall be determined for two orthogonal axes; one for the axis of symmetry of the reinforcing rim (axis passing through the center of the opening) and one for an axis normal to the axis of symmetry. The minimum section modulus shall be used for determining the nominal stress on the pole wall.

The pole wall surface around the perimeter of the cutout for the reinforcing rim shall have a surface roughness profile not exceeding 1,000 microinches [25 μm]. The surface roughness requirement shall be considered to be satisfied for thermally cut surfaces when ground smooth.

The inside corner radius of a reinforcing rim shall not be less than 30% of the inside clear width. The inside clear width shall not exceed 50% of the outside pole diameter or flat-to-flat width at the mid-height elevation of the opening.

Reinforcement rims shall be continuous and shall be sized to satisfy all strength requirements. Reinforcement rims shall not be less than 1/4 in. [6 mm] nominal thickness and shall be connected to the pole wall with a minimum 5/16 in. [8 mm] fillet-weld.

The vertical clear distance from a reinforcing rim to the surface of a flange plate shall not be less than the outside pole diameter or flat-to-flat width at the mid-height elevation of the opening.

The fatigue threshold, ΔF_{TH}, for details not covered in this Section shall be determined in accordance with AASHTO LRFD or other recognized standards.

17.12.4.5 Unreinforced Hand-holes, Cutouts and Ports in Tubular Pole Structures

The fatigue threshold, ΔF_{TH}, for stresses in the pole wall at the mid-height elevation of the opening shall be equal to 24 ksi [165 MPa]. The nominal stress shall be based on the moment and cross sectional pole properties with the cutout at the mid-height elevation of the opening. A stress concentration factor equal to 4.0 shall be applied to the nominal stress when the clear width of the opening does not exceed 40% of the pole outside diameter or flat-to-flat width at the mid-height elevation of the opening and equal to 6.0 for larger clear widths.

The pole wall surface around the perimeter of the opening shall have a surface roughness profile not exceeding 1,000 microinches [25 μm]. The surface roughness requirement shall be considered to be satisfied for all drilled holes and for thermally cut surfaces when ground smooth.

The inside corner radius of an unreinforced opening shall not be less than 30% of the inside clear width. The inside clear width shall not exceed 50% of the outside pole diameter or flat-to-flat width at the mid-height elevation of the opening.

The vertical clear distance from an unreinforced opening to the surface of a flange plate shall not be less than the outside pole diameter or flat-to-flat width at the mid-height elevation of the opening.

The fatigue threshold, ΔF_{TH}, for details not covered in this Section shall be determined in accordance with AASHTO LRFD or other recognized standards.

17.12.4.6 Tubular Pole Structure Exterior Flange Plates

The fatigue threshold, ΔF_{TH}, for pole exterior flange plates, except as specified in 17.12.4.1 and 17.12.4.2, shall be determined in accordance with this Section.

The steps required for the determination of fatigue thresholds, ΔF_{TH}, are as follows:

1. Determine the flange plate geometry coefficient from Table 17-4.
2. Determine the flange plate stress concentration factor from Table 17-5.
3. Determine the fatigue threshold, ΔF_{TH}, at the flange plate from Table 17-6.
4. For stiffened joints, determine the fatigue threshold, ΔF_{TH}, at the interface of the stiffeners with the flange plate in accordance with 17.12.4.6.1.

Notes:

1. The inside corner bend radius of a polygonal section is required to determine the fatigue threshold, ΔF_{TH}. For the purpose of determining the fatigue threshold, ΔF_{TH}, at flange plates when the inside corner bend radius is not known, it shall be permissible to assume an inside bend radius equal 1.5 times the pole wall thickness.
2. The center opening diameter is required to determine the fatigue threshold, ΔF_{TH}, of butt joints. The use of holes or slots not exceeding 2 in. [50 mm] in diameter or width with a 90 degree minimum separation for galvanizing drainage or venting shall be considered to not reduce the fatigue threshold, ΔF_{TH}. For other hole or slot geometries, the fatigue threshold, ΔF_{TH}, shall be determined assuming a center opening diameter equal to the diameter of a circle inscribing the openings. Diamond shaped center openings shall be considered as a circular center opening with a diameter equal the longer dimension of the opening.

The fatigue threshold, ΔF_{TH}, for external flange plates not covered in this Section shall be determined in accordance with AASHTO LRFD or other recognized standards.

17.12.4.6.1 Longitudinal Stiffener Interface with Flange Plates

The fatigue threshold, ΔF_{TH}, at the interface with a flange plate (through the stiffener thickness and through the weld throat of fillet-welds when utilized) for longitudinal stiffeners with a thickness greater than 0.50 in. [13 mm], shall be determined from the following equations:

$$\Delta F_{TH} = 10.0\left[0.0055 + 0.72\left(\frac{E_w}{t_{ST}}\right)\right]\left(t_{ST}^{-0.17}\right) \le 10.0 \text{ (ksi)}$$

$$\Delta F_{TH} = 69.0\left[0.0940 + 1.23\left(\frac{E_w}{t_{ST}}\right)\right]\left(t_{ST}^{-0.17}\right) \le 69.0 \text{ [MPa]}$$

where:

t_{ST} = stiffener thickness, in. [mm]

E_w = effective weld throat per unit length at the stiffener to flange plate interface, but not larger than t_{ST}

The nominal stress at the stiffener interface with a flange plate shall be based on the gross cross section of the pole including the stiffeners. The nominal stress shall be calculated at the outermost tip of the stiffener based on an elastic distribution of stresses. Reinforcing fillet-welds shall be ignored in the calculation of the elastic section modulus for determining elastic stresses.

The fatigue threshold, ΔF_{TH}, at the interface with the flange plate applies to both the nominal stress in the stiffener and to the nominal stress on the effective throat of a fillet-weld connecting a stiffener to the flange plate. (Example: for an all-around fillet-weld with a combined effective throat equal to 50% of the stiffener thickness, a 5 ksi [34 MPa] calculated nominal stress for the stiffener would result in a 10 ksi [69 MPa] nominal stress on the effective weld throat.)

17.12.4.7 Anchor Rods

The fatigue threshold, ΔF_{TH}, for anchor rods shall be equal to 7.0 ksi [48 MPa]. Anchor rod forces shall be determined based on an elastic distribution. Nominal stresses shall be calculated using the tensile root area of the anchor rod.

For base plates supported on leveling nuts, when the clear distance between the bottom of the leveling nut and the top of concrete is less than the nominal anchor rod diameter, bending stresses in the anchor rod may be ignored. For larger clear distances, the distance between the top of concrete and the bottom of the leveling nut (gap dimension) shall be used to determine individual anchor rod bending moments. It shall be permissible to assume an inflection point equal to 0.65 times the gap dimension for determining individual anchor rod bending moments. Anchor rod bending stresses shall be determined using the elastic section modulus of the anchor rod based on the tensile root diameter of the anchor rod. Grout, when utilized, shall be ignored when determining anchor rod stresses.

For anchor rods arranged in a symmetrical circular pattern, the following equations apply:

d_n = tensile root diameter of anchor rod

$= d - 0.9743 / n$ in.

$= d - 0.9382(p)$ [mm]

A_n = net area of anchor rod through the threaded portion

$= [\pi(d_n)^2] / 4$ (in.)^2 [mm]^2

S_{AR} = elastic section modulus of anchor rod

$= [\pi(d_n)^3] / 32$ (in.)^3 [mm]^3

P_{AR1} = anchor rod axial force due to an applied vertical reaction

$= P_F / N_{AR}$ kips [kN]

P_{AR2} = anchor rod axial force due to an applied resultant overturning moment reaction

$= 4(M_F) / [N_{AR}(D_{BC})]$ kips [kN]

V_{AR1} = anchor rod shear force due to an applied resultant shear reaction

$= 2(V_F) / N_{AR}$ kips [kN]

V_{AR2} = anchor rod shear force due to an applied torsional moment reaction

$= 2(T_F) / [N_{AR}(D_{BC})]$ kips [kN]

M_{AR} = anchor rod bending moment due to anchor rod shear forces

$= (V_{AR1} + V_{AR2})(0.65)(I_{ar})$ inch-kips [kN-mm]

F_{AR} = nominal stress in anchor rod

$= (P_{AR1} + P_{AR2}) / (A_n) + M_{AR} / S_{AR}$ ksi [MPa]

where:

d = nominal diameter of anchor rod, in. [mm]

n = number of threads per inch

p = pitch of threads, mm

P_F = applied vertical reaction on anchor rod group, kips [kN]

N_{AR} = number of anchor rods

M_F = applied resultant overturning moment reaction on anchor rod group, inch-kips [kN-mm]

D_{BC} = anchor rod bolt circle, in. [mm]

V_F = applied resultant shear reaction on anchor rod group, kips [kN]

T_F = applied torsional moment reaction on anchor rod group, inch-kips [kN-mm]

l_{ar} = length from top of concrete to bottom of anchor rod leveling nut, in. [mm]

Anchor rods that are not supported on leveling nuts and that are post tensioned to reduce cyclic stresses to insignificance are exempt from the requirements of this Section.

17.12.5 Miscellaneous Fatigue Strength Requirements

17.12.5.1 Connection Bolts for Turbine Bases

The number, size, arrangement and grade of turbine base connection bolts shall be specified by the turbine manufacturer. The minimum grade of connection bolts shall be ASTM A325. Bolts shall be pre-tensioned to a minimum of 70% of their ultimate tensile strength.

17.12.5.2 Anchor Rods

Anchor rods shall conform to the requirements of ASTM F1554 for smooth anchor rods and ASTM A615 for deformed anchor rods. ASTM F1554 Grade 105 and A615 anchor rods shall have a minimum Charpy V-Notch impact strength of 15 ft-lbs at -20 degrees F [20 J at -29 degrees C].

Beveled washers shall be provided for the lower and upper nuts when anchor rods are installed with a misalignment greater than 1:40 (1.43 degrees from vertical).

17.12.5.3 Latticed Structures

The maximum effective slenderness ratios for members, (KL/r), and the minimum gusset plate thicknesses for member connections shall be determined from Table 17-2.

17.12.5.4 Guyed Mast Guy Anchorages

Guy connection plates used to connect guys to an anchorage shall be limited to designs using pinned guy connection plates.

17.12.5.5 Tubular Pole Structures

Tubular pole structures shall be of round or polygonal cross sections. Polygonal cross sections shall have a minimum of 8 sides.

17.12.5.6 Tubular Pole Structure Flange Plates

Flange plate connection bolts and anchor rods shall be on a symmetrical circular pattern. A minimum of 8 connection bolts or anchor rods shall be used with a minimum nominal diameter equal to 1 in. [25 mm] for pole structures with an outside diameter or flat-to-flat width 8 in. [200 mm] or greater. The spacing between connection bolts or anchor rods measured circumferentially along the bolt circle shall not exceed 6 times the flange plate thickness but in no case shall the spacing exceed 15 in. [380 mm].

Flange plate thickness shall not be less than 1.5 in. [38 mm] for outside pole diameters up to 8 in. [200 mm] and not less than 2 in. [50 mm] for larger outside diameters. The diameter of a polygonal cross section shall be based on the outside flat-to-flat width. In addition, flange plate thickness shall not be less than the connection bolt or anchor rod nominal diameter for anchor rods with required minimum guaranteed yield strengths exceeding 75 ksi [520 MPa] and for lower strengths anchor rods, not less than the anchor rod diameter minus 0.25 in. [6 mm].

Butt joints shall be full-penetration groove-welds with a reinforcing fillet-weld.

For unstiffened external flange plates and all internal flange plates, the bolt circle diameter shall be within 3 times the thickness of the flange plate from the center of the adjacent pole reinforcing fillet-weld.

The center opening for butt joints shall not exceed 75% of the pole outside diameter or flat-to-flat width. The perimeter of round center openings shall not be closer than 2 times the flange plate thickness to the inside pole wall.

The minimum inside corner bend radius for polygonal cross section poles with flange plates shall not be less than 3 times the wall thickness of the cross section and shall be uniform throughout the arc of the bend.

The fatigue thresholds specified for flange plates with longitudinal stiffeners are based on stiffeners that are designed for the extreme loads specified in this Section based on a rational method to limit stiffener stresses applied to the pole wall. This includes both normal and parallel stresses applied to the pole wall from eccentric anchor rod or connection bolt forces. Stresses shall be limited to prevent buckling or tear-out of the pole wall and to prevent local buckling or rupture of the stiffeners as a load bearing component.

The stiffener weld to the pole wall and to the flange plate may be a full-penetration weld, a partial-penetration weld or an all-around fillet-weld with an effective throat as required for strength requirements. When a fillet-weld is utilized for the connection to the pole wall, a transition radius shall not be utilized and the wrap around fillet-weld shall not be ground.

Backing rings for use with butt joints shall not exceed a 2 in. [50 mm] height and shall be attached with a continuous weld to either the pole wall at the top of the backing ring or to the flange plate at the bottom of the backing ring or at both locations. The connection shall be permitted to be a continuous fillet-weld, a partial-penetration weld or a full-penetration weld.

17.12.5.6.1 Butt Welded Flange Plate Connections

Butt welded connections shall be used for all external flanges, with or without the use of stiffeners, for tubular pole sections greater than 24 in. [610 mm] in outside diameter or outside flat-to-flat width and for all internal flanges. Butt welded connections shall be full-penetration groove welds with reinforcing fillet-welds.

17.12.5.6.2 Socketed Flange Plate Connections

Socketed connections, with or without the use of stiffeners, shall be limited to exterior flange plates and to tubular pole sections with outside diameters or outside flat-to-flat widths 24 in. [610 mm] or less.

17.13 Other Structural Materials

This Section has been developed primarily for steel SWT supporting structures but may also be applied to other materials using appropriate resistance factors to result in an equivalent level of reliability. Resistance factors shall not be higher than the values specified in 17.13.1 and 17.13.2.

Appropriate considerations unique to a material such as UV degradation, temperature, moisture, aging, effect of lightning strikes, etc. shall be based on industry standards for the material.

The stiffness requirements specified in section 17.10 may be waived when the turbine and the supporting structure are modeled as a system using a structural dynamic and aeroelastic model and/or when full-scale testing in accordance with IEC is used to establish appropriate design criteria and to verify that there are no expected blade strikes or damaging dynamic or resonant conditions. The limit state deformations specified in Section 2.0 for the service load condition shall apply as a minimum requirement of stiffness (4 degrees maximum twist or sway and maximum horizontal displacement equal to 3% of height).

17.13.1 Extreme Loading Conditions

The nominal strengths for extreme loading conditions for material other than steel shall be based on the minimum strengths guaranteed by the manufacturer of the material or alternately, based on tests to determine strengths of 95% survival probability with a 95% confidence limit. Resistance factors applied to nominal strengths shall be in accordance with Table 17-8.

17.13.2 Fatigue Loading Condition

Fatigue thresholds, ΔF_{TH}, for all components shall be equal to the fatigue stress range at or below the stress range that may be repeated for an infinite number of cycles without initiating fatigue damage. Nominal fatigue thresholds, ΔF_{TH}, shall be determined by testing. The appropriate resistance factors from Table 17-9 shall be applied to nominal values to determine fatigue thresholds, ΔF_{TH}, for use with this Section.

For materials that do not display a fatigue threshold, ΔF_{TH}, the nominal fatigue stress range for use with this Section shall be equal to the limiting stress range determined for 5 million cycles.

Note: The fatigue thresholds, ΔF_{TH}, specified in 17.12.4 for steel components include appropriate resistance factors for steel components manufactured in accordance with this Standard and maintained and inspected in accordance with section 17.15.

17.14 Foundations

Foundations shall be designed in accordance with this Standard to support the reactions from the loading conditions specified in sections 17.6 through 17.9.

In addition, for the reactions from the stiffness investigation load condition specified in section 17.10, mat foundations for self-supporting structures shall be sized such that compressive soil bearing stresses exist over the full plan dimension of the foundation and drilled shafts, pile foundations and direct embed poles subjected to lateral load shall be sized considering repetitive lateral loading soil conditions.

17.15 Maintenance and Condition Assessment

The maintenance and condition assessment of SWT supporting structures shall be in accordance with this Standard except that the interval period shall be 6 months for all supporting structure types during the first 5 years of continuous operation. It shall be permissible to extend the interval period to 12 months in the event that no fatigue damage is discovered during the first 5 years of continuous operation.

Additional condition assessments shall be performed after any unusual loading events due to turbine operational issues such as a loss of a blade, bearing failures and other malfunctions.

Notes:

1. Fatigue cracks may be expected to occur in structural components after rare severe loading conditions due to turbine malfunctions.
2. Although the fatigue design provisions of this section are believed to be conservative, the maintenance and condition assessment recommendations are intended to identify and mitigate fatigue and other issues in a structure prior to a catastrophic failure.

17.16 Modification of Support Structures

Modifications to SWT support structures including the addition of appurtenances may affect natural frequencies or the fatigue thresholds of structural members and components. Modifications shall not be performed without confirming compliance with the requirements of this Section.

Table 17-1: Importance Factors

Risk Category	Critical Moment Importance Factor, I_{CM}	Fatigue Importance Factor, I_f
I	0.87	0.90
II	1.00	1.00
III	1.15	1.35
IV	1.25	1.50

Table 17-2: Latticed Structure Limitations

AWEA Turbine Power Rating	Maximum Effective Slenderness Ratio of Members (KL/r)	Minimum Gusset Plate Thickness
Up to 10 kW	200	3/16 in. [5 mm]
Over 10 kW to 25 kW	185	1/4 in. [6 mm]
Over 25 kW	175	3/8 in. [10 mm]

Table 17-3: Fatigue Thresholds, ΔF_{TH}, at Longitudinal Attachments to Tubular Pole Structures

Length of Longitudinal Attachment	ΔF_{TH}
2 in. [50 mm] $\leq L \leq 12(t_A)$ and 4 in. [100 mm]	7.0 ksi [48 MPa]
$L > 12(t_A)$ or 4 in. [100 mm] when $t_A \leq 1$ in. [25 mm]	4.5 ksi [31 MPa]
$L > 12(t_A)$ or 4 in. [100 mm] when $t_A > 1$ in. [25 mm]	2.6 ksi [18 MPa]
L = length of attachment measured along longitudinal axis of pole, in. [mm] t_A = attachment thickness, in. [mm]	

Table 17-4: Geometry Coefficients for Tubular Pole Structure External Flange Plates

Joint Type/Location	Geometry Coefficients (refer to limiting values of variables below)
Round Pole Cross Sections at Flange Plate	$G_r = 1.00$
Polygonal Pole Cross Sections at Flange Plate	$G_r = 1.00 + (D_T - r_b)(N_S^{-2.00})$
Socketed Joints	$G_a = G_r[2.20 + 4.6(15t_T + 2)(D_T^{1.20} - 10)(t_{TP}^{-2.50})(C_{BC}^{0.03} - 1)]$
Butt Joints	$G_b = G_r\left[1.35 + 16(15t_T + 1)(D_T - 5)(t_{TP}^{-2.00})\left(\dfrac{C_{BC}^{0.02} - 1}{4C_{OP}^{-0.70} - 3}\right)\right]$
Butt Joints with Stiffeners at Stiffener Terminations	$G_c = \left(\dfrac{t_{ST}^{0.40}}{t_T^{0.70}} + 0.3\right)\left(\dfrac{0.4D_T^{0.80}}{N_{ST}^{1.20}} + 0.9\right)$
Butt Joints with Stiffeners at Stiffener Base	$G_d = G_a\left(130\dfrac{D_T^{0.15}}{N_{ST}^{1.50}} + 1\right)\left(\dfrac{0.13}{h_{ST} + 7}\right)\left(\dfrac{6.5}{t_{ST}^{0.50}} - 1\right)$

$C_{BC} = D_{BC}/D_T$ (when ratio is < 1.25, use 1.25 to calculate G_a and G_b)

$C_{OP} = D_{OP}/D_T$ (when ratio is < 0.30, use 0.30 to calculate G_b)

D_{BC} = Bolt circle diameter, in.

D_{OP} = Inside center opening diameter, in. (refer to 17.12.4.6)

D_T = pole outside diameter for round cross sections or outside flat-to-flat width for polygonal cross sections at the intersection of the pole with the flange plate, in.

N_S = number of sides for polygonal cross sections (when > 16, use 16 to calculate G_r)

N_{ST} = number of longitudinal stiffeners (when > 12, use 12 to calculate G_c and G_d)

h_{ST} = longitudinal stiffener height, in.

r_b = inside corner bend radius (when > 4 in., use 4 in. to calculate G_r)

t_{ST} = longitudinal stiffener thickness, in.

t_T = pole wall thickness, in.

t_{TP} = flange plate thickness, in.

Notes:

1. The limiting values specified in the terms defined above apply to the determination of geometry coefficients. Example: For an 18-sided polygonal section, use 16 as the number of sides to calculate G_r and for a 12-sided polygonal section, use 12 as the number of sides to calculate G_r
2. The term Ga is used in the determination of the geometry coefficient for butt joints with stiffeners as the increase in fatigue strength using appropriately designed and detailed stiffeners (refer to Table 17-7) has been correlated to the increase in fatigue strength for socketed joints.
3. For SI units, conversion from millimeters to inches is required.

Table 17-5: Stress Concentration Factors for Tubular Pole Structure External Flange Plates

Joint Type/Location	Stress Concentration Factors
Socketed Joints	$K_I = G_a[(1.76 + 1.83t_T) - 4.76(0.22)^{G_a}]$
Butt Joints	$K_I = G_b[(1.76 + 1.83t_T) - 4.76(0.22)^{G_b}]$
Butt Joints with Stiffeners at Stiffener Terminations	$K_I = G_c[(1.76 + 1.83t_T) - 4.76(0.22)^{G_c}]$
Butt Joints with Stiffeners at Stiffener Base	$K_I = G_d[(1.76 + 1.83t_T) - 4.76(0.22)^{G_d}]$

t_T = pole wall thickness, in.

$G_{(n)}$ = geometry coefficients from Table 17-4

Note: For SI units, conversion from millimeters to inches is required.

Table 17-6: Fatigue Thresholds, ΔF_{TH}, for Tubular Pole Structure External Flange Plates

Joint Type/Location	Table 17-7 Limitations Satisfied		Table 17-7 Limitations Not Satisfied
	K_I	ΔF_{TH}	ΔF_{TH} [(1, 2)]
Socketed Joints	$K_I \leq 4.0$	7.0 ksi [48 MPa]	2.6 ksi [18 MPa]
	$4.0 < K_I \leq 6.5$	4.5 ksi [31 MPa]	
	$6.5 < K_I \leq 7.7$	2.6 ksi [18 MPa]	
Butt Joints	$K_I \leq 3.0$	10.0 ksi [69 MPa]	4.5 ksi [31 MPa]
	$3.0 < K_I \leq 4.0$	7.0 ksi [48 MPa]	
	$4.0 < K_I \leq 6.5$	4.5 ksi [31 MPa]	
Butt Joints with Stiffeners at Stiffener Terminations	$K_I \leq 5.5$	7.0 ksi [48 MPa]	2.6 ksi [18 MPa] [(3)]
Butt Joints with Stiffeners at Stiffener Base	$K_I \leq 4.0$	7.0 ksi [48 MPa]	4.5 ksi [31 MPa]
	$4.0 < K_I \leq 7.7$	4.5 ksi [31 MPa]	

Notes:

1. The fatigue threshold, ΔF_{TH}, applies to the nominal stress in the pole cross section at the intersection of the pole with the flange plate.
2. The ΔF_{TH} value for the stiffener termination location applies to the nominal stress in the pole wall at the termination of the longitudinal stiffeners on the pole wall.
3. The ΔF_{TH} value for the stiffener base location applies to the nominal stress in the pole wall at the flange plate.
4. The nominal stress at both stiffener locations shall be based on the cross section of the pole without the stiffeners. (The geometry coefficients specified in Table 17-4 account for the effects of stiffeners.)
5. Backing rings shall be ignored for all nominal stress calculations.

Footnotes:

1. ΔF_{TH} values may conservatively be used in lieu of calculating stress concentration factors.
2. Conformance to the limitations of 17.12.5.5 and 17.12.5.6 is required.
3. When the outside pole diameter or flat-to-flat width is the only limiting value exceeded in Table 17-7, the value of ΔF_{TH} shall be equal to 4.5 ksi [31 MPa] at the termination of a longitudinal stiffener.

Table 17-7: Limitations [1] for the Use of Stress Concentration Factors to Determine ΔF_{TH}

Criteria	Socketed Joint (Fillet-Welded)	Butt Joint (Full-Penetration)	Stiffened Butt Joint (Full-Penetration)
Pole Diameter (D_T)	8 to 24 in. [200 to 610 mm]	8 to 50 in. [200 to 1,270 mm]	24 to 50 in. [610 to 1,270 mm]
Pole Wall Thickness (t_T)	0.18 to 0.50 in. [5 to 13 mm]	0.18 to 0.63 in. [5 to 16 mm]	0.25 to 0.63 in. [6 to 16 mm]
Number of Sides For Polygonal Sections	≥ 8 or $(5.0D_T)^{0.5}$ for D_T (in.) $(0.2D_T)^{0.5}$ for D_T [mm]	≥ 8 or $(5.0D_T)^{0.5}$ for D_T (in.) $(0.2D_T)^{0.5}$ for D_T [mm]	≥ 8 or $(5.0D_T)^{0.5}$ for D_T (in.) $(0.2D_T)^{0.5}$ for D_T [mm]
Inside Corner Bend Radius for Polygonal Sections	≥ $5(t_T)$ 1 in. [25 mm] minimum	≥ $5(t_T)$ 1 in. [25 mm] minimum	≥ $5(t_T)$ 1 in. [25 mm] minimum
Bolt Circle Diameter	≤ $2.5(D_T)$	≤ $2.5(D_T)$	≤ $2.5(D_T)$
Number of Bolts or Anchor Rods	≥ 8	≥ 8	≥ 8
Flange Plate Maximum Thickness	4 in. [100 mm]	4 in. [100 mm]	4 in. [100 mm]
Center Opening Diameter	N/A	≤ $0.9(D_T)$	≤ $0.9(D_T)$
Termination of Fillet-Welds on Pole Wall [2]	30 degrees	30 degrees	30 degrees
Number of Stiffeners (Equally Spaced)	None	None	≥ 8
Stiffener Type	N/A	N/A	15 degree termination angle to pole wall and coped in corner [3]
Stiffener Spacing [4]	N/A	N/A	≤ 16 in. [400 mm]
Stiffener Thickness	N/A	N/A	0.25 to 0.75 in. [6 to 19 mm] but ≤ $1.25(t_T)$
Stiffener Height [5]	N/A	N/A	12 to 42 in. [300 to 1,070 mm]
Stress Concentration Factor	≤ 7.7	≤ 6.5	At termination ≤ 5.5 At stiffener base ≤ 7.7

D_T = outside pole diameter or flat-to-flat width, in. [mm]

t_T = pole wall thickness, in. [mm]

Footnotes:

1. The limitations are used in Table 17-6 to determine when higher ΔF_{TH} values may be justified.
2. Fillet-welds, including reinforcing fillet-welds at butt joints, shall be unequal leg welds with the long leg along the pole wall with an approximate 30 degree angle between the fillet-weld and the pole wall.
3. Cope shall be 1 in. [25 mm] minimum or as required to provide 3/8 in. [10 mm] minimum clearance from the stiffener weld toes to the unequal fillet-weld toes around the perimeter of the flange plate.
4. Stiffener spacing is measured circumferentially along a perimeter defined by D_T.
5. The optimum stiffener height is equal to 1.6 times the stiffener spacing.

Table 17-8: Resistance Factors for Extreme Loading (Structural Materials Other Than Steel)

Type of Failure	Resistance Factor
Yielding of ductile material	0.90
Local or global buckling	0.85
Fracture of brittle or ductile material	0.75

Table 17-9: Resistance Factors for Fatigue Loading (Structural Materials Other Than Steel)

Basis of Fatigue Threshold (ΔF_{TH})	Resistance Factor
50% survival probability with coefficient of variation ≥ 15%	0.60
50% survival probability with coefficient of variation < 15%	0.67
Test data with basis of 95% survival probability with a 95% confidence level	0.85

Table 17-10: Fatigue Details

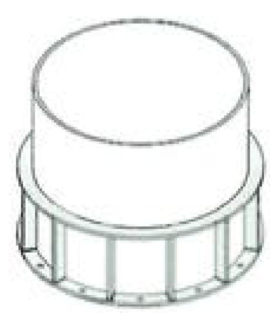
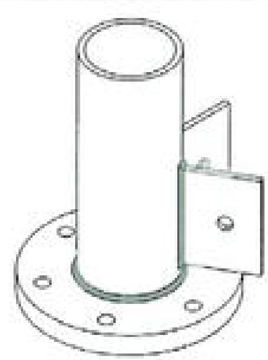
Category 1 Component Examples (17.12.4.1)			
Description	**Fatigue Threshold ΔF_{TH} ksi [MPa]**	**Potential Crack Location**	**Typical Illustrative Examples**
Category 1 – Item 1 Tubular pole structures at full-penetration groove-welded splices (with or without backing ring removed).	4.5 [31]	In pole wall along weld toe.	
Category 1 – Item 2 Tubular pole structures at all-around fillet-welds at ends of ground sleeves or collars.	4.5 [31]	In pole wall at ends of ground sleeves or collars.	
Category 1 – Item 3 Tubular pole structures or latticed tower legs at flanges where the termination of stiffeners are connected to a continuous annular ring plate welded to the pole or leg.	4.5 [31]	In pole wall or tower leg along weld toe at annular ring plates.	
Category 1 – Item 4 Latticed tower legs at flanges.	4.5 [31]	In tower leg along weld toe at flange plates.	

Table 17-10: Fatigue Details (Continued)

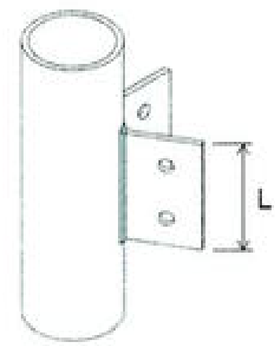
Category 1 Component Examples (17.12.4.1)			
Description	Fatigue Threshold, ΔF_{TH}, ksi [MPa]	Potential Crack Location	Typical Illustrative Examples
Category 1 – Item 5 Latticed tower legs at the ends of welded attachments exceeding a 2 in. [50 mm] length (L) along the longitudinal axis of the leg.	4.5 [31]	In tower leg at ends of attachment.	

Table 17-10: Fatigue Details (Continued)

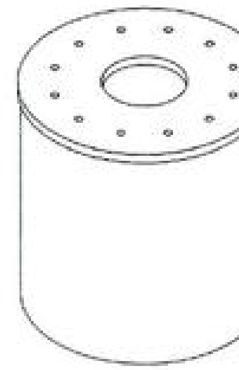
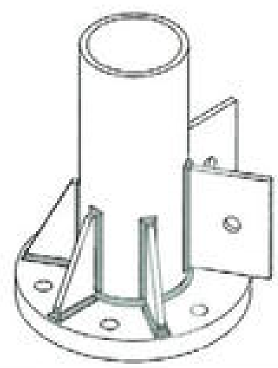
Category 2 Component Examples (17.12.4.2)			
Description	**Fatigue Threshold, ΔF_{TH}, ksi [MPa]**	**Potential Crack Location**	**Typical Illustrative Examples**
Category 2 – Item 1 Tubular pole structures at internal flange plates.	2.6 [18]	In pole wall along weld toe.	
Category 2 – Item 2 Tubular pole structures at socketed external flange plates with longitudinal stiffeners.	2.6 [18]	In pole wall at termination of stiffeners.	
Category 2 – Item 3 Latticed tower legs at flanges with longitudinal stiffeners.	2.6 [18]	In tower leg at termination of stiffeners.	

Table 17-10: Fatigue Details (Continued)

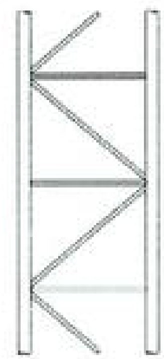
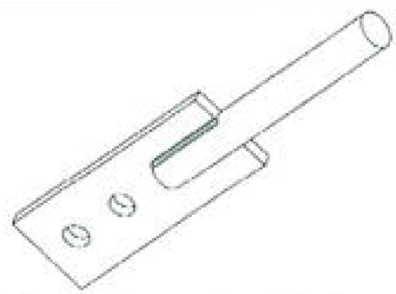
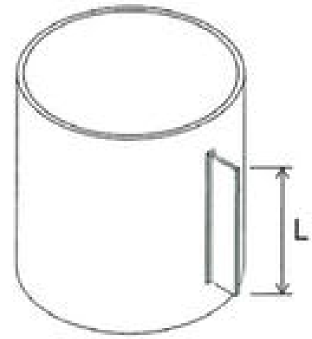
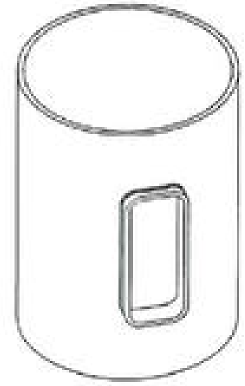
Category 2 Component Examples (17.12.4.2)			
Description	Fatigue Threshold, ΔF_{TH}, ksi [MPa]	Potential Crack Location	Typical Illustrative Examples
Category 2 – Item 4 Latticed tower main load carrying bracing members with effective slenderness ratios (KL/r) less than 60 with welded end connections.	2.6 [18]	In bracing member at end connections.	
Category 2 – Item 5 Latticed tower tension-only bracing members with welded end connections.	2.6 [18]	In tension-only member at end plate terminations.	

Table 17-10: Fatigue Details (Continued)

Description	Fatigue Threshold, ΔF_{TH}, ksi [MPa]	Potential Crack Location	Typical Illustrative Examples
Welded Attachments to Tubular Pole Structures (17.12.4.3)			
Ends of longitudinal attachments 2 in. [50 mm] or greater length (L).	2.6 – 7.0 [18 – 48]	In pole wall at terminations of attachment.	
Reinforced Openings in Tubular Pole Structures (17.12.4.4)			
Reinforced openings. (1)	7.0 [48] 16 [110]	In pole wall above and below opening based on gross cross section of pole without opening and reinforcing rim. In pole wall at center elevation of opening at intersection of pole wall and reinforcing rim. (2)	

(1) Refer to 17.12.4.4 for limitations.

(2) ΔF_{TH} applies to nominal stress multiplied by a stress concentration factor in accordance with 17.12.4.4.

Table 17-10: Fatigue Details (Continued)

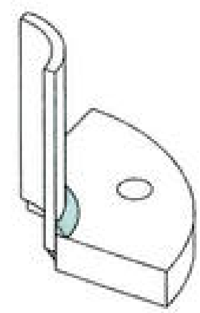
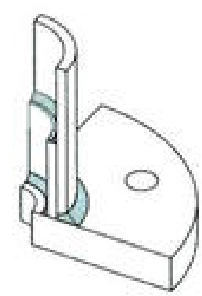
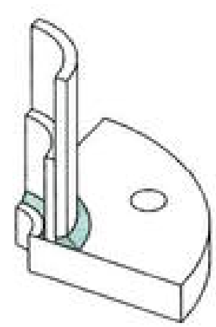
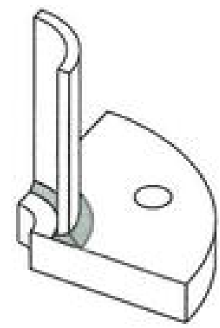
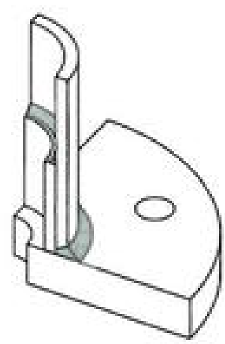
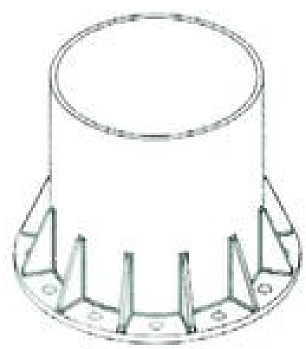
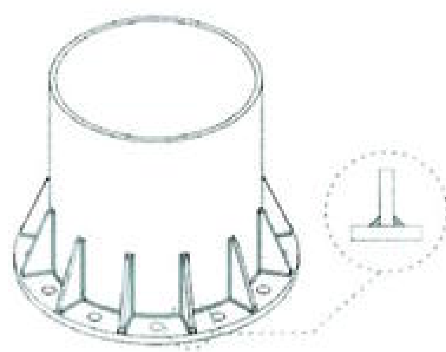
Tubular Pole Structure Exterior Flange Plates (17.12.4.6) [(1)]			
Description	**Fatigue Threshold, ΔF_{TH}, ksi [MPa]**	**Potential Crack Location**	**Typical Illustrative Examples**
		Socketed Joints	
Socketed joints.	2.6 – 7.0 [18 – 48]	In pole wall along fillet-weld toe.	
		Butt Joints	
Full-penetration groove-welded butt joints with backing ring attached at the top to the pole wall and at the bottom to the flange plate. Full-penetration groove-welded butt joints with backing ring attached at the top to the pole wall and at the bottom to the flange plate.	4.5 – 10.0 [31 – 69]	In pole wall along groove-weld or along upper fillet-weld toe.	
Full-penetration groove-welded butt joints with backing ring only attached at the bottom Full-penetration groove-welded butt joints with backing ring only attached at the bottom to the flange plate.	4.5 – 10.0 [31 – 69]	In pole wall along groove-weld.	

Table 17-10: Fatigue Details (Continued)

Tubular Pole Structure Exterior Flange Plates (17.12.4.6) (1)			
Description	**Fatigue Threshold, ΔF_{TH}, ksi [MPa]**	**Potential Crack Location**	**Typical Illustrative Examples**
Full-penetration groove-welded butt joints welded from both sides with back-gouging (without backing ring).Full-penetration groove-welded butt joints welded from both sides with back-gouging (without backing ring).	4.5 – 10.0 [31 – 69]	In pole wall along groove-weld.	
Full-penetration groove-welded butt joints with backing ring only attached at the top to the pole wall.	4.5 – 10.0 [31 – 69]	In pole wall along groove-weld or along upper fillet-weld toe.	

(1) Refer to 17.12.5.6 for limitations.

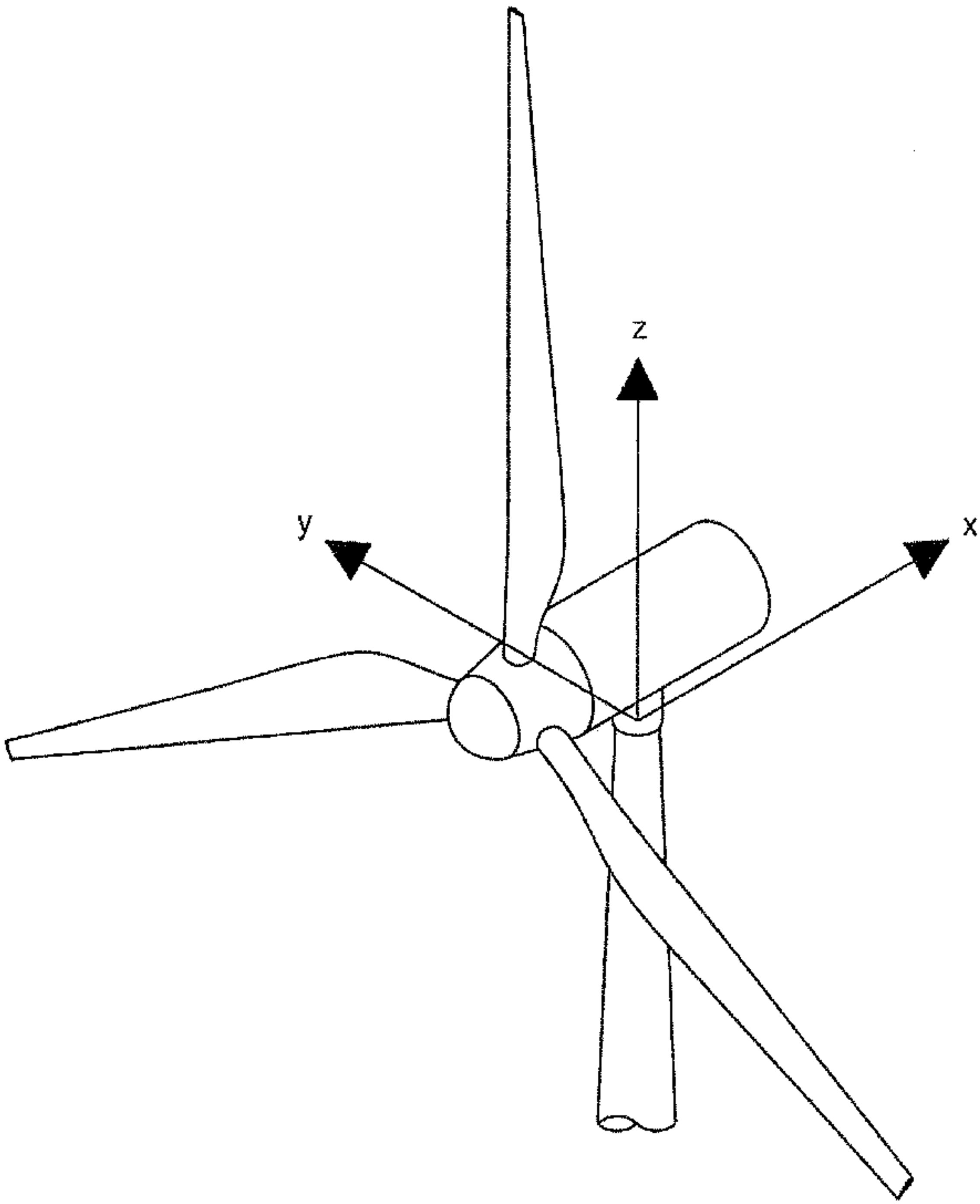
Table 17-10: Fatigue Details (Continued)

Tubular Pole Structure Exterior Flange Plates with Stiffeners (17.12.4.6) [1]			
Description	Fatigue Threshold, ΔF_{TH}, ksi [MPa]	Potential Crack Location	Typical Illustrative Examples
Butt joints with stiffeners.	2.6 - 7.0 [18 - 48] 4.5 - 7.0 [31 - 48]	In pole wall at termination of stiffeners. In pole wall along groove-weld at flange plate. flange plate.	
Butt joints with stiffeners greater than 0.50 in. [13 mm] thickness. [2]	≤ 10.0 [69]	Through stiffener thickness at flange plate and through weld throat when a fillet-weld is used for the stiffener connection to the flange plate.	

(1) Refer to 17.12.5.6 for limitations.

(2) Refer to 17.12.4.6.1 for ΔF_{TH} values based on stiffener thickness.

Figure 17-1: Coordinate System for SWT Support Structures



18.0 INSTALLATION

Rigging and temporary supports such as temporary guys, braces, false work, cribbing or other elements required for erection/modification shall be determined, documented, furnished and installed by the erector accounting for the loads imposed on the structure due to the proposed construction method.

The installation of appurtenances on a structure shall not degrade the climbing facility.

Installation shall be in accordance with design documents, installation documents and the ANSI/TIA-322 and ANSI/ASSE A10.48 Standards.

Inspection of installation shall be required for Risk Category III or IV structures, and is preferred for Risk Category I and II structures. Inspections, as a minimum, shall be in accordance with Annex N or Annex O.

ANNEX A: USER AND PROCUREMENT GUIDELINES (Normative)

This Annex is intended to provide user guidelines for the use of this Standard and to assist in the procurement of structures addressed in the scope of this Standard. Sections referenced in this Annex correspond to sections in this Standard with an A prefix.

Default design parameters appropriate for the referenced sections are provided to simplify the procurement specifications for a structure. It is intended that the default design parameters presented in this Annex be used for design unless otherwise specified in the procurement specifications for a structure. In addition, sections are referenced where site-specific or supplementary design requirements are often required when preparing procurement specifications for a structure.

A.2.0 Loads

Site-specific loading or local building code requirements may be more stringent than the minimum loading requirements specified in the Standard. These and other unique load or loading combination requirements are to be included in the procurement specifications.

For sites located in special wind or ice regions indicated in Annex B, the authority having jurisdiction may require higher wind speeds and ice thicknesses than the adjacent regions to account for local conditions.

A.2.2 Risk Category of Structures

The standard establishes four risk categories of structures based on reliability criteria.

The default Risk Category is Category II.

A.2.3.2 Strength Limit State Load Combinations

The Standard is based on limit states design and specifies appropriate limit state loads or load factors to be applied to nominal loads. For locations not included in Annex B, the minimum basic wind speed shall be 85 mph [38 m/s] 3-sec gust or as determined in accordance with 2.6.4.1 for the site location.

A.2.4 Temperature Effects

The Standard specifies a 50 degrees F [28 degrees C] reduction in temperature for loading conditions that include ice. Based on site-specific requirements, a greater reduction may be appropriate and is to be included in the procurement specifications.

A.2.6.4 Basic Wind Speed and Design Ice Thickness

The Standard is based on 3-second gust wind speeds and radial glaze ice thicknesses for defined return periods based on risk category. Wind speeds averaged over different time periods are to be converted to 3-second gust wind speeds for use with the Standard. (Refer to Annex L for wind speed conversion factors.) Wind speeds and ice thicknesses must be converted to appropriate return periods based on risk category.

Supplementary rime ice and in-cloud ice loadings (including thickness, density, escalation with height and corresponding wind speed) are to be included in the procurement specification when appropriate for a given site location.

A.2.6.5 Exposure Categories

The default Exposure Category is Exposure C.

Detailed site information is required for use of a Site-Specific Exposure category.

A.2.6.6 Topographic Effects

The default Topographic Category is Category 1 for Method 1.

For Topographic Categories 2, 3 and 4, the height of the topographic feature is to be included in the procurement specifications.

Detailed site information is to be included in the procurement specification for the use of Topographic Factor Procedure Methods 2 and 3.

For guyed masts in any topographic category, the elevation of the ground guy anchor supports is to be assumed at the same elevation as the base of the mast unless otherwise specified in the procurement specifications.

When relative differences exist in elevation between the base of a guyed mast and the ground guy anchor supports, the differences are to be included in the procurement specifications. Alternatively, a topographic survey for the site may be provided with the procurement specifications. The relative elevations are required in order to properly model a guyed mast in accordance with the Standard. Note that although a structure may be located on terrain with no abrupt changes, there may be significant differences in elevation between the ground anchor supports and the base of the mast.

A.2.6.7 Rooftop Wind Speed-Up Factor

For structures or appurtenances supported on enclosed buildings the following criteria is to be included in the procurement specifications:

1. Building height and projection above the average height of immediately adjacent buildings;
2. Building plan dimensions;
3. Parapet height;
4. Location of structure on roof;
5. For guyed masts, the locations of the guy anchor supports.

A.2.6.8 Ground Elevation Factor

The default ground elevation factor is 1.0.

The ground elevation is required for the use of ground elevation factors less than 1.0.

A.2.6.11.5 Transmission Lines Mounted in Clusters or Blocks

The distribution of lines on a latticed structure has a significant effect on the wind loads applied to the structures. As a default, lines may be distributed on multiple faces in clusters or blocks in accordance with the Standard in order to minimize wind loads on the structure. When other arrangements are required, the distribution and location of lines are to be specified in detail in the procurement specifications.

A.2.7.3 Seismic Load Effect Parameters

The default Site Class for earthquake analysis is Class D.

A.2.8 Serviceability Requirements

The service loads and deformation limits specified in the Standard are the minimum requirements for communication structures. When more stringent requirements are required for a specific application, the serviceability limit state basic wind speed and if required, the serviceability limit state design ice thickness; the deformation limitations (twist, sway and horizontal displacement) and the location/elevation where the deformation limitations apply are to be included in the procurement specifications.

A.5.6.6 Guy Anchorages (Corrosion Control)

The default soil condition is non-corrosive soil.

The Standard specifies minimum corrosion control when steel anchors are in direct contact with soil. Requirements for supplemental corrosion control are to be included in the procurement specifications. (Refer to Annex H).

A.5.6.7 Ground Embedded Poles (Corrosion Control)

The default soil condition is non-corrosive soil.

The Standard specifies minimum corrosion control for ground embedded poles. Requirements for supplemental corrosion control are to be included in the procurement specifications. (Refer to Annex H).

A.7.5 Guy Dampers

The Standard specifies minimum requirements for high frequency guy dampers. High frequency low amplitude (Aeolian) and low frequency high amplitude (galloping) vibrations are difficult to predict prior to the installation of a structure due to unique site and environmental variables. High and Low frequency dampers can be retrofitted when necessary. Additional damper requirements are to be included in the procurement specifications.

A.9.0 Foundations and Anchorages

The Standard refers to Annex F for presumptive soil parameters, which are intended to be used for foundation and anchorage designs in the absence of a geotechnical report for Risk Category I or II structures. It is intended that actual site conditions will be investigated by the owner or the owner's representative prior to the installation of foundations and anchorages designed in accordance with presumptive soil parameters. The default soil type is clay unless otherwise specified in the procurement specifications. The default frost depth is 3.5 ft. [1.1 m] when the site location is undetermined unless otherwise specified in the procurement specifications.

Foundations and anchorages may be designed by a third party subsequent to the design of the structure. It is intended the proper development of anchor rods and anchorages supplied with a structure be verified by the foundation engineer. Site-specific requirements for anchorages, such as roof-mounted structures, pile caps or other similar situations are to be included in the procurement specifications.

The frost depth for sites located in Alaska shall be included in the procurement specifications based on regional climatic data and knowledge of local conditions.

A.10.0 Protective Grounding

The Standard provides grounding requirements to limit damage to the structure or foundation.

Additional or alternate grounding materials or requirements other than specified in Section 10.0 are to be included in the procurement specifications.

It is intended that the owner or the owner's representative will verify the adequacy of a grounding system based on actual site soil conditions. Additional or alternate grounding material may be required.

The following are common examples where additional or alternate grounding materials may be required:

1. The presence of high resistivity soil.
2. The presence of highly corrosive soil.
3. The presence of rock near the surface.
4. Grounding systems installed in highly conductive soils in order to minimize galvanic corrosion of the structure or grounding system components.
5. AM radio installations.

The requirements for lightning rods for the protection of equipment or lighting systems are to be included in the procurement specifications.

A.11.0 Obstruction Markings

Requirements for obstruction markings are to be included in the procurement specifications. The default condition is that no obstruction markings are required.

A.12.0 Climbing Facilities

The default climbing facility classification is Class B.

The Standard specifies minimum climbing facility requirements considering that only competent or authorized climbers will be accessing the structure. Additional climbing facilities, climber attachment points and platform requirements are to be included in the procurement specifications.

The requirement for cages or barriers, including dimensions and strength requirements, are to be included in the procurement specification.

The Standard specifies minimum strength requirements for the anchorage and support of safety climb systems. Strength requirements for safety climb systems with higher anchorage or support strength requirements shall be included in the procurement specifications.

A.14.4 Guy Anchor Shafts

The interval and extent of inspections of guy anchor shafts with steel in direct contact with soil shall be in accordance with a corrosion management plan established by the owner based on site-specific corrosion conditions.

A.15.0 Existing Structures

The default Risk Category for existing structures is Category II.

A site-specific management plan for periodic inspections required for the use of Annex S shall be established by the owner based on site-specific conditions.

A.15.7.4 Verification of Installation

It is intended that the inspection of modifications to a structure will be performed by the owner or the owner's representative.

A.17.4 Turbine Data

Turbine data required for design or analysis is to be included in the procurement specification or otherwise provided by the owner or owner's representative.

A.17.9 Critical Turbine Moments

Critical turbine moments required for design or analysis are to be included in the procurement specification or otherwise provided by the owner or owner's representative.

A.17.10 Stiffness Requirements

Stiffness requirements for a supporting structure are to be included in the procurement specification or otherwise provided by the owner or owner's representative.

A.17.11 Dynamic Requirements

Natural frequency limitations or other structural requirements for a supporting structure are to be included in the procurement specification or otherwise provided by the owner or owner's representative.

A.17.12 Design for Fatigue

Fatigue strength criteria more stringent than section 17.12 for a supporting structure is to be included in the procurement specification or otherwise provided by the owner or owner's representative.

A.17.12.2 Vertical Axis Turbines

Equivalent constant-range fatigue loads for vertical axis turbines are to be included in the procurement specification or otherwise provided by the owner or owner's representative.

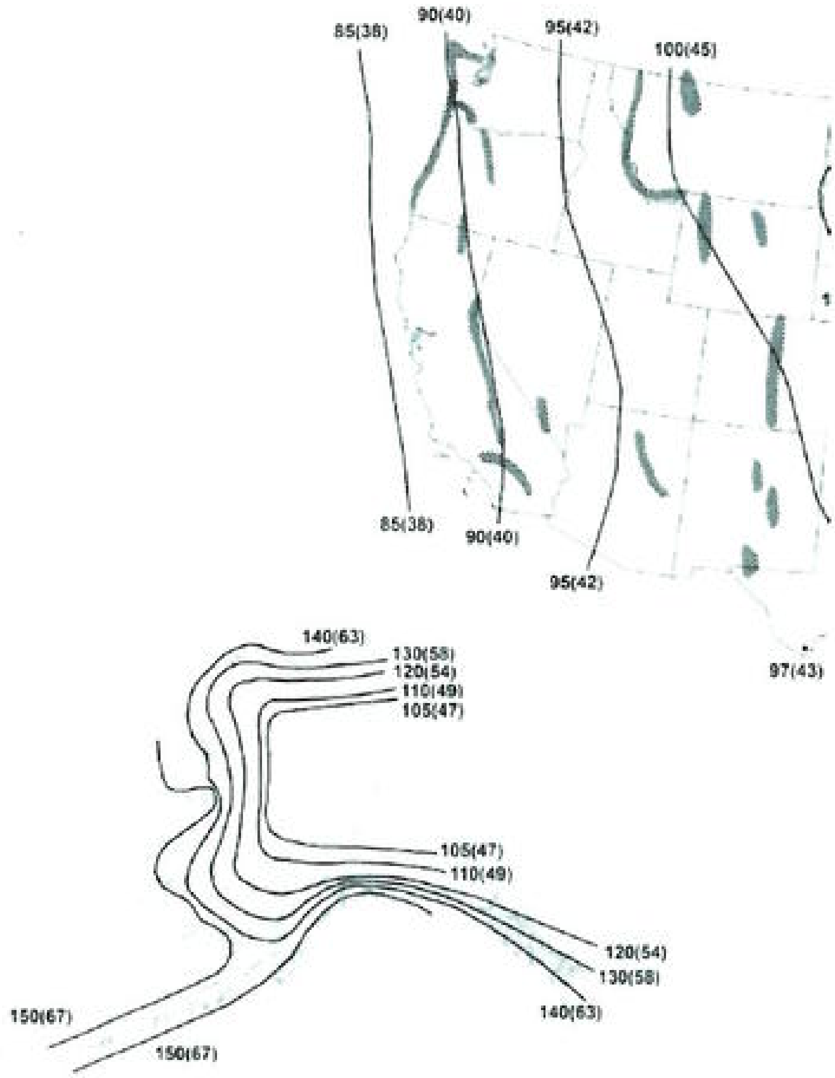
A.18.0 Installation

It is intended that the inspection of a structure will be performed by the owner or the owner's representative.

ANNEX B: WIND, ICE, EARTHQUAKE AND FROST DEPTH MAPS (Normative)

(Reference ASCE 7-16, NAVFAC DM 7.01)

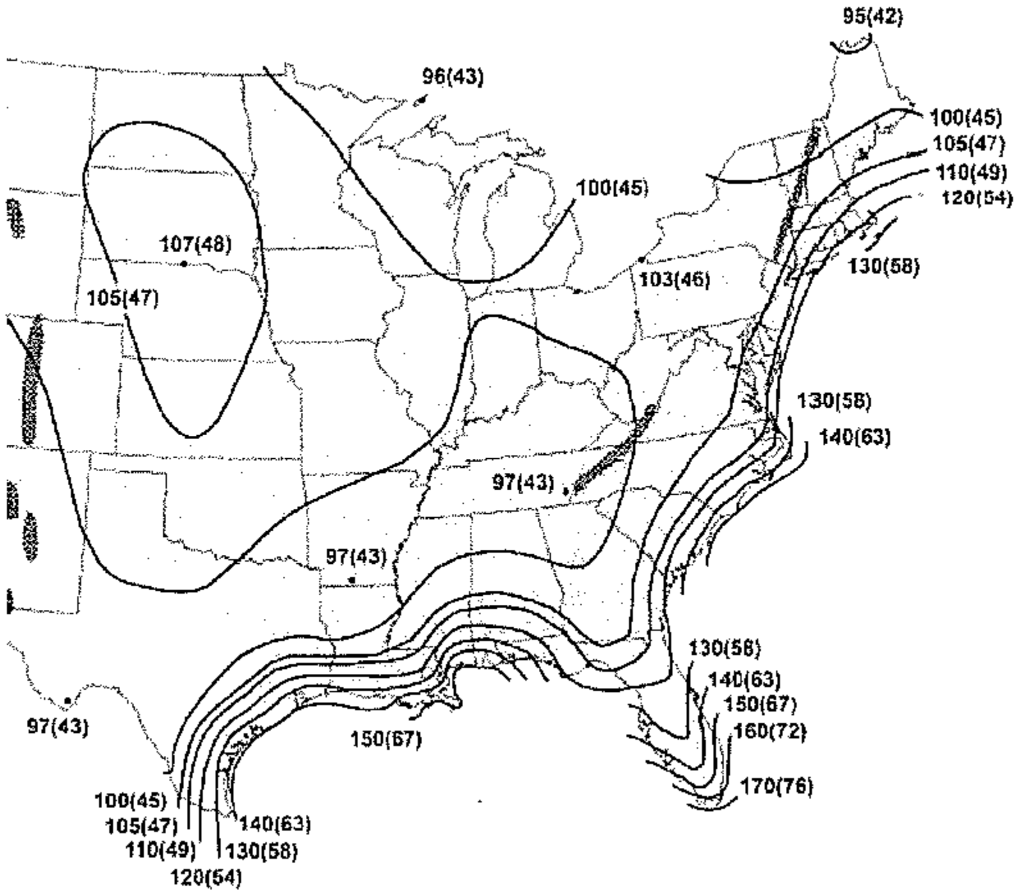
Figure B-1: Basic Wind Speeds for Risk Category I Buildings and Other Structures



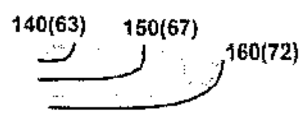
Notes:

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 Category C.
2. Linear interpolation is permitted between contours. Point values are provided to aid with interpolation.
3. Islands, coastal areas, and land boundaries outside the last contour shall use the last wind speed contour.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 15% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00333, MRI = 300 years).

Figure B-1: Basic Wind Speeds for Risk Category I Buildings and Other Structures (Continued)



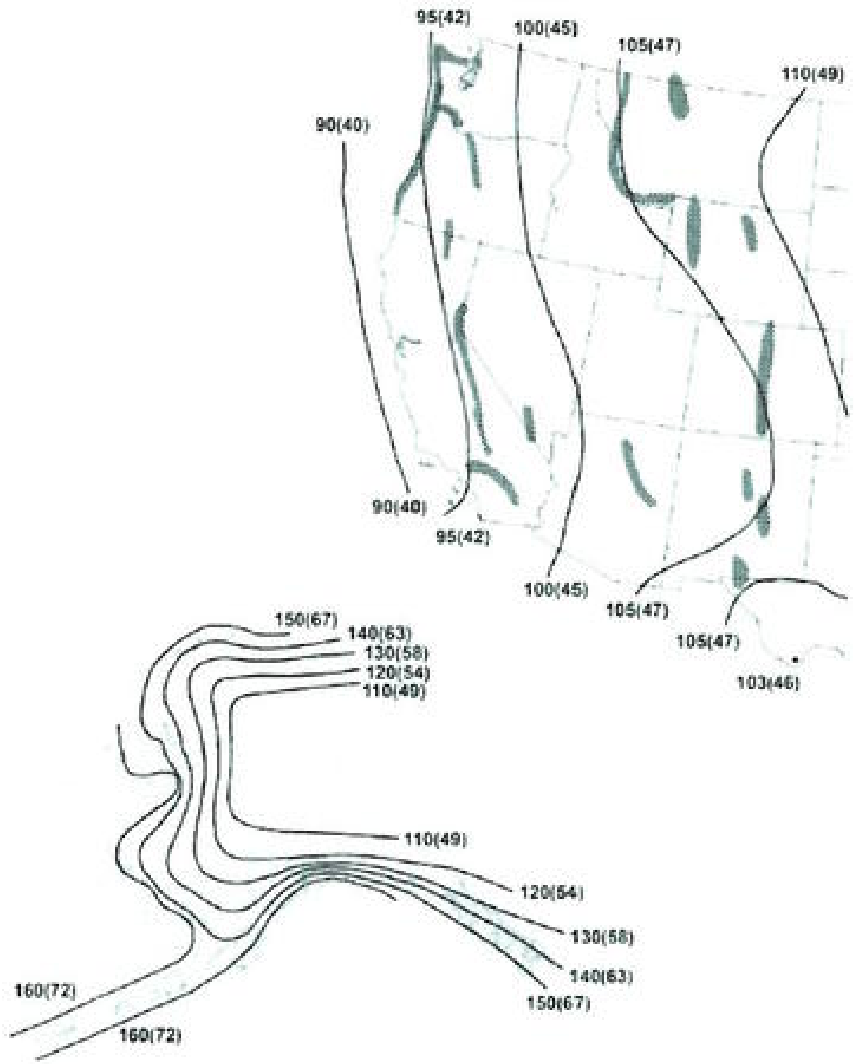
Special Wind Region



Puerto Rico

Location	Vmph	(m/s)
Guam	180	(80)
Virgin Islands	150	(67)
American Samoa	150	(67)
Hawaii	Figure B-5	

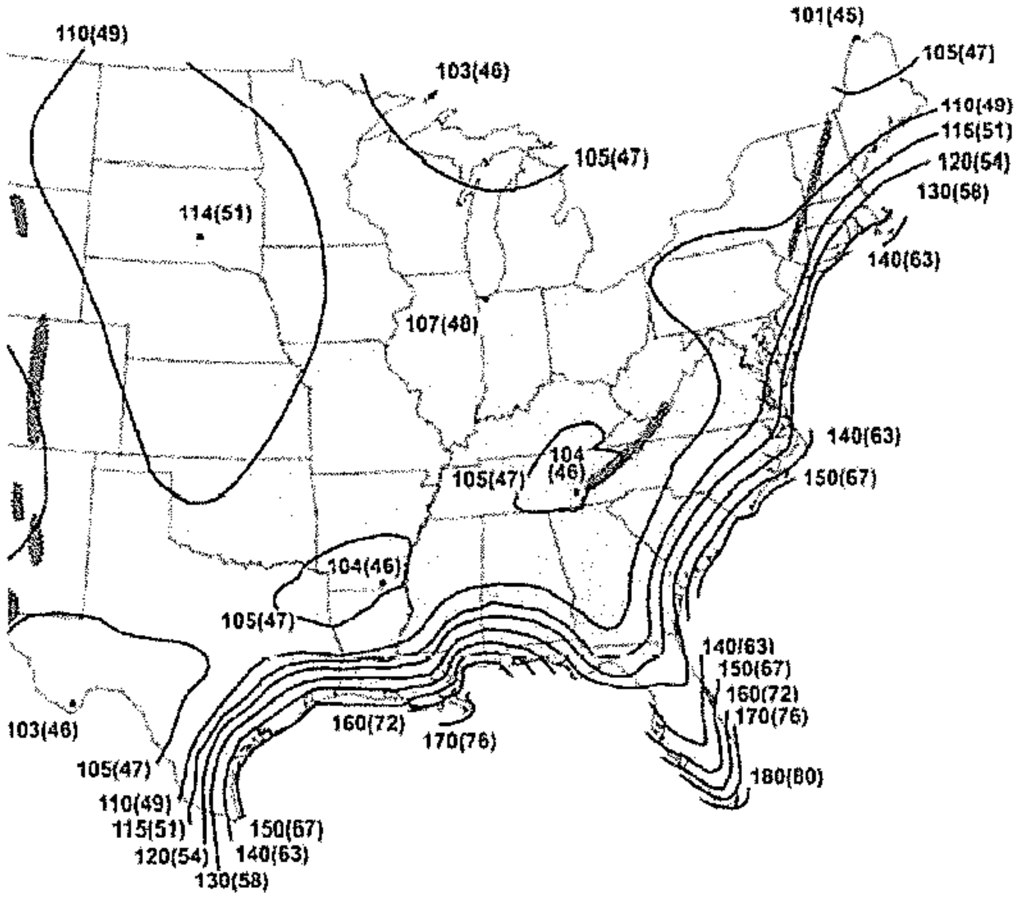
Figure B-2: Basic Wind Speeds for Risk Category II Buildings and Other Structures



Notes:

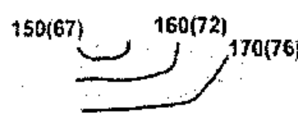
1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft. [10 m] above ground for Exposure Category C.
2. Linear interpolation is permitted between contours. Point values are provided to aid with interpolation.
3. Islands, coastal areas, and land boundaries outside the last contour shall use the last wind speed contour.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 years).

Figure B-2: Basic Wind Speeds for Risk Category II Buildings and Other Structures
(Continued)



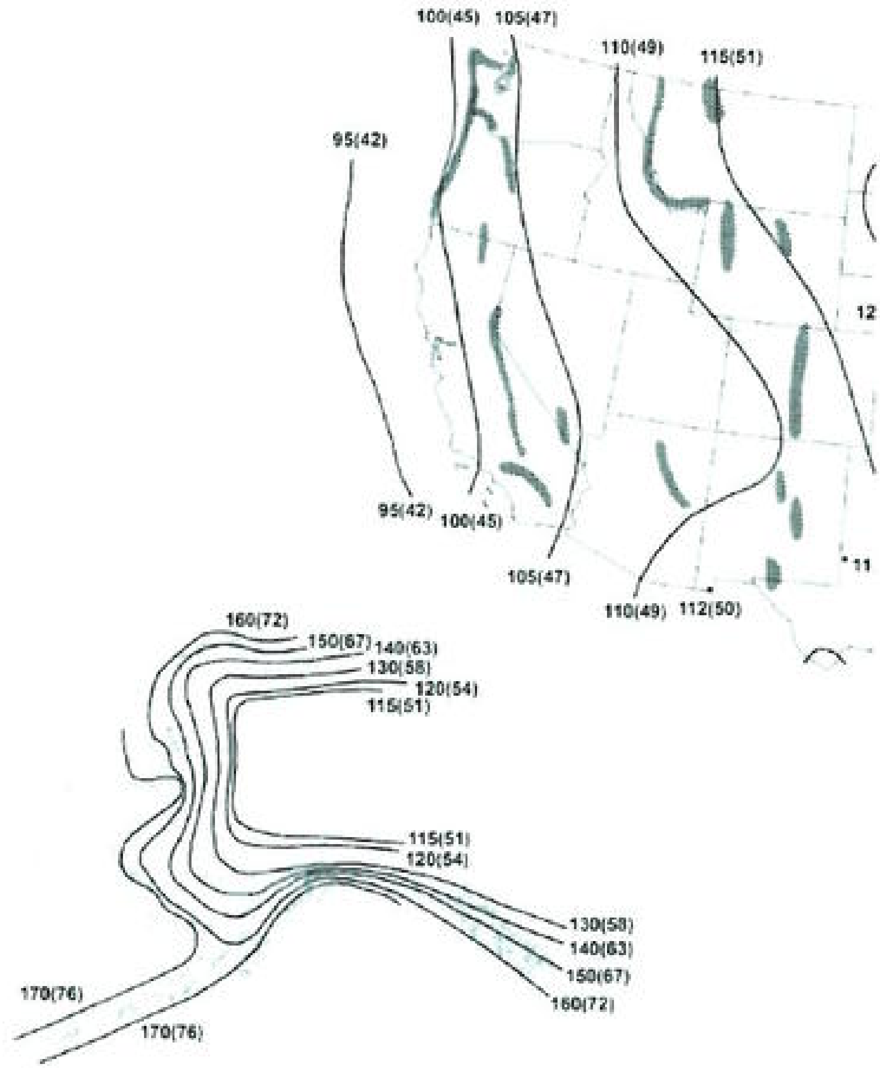
Special Wind Region

Location	Vmph	(m/s)
Guam	195	(87)
Virgin Islands	150	(74)
American Samoa	150	(72)
Hawaii	Figure B-6	



Puerto Rico

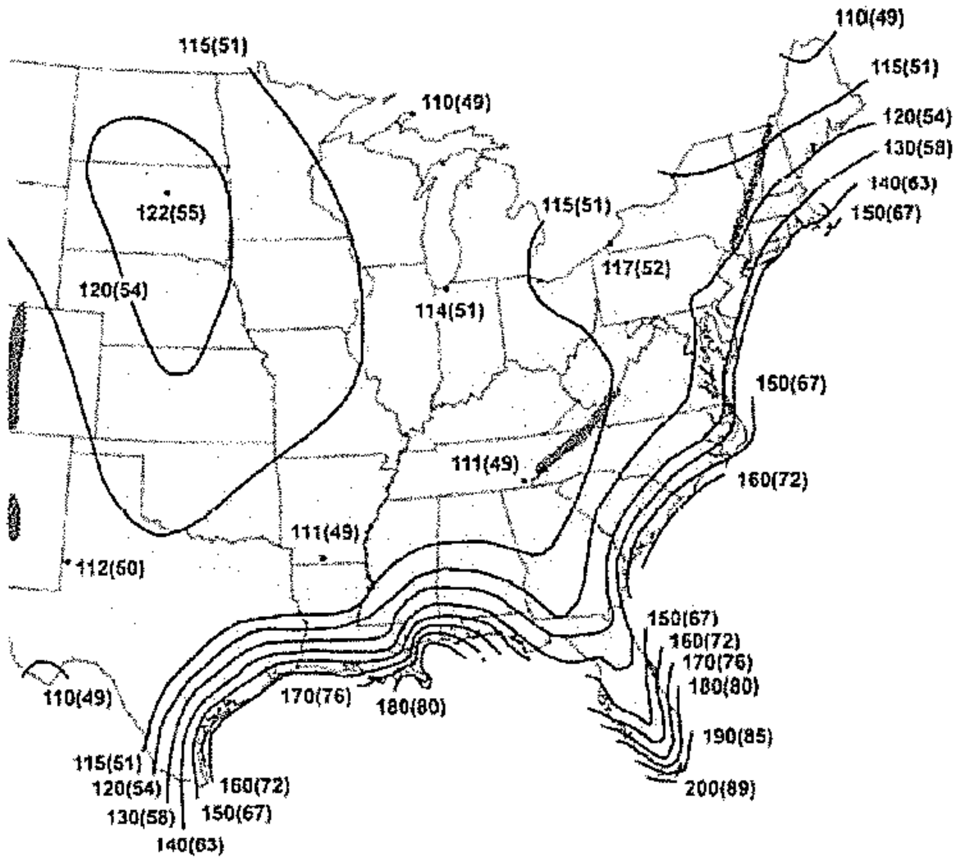
Figure B-3: Basic Wind Speeds for Risk Category III Buildings and Other Structures



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 Category C.
2. Linear interpolation is permitted between contours. Point values are provided to aid with interpolation.
3. Islands, coastal areas, and land boundaries outside the last contour shall use the last wind speed contour.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years (Annual Exceedance Probability = 0.000588, MRI = 1,700 years).

Figure B-3: Basic Wind Speeds for Risk Category III Buildings and Other Structures (Continued)



Location	Vmph	(m/s)
Guam	210	(94)
Virgin Islands	175	(78)
American Samoa	170	(76)
Hawaii	Figure B-7	

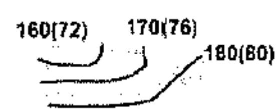
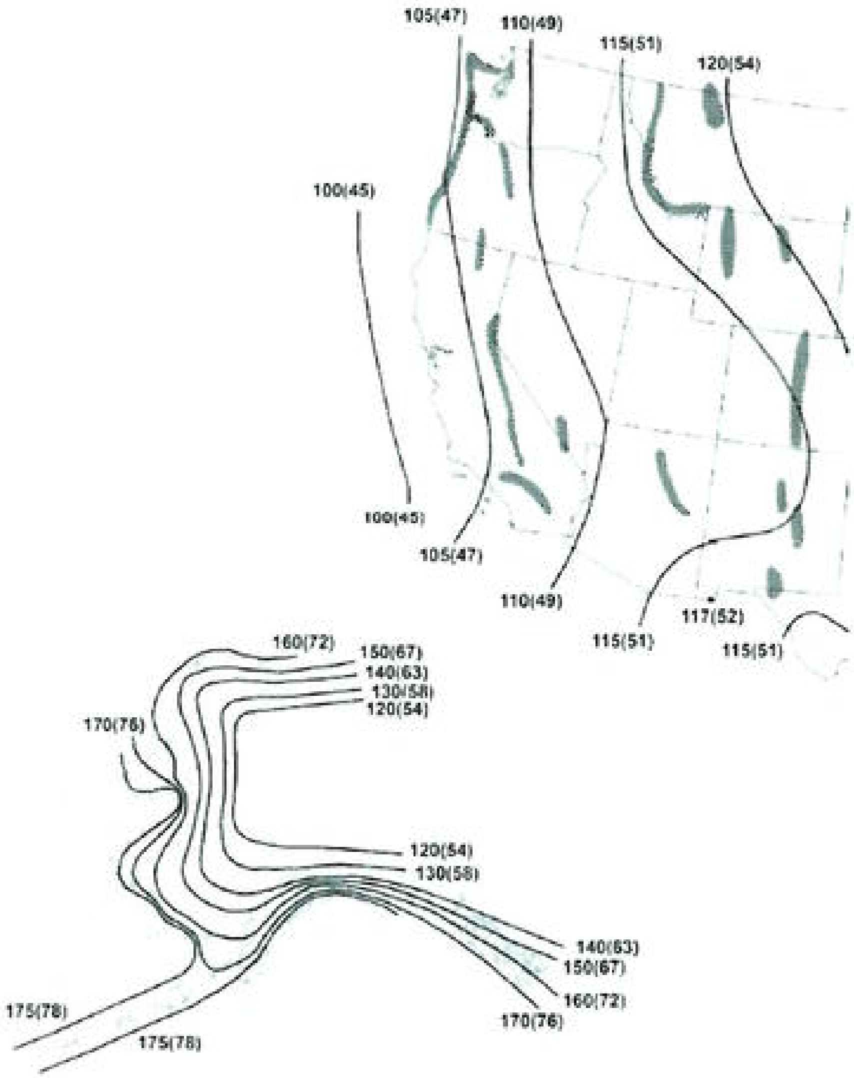


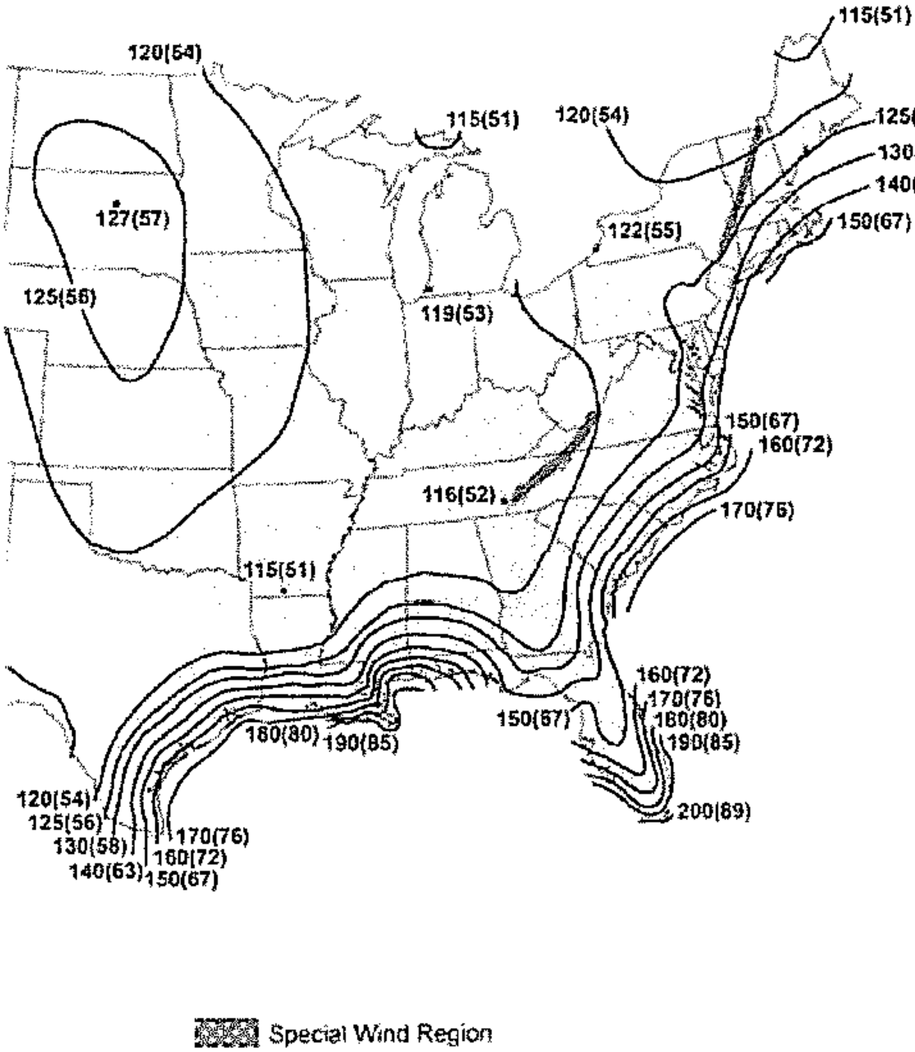
Figure B-4: Basic Wind Speeds for Risk Category IV Buildings and Other Structures



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 Category C.
2. Linear interpolation is permitted between contours. Point values are provided to aid with interpolation.
3. Islands, coastal areas, and land boundaries outside the last contour shall use the last wind speed contour.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 1.6% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00033, MRI = 3,000 years).

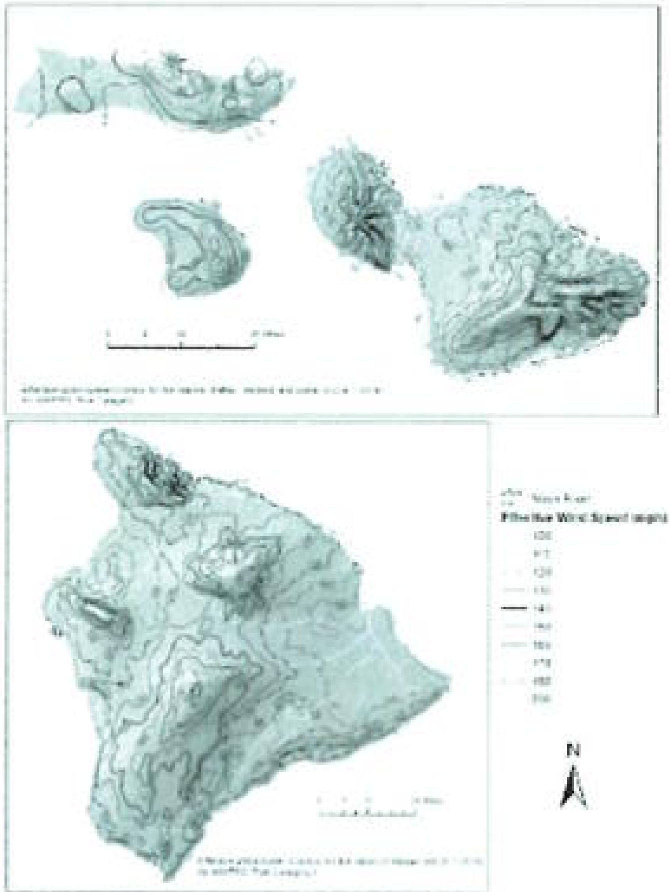
Figure B-4: Basic Wind Speeds for Risk Category IV Buildings and Other Structures
(Continued)



Location	Vmph	(m/s)
Guam	220	(98)
Virgin Islands	180	(80)
American Samoa	180	(80)
Hawaii	Figure B-8	

Puerto Rico

Figure B-5: Basic Wind Speeds for Risk Category I Buildings and Other Structures (Hawaii)



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 Category C.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. It is permitted to use the standard values of K_{zt} of 1.0 (special topographic wind speed-up effects included in above Figure) and K_d as given in this Standard.
5. Ocean promontories and local escarpments shall be examined for unusual wind conditions.
6. Wind speeds correspond to approximately a 15% probability of exceedance in 50 years (Annual Exceedance Probability = 0.0033, MRI = 300 years).

Figure B-5: Basic Wind Speeds for Risk Category I Buildings and Other Structures (Hawaii) (Continued)

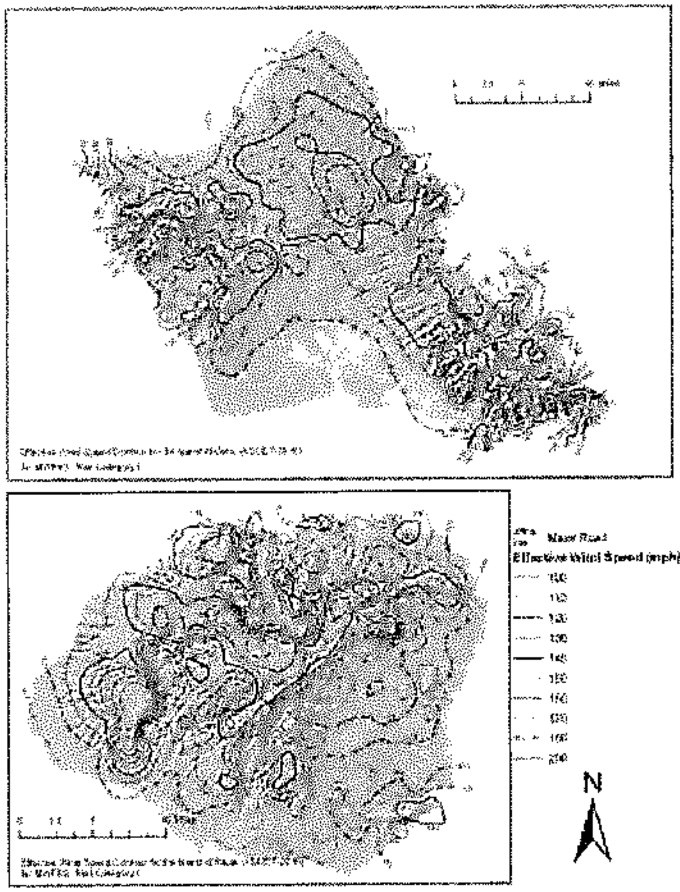
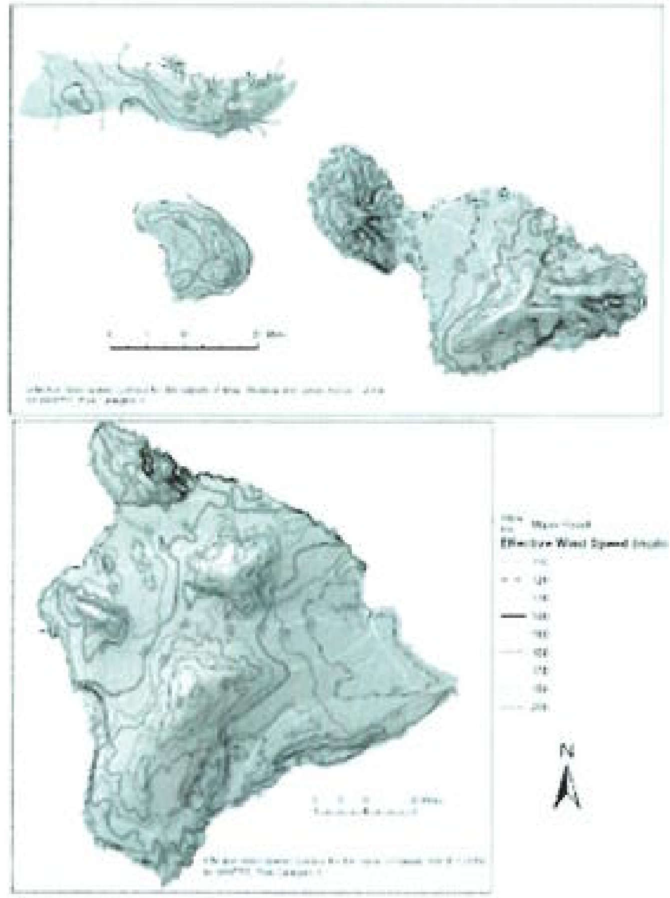


Figure B-6: Basic Wind Speeds for Risk Category II Buildings and Other Structures (Hawaii)



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 Category C.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. It is permitted to use the standard values of K_{zt} of 1.0 (special topographic wind speed-up effects included in above Figure) and K_d as given in this Standard.
5. Ocean promontories and local escarpments shall be examined for unusual wind conditions.
6. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 years).

Figure B-6: Basic Wind Speeds for Risk Category II Buildings and Other Structures (Hawaii) (Continued)

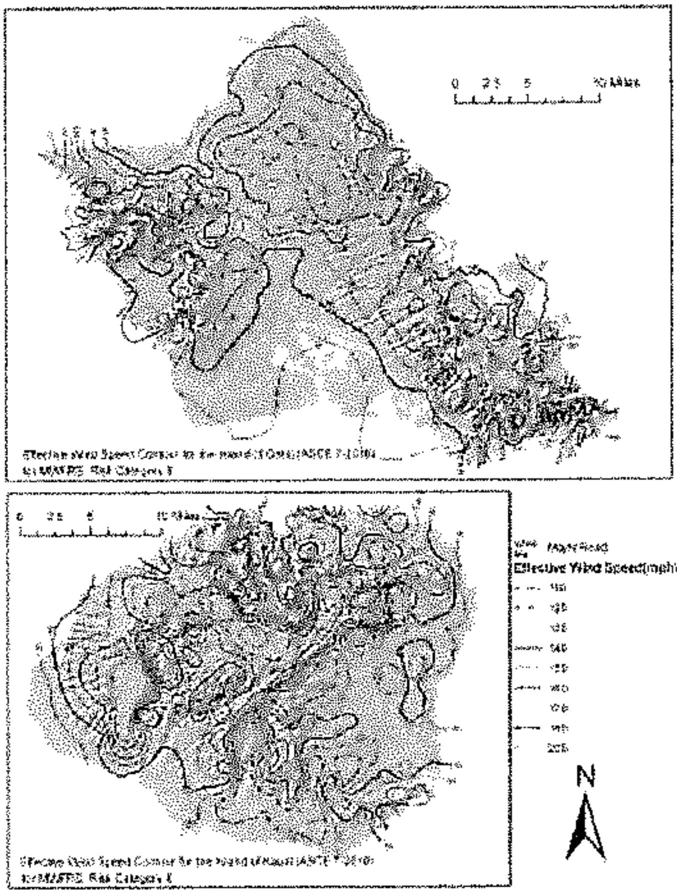
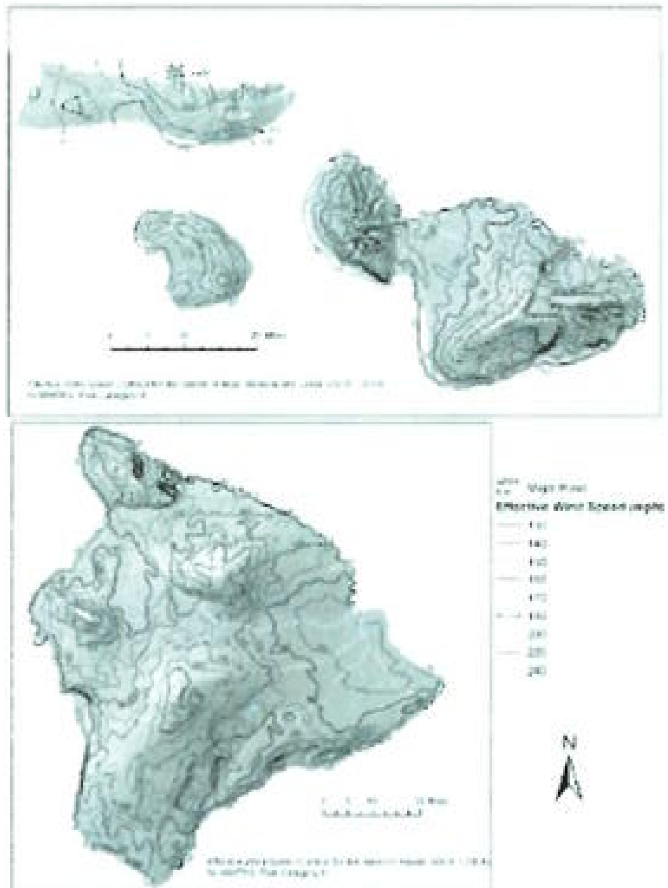


Figure B-7: Basic Wind Speeds for Risk Category III Buildings and Other Structures (Hawaii)



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 Category C.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. It is permitted to use the standard values of K_{zt} of 1.0 (special topographic wind speed-up effects included in above Figure) and K_d as given in this Standard.
5. Ocean promontories and local escarpments shall be examined for unusual wind conditions.
6. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years (Annual Exceedance Probability = 0.000588, MRI = 1,700 years).

Figure B-7: Basic Wind Speeds for Risk Category III Buildings and Other Structures (Hawaii) (Continued)

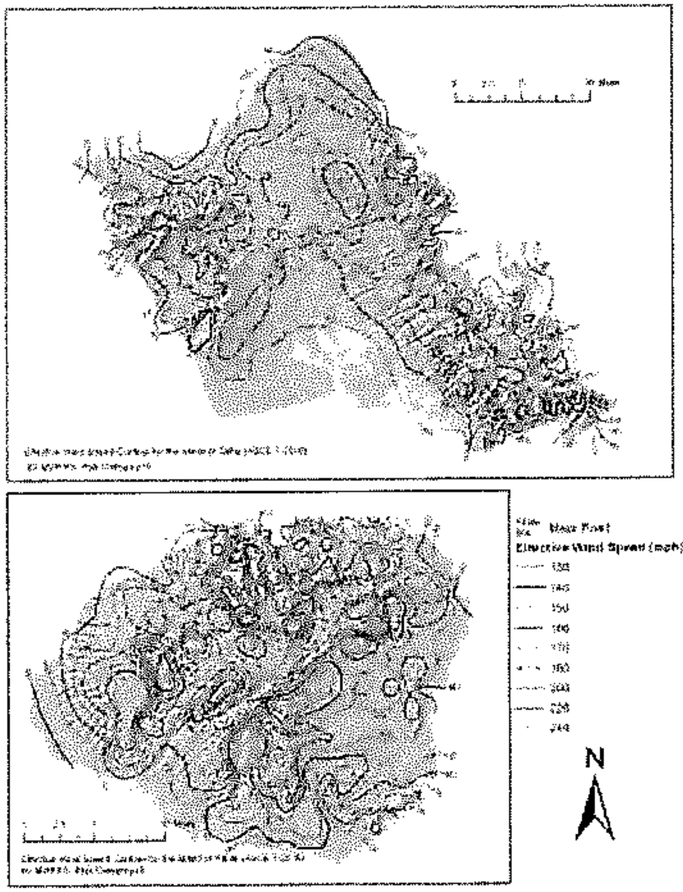
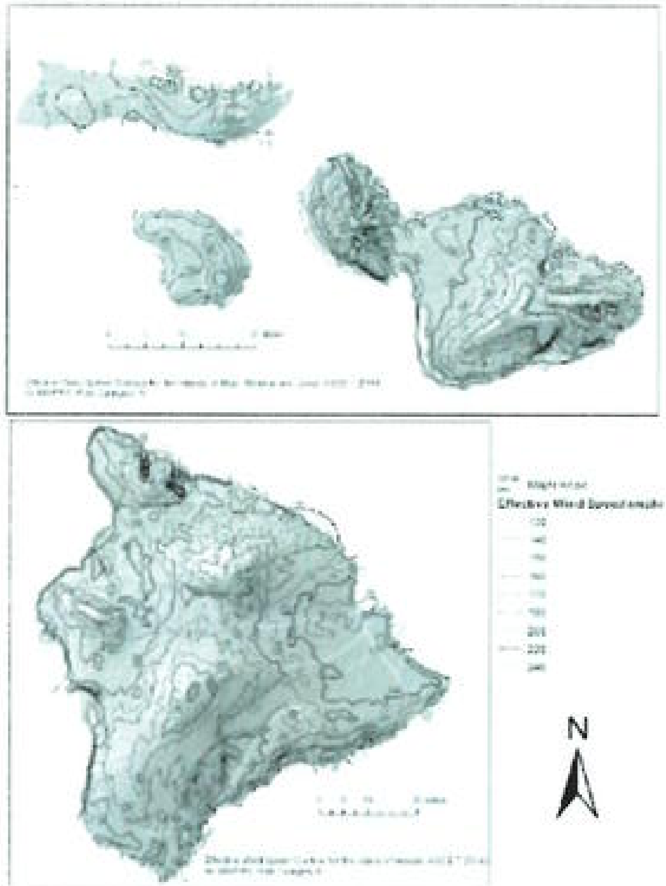


Figure B-8: Basic Wind Speeds for Risk Category IV Buildings and Other Structures (Hawaii)



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 Category C.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. It is permitted to use the standard values of K_{zt} of 1.0 (special topographic wind speed-up effects included in above Figure) and K_d as given in this Standard.
5. Ocean promontories and local escarpments shall be examined for unusual wind conditions.
6. Wind speeds correspond to approximately a 1.7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.000333, MRI = 3,000 years).

Figure B-8: Basic Wind Speeds for Risk Category IV Buildings and Other Structures (Hawaii) (Continued)

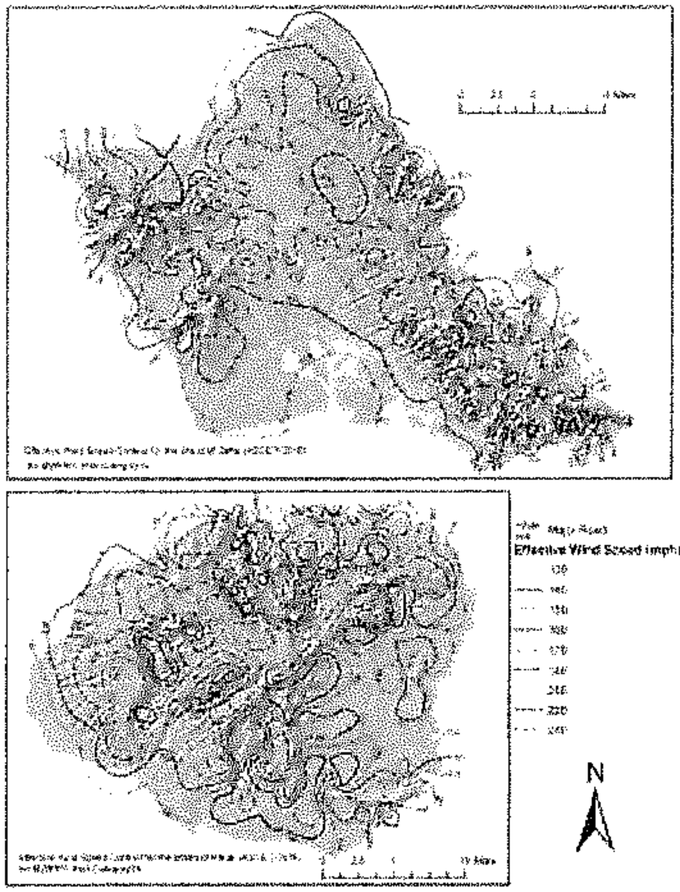
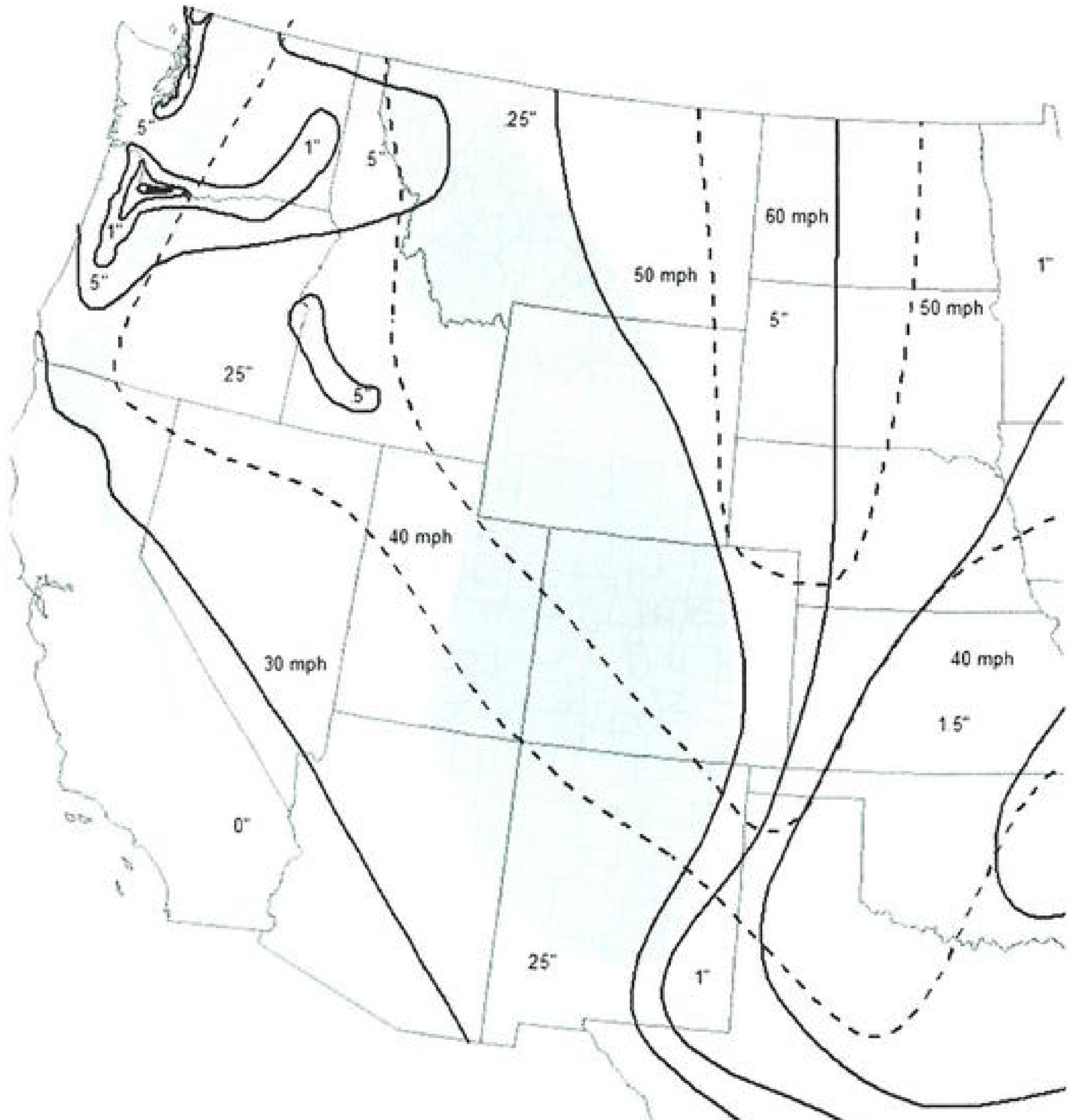


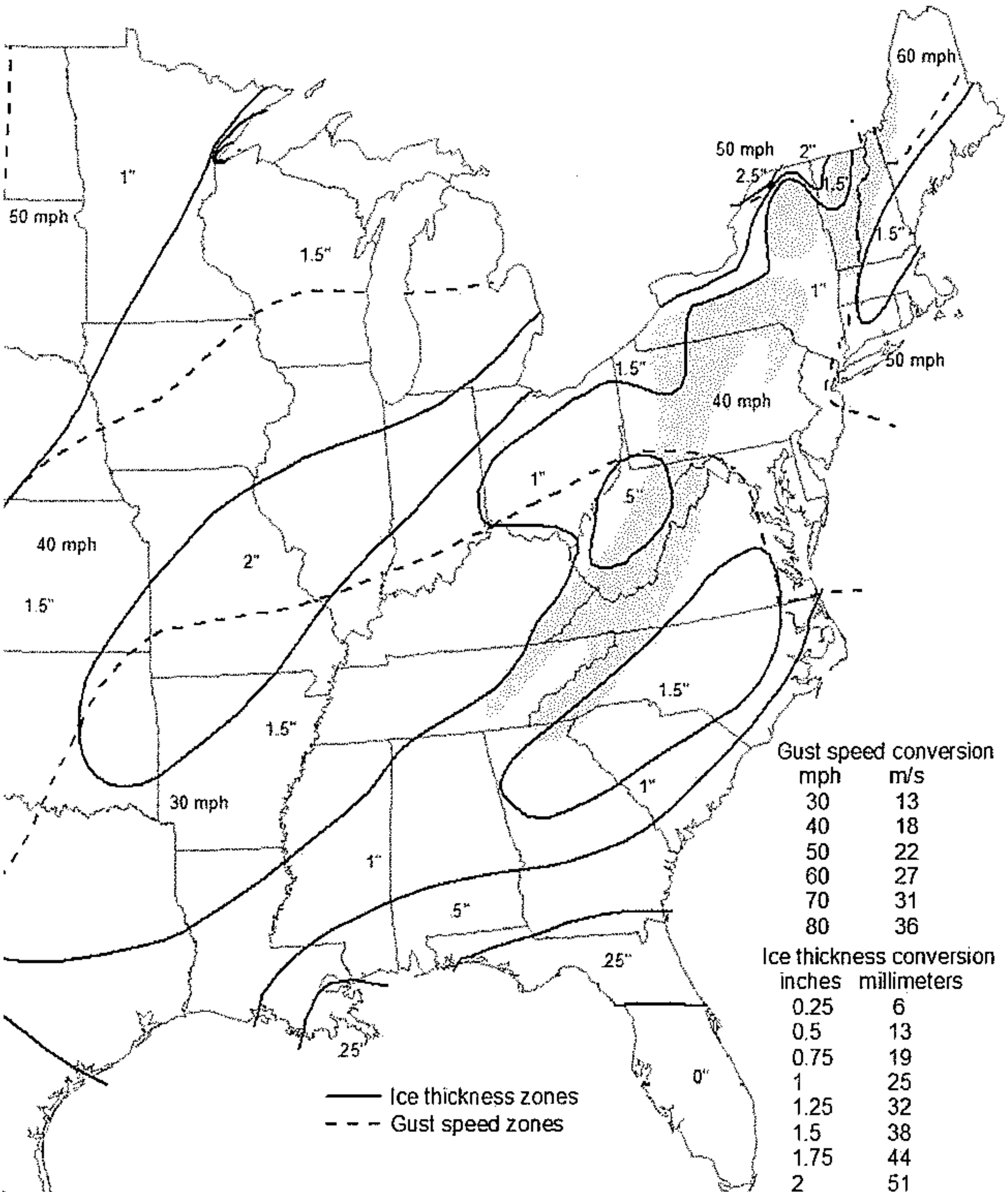
Figure B-9: Equivalent Radial Ice Thicknesses Caused by Freezing Rain with Concurrent 3-Second Gust Speeds, for a 500-Year Mean Recurrence Interval



Notes:

1. Ice thicknesses on structures in exposed locations at elevations higher than the surrounding terrain and in valleys and gorges may exceed the mapped values.
2. In the mountain west, indicated by the shading, ice thicknesses may exceed the mapped values in the foothills and passes. However, at elevations above 5,000 ft, freezing rain is unlikely.
3. In the Appalachian Mountains, indicated by the shading, ice thicknesses may vary significantly over short distances.

Figure B-9: Equivalent Radial Ice Thicknesses Caused by Freezing Rain with Concurrent 3-Second Gust Speeds, for a 500-Year Mean Recurrence Interval (Continued)



Gust speed conversion

mph	m/s
30	13
40	18
50	22
60	27
70	31
80	36

Ice thickness conversion

inches	millimeters
0.25	6
0.5	13
0.75	19
1	25
1.25	32
1.5	38
1.75	44
2	51
2.5	64

Figure B-9: Lake Superior Detail

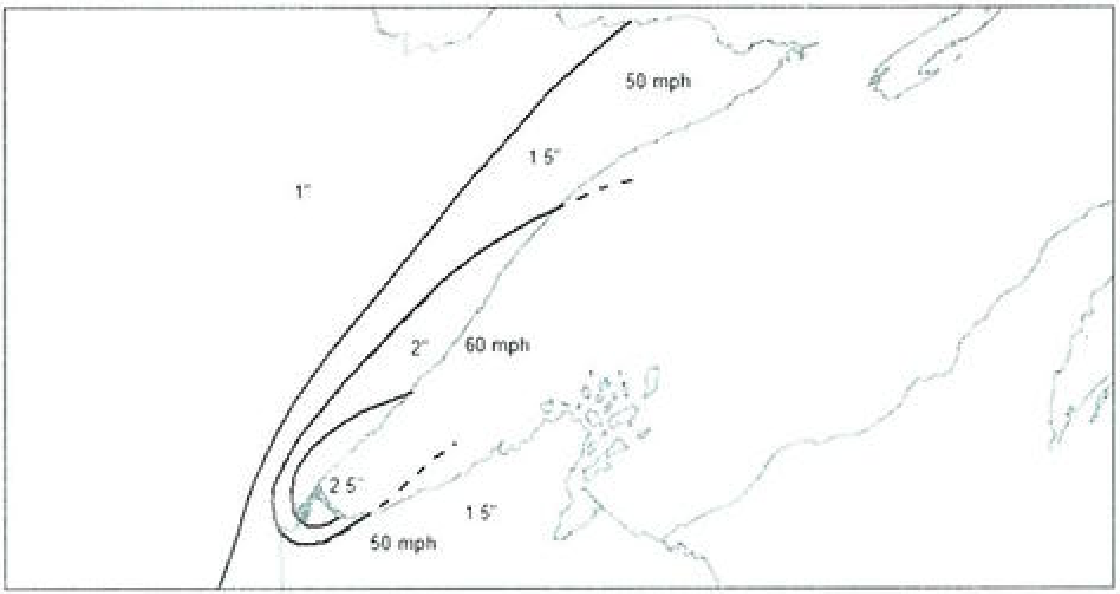


Figure B-9: Fraser Valley Detail (NW Washington State)

Figure B-9: Columbia River Gorge Detail (Washington and Oregon States)

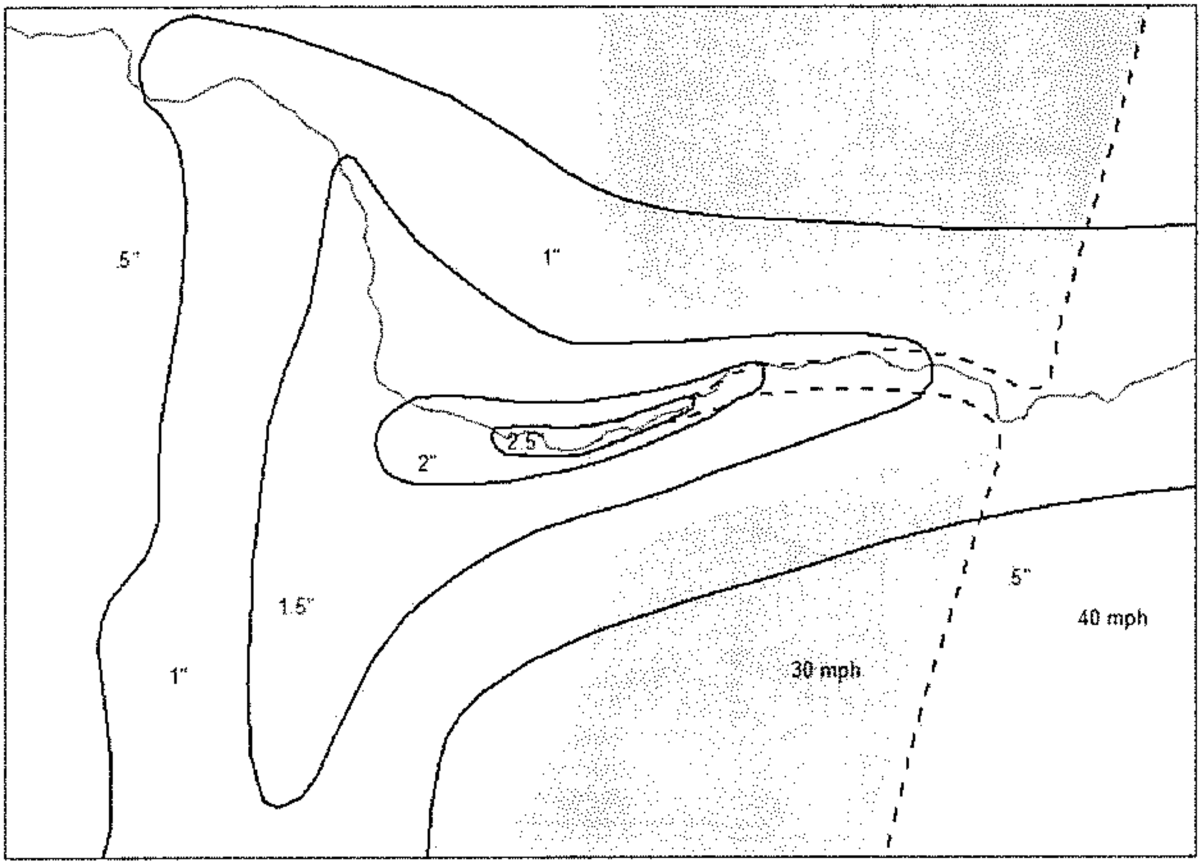


Figure B-10: Equivalent Radial Ice Thicknesses Caused by Freezing Rain with Concurrent 3-Second Gust Speeds, for a 500-Year Mean Recurrence Interval (Alaska)

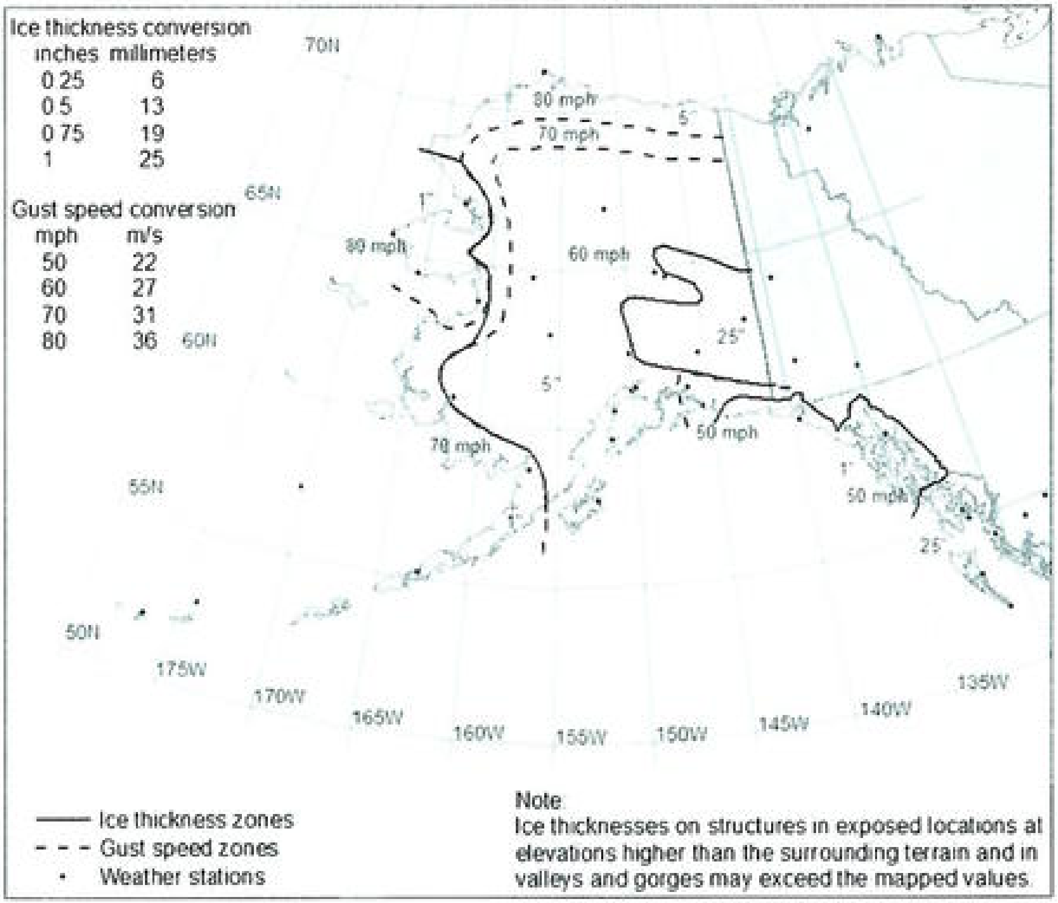


Figure B-11: S_s Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for 0.2 *s* Spectral Response Acceleration (5% of Critical Damping), Site Class B

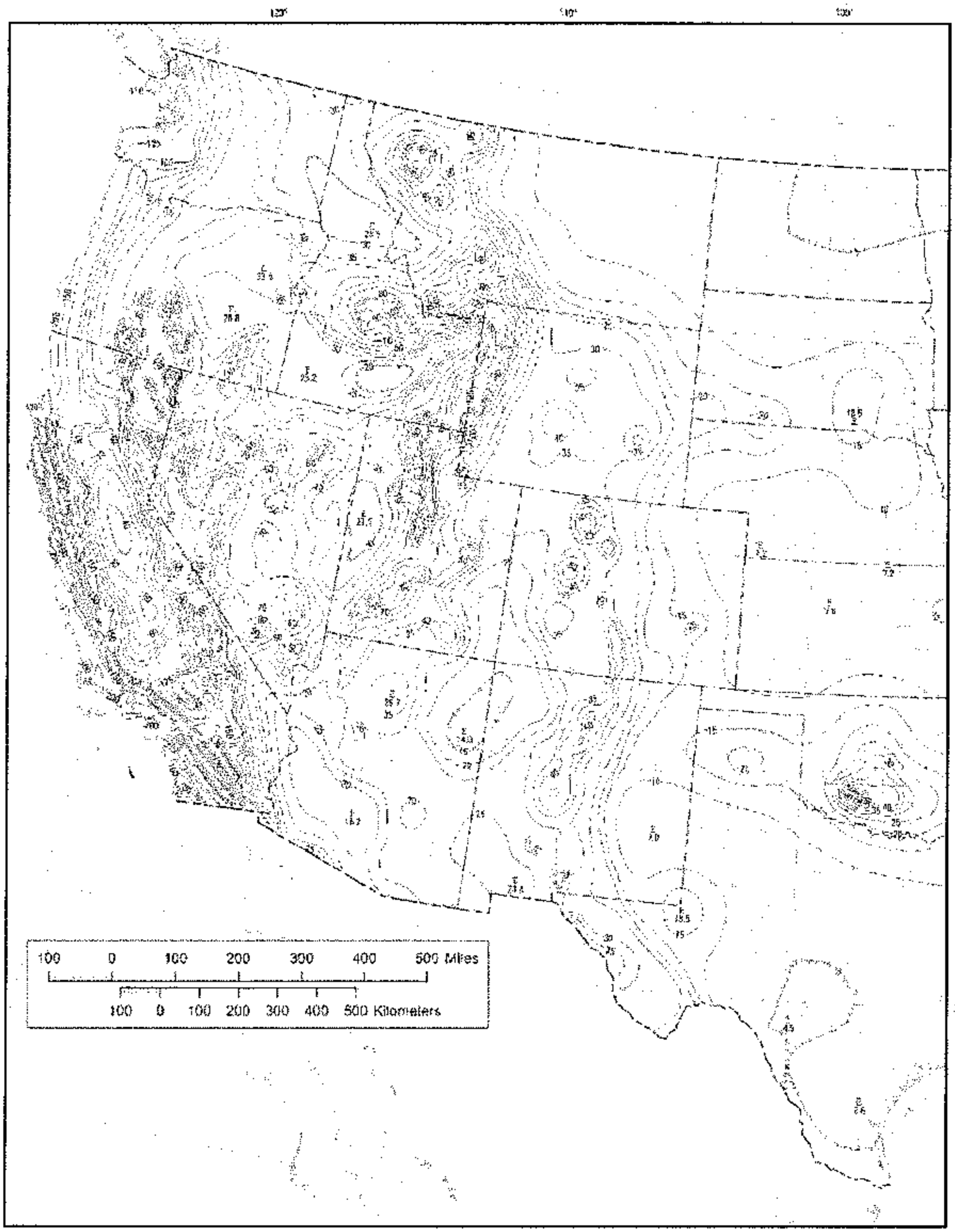


Figure B-11: S_s Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for 0.2 s Spectral Response Acceleration (5% of Critical Damping), Site Class B (Continued)

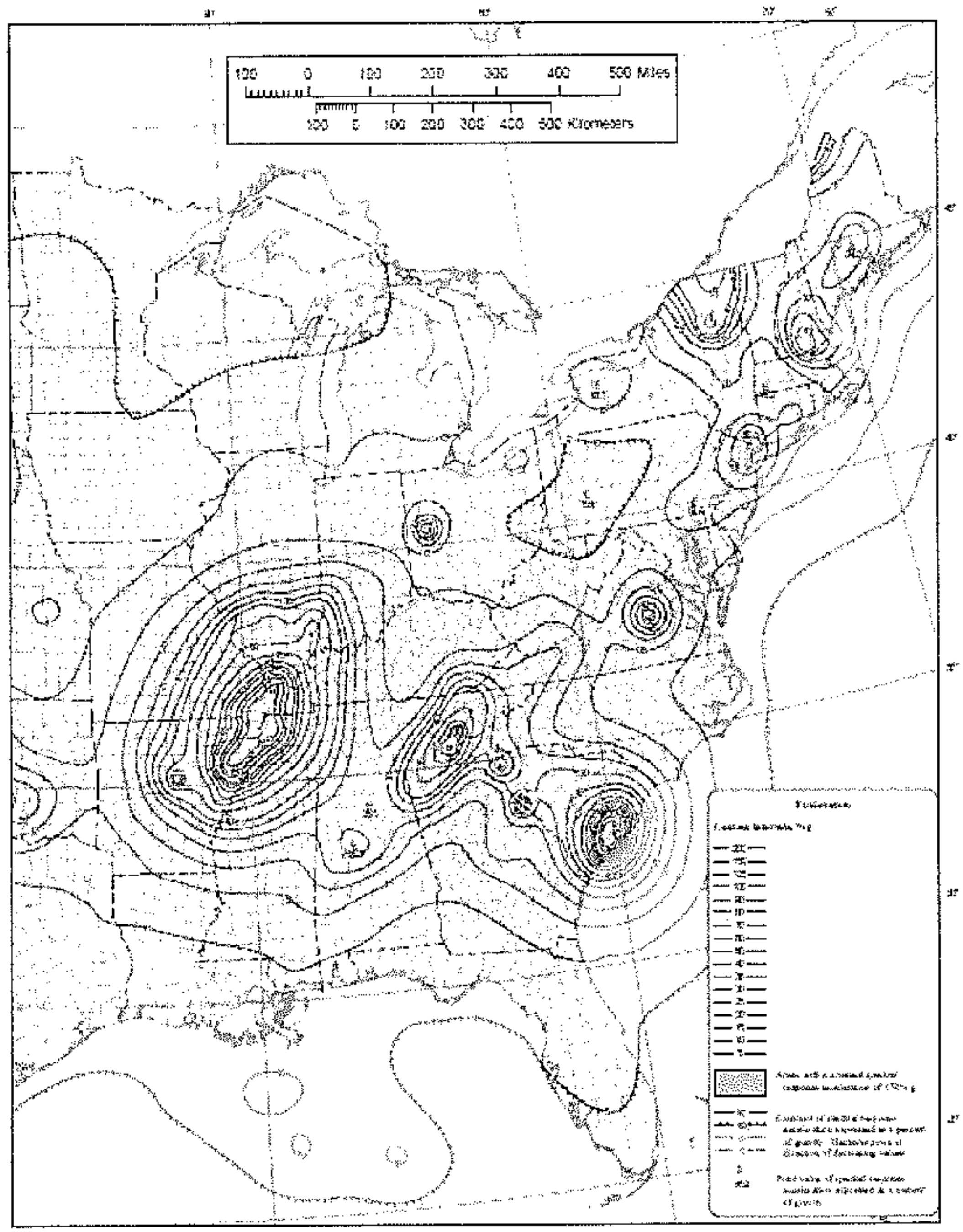


Figure B-12: S_1 Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for 1.0 s Spectral Response Acceleration (5% of Critical Damping), Site Class B

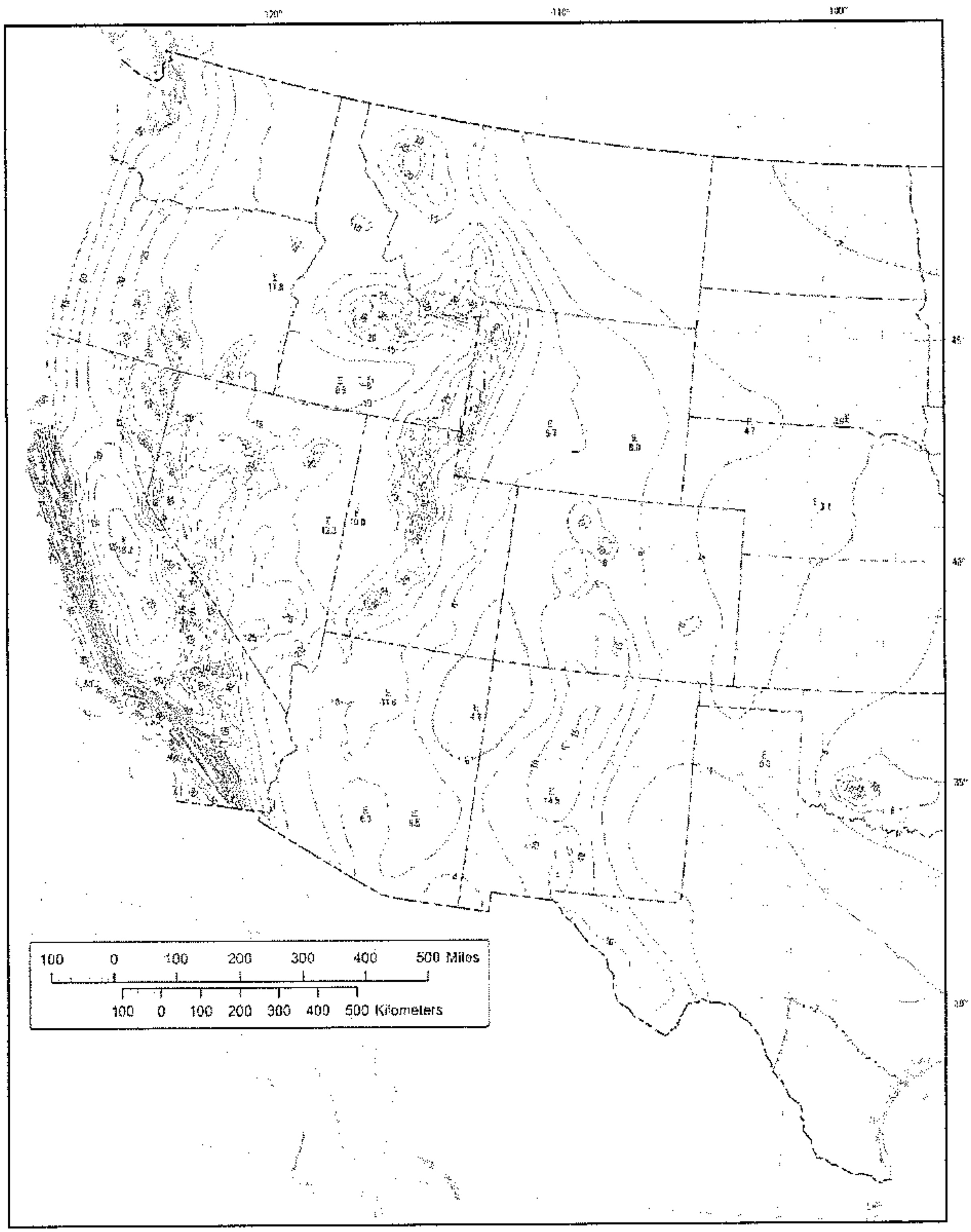


Figure B-12: S_1 Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for 1.0 *s* Spectral Response Acceleration (5% of Critical Damping), Site Class B (Continued)

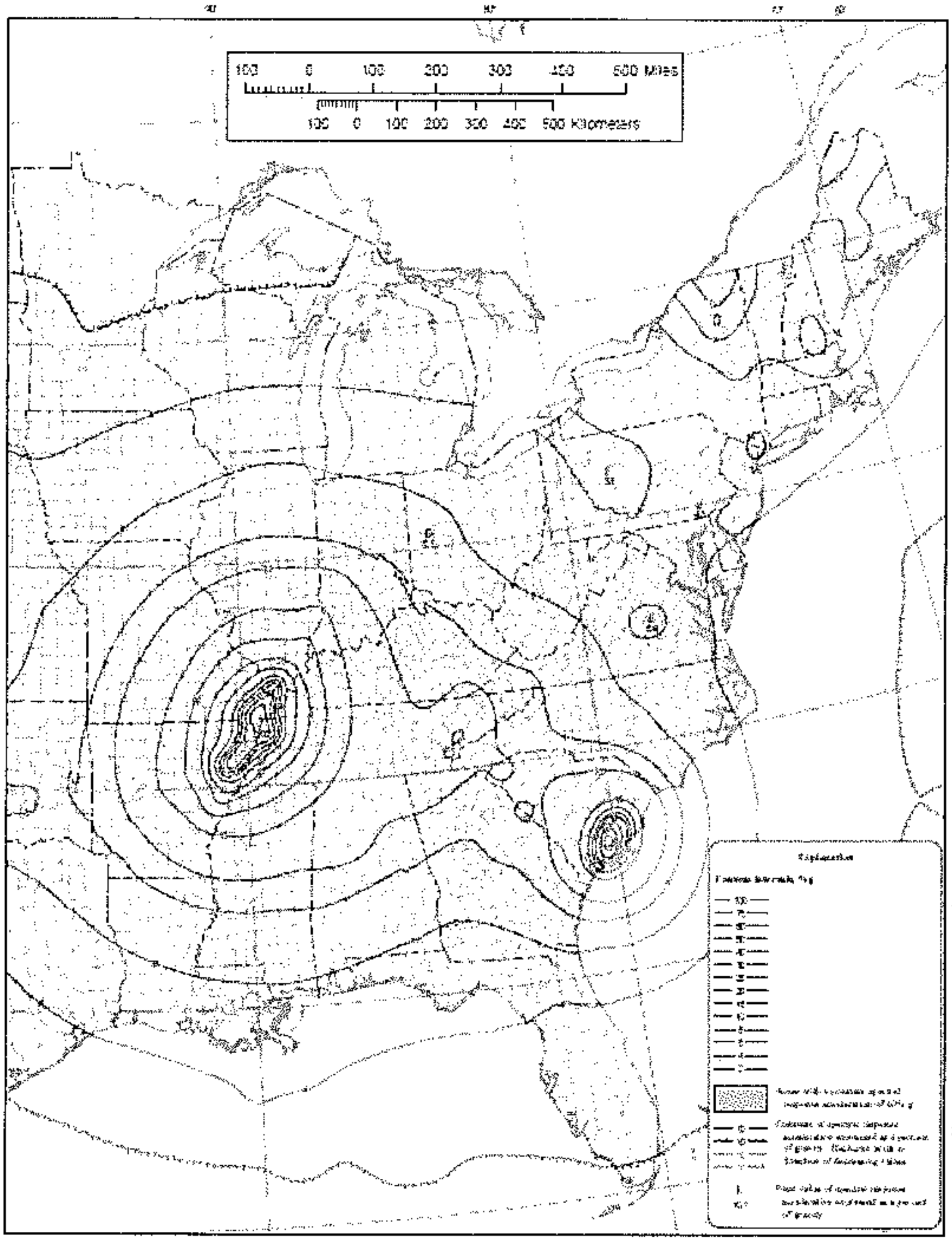


Figure B-13: S_S and S_1 Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for Guam and the Northern Mariana Islands for 0.2 and 1.0 *s* Spectral Response Acceleration (5% of Critical Damping), Site Class B

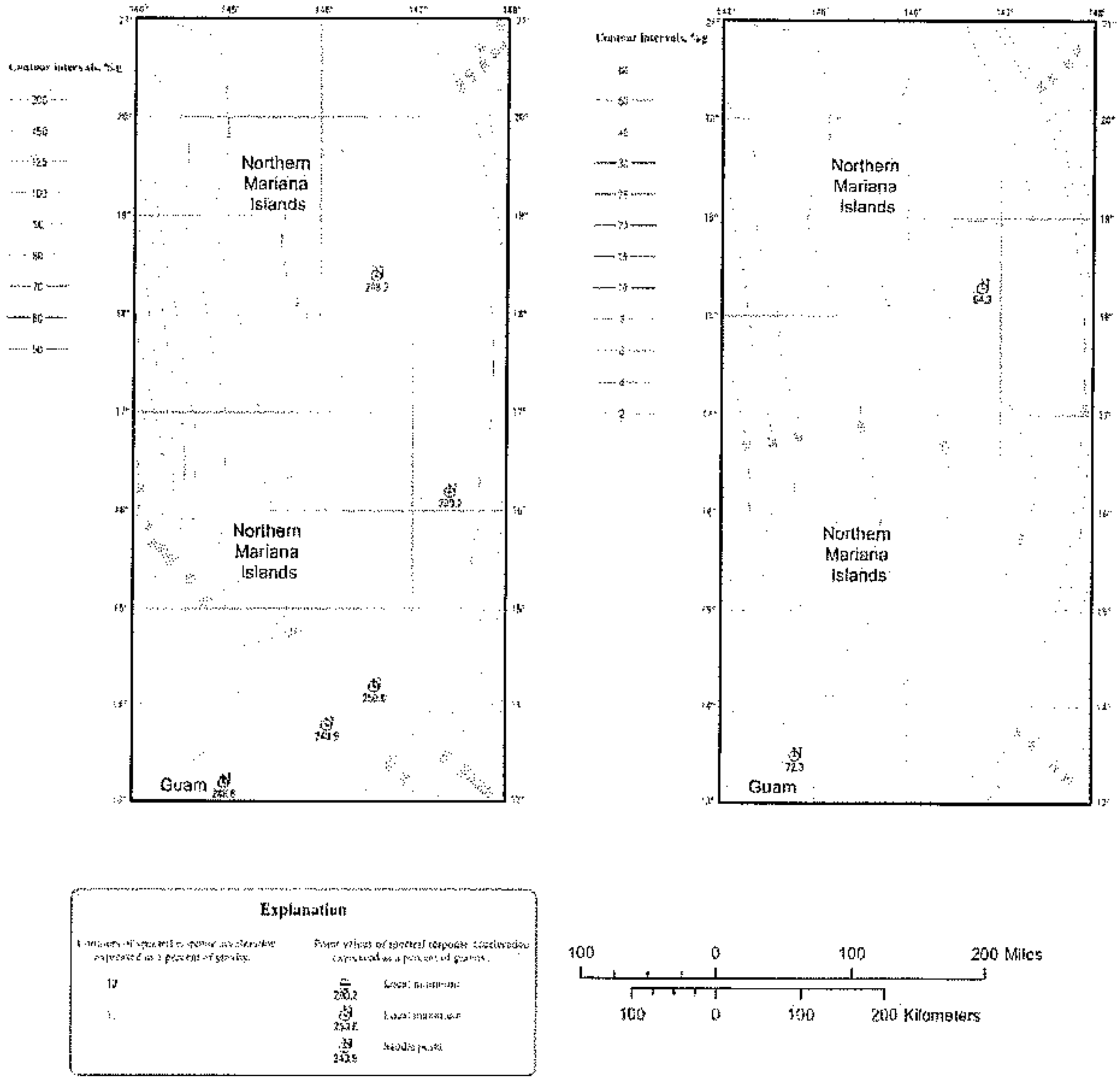


Figure B-14: S_S and S_1 Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for American Samoa for 0.2 and 1.0 *s* Spectral Response Acceleration (5% of Critical Damping), Site Class B

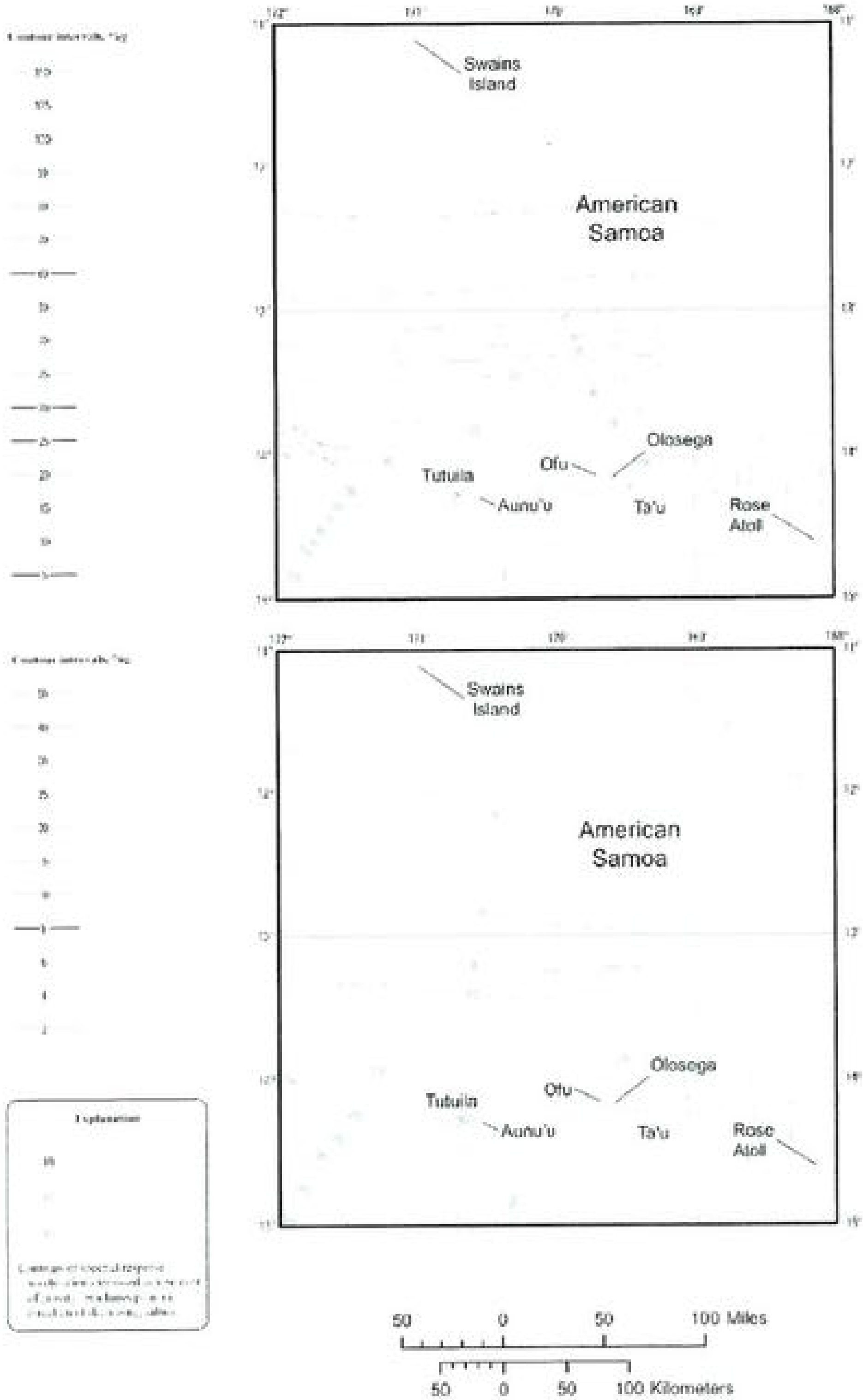


Figure B-15: S_S and S_1 Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for Puerto Rico and the United States Virgin Islands for 0.2 and 1.0 *s* Spectral Response Acceleration (5% of Critical Damping), Site Class B

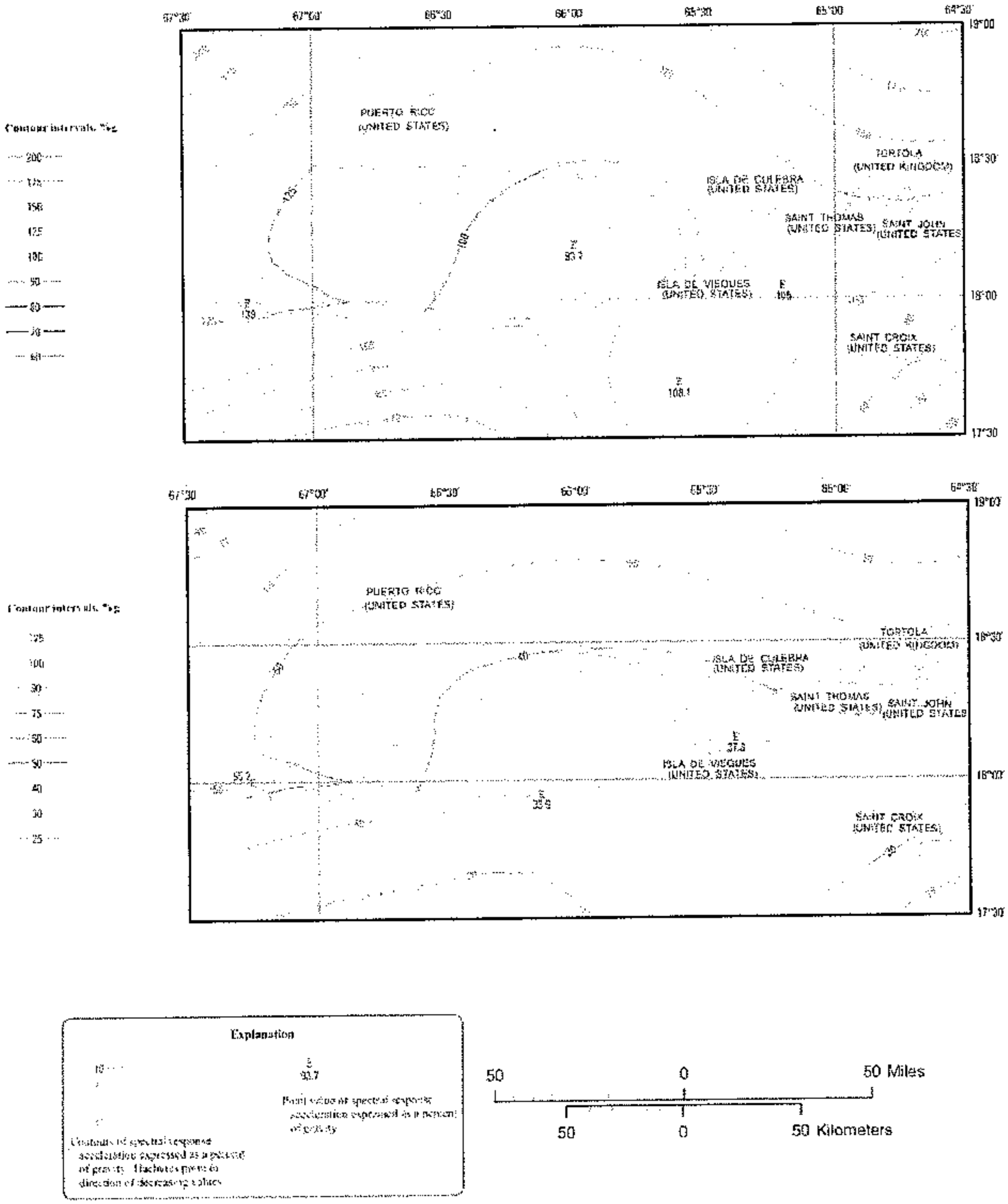
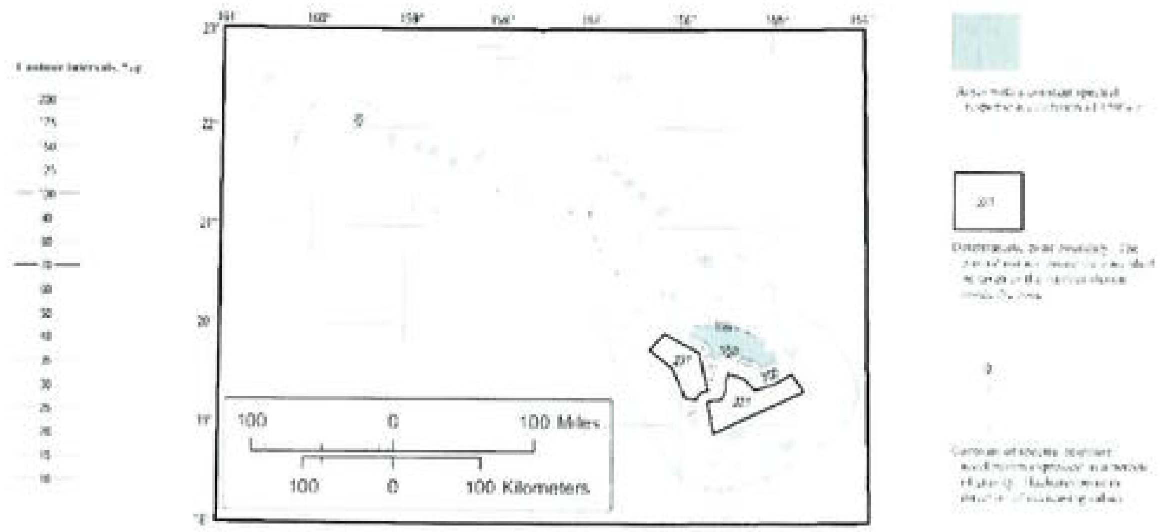


Figure B-16: S_S and S_1 Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for Hawaii for 0.2 and 1.0 *s* Spectral Response Acceleration (5% of Critical Damping), Site Class B



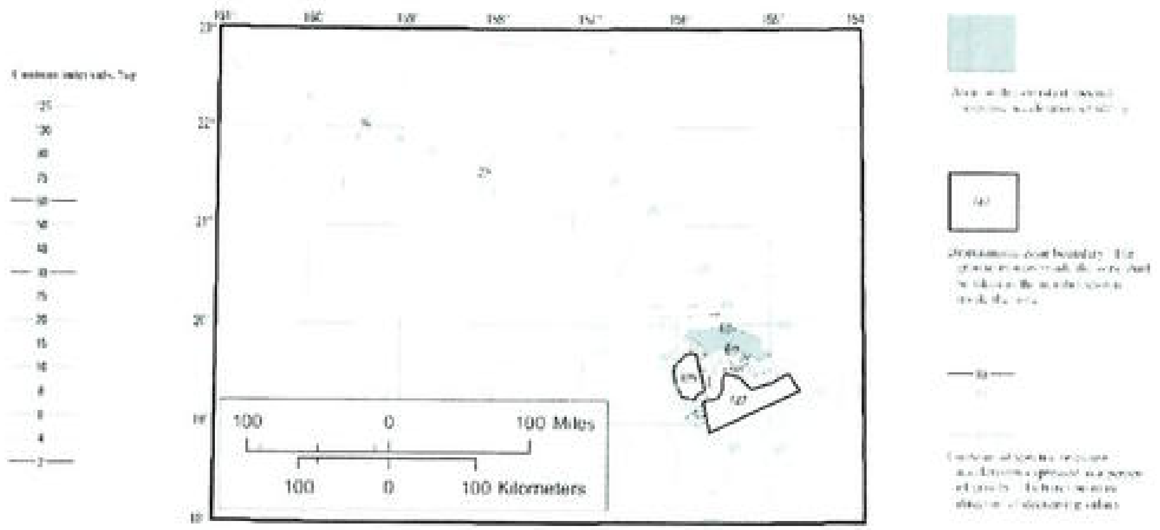


Figure B-17: S_S Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for Alaska for 0.2 *s* Spectral Response Acceleration (5% of Critical Damping), Site Class B

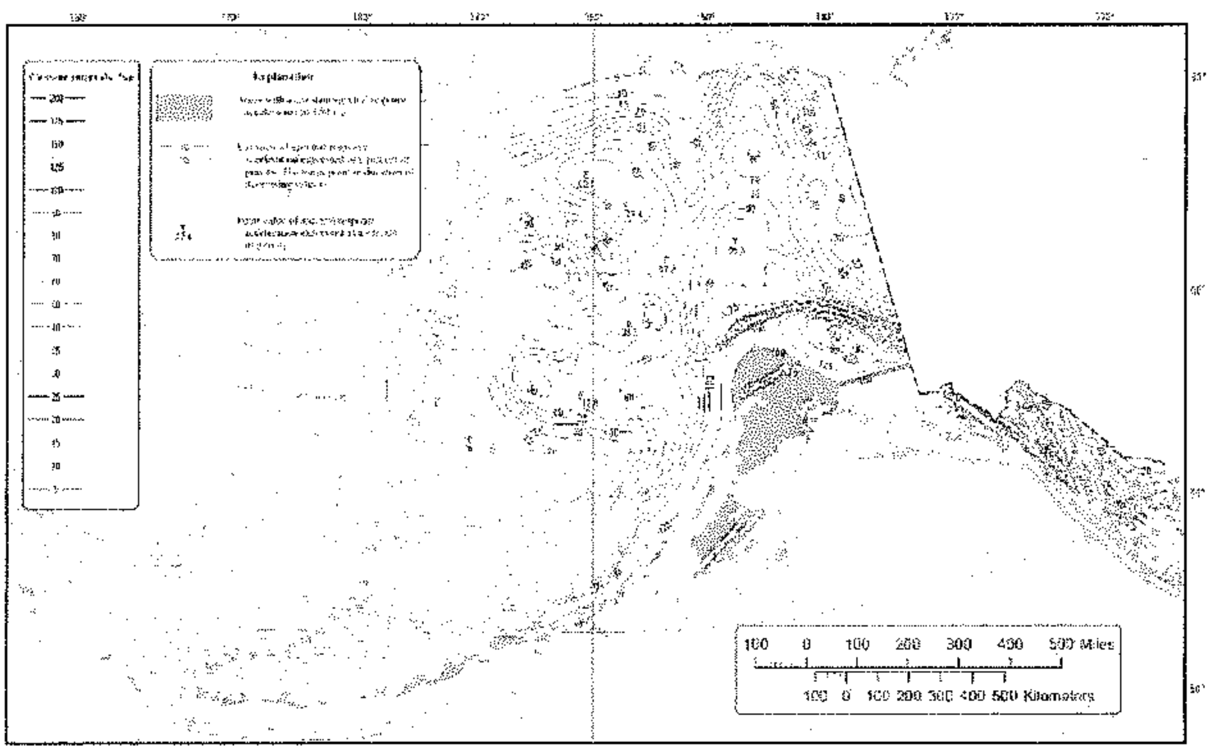


Figure B-18: S_1 Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for Alaska for 1.0 *s* Spectral Response Acceleration (5% of Critical Damping), Site Class B

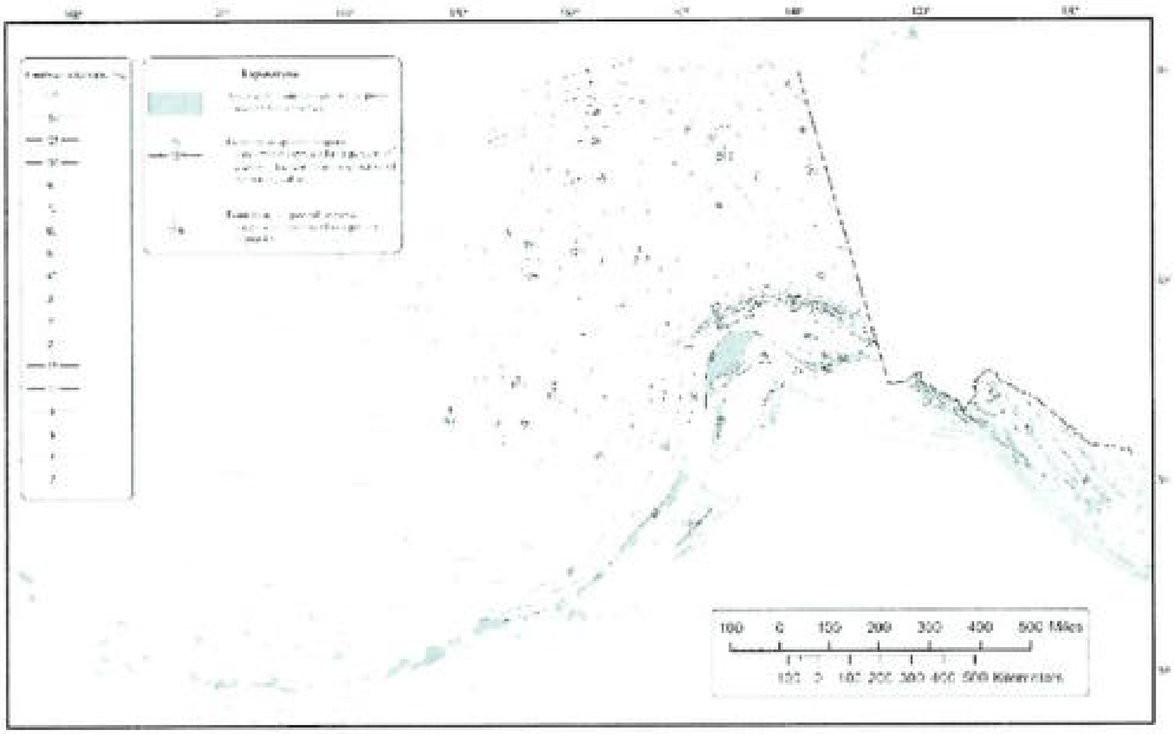


Figure B-19: Mapped Long-Period Transition Period, T_L (s), for the Conterminous United States

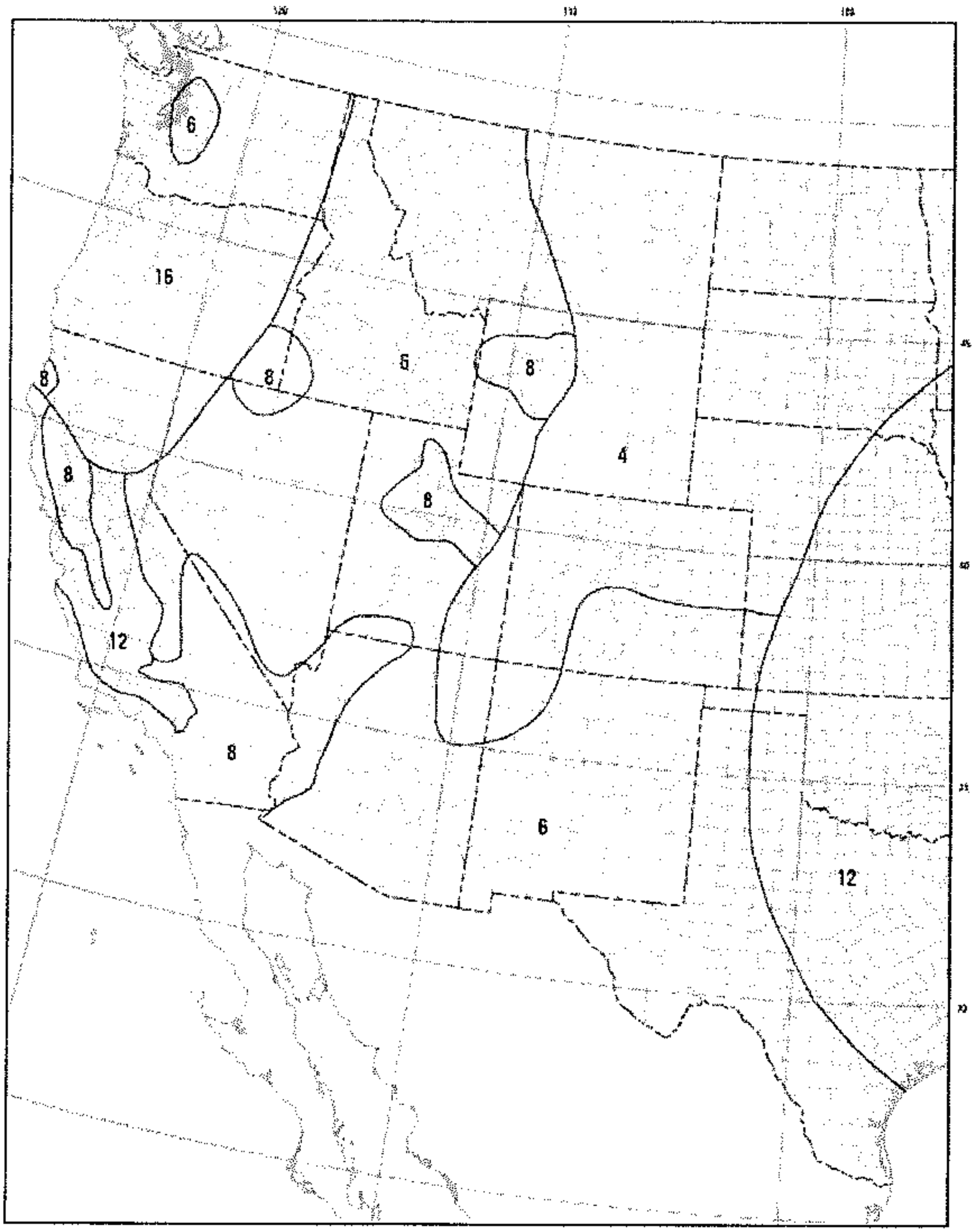


Figure B-19: Mapped Long-Period Transition Period, T_L (s), for the Conterminous United States (Continued)

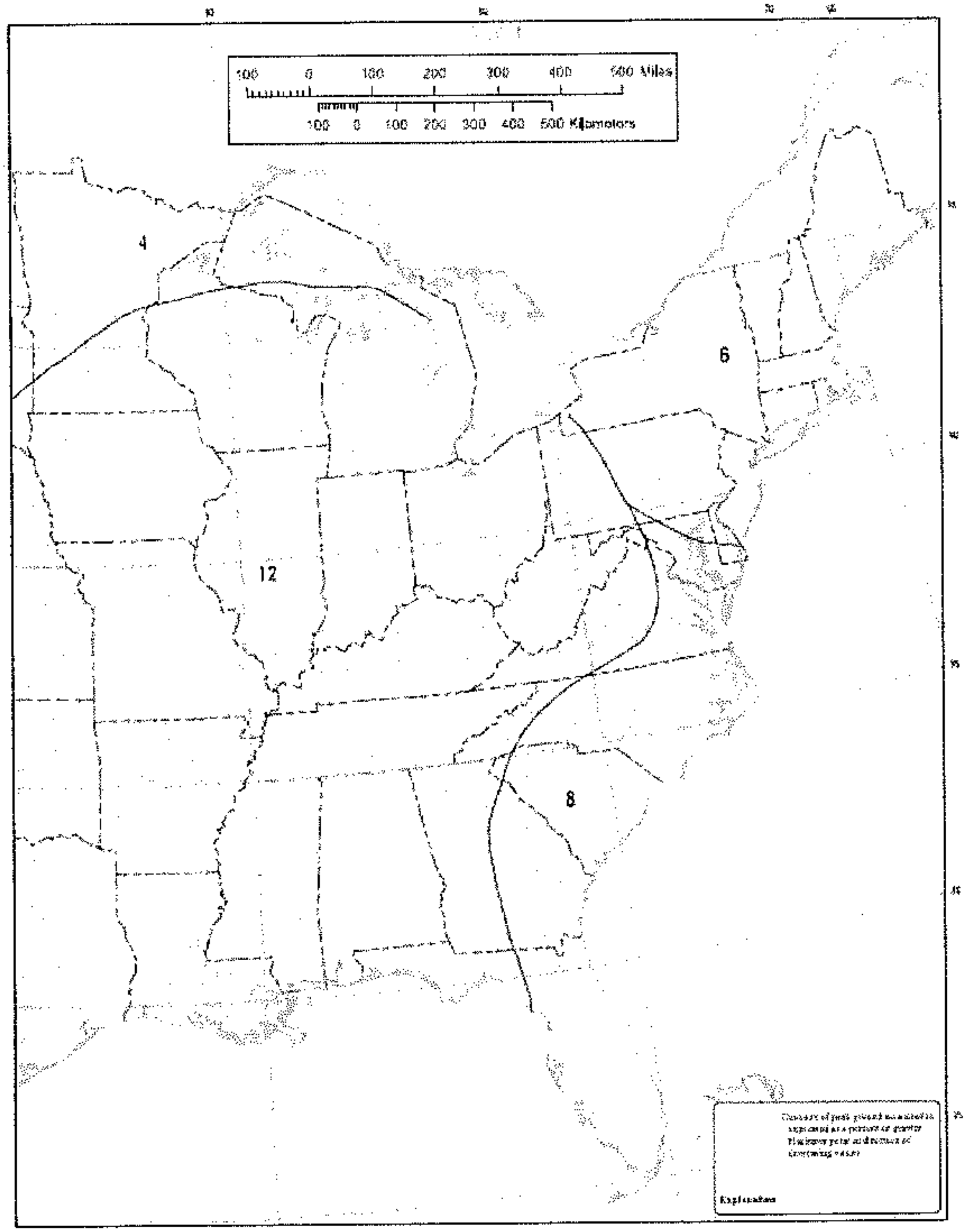


Figure B-20: Mapped Long-Period Transition Period, T_L (s), for Hawaii

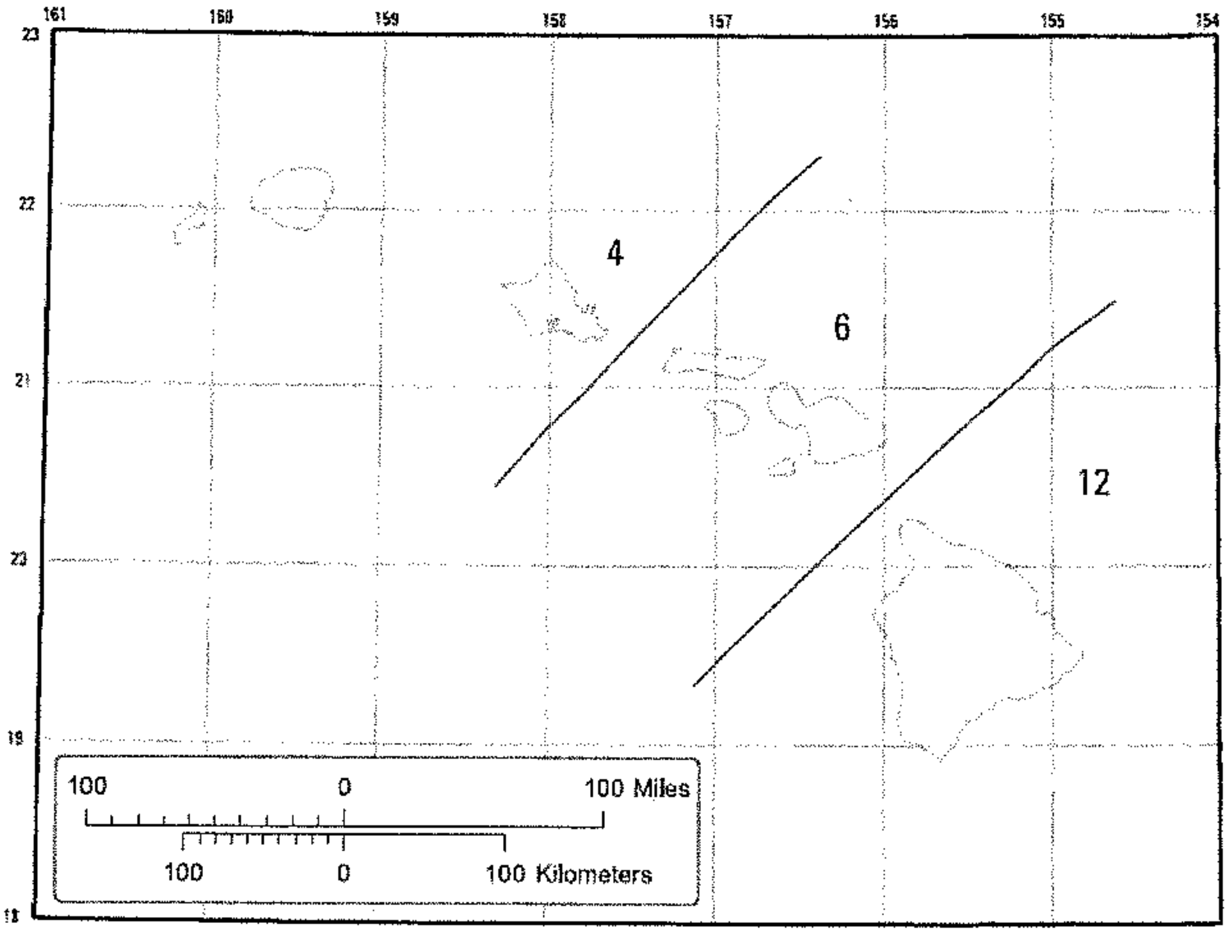


Figure B-21: Mapped Long-Period Transition Period, T_L (s), for Alaska

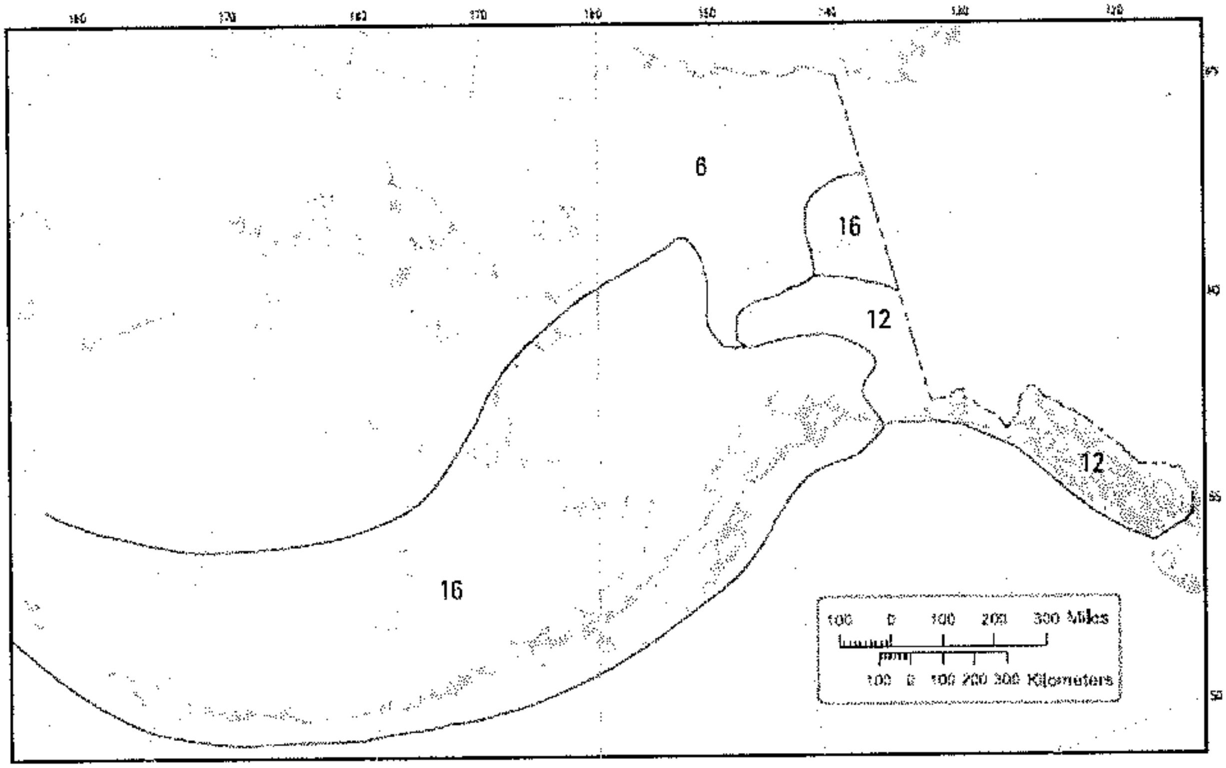


Figure B-22: Mapped Long-Period Transition Period, T_L (s), for Puerto Rico and the United States Virgin Islands

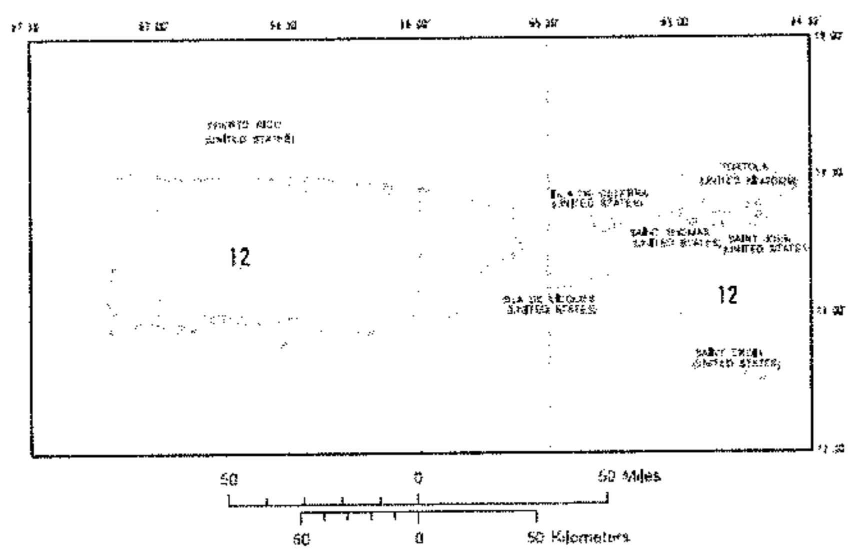
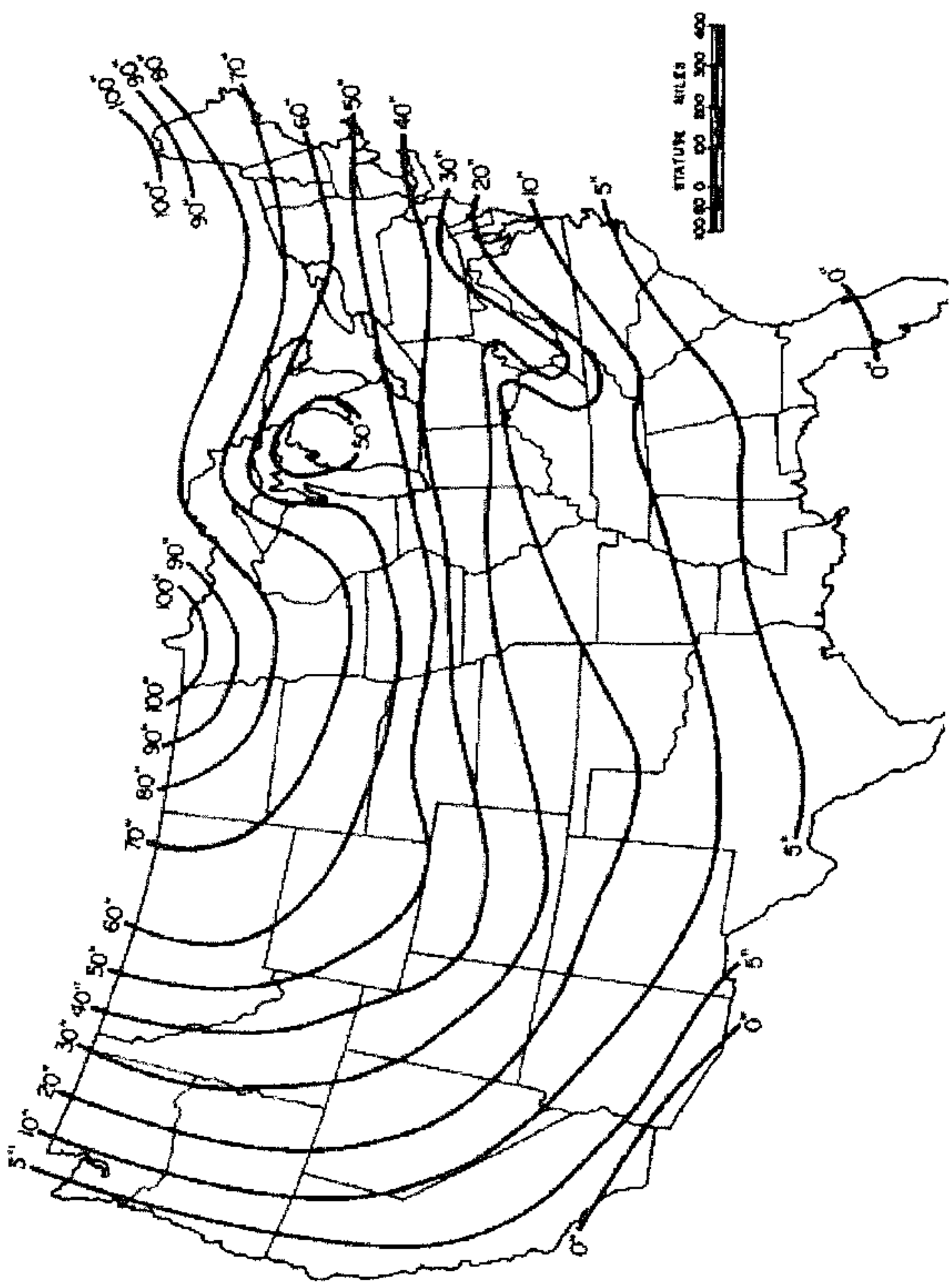


Figure B-23: Design Frost Depth



Note:

The frost depth for Alaska shall be based on regional climatic data and knowledge of local conditions.

ANNEX C: DESIGN WIND FORCE ON TYPICAL MICROWAVE ANTENNAS (Normative)

This Annex contains wind load data for typical microwave antennas.

When the azimuth orientations of antennas located on the same relative elevation on a structure are not specified, the antennas shall be assumed to radiate symmetrically about the structure.

Wind force data presented in this Annex for typical microwave antennas (including grid antennas) are described in the antenna axis system having the origin at the vertex of the reflector. The axial force, F_{AM}, acts along the axis of the antenna. The side force, F_{SM}, acts perpendicular to the antenna axis in the plane of the antenna axis and the wind vector. The twisting moment, M_M, acts in the plane containing F_{AM} and F_{SM} (refer to Figure C-1).

In all cases, the magnitude of F_{AM}, F_{SM} and M_M depend on the dynamic pressure of the wind, the projected frontal area of the antenna, and the aerodynamic characteristics of the antenna body. The aerodynamic characteristics vary with wind angle. The values of F_{AM}, F_{SM} and M_M shall be determined from the following equations:

$$F_{AM} = q_z\, G_h\, C_A\, A \qquad F_{SM} = q_z\, G_h\, C_S\, A \qquad M_M = q_z\, G_h\, C_M\, A\, D$$

where:

q_z = velocity pressure at vertex of the antenna from 2.6.11.6

G_h = gust effect factor from 2.6.9 (depending on the type of structure supporting the antenna)

C_A, C_S and C_M are the coefficients contained in Tables C-1 through C-4 as a function of wind angle, θ.

θ = wind angle, refer to Figure C-1 for positive sign conventions

A = outside aperture area of microwave antenna

D = outside diameter of microwave antenna

Table C-1: Wind Force Coefficients for Typical Microwave Antenna without Radome

WIND ANGLE θ (DEG.)	C_A	C_S	C_M
0	1.5508	0.0000	0.0000
10	1.5391	-0.0469	-0.0254
20	1.5469	-0.0508	-0.0379
30	1.5547	-0.0313	-0.0422
40	1.5938	0.0078	-0.0535
50	1.6641	0.0898	-0.0691
60	1.6484	0.2422	-0.0871
70	1.3672	0.4570	-0.0078
80	0.7617	0.3789	0.1000
90	-0.0117	0.3438	0.1313
100	-0.4023	0.3828	0.1320
110	-0.4609	0.4141	0.1340
120	-0.4570	0.4570	0.1430
130	-0.4688	0.4688	0.1461
140	-0.5742	0.4453	0.1320
150	-0.7734	0.3906	0.1086
160	-0.8672	0.2930	0.0836
170	-0.9453	0.1445	0.0508
180	-1.0547	0.0000	0.0000
190	-0.9453	-0.1445	-0.0508
200	-0.8672	-0.2930	-0.0836
210	-0.7734	-0.3906	-0.1086
220	-0.5742	-0.4453	-0.1320
230	-0.4688	-0.4688	-0.1461
240	-0.4570	-0.4570	-0.1430
250	-0.4609	-0.4141	-0.1340
260	-0.4023	-0.3828	-0.1320
270	-0.0117	-0.3438	-0.1313
280	0.7617	-0.3789	-0.1000
290	1.3672	-0.4570	0.0078
300	1.6484	-0.2422	0.0871
310	1.6641	-0.0898	0.0691
320	1.5938	-0.0078	0.0535
330	1.5547	0.0313	0.0422
340	1.5469	0.0508	0.0379
350	1.5391	0.0469	0.0254

Table C-2: Wind Force Coefficients for Typical Microwave Antenna with Radome

WIND ANGLE θ (DEG.)	C_A	C_S	C_M
0	0.8633	0.0000	0.0000
10	0.8594	0.1484	-0.0797
20	0.8203	0.2969	-0.1113
30	0.7617	0.4102	-0.1082
40	0.6641	0.4883	-0.0801
50	0.5469	0.5313	-0.0445
60	0.4180	0.5000	-0.0008
70	0.3125	0.4609	0.0508
80	0.2266	0.4375	0.1047
90	0.1328	0.4063	0.1523
100	0.0313	0.3906	0.1695
110	-0.0664	0.3711	0.1648
120	-0.1641	0.3477	0.1578
130	-0.2930	0.3203	0.1395
140	-0.4102	0.3047	0.0906
150	-0.5195	0.2734	0.0516
160	-0.6016	0.2266	0.0246
170	-0.6563	0.1484	0.0086
180	-0.6914	0.0000	0.0000
190	-0.6563	-0.1484	-0.0086
200	-0.6016	-0.2266	-0.0246
210	-0.5195	-0.2734	-0.0516
220	-0.4102	-0.3047	-0.0906
230	-0.2930	-0.3203	-0.1395
240	-0.1641	-0.3477	-0.1578
250	-0.0664	-0.3711	-0.1648
260	0.0313	-0.3906	-0.1695
270	0.1328	-0.4063	-0.1523
280	0.2266	-0.4375	-0.1047
290	0.3125	-0.4609	-0.0508
300	0.4180	-0.5000	0.0008
310	0.5469	-0.5313	0.0445
320	0.6641	-0.4883	0.0801
330	0.7617	-0.4102	0.1082
340	0.8203	-0.2969	0.1113
350	0.8594	-0.1484	0.0797

Table C-3: Wind Force Coefficients for Typical Microwave Antenna with Cylindrical Shroud

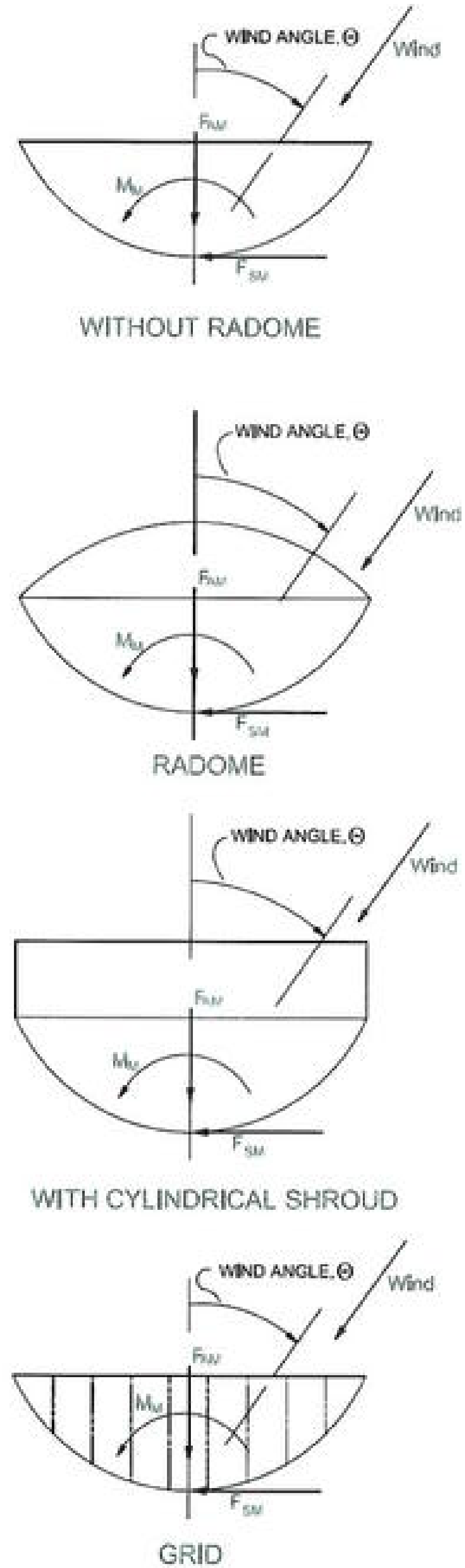
WIND ANGLE θ (DEG.)	C_A	C_S	C_M
0	1.2617	0.0000	0.0000
10	1.2617	0.0977	-0.0281
20	1.2500	0.1758	-0.0453
30	1.2109	0.2344	-0.0520
40	1.1563	0.2813	-0.0488
50	1.0859	0.3047	-0.0324
60	0.9453	0.3672	-0.0086
70	0.6719	0.4766	0.0227
80	0.2734	0.5820	0.0695
90	-0.1094	0.6250	0.0980
100	-0.3438	0.6016	0.1125
110	-0.5391	0.5313	0.1141
120	-0.7109	0.4375	0.1039
130	-0.8594	0.3125	0.0926
140	-0.9336	0.2305	0.0777
150	-0.9570	0.1758	0.0617
160	-0.9727	0.1484	0.0438
170	-0.9961	0.0977	0.0230
180	-1.0156	0.0000	0.0000
190	-0.9961	-0.0977	-0.0230
200	-0.9727	-0.1484	-0.0438
210	-0.9570	-0.1758	-0.0617
220	-0.9336	-0.2305	-0.0777
230	-0.8594	-0.3125	-0.0926
240	-0.7109	-0.4375	-0.1039
250	-0.5391	-0.5313	-0.1141
260	-0.3438	-0.6016	-0.1125
270	-0.1094	-0.6250	-0.0980
280	0.2734	-0.5820	-0.0695
290	0.6719	-0.4766	-0.0227
300	0.9453	-0.3672	0.0086
310	1.0859	-0.3047	0.0324
320	1.1563	-0.2813	0.0488
330	1.2109	-0.2344	0.0520
340	1.2500	-0.1758	0.0453
350	1.2617	-0.0977	0.0281

Table C-4: Wind Force Coefficients for Typical Microwave Grid Antenna without Ice

WIND ANGLE θ (DEG.)	C_A	C_S	C_M
0	0.5352	0.0000	0.0000
10	0.5234	0.1016	0.0168
20	0.5078	0.1797	0.0289
30	0.4609	0.2305	0.0383
40	0.4063	0.2617	0.0449
50	0.3438	0.2734	0.0496
60	0.2344	0.2813	0.0527
70	0.1289	0.2734	0.0555
80	0.0391	0.2500	0.0492
90	-0.0508	0.2422	0.0434
100	-0.1172	0.2734	0.0469
110	-0.1875	0.2852	0.0504
120	-0.2656	0.2773	0.0512
130	-0.3359	0.2617	0.0496
140	-0.4063	0.2344	0.0445
150	-0.4766	0.2031	0.0371
160	-0.5469	0.1563	0.0273
170	-0.5859	0.0859	0.0148
180	-0.5938	0.0000	0.0000
190	-0.5859	-0.0859	-0.0148
200	-0.5469	-0.1563	-0.0273
210	-0.4766	-0.2031	-0.0371
220	-0.4063	-0.2344	-0.0445
230	-0.3359	-0.2617	-0.0496
240	-0.2656	-0.2773	-0.0512
250	-0.1875	-0.2852	-0.0504
260	-0.1172	-0.2734	-0.0469
270	-0.0508	-0.2422	-0.0434
280	0.0391	-0.2500	-0.0492
290	0.1289	-0.2734	-0.0555
300	0.2344	-0.2813	-0.0527
310	0.3438	-0.2734	-0.0496
320	0.4063	-0.2617	-0.0449
330	0.4609	-0.2305	-0.0383
340	0.5078	-0.1797	-0.0289
350	0.5234	-0.1016	-0.0168

Note: For iced conditions, in the absence of more accurate data, coefficients from Table C-1 shall be used.

Figure C-1: Wind Forces on Typical Microwave Antennas



ANNEX D: TWIST AND SWAY LIMITATIONS FOR MICROWAVE ANTENNAS (Normative)

This Annex provides twist and sway deformation limits for structures supporting microwave antennas under the serviceability limit state condition.

The twist and sway limits of the structure at the elevation of an antenna, θ, shall be calculated in accordance with the following:

a) For a microwave antenna with an allowable 10 dB degradation in radio frequency signal level:

$$\theta = \frac{C_{10}}{D \quad \alpha}$$

b) For a microwave antenna with an allowable 3 dB degradation in radio frequency signal level:

$$\theta = \frac{C_3}{D \quad \alpha}$$

where:

θ = twist or sway deformation limit, degrees

C_{10} = 53.1 GHz-ft-deg. [16.2 GHz-m-deg.]

C_3 = 31.0 GHz-ft-deg. [9.45 GHz-m-deg.]

D = diameter of dish, ft. [m]

α = dish frequency, GHz

Notes:

1. It is not intended that the calculated values of θ imply an accuracy of beam width determination or structural rigidity calculation beyond known practical values and computational procedures. For most microwave antenna supporting structures, it is not practical to specify a calculated structural rigidity less than 0.25 degrees twist or sway with a 60 mph [27 m/s] basic wind speed.
2. Section A2.8 requires default limit state deformations to be based on an overall allowable 10 dB degradation. The equation based on 3 dB is provided for reference purposes.

ANNEX E: GUY RUPTURE (Normative)

E.1 Scope

This Annex defines a simplified equivalent static method that shall be used when it is a specified requirement to check for a guy rupture condition.

E.2 Introduction

An accurate analysis of a guyed mast for the dynamic effects caused by the sudden rupture of a guy is very complicated as it depends on the characteristics of the rupture, the damping of the structure, the vibration of the guys and the mast, etc. The following equivalent static method is provided to simulate the behavior of the structure immediately after a guy rupture.

The method presented herein utilizes the following simplifying criteria:

1. The rupture is a simple cut through the guy.
2. The elastic energy stored in the broken guy before the rupture is neglected.
3. Damping is not considered.
4. The wind loading at the time of guy rupture is minimal and is neglected.
5. For face guying or guy levels involving torsion stabilizers, the two guys in the same general direction shall be assumed to be broken. (Note: the torsional effects of a single broken guy under these guying configurations are beyond the scope of this Annex.)

Guyed masts with only one guy level must have fixed bases in order to provide any resistance to guy rupture.

This method replaces the dynamic forces acting on the mast just after a guy rupture with an equivalent horizontal static force, F_{dyn}, acting on the mast at the attachment level of the ruptured guy (refer to Figure E-1).

E.3 Analysis Method

1. The remaining guys excluding the ruptured guy (guy 1) are analyzed as a system with the mast replaced by a vertical only support under an applied horizontal force, F, acting in the direction of the broken guy. Curve 1 (refer to Figure E-2) is generated for different values of F by plotting the sum of the horizontal components in the direction parallel to F of the non-ruptured guys at the ruptured guy level with the corresponding deflections of the guy system at the ruptured guy level from the initial tension condition. (Note that the deflection increases as the applied horizontal force, F, decreases.)
2. The structure is analyzed with all guys removed at the ruptured guy level for different values of F, acting in the opposite direction of the broken guy. Curve 2 (refer to Figure E-2) is generated for different values of F and plotted with the corresponding deflections of the mast from the initial tension condition. (Note that the deflection increases as the applied horizontal force, F, increases.)
3. The area under curve 1 represents the energy that is lost in the non-ruptured guys as the mast deflects in their direction. The area under Curve 2 represents the energy absorbed by the structure as it deflects due to an external horizontal force. The

equivalent static force for the guy rupture condition, F_{dyn}, corresponds to the magnitude of the applied horizontal force, F, required to result in the area under Curve 2 to equal the area under Curve 1 (refer to Figure E-2).

4. F_{dyn} is applied to the structure (with a load factor equal to 1.0) with all guys removed at the same level and in the opposite direction of the ruptured guy (refer to Figure E-1). (Note that under this condition, the structure absorbs the energy lost in the non-ruptured guy system under the movement associated with the guy rupture. This conservation of energy is required to maintain equilibrium of the structure. The resulting member forces in the structure therefore simulate the member forces that would occur under a ruptured guy condition.)

Figure E-1: Guy Rupture Condition

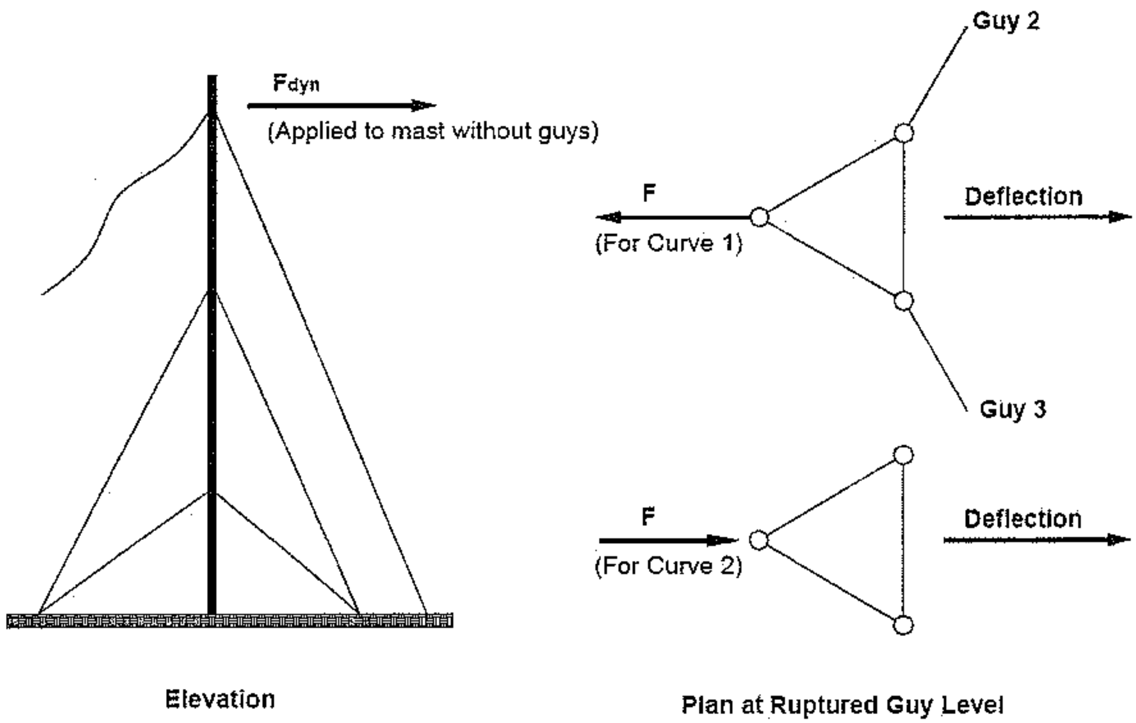
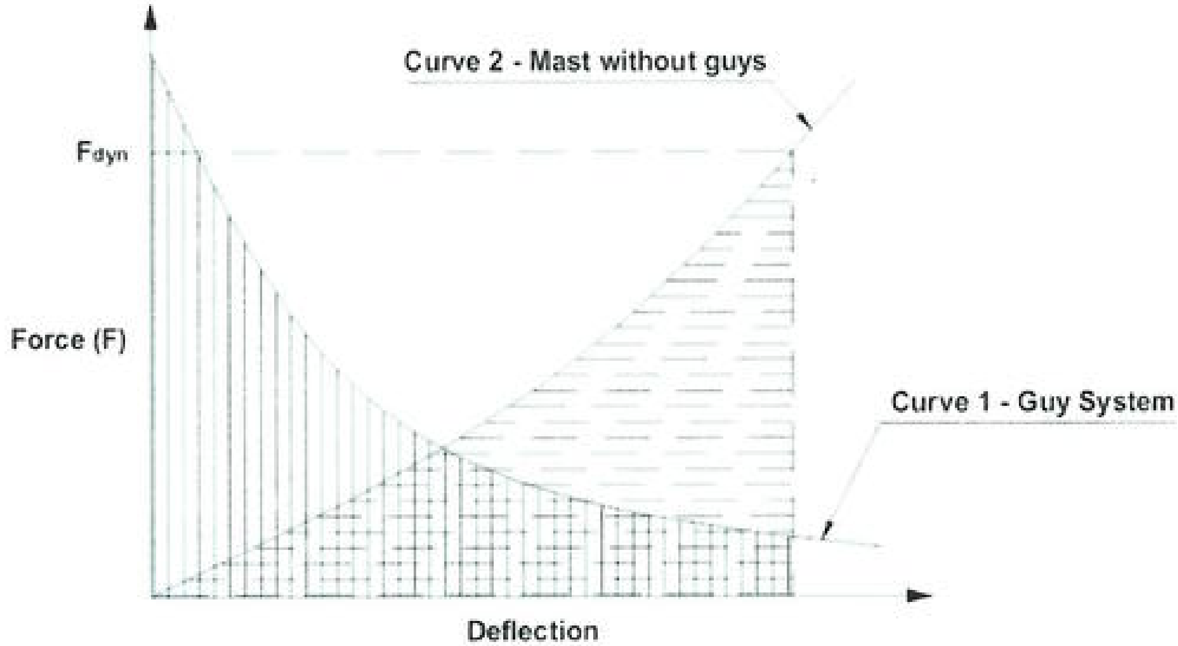


Figure E-2: Determination of F_{dyn}



F_{dyn} is defined as the force associated with the point along Curve 2 where the area under Curve 2 equals the area under Curve 1.

ANNEX F: PRESUMPTIVE SOIL PARAMETERS (Normative)

This Annex provides presumptive soil parameters to be used in the absence of a geotechnical report. Clay soils are assumed to be non-expansive. The presumptive soil parameters in this Annex assume non-corrosive and dry conditions (non-buoyant). Site Class D shall apply for determining seismic load effects. When the site location is unknown, the frost depth shall be equal to 3.5 ft. [1.1 m]. Presumptive soil parameters and assumptions shall be validated for a specific site prior to installation.

Table F-1: Presumptive Soil Parameters

Soil Type	N (blows/ft.) [blows/m]	ϕ (deg.)	γ (lb/ft3) [kN/m³]	c (psf) [kPa]	Nominal Ultimate Net Bearing Strength (psf) [kPa]		S_f (psf) [kPa]	k (pci) [kN/m³]	ε_{50}
					Shallow Foundations	Deep Foundations			
Clay	8 [26]	0	110 [17]	1000 [48]	5000 [240]	9000 [430]	500 [24]	150 [41,000]	0.01
Sand	10 [33]	30	110 [17]	0	4000 [190]	9000 [430]	500 [24]	35 [9,500]	N/A

where:

N = standard penetration value

ϕ = angle of internal friction

γ = effective unit weight of soil

c = cohesion

S_f = ultimate skin friction

k = lateral modulus of soil reaction

ε_{50} = strain at 50% of ultimate compression

Shallow Foundations: are isolated foundations such as pier on pads and mats.

Deep Foundations: are drilled piers, piles, and drill and bell foundations.

Note: Actual soil design parameters based on a geotechnical report with similar standard penetration values may vary from the tabulated values.

ANNEX G: GEOTECHNICAL INVESTIGATIONS (Informative)

This Annex contains information that should be contained in a geotechnical investigation.

G.1 Boring logs and report should include the following:

1. Date, sampling methods, number and type of samples.
2. Description of the soil strata according to the Unified Soil Classification System.
3. Depths at which strata changes occur referenced to a site benchmark elevation.
4. Standard Penetration Test blow counts for each soil layer.
5. Soil density for each soil layer.
6. Internal angle of friction for each soil layer.
7. Cohesion for each soil layer.
8. Ultimate bearing capacities for each soil layer or at the recommended bearing depth(s).
9. For expansive soil conditions, the active zone of influence and recommendations for design.
10. Elevation of free water encountered and the ground water depth below grade to be considered for design.
11. Frost depth to be considered for design.
12. Soil electrical resistivity, pH values and corrosive nature of soil.
13. Other pertinent soil design data and recommendations.
14. Recommendations for alternate foundation types.
15. Topographic information for the site.
16. Note the location within 1,000 ft. [300 m] of the structure of underground pipelines, buried concentric neutral power wires and electrical substations as these may affect electrolytic corrosion.

G.2 For drilled piers the following information shall also be provided:

1. Ultimate tip bearing capacity.
2. Ultimate skin friction for each soil layer.
3. Lateral modulus of soil reaction for each soil layer.
4. Ultimate soil strain at 50% of ultimate compression, ε_{50}, for each soil layer.

G.3 For rock anchors the following information shall also be provided:

1. Type and condition of rock.
2. Rock quality designation, RQD.
3. Percent rock sample recovered.
4. Ultimate bond stress in the interface between the rock and grout.
5. Ultimate shear strength.

ANNEX H: ADDITIONAL CORROSION CONTROL (Normative)

This Annex provides additional corrosion control methods for steel guy anchorages and ground embedded poles in direct contact with soil.

Additional corrosion control methods are required for steel in direct contact with soil when the measured soil electrical resistivity is less than 50 ohm-m and/or the measured soil pH values are below 3 or greater than 9 for Risk Categories II, III and IV structures.

Additional corrosion control methods are also recommended for AM antenna sites and sites known to be in close proximity to underground buried pipelines, underground buried cables that utilize a concentric neutral or located within 1,000 ft. [300 m] of an electrical substation.

Sites with soils with a high salt or organic content, oxygen differential or transfer, significant moisture content fluctuation, or with high redox potential (microbiological corrosion potential) may be susceptible to accelerated corrosion and it is recommended that a corrosion control expert establish the control measures for the site.

Additional Corrosion Control Methods

1. Cathodic protection utilizing sacrificial anodes: the size, type and placement of anodes are to be determined by a competent corrosion control specialist or firm.
2. Cathodic protection utilizing an impressed current: a competent corrosion specialist or corrosion control firm shall determine the impressed current system to be utilized.
3. Concrete encasement or backfill: sulfate resisting concrete mix designs should be used for all concrete below grade depending upon the concentration of soluble sulfates that exist in the soil or groundwater.

   When a concrete deadman is used with an anchor, the reinforcing in the concrete encasement shall be properly developed into the concrete deadman to prevent excess cracking and the concrete encasement shall extend a minimum of 6 in. [150 mm] above grade.
4. Taping or coating of steel in direct contact with soil with special corrosion control products that are designed to remain crack free and chemically stable over the anticipated life of the structure. Special precautions are required for this method during installation and backfill operations to avoid damage to the coating. Accelerated corrosion may occur at the damaged location.

ANNEX I: CLIMBER ATTACHMENT ANCHORAGES (Informative)

This Annex is intended to provide examples of climber attachment anchorages that may be judged by a competent climber after inspection to be capable of supporting the potential forces that could be encountered during a fall in accordance with the ASSE A10.48 definition for a Fall Protection Non-Certified Anchorage.

Caution shall be used to ensure that attachments are made to sound members that do not exhibit signs of damage and/or excessive corrosion.

Figure I-1: Examples of Climber Attachment Anchorages (All Welded Sections)

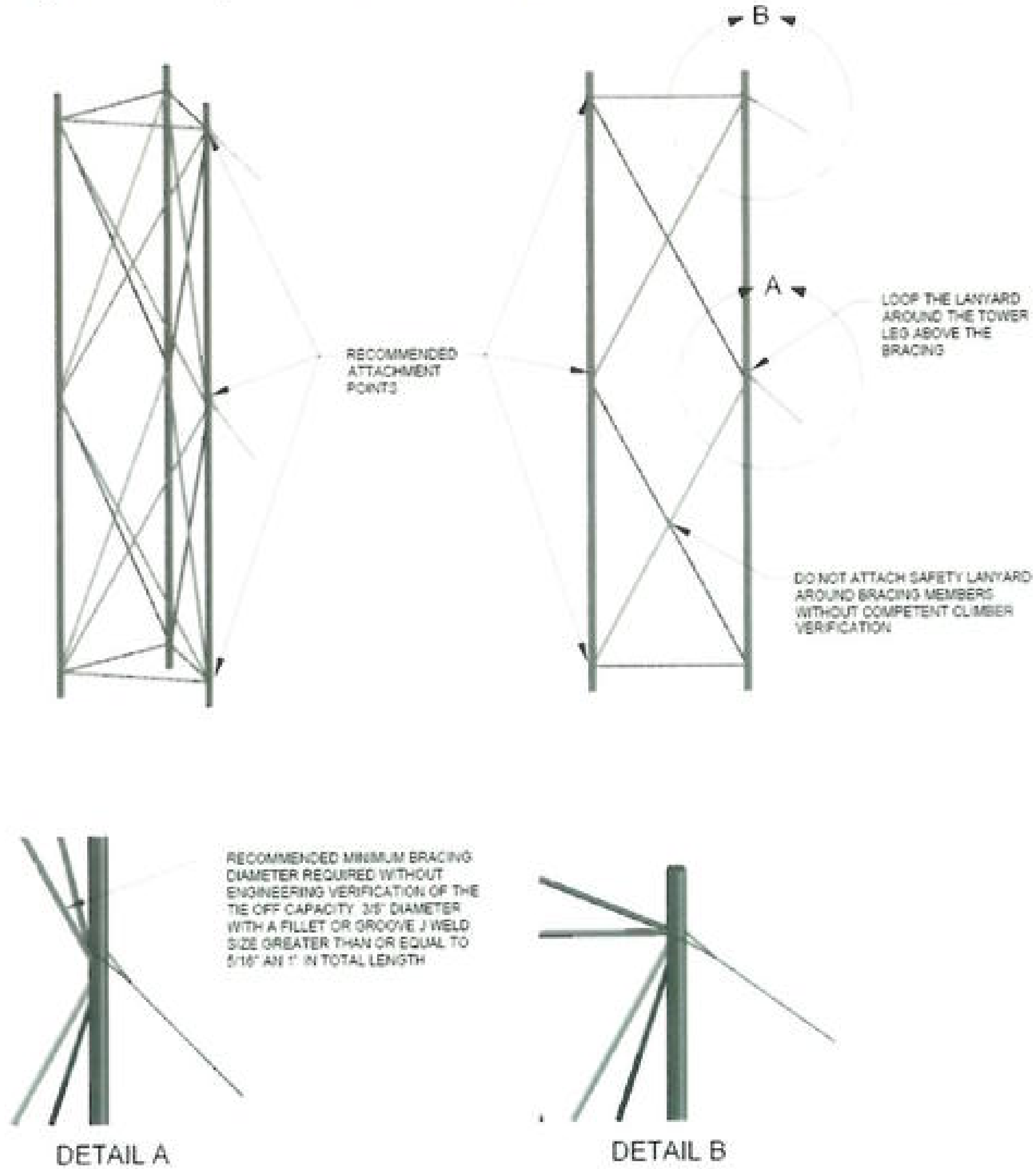
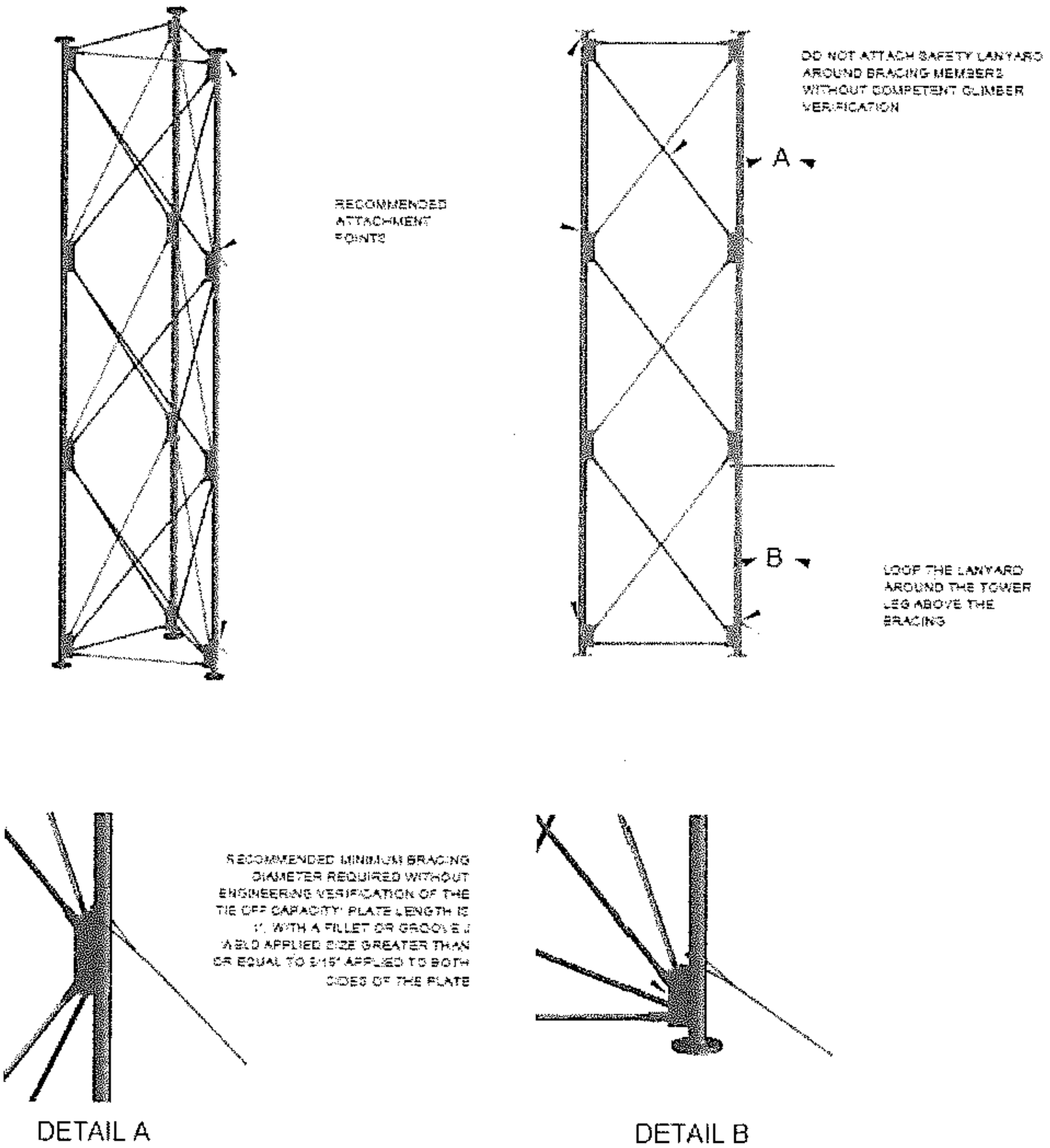


Figure I-2: Examples of Climber Attachment Anchorages (Bolted Sections)



Note: Similar climbing attachment anchorages exist at the apex of K-type bracing patterns with horizontal plan bracing connection details matching those illustrated.

ANNEX J: MAINTENANCE AND CONDITION ASSESSMENT (Normative)

This Annex provides guidelines for maintenance and condition assessments and for field mapping of structures and appurtenances. The maintenance and condition assessment for a site-specific structure may vary depending upon the type of structure and site-specific conditions.

It is not the intent of this Annex to provide guidelines for reconciliation of items identified in a maintenance and condition assessment. When a maintenance and condition assessment is used as the basis of a comprehensive structural analysis (refer to Section 15.0), items requiring reconciliation shall be addressed in the structural analysis report.

Refer to Annex P for reconciliation of cracks observed in base or flange plates of tubular pole structures.

The criteria for safety practices for maintenance and condition assessments is not within the scope of this Annex (refer to the ANSI/ASSE A10.48 Standard). Safety climb systems shall be inspected prior to use when performing a maintenance and condition assessment. For guyed masts, prior to climbing, the condition of guy anchor shafts with steel in direct contact with soil shall be assessed in accordance with the corrosion management plan for the site prior to a maintenance and condition assessment (refer to section 14.4 and Annex A).

J.1 Maintenance and Condition Assessment

The location of all items identified in a maintenance and condition assessment shall be noted.

A) Structure Condition

1. Damaged members (legs and bracing).
2. Loose members.
3. Missing members.
4. Loose and/or missing bolts and/or nut locking devices.
5. Visible cracks in welded connections including cracks underneath canister mounts for flag poles and other similar connections.
6. Pole flange and base plate cracks visible in base metal or at ends of plate stiffeners (cracks in base metal may only be visible on the inside surface of a pole).
7. Record temperature, wind speed and direction, and other environmental conditions.

B) Finish

1. Paint and/or galvanizing condition.
2. Rust and/or corrosion condition including mounts and accessories.
3. FAA or ICAO color marking conditions.
4. Water collection in members (to be remedied, e.g., unplug drain holes, etc.).

C) Lighting (external portions of components only)

1. Conduit, junction boxes, and fasteners (weather tight and secure).

2. Drain and vent openings (unobstructed).
3. Wiring condition.
4. Light lenses.
5. Bulb condition
6. Controllers:
   a) Flasher.
   b) Photo control.
   c) Alarms.
7. Obstructions to lighting system.

D) Grounding

1. Connections.
2. Corrosion.
3. Lightning protection.

Note: Lightning rods are not required for the protection of the structure in accordance with this Standard but may be required at or near the top of the structure for the protection of equipment or lighting systems.

E) Appurtenances such as Mounts, Antennas and Lines

1. Antenna and Mount condition:
   a) Proper tie-back of microwave dishes.
   b) Damage to supporting structure at connections.
   c) Defects, deformations, loose, missing members, etc.
   d) Loose or missing hardware.
   e) Condition of antenna covers.
2. Feed line condition:
   a) Flanges, seals, dents, jacket damage, grounding, etc.
   b) Properly secured/supported on the structure and mount.
   c) Hanger condition (snap-ins, bolt on, kellum grips, etc.).
   d) Secured to structure (waveguide ladder).

F) Other appurtenances (Ice shields, walkways, platforms, climbing facilities, sensors, floodlights, etc.)

1. Condition.
2. Obstructions to climbing path or safety climb systems.
3. Defects, deformations, loose or missing members, etc.
4. Loose or missing hardware.

5. Secured to structure.

G) Insulators (Base insulator, AM detuning kits, fiberglass rods, porcelain insulator, non-metallic guys, etc.)

1. Cracking and chipping.
2. Cleanliness of insulators.
3. Spark gaps.
4. Isolation transformer.
5. Bolts and connections.
6. Delamination, UV degradation, rod slippage.

H) Guys

1. Strand condition (corrosion, breaks, nicks, kinks, etc.).
2. Guy hardware conditions:
   a) Turnbuckles or equivalent:
      (i) Thread extended past body.
      (ii) Secured with safety cable or equivalent.
      (iii) Cracks, defects, damage, etc.

   b) Cable thimbles.
   c) Ice clips.
   d) Cable connectors (end fittings):
      (i) Cable clamps applied properly and bolts tight.
      (ii) Wire serving.
      (iii) Slippage or damaged strands.
      (iv) Deadend grips – fully wrapped, end sleeve/ice clips (on anchor end).
      (v) Poured sockets – signs of separation, twisting, etc.
      (vi) Shackles, bolts, pins and cotter pins.

   e) Inspect tension rods / anchor rods welded to fan plates for fatigue cracks.

3. Guy tensions.
   a) Measure guy tensions (refer to Annex K).
   b) Record temperature, wind speed and wind direction.

   Note: Minor variations in guy tensions are to be expected due to temperature, wind speed conditions, anchor elevation differences, etc.

I) Concrete Foundations

1. Ground condition:

a) Settlement, movement or earth cracks.

b) Erosion.

c) Site condition (standing water, drainage, trees, etc.).

2. Anchorage condition:

   a) Top and bottom base plate nuts tight.

   b) Nut locking device.

   c) Grout condition.

   d) Anchorages.

   e) Anchor rods.

3. Concrete condition:

   a) Cracking, spalling, or splitting.

   b) Chipped or broken concrete.

   c) Honeycombing.

   d) Low spots to collect moisture.

J) Guyed Mast Anchors

a) Settlement, movement or earth cracks.

b) Grade sloped away from anchors.

c) Anchor shaft condition below grade.

d) Corrosion control measures (galvanizing, coating, concrete encasement, cathodic protection systems, etc.).

e) Anchor heads above grade, clear of vegetation, obstructions, etc. and turnbuckles free to articulate.

K) Structure Alignment

1. Structure Plumb and Twist (refer to Figures J-1 and J-2 for latticed structures).

Note: The assembly tolerances specified in Section 13.0 represent tolerances that are readily obtainable using ordinary installation methods as opposed to tolerances beyond which structural capacity is reduced to insignificance. Measured misalignments within the tolerances specified in Section 13.0 may be ignored in the analysis of a structure; however, when measured misalignments exceed the specified tolerances, the measured misalignments must be accounted for in the analysis model of that structure.

L) Structure Modifications

1. Refer to Annex O for inspection guidelines for post modification inspections.

J.2 Field Mapping

J.2.1 Mapping of Appurtenances

The mapping of appurtenances shall provide sufficient dimensional data in order to calculate the effective projected area, weight and location of all appurtenances.

The mapping of appurtenances shall include, as a minimum:

1. Inventory of existing antennas: elevation, antenna type and dimensions/model number, support mount and location, spacing and orientation on cross-section, and corresponding transmission line(s).
2. Inventory of other appurtenances (such as climbing ladders, safety climb systems, platforms, etc.): elevation, appurtenance type and dimensions, location, spacing and orientation on cross-section.
3. A cross-section sketch locating and labeling the transmission lines (size and spacing) and showing the orientation of the lines and the structure with respect to North. For transmission lines in clusters: number of lines per row, number of rows, and separation between the lines, overall width and depth dimensions.

J.2.1.1 Mounting Systems

The tolerances for these systems shall be in accordance with the structural tolerances specified in J.2.3.2.

The mapping of mounting systems shall include, as a minimum:

1. Inventory of existing antennas and tower mounted equipment: elevation, antenna type and/or dimensions/model number, support mount and location, and spacing and orientation on cross-section.
2. Azimuth of the mounting system – The radial direction in plan view that a mount projects outward from the supporting structure. For a sector mount, the direction normal to the plane formed by the mounting pipes. For a side arm mount, the direction parallel to the plane formed by the side arm.
3. Tie back (strut, stabilizer) location and attachment point to the structure.
4. Sketches of the front, side, and top profiles of the mounting system. The sketches shall include all mount member sizes, overall dimensions and orientation.
   a) Each mounting assembly at an elevation shall be mapped separately.
5. Centerline elevation of the mounting system.
6. Sketches detailing the location, size, dimensions and orientation of mount modifications or reinforcements (e.g. horizontal support brace (tie-back), vertical support brace (kicker) and support rail).
7. Sketch detailing the connection of the mount to the structure:
   a) Size, thickness and shape of connecting plates.
   b) Bolt quantities, grade/head markings, sizes and configuration.

c) Details of mount collar, including size, thickness and shape of bent plates; and size and quantity of all-thread rods and nuts.

8. Flange connection information for mounts connected directly to the top flange of the structure including bolt size and flange plate thickness and diameter.
9. Sketch detailing the connection of the mount members:
   a) Size, thickness and shape of connecting plates.
   b) Bolt quantities, grade/head markings, sizes and configuration.

Note: When available, the mount manufacturer ID tag information and name of manufacturer.

J.2.2 Mapping of Structural Members and Connections

In order to perform an analysis of a structure, the structural configuration and the size of all structural members must be mapped in order to calculate wind loading and member capacities.

The mapping of the structure and its main structural members shall include, as a minimum:

J.2.2.1 Latticed Structures

1. Sketch of overall structure numbering all sections.
2. A sketch of each typical section indicating the following:
   a) Section height.
   b) Panel height and number of panels.
   c) Configuration of the panels (Single, X, X with horizontal, K, etc.).
   d) Face width (center-to-center of legs) at all taper change locations.
   e) Bracing offsets (distance above and below leg connection).
3. Member sizes for each section:
   a) Leg member sizes - i.e. pipe diameter (outside diameter & wall thickness), solid round diameter, or angle size & thickness (60 degrees or 90 degrees).
   b) Diagonal member sizes - i.e. pipe diameter (outside diameter & wall thickness), angle size, thickness and orientation (long leg back to back, LLBB, or short leg back to back, SLBB).
   c) Horizontal member sizes - i.e. pipe diameter (outside diameter & wall thickness), angle size, thickness and orientation (LLBB, SLBB).
   d) Sub-brace member sizes (if applicable) - i.e. pipe diameter (outside diameter & wall thickness), angle size, thickness and orientation (LLBB, SLBB).

J.2.2.2 Guyed Masts

1. Structure base type (fixed or pinned) and tapered or flat base.
2. Guy anchor dimensions: distance from base to guy anchors and their relative elevations to base and their orientation.
3. Sketch of overall structure and member sizes (refer to J.2.2.1):
   a) Location of all guy levels and torque arms.

b) Guy diameter, type and number of strands at each level.

c) Configuration and member sizes of torque arms.

d) Guy strap and lug details.

J.2.2.3 Pole Structures

1. Sketch of overall structure numbering all sections.
2. Configuration of each section:
   a) Section height - For flanged type, the length from splice to splice. For telescoping poles, the length from butt to butt.
   b) If multi-sided, number of sides.
   c) Flat-to-flat dimension or diameter and circumference at top and bottom of each section.
   d) Outside corner radius of adjacent flat sides.
   e) Port hole opening size, reinforcing dimensions, welds and location.
3. Size for each section:
   a) Wall thickness of each section.

J.2.2.4 Connections

In order to perform a comprehensive structural analysis of a structure, the details of all structural connections shall be mapped.

The mapping of the structure connections shall include, as a minimum, the following:

1. Member end connection details:
   a) If Bolted: number, type and size of end bolts and center bolts along with edge distances, gauges and coping.
   b) Size and thickness of gusset plate with related details (hole sizes, edge distances, weld size and length).
   c) If Welded: weld size and length of end and center connections.
   d) Splice connection details:
      (i) Number, type and size of bolts.
      (ii) Size and thickness of splice plate with related details (hole sizes, edge distances, weld size and length) and distance from panel intersection point.
   e) Anchor rod type, size, number and bolt circle.
2. Guy assembly and connection details:
   a) Deadend grips size/type, turnbuckle size, shackle size.
   b) Socket size, pin size, link plate dimensions with related details.
   c) Size and thickness of guy pull-off plate with related details (hole sizes, edge distances, weld size and length, stiffener size).

d) Guy anchor head plate size, thickness, holes size, spacing, and edge distances of holes, shaft type, size and extension length and angle from horizontal plane and weld size and length of connection between shaft and fan plate.

J.2.3 Tolerances

Measurements for mapping of a structure shall be in accordance with the following tolerances.

J.2.3.1 Dimensions

1. Elevation: +/- 6 in. [152 mm]
2. Member length: +/- 1/2 in. [13 mm]
3. Member diameter/width: +/- 1/16 in. [2 mm]
4. Member thickness: +/- 1/32 in. [1 mm]
5. Guy diameter: +/- 1/32 in. [1 mm]
6. Tubular pole circumference: +/- 1/2 in. [13 mm]
7. Panel antenna height, width and depth: +/- 1/2 in. [13 mm]
8. Other antennas: height +/- 1 in. [25 mm], width and depth: +/- 1/2 in. [13 mm]
9. Dish antenna diameter: +/- 1 in.[25 mm]
10. Cylindrical circumference: +/- 1/2 in. [13 mm]
11. Bolt diameter: +/- 1/16 in. [2 mm]
12. Bolt length: +/- 1/8 in. [3 mm]

J.2.3.2 Twist, Plumb and Guy Tensions

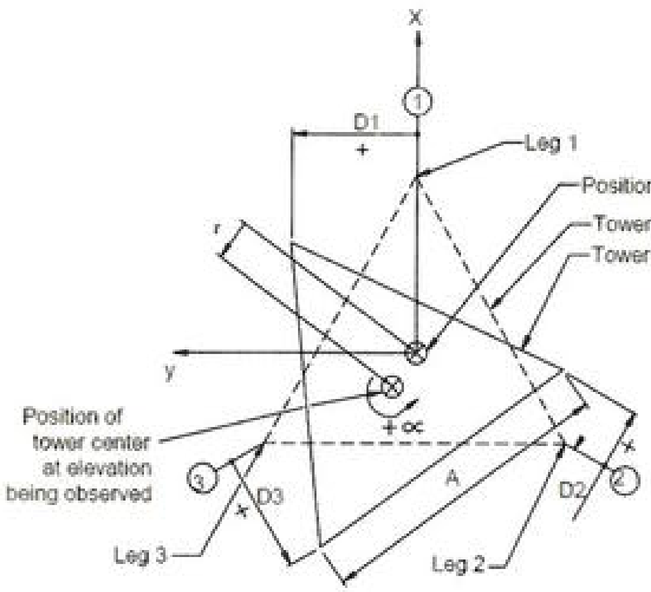
1. Twist: +/- 0.50 degree
2. Plumb: +/- 0.10% of structure height
3. Guy tensions: +/- 1% of ultimate guy breaking strength

Figure J-1: Twist and Out-of-Plumb Determination for Triangular Towers

Site Name: ______________________________ Date: ____________________________

Wind: __________________________________ Temperature: _____________________

The transit is to be set up on each leg azimuth at the base of the tower. The corresponding tower leg at the base of the tower is used to set the vertical baseline. The deflection at each point of interest on the tower is measured from this vertical baseline, as shown below.



$d = (D1 + D2 + D3) / 3$

$e = (d\sqrt{3}) / A$

$\alpha = \arcsin(e)$

$x = (D2 - D3) / \sqrt{3}$

$y = (2 \times D1 - D2 - D3) / 3$

$r = \sqrt{x^2 + y^2}$

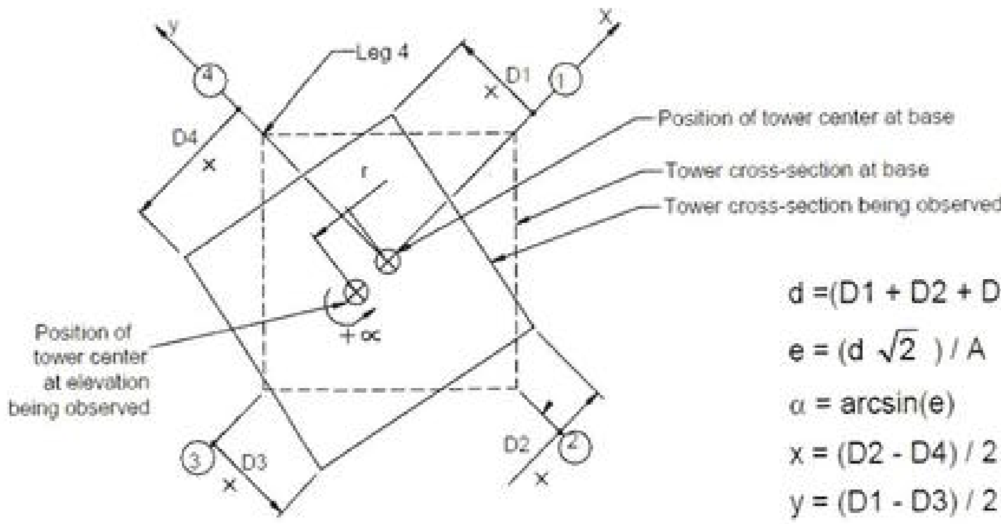
OBSERVED LEG DISPLACEMENTS					CALCULATED TWIST			CALCULATED OUT-OF-PLUMB		
SIGHTED ELEV. ft. [m]	A in. [mm]	D1 in. [mm]	D2 in. [mm]	D3 in. [mm]	d in. [mm]	e	α deg.	x in. [mm]	y in. [mm]	r in. [mm]

Figure J-2: Twist and Out-of-Plumb Determination for Square Towers

Site Name: ______________________ Date: ______________________

Wind: ______________________ Temperature: ______________________

The transit is to be set up on each leg azimuth at the base of the tower. The corresponding tower leg at the base of the tower is used to set the vertical baseline. The deflection at each point of interest on the tower is measured from this vertical baseline, as shown below.



$$d = (D1 + D2 + D3 + D4) / 4$$

$$e = (d\sqrt{2}) / A$$

$$\alpha = \arcsin(e)$$

$$x = (D2 - D4) / 2$$

$$y = (D1 - D3) / 2$$

$$r = \sqrt{x^2 + y^2}$$

OBSERVED LEG DISPLACEMENTS						CALCULATED TWIST			CALCULATED OUT-OF-PLUMB		
SIGHTED ELEV. ft. [m]	A in. [mm]	D1 in. [mm]	D2 in. [mm]	D3 in. [mm]	D4 in. [mm]	d in. [mm]	e	α deg.	x in. [mm]	y in. [mm]	r in. [mm]

ANNEX K: MEASURING GUY TENSIONS (Informative)

This Annex provides guidelines for field measuring guy tensions. There are two basic methods for measuring guy initial tensions in the field: the direct method and the indirect method.

All guy tension measurements may need to be adjusted to account for temperature. All guy tension measurements should be recorded with the temperature at the time of measurement.

Manufacturers of equipment utilized for the purpose of measuring guy tensions shall provide upon request documentation verifying the accuracy of the equipment for the guy sizes and types to be measured.

A. The Direct Method (Refer to Figure K-1)

A dynamometer (load cell) with a length adjustment device, such as a come-along, is attached to the guy system by clamping onto the guy just above the turnbuckle and onto the anchor shaft below the turnbuckle.

The come-along is then tightened until the original turnbuckle begins to slacken. At this point the dynamometer carries all of the guy load to the anchor and the guy tension may be read directly off the dynamometer dial.

One may use this method to set the correct tension by adjusting the come-along until the proper tension is read on the dynamometer. The control points are marked, one above the clamping point on the guy and one on the anchor shaft, and the control length is measured. The dynamometer and come-along are then removed, and the original turnbuckle is adjusted to maintain the control length previously measured.

B. The Indirect Methods

There are three common techniques for the indirect measurements of guy initial tensions; the pulse method (vibration), the tangent intercept or sag method (geometry) and the shunt dynamometer method (mechanical).

1. The Pulse Method (Refer to Figures K-1 and K-3)

One sharp jerk is applied to the guy near its connection to the anchor causing a pulse or wave to travel up and down the guy. On the first return of the pulse to the lower end of the guy the stopwatch is started. A number of returns of the pulse to the anchor are then timed, and the guy tension is calculated from the following equations:

$$T_M = \frac{WLN^2}{8.05P^2} \qquad \left[T_M = \frac{WLN^2}{5.94P^2}\right]$$

$$T_A = \sqrt{\left(T_M - \frac{WV}{2L}\right)^2 + \left(\frac{WH}{2L}\right)^2}$$

where:

T_M = guy tension at mid-guy, lb [N] (refer to Figure K-3)

W = total weight of guy, including insulators, etc., lb [N]

L = guy chord length, ft. [m]

$L = \sqrt{H^2 + V^2}$

N = number of complete pulses or swings counted in P seconds

P = period of time measured for N pulses or swings, seconds

T_A = guy tension at anchor, lb [N] (refer to Figure K-3)

H = horizontal distance from guy attachment on mast to guy attachment at anchor, ft. [m]

V = vertical distance from guy attachment on mast to guy attachment at anchor, ft. [m]

2. The Tangent Intercept Method (Refer to Figure K-2)

A line if sight is established which is tangential to the guy near the anchor end and which intersects the mast a distance (tangent intercept) below the guy attachment point on the mast. This tangent intercept distance is either measured or estimated and the tension is calculated from the following equation.

$$T_A = \frac{WC\sqrt{H^2 + (V - I)^2}}{HI}$$

where:

C = distance from guy attachment on mast to the center of gravity of the weight W, ft. [m]

I = tangent intercept, ft. [m]

When the weight is uniformly distributed along the guy, C will be approximately equal to H/2. When the weight is not uniformly distributed, the guy may be subdivided into n segments and the following equation may be used:

$$T_A = \frac{S\sqrt{H^2 + (V - I)^2}}{HI}$$

where:

$$S = \sum_{i=1}^{N} W_i C_i$$

W_i = weight of segment i, lb [N]

C_i = horizontal distance from the guy attachment on the mast to the center of gravity of segment i, ft. [m]

N = number of segments

If the intercept is difficult to establish, one may use the guy slope at the anchor end with the following equation:

$$T_A = \frac{WC\sqrt{1+\tan^2\alpha}}{(V - H\tan\alpha)}$$

where:

α = guy angle at the anchor (refer to Figure K-2)

$I = V - H\tan\alpha$

and

$$\frac{\sqrt{H^2 + (V-I)^2}}{H} = \sqrt{1+\tan^2\alpha}$$

WC may be replaced with S.

3. The Shunt Dynamometer Method (Refer to Figures K-1 and K-4)

A shunt dynamometer is a deflection-type indicator that is clamped onto a guy and measures deflection as it applies a force to the guy. Shunt dynamometers require periodic calibration and should be used, maintained and calibrated in accordance with the manufacturer's recommendations and specifications.

Figure K-1: Methods of Measuring Guy Tensions

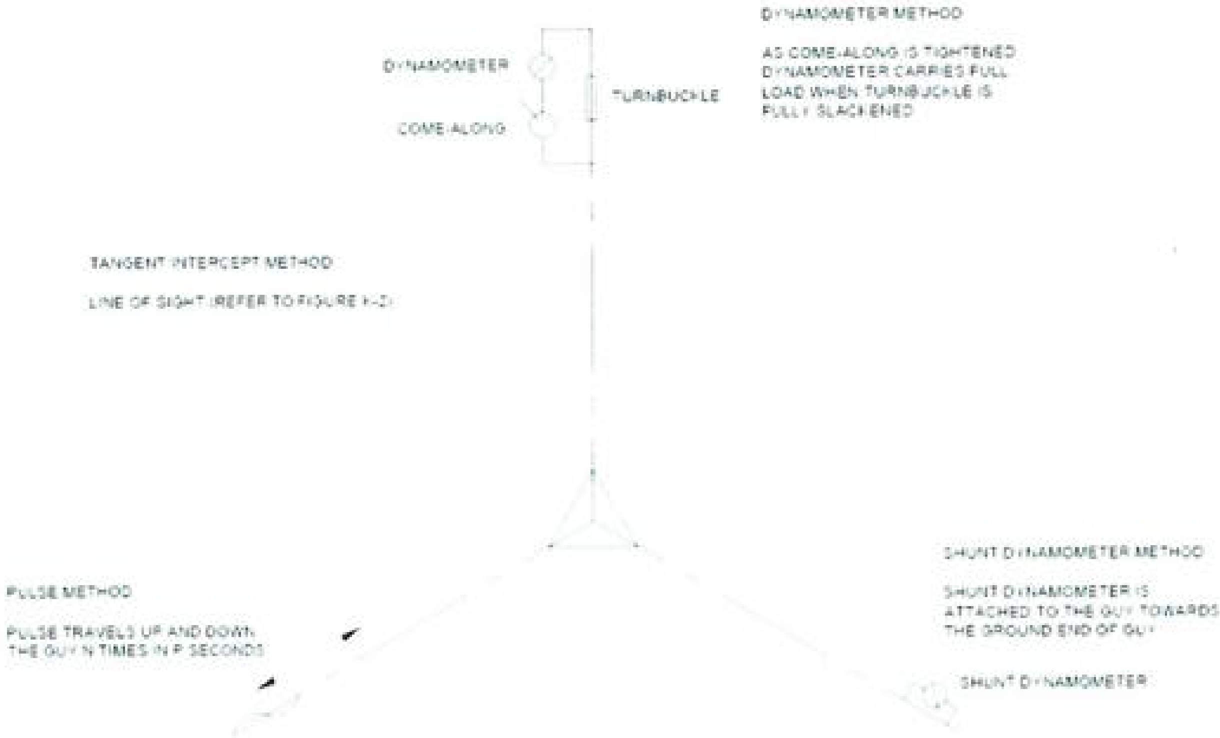


Figure K-2: Tangent Intercept Method

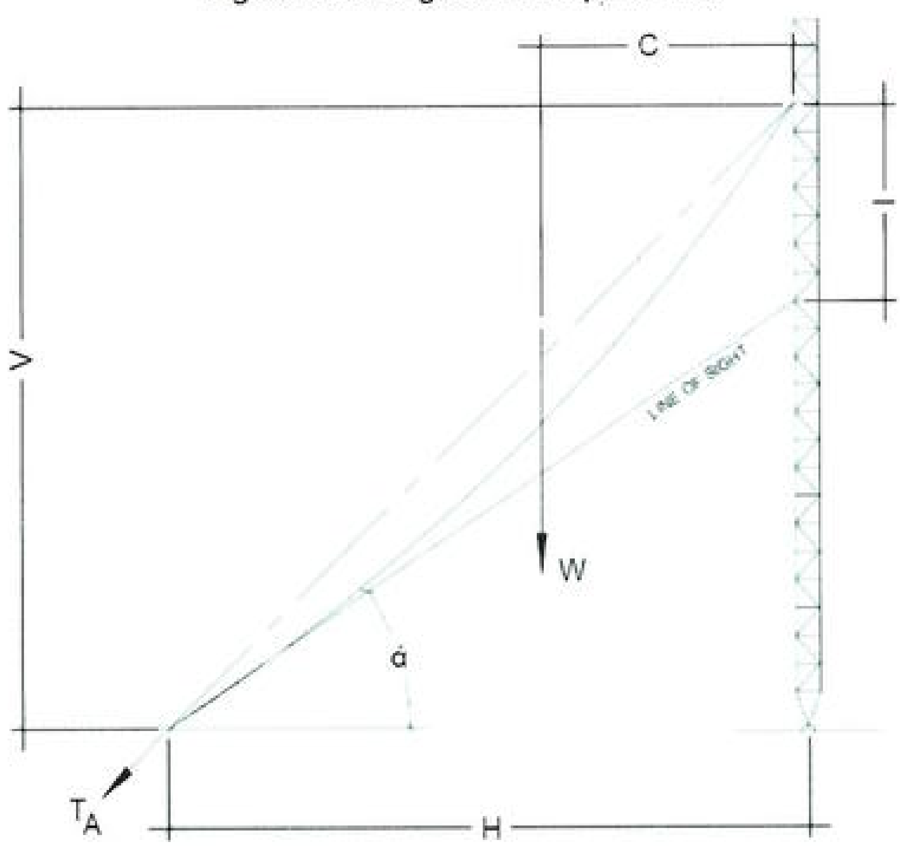


Figure K-3: Guy Tensions at Anchor and at Mid-Guy

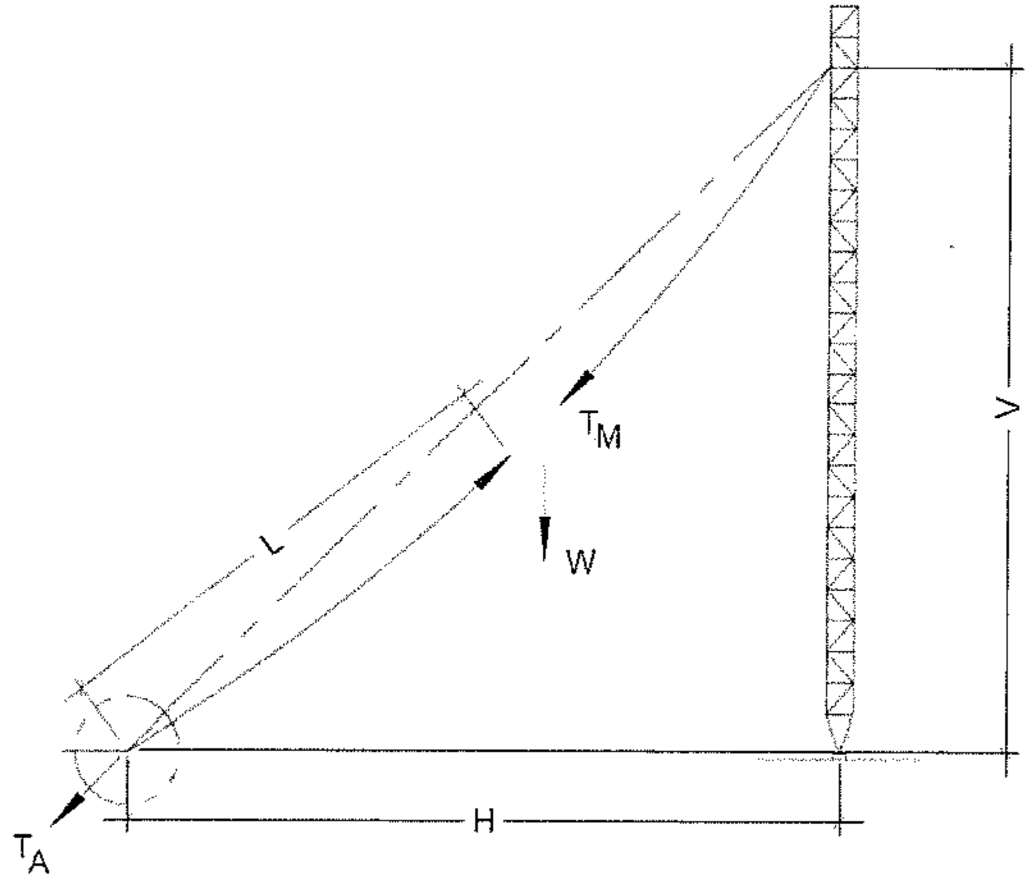
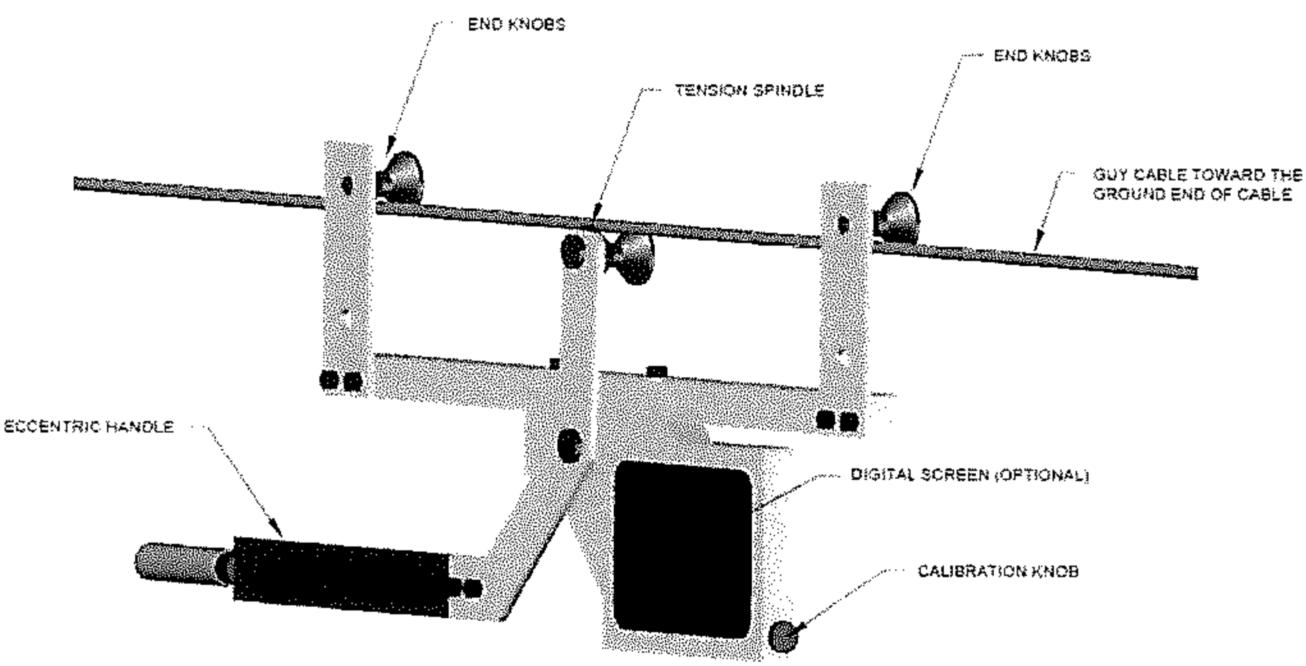


Figure K-4: Shunt Dynamometer Method



ANNEX L: WIND SPEED CONVERSIONS (Normative)

This Annex provides conversion of wind speeds based on various averaging periods to the 3-second gust wind speed with return periods associated with different risk categories. Wind data based on other averaging periods are to be converted to a 3-second gust wind speed with a return period corresponding to the risk category of the structure for use with the Standard.

Equivalent Ultimate Wind Speed (mph) Based on Risk Category (return period)				50-year Return Period			
I (300-yr)	II (700-yr)	III (1,700-yr)	IV (3,000-yr)	3-sec gust (mph)	Fastest-mile (mph)	10-min. avg. (mph)	Hourly mean (mph)
71	76	81	85	60	47	42	40
83	89	95	99	70	57	49	46
94	101	109	113	80	66	56	53
100	108	115	120	85	71	59	56
106	114	122	127	90	76	62	60
112	120	129	134	95	80	66	63
118	126	136	141	100	85	69	66
124	133	142	148	105	90	73	70
130	139	149	156	110	95	76	73
136	145	156	163	115	100	80	76
142	152	163	170	120	104	83	79
147	158	170	177	125	109	87	83
153	164	176	184	130	114	90	86
159	171	183	191	135	119	94	89
165	177	190	198	140	123	97	93
171	183	197	205	145	128	101	96
177	190	203	212	150	133	104	99
183	196	210	219	155	138	108	103
189	202	217	226	160	142	111	106
195	209	224	233	165	147	115	109
201	215	231	240	170	152	118	113

Notes:

1. The above conversion for risk category wind speeds assumes constant ratios between the Risk Category return period wind speed to the 50-year return period wind speed using ratios of 1.18, 1.26, 1.36 and 1.41 for Risk Categories I, II, III, and IV respectively. Site-specific ratios may vary.
2. For conversion to [m/s] multiply the above values by 0.447.
3. Linear interpolation may be used between the values shown.

ANNEX M: WIND INDUCED STRUCTURAL OSCILLATIONS (Informative)

Cantilevered tubular round or multisided sections used for pole structures, spines, antennas and shrouds can be subjected to significant wind induced oscillations, especially when there are minimal appurtenances or lines supported by the structure. Guyed masts and self-supporting latticed towers can also be subjected to wind induced oscillations.

Wind induced oscillations may appear to be random or may occur with repetitive constant amplitudes. The oscillations can occur under a wide range of wind speeds and can oscillate in a direction normal, parallel or oblique to the wind direction. Torsional oscillations may also occur about the vertical axis of a structure. Often, wind induced oscillations are barely noticeable by the naked eye and at other times the amplitudes are significant, resulting in the concerns of owners and the general public due to observed movements or noise from the oscillations.

There are three general types of wind induced oscillations: vortex-shedding, buffeting and galloping. Vortex-shedding occurs as wind vortices are shed alternatively on each side of a blunt object (non-streamlined) under low steady laminar wind speed conditions. Buffeting occurs as the result of moderate wind gusts interacting with the displacement of a structure. Galloping occurs due to an instability from wind flow which may occur when a steady wind blows across a guy of a guyed mast during heavy rain or where ice accumulation has occurred or for a structure in the downwind turbulent wake of an adjacent structure.

The significance of wind induced oscillations depends upon the amplitude of the oscillations, how frequently the oscillations occur and the duration of each event. For many structures, the stresses resulting from the oscillations are very low and there is no need for remedial action. For other structures, the amplitudes may result in significant cyclic stresses in the structure which can lead to fatigue cracks. The fatigue cracks may take years to initially develop and propagate or may develop in a relatively short period of time.

Although fatigue damage may occur at any location where a stress concentration exists (i.e. at openings in pole structures, at abrupt changes in cross sections (stiffness) or at welded attachments to the structure) the most frequent damage occurs at the following locations:

1. Spines and cantilevered antennas: at the base or in the connection bolts.
2. Cylindrical shrouded spines: at the base or in the connection bolts.
3. Self-supporting structures: at the base or in the anchor rods.
4. Guyed masts: in the guys or at their connection to the mast or guy anchor.

Although there has been considerable research in the science of wind induced structural oscillations, no practical analytical method has been found to predict, in advance, if significant wind induced structural oscillations will occur at a particular site. It has been demonstrated that wind induced oscillations result from a complex combination of variables, which are beyond the control of the manufacturer or the structure designer. Some of the variables include the stiffness and mass of the structure, the characteristics of a building supporting a structure, the amount of damping, the location and shape of appurtenances, prevailing wind directions, site terrain, laminar wind patterns, localized turbulence effects, site-specific wind flow characteristics and the size, type and proximity of adjacent structures or topographic features.

There are methods to reduce the magnitude and duration of wind induced oscillations. The methods involve complex and unpredictable variables and may require an iterative process for a successful implementation. When oscillations are observed, the owner should contact a qualified engineer experienced in the area of structural oscillations for evaluation, monitoring and possible mitigation. Some of the solutions proven for mitigation include one or a combination of the following: installation of mass or liquid dampers, installation of helical strakes or additional appurtenances intended to introduce turbulence, changing the stiffness or mass of the structure or adjusting the still-air tensions in guy assemblies. When wind induced oscillations are known to occur, more frequent maintenance and condition assessments should be considered to identify early signs of fatigue allowing implementation of repair and mitigation plans before fatigue cracks propagate to a length that could result in a catastrophic failure.

The Standard includes a requirement for tightening top and leveling nuts for base plates. This requirement has been added to address fatigue related issues reported for structures with loose anchor rod nuts (particularly leveling nuts) that have resulted in excessive deformations of base plates under cyclic loading leading to the generation of fatigue cracks.

Fatigue cracks most often originate and propagate from weld metal due to inherent microscopic imperfections unavoidable in weld metal in combination with the stress concentration associated with the geometry of a weldment. The occurrence of a fatigue crack from a weld often is initially attributed to faulty material or welding when the actual cause is cyclic loading from wind induced oscillations.

Section 17.0 of this Standard includes detailing criteria intended to result in components with improved fatigue performance that minimize the occurrence of fatigue cracks for small wind turbine support structures. The detailing criteria of Section 17.0 may also be appropriate for structures, such as those noted above, susceptible to wind induced oscillation.

ANNEX N: INITIAL CONSTRUCTION INSPECTION (Normative)

This Annex provides guidelines for inspection of steel structures during and after the initial construction phase. The inspection guidelines for a site-specific structure may vary depending upon the type of structure and site-specific conditions. The structure, foundation, and construction materials shall be field inspected in accordance with the design documents and the applicable referenced standards.

This Annex addresses the structure and foundation only and is not applicable for other site facilities such as shelters, generator slabs, and fencing. Reports indicating conformance with the design documents including photographic evidence should be included in an inspection report upon construction completion along with any recommended remediation.

N.1 Foundation and Grounding Inspection

1. Parameters that require verification in accordance with the recommendations of the geotechnical report for the site or as noted in the design documents.
2. Foundation dimensions.
3. Reinforcing steel grade, size, condition, support, placement and cover.
4. Concrete mix design documentation matches strength, durability requirements, etc.
5. Concrete tests required to be performed prior to placement of concrete (i.e. slump, temperature, air content, test cylinder, etc.).
6. Anchorage dimensions and placement (i.e. size, embedment depth, projection above concrete, orientation, pattern, alignment, etc.).
7. Condition of subgrade immediately prior to concrete placement.
8. Proper concrete placement (i.e. avoid segregation of aggregates, etc.) and curing.
9. Grounding material, placement and required resistance tests.
10. Structural backfill material and placement (i.e. maximum lift thickness, moisture content and density).

N.2 Post-Installation Inspection

1. Perform a Maintenance and Condition Assessment within 180 days of installation (refer to Annex J).
2. Structural members, sections and assemblies (i.e. torque arms, mounts, etc.) are in correct orientation and placement.
3. Installation tolerances are in accordance with Section 13.0 of Standard.
4. Maximum height of base plate leveling nuts above concrete and top and leveling nuts tightened in accordance with 4.9.9 of this Standard.
5. Perform required bolt pre-tensioning tests and installation of nut-locking devices on non-pretensioned connections for the structure.
6. Jacking and lap lengths for tubular pole structure splices.
7. Repair of galvanized and painted surfaces in accordance with the design documents.
8. Initial tension of guys for guyed masts in accordance with the design documents.

a. Cable or other devices shall be installed on turnbuckles to prevent disengagement under wind loading.

b. Striped cables shall be erected such that the paint stripe applied during measuring is straight after erection.

c. Initial tensions shall be measured by either direct or indirect methods (examples are provided in Annex K).

9. Climbing facilities, including safety climbs, are installed in accordance with section 12.0 of this Standard.

10. Warning signs and labels visible and undamaged.

ANNEX O: EXISTING STRUCTURES MODIFICATION INSPECTION (Normative)

This Annex provides guidelines for inspection of modifications to existing steel structures before, during and after the construction phase. The inspection guidelines for a site-specific structure may vary depending upon the type of structure and site-specific conditions. The structure, foundation, and construction materials shall be field inspected in accordance with the design documents and the applicable referenced standards.

This Annex addresses the structure and foundation only. Reports indicating conformance with the design documents including photographic evidence should be included in an inspection report upon construction completion along with any recommended remediation.

PRE-CONSTRUCTION	
Fabricator Certified Weld Inspection	When material is not provided by a fabricator certified in accordance with AISC 201, visual pre/during/post observation in accordance with AWS D1.1 by a CWI for critical weldments identified in the design documents.
Material Test Report (MTR)	Material test reports shall be provided for steel components (members, connection plates, etc.) when specified in the design documents.
Fabricator NDE Inspection	NDT shall be performed in accordance with the design documents when specified.
Field Welding	Documentation of WPS(s), welder qualification and welding inspector for the proposed field welding process, joint configuration and position.
DURING / POST-CONSTRUCTION	
Foundation and Geotechnical Inspections	Refer to N.1 in Annex N.
Concrete Placement and Tests	Refer to N.1 in Annex N.
Structural Backfill	Refer to N.1 in Annex N.
Post-Installed Anchor Rods	Post-installed anchor rods verification shall be performed including pull tests, hole preparation and depth, hole diameter, adhesive medium, anchor rod length, anchor rod projection above top of foundation, placement and location, curing conditions, and coating if specified, etc. as specified in the design documents.
Base Plate Grout	Grout material and placement in accordance with the design documents.
Field Weld Inspections	Weld inspections conducted for all structural field welds pre/during/post welding operations, including protection of lines, appurtenances, equipment, etc. in accordance with the design documents.

DURING / POST-CONSTRUCTION (Continued)	
Structural Components	Refer to N.2 in Annex N.
Galvanized & Painted Surfaces	Refer to N.2 in Annex N.
Guy Tension, Twist, and Plumb Report	Refer to N.2 in Annex N.
Structural Bolted Connection	Refer to N.2 in Annex N.
Grounding	Refer to N.1 in Annex N.
Climbing Facilities	Refer to N.2 in Annex N.

ANNEX P: TUBULAR POLE STRUCTURE WELD TOE CRACK EVALUATION (Informative)

This Annex provides an approach for evaluating weld toe cracks for flange and base plates of tubular pole structures. A classification system is presented based on the age of the structure, the length and depth of toe cracks and the recommended repair schedule.

P1.0 Definitions

Crack: a fracture-type discontinuity characterized by a sharp tip and high ratio of length and width to opening displacement.

CWI: Certified Welding Inspector in accordance with AWS D1.1.

MT: magnetic particle weld inspection (surface/near surface) per AWS D1.1.

NDE: Non-Destructive Examination is the act of evaluating a welded component without affecting the serviceability of the part or material.

Toe crack: a defect observed at the upper weld toe of a flange or base plate welded connection.

UT: ultrasonic weld inspection (volumetric) per AWS D1.1.

Visual: visual weld inspection per AWS D1.1.

P2.0 Symbols and Notations

C_S = circumference of pole shaft approximated as (π D_F) for polygonal cross-sections;

D_T = outside diameter of pole shaft or outside flat-to-flat width for polygonal cross-sections;

d_c = maximum measured depth of crack into the pole wall thickness;

L_C = total accumulated length of all weld cracks measured around circumference $\sum (L_1 + L_2 + L_3 \ldots)$;

t_T = thickness of pole shaft.

P3.0 General

Toe crack classification and the recommended appropriate repair schedule shall be determined in accordance with the following:

1. Determine the total length of weld toe cracks by summing the lengths of each individual toe crack around the circumference of the pole.
2. Determine the maximum weld toe crack depth considering all toe cracks around the circumference of the pole.
3. Calculate the weld toe crack length ratio by dividing the total length of toe cracks by the circumference of the pole.
4. Calculate the weld toe crack depth ratio by dividing the maximum toe crack depth by the pole wall thickness.

5. Determine the weld toe Crack Category from Table P-1 based on the calculated weld toe crack length and depth ratios.
6. Determine the weld toe Crack Category from Table P-2 based on the calculated weld toe crack length and depth ratios and the age of the pole. When the age of the pole is not known, the age shall be considered to be less than 5 years.
7. Determine the recommended corrective actions and repair schedule from Table P-3 using the higher weld toe Crack Category from step 5 and step 6.

Table P-1: Weld Toe Crack Category Based on Crack Length

Weld Toe Crack Length Ratio	Weld Toe Crack Depth Ratio			
	$d_c / t_T < 1/4$	$d_c / t_T < 1/2$	$d_c / t_T < 3/4$	$d_c / t_T \geq 3/4$
$L_C / C_S < 1/4$	1	1	2	3
$1/4 \leq L_C / C_S \leq 1/2$	1	2	3	4
$L_C / C_S > 1/2$	2	3	4	4

Table P-2: Weld Toe Crack Category Based on Pole Age

Age of Pole (Years)	Weld Toe Crack Length and Depth Ratios					
	$L_C / C_S < 1/4$		$1/4 \leq L_C / C_S \leq 1/2$		$L_C / C_S > 1/2$	
	$d_c / t_T < 1/2$	$d_c / t_T \geq 1/2$	$d_c / t_T < 1/2$	$d_c / t_T \geq 1/2$	$d_c / t_T < 1/2$	$d_c / t_T \geq 1/2$
< 5	2	3	3	4	3	4
5 to 10	1	2	3	4	3	4
> 10	1	2	2	3	3	4

Table P-3: Weld Toe Crack Classification for Corrective Actions

Crack Category	Description
0	**No toe crack indications identified.** No cracks identified during a complete CWI inspection using visual and NDE techniques (typically MT & UT). **Corrective actions:** No corrective action required.
1	**Small toe crack indications identified.** **Corrective actions:** a) Remove cracks via grinding. b) Repair welds and install stiffeners when required. **Repair schedule:** Complete all repairs within 60 days.
2	**Moderate toe crack indications identified.** **Corrective actions:** a) Remove cracks via grinding. b) Repair welds and install stiffeners when required. **Repair schedule:** Complete all repairs within 30 days.
3	**Extensive toe crack indications identified.** **Corrective actions:** a) Drill holes at ends to prevent further crack propagation. b) Remove cracks via grinding. c) Repair welds and install stiffeners when required. **Repair schedule:** Complete all repairs within 14 days.
4	**Severe toe crack indications identified.** **Corrective actions:** a) Immediately stabilize the pole. b) Drill holes at ends to prevent further crack propagation. c) Remove cracks via grinding. d) Repair welds and install stiffeners when required. **Repair schedule:** Immediately stabilize the pole and complete all repairs within 5 days.

ANNEX Q: TUBULAR POLE BASE PLATES (Informative)

This Annex provides supplementary requirements that pertain specifically to the unique characteristics of tubular pole base plates.

Note: This Annex is not intended to apply to miscellaneous ancillary structures such as ice bridge supports, equipment stands, etc.

Q1.0 Definitions

External base plate: a base plate with a bolt circle external to the pole cross section.

Internal base plate: a base plate with a bolt circle internal to the pole cross section.

Pole diameter: outside diameter of a round cross section pole or the outside flat-to-flat width of a polygonal cross section pole.

Q2.0 Symbols and Notations

B_{eff} = total effective base plate width resisting bending;

B_{er} = effective base plate width resisting bending from radial bend lines;

B_{et} = effective base plate width resisting bending from transverse bend line;

D_{BC} = anchor rod bolt circle diameter;

D_{ce} = effective base plate center opening diameter;

D_e = effective pole outside diameter;

D_i = effective pole inside diameter;

D_{OD} = base plate outside diameter;

D_{oe} = effective base plate outside diameter;

D_{OP} = base plate center opening diameter;

D_T = outside diameter of a round cross section pole, outside flat-to-flat width of a polygonal cross section pole;

d_s = the diameter of a hole or the length of a slot used for galvanizing drainage or venting;

F_{yf} = base plate design yield strength;

F_{yp} = pole design yield strength;

M_u = pole resultant overturning moment reaction due to factored loads on anchor rod group;

n = number of anchor rods;

n_c = anchor rod force correction factor;

P_u = maximum anchor rod axial force due to factored pole reactions on anchor rod group;

R_u = pole vertical reaction due to factored loads on anchor rod group;

t_T = pole wall thickness;

t_{TP} = base plate thickness;

w_1 = outside fillet-weld horizontal leg dimension;

w_2 = inside fillet-weld horizontal leg dimension;

x = effective moment arm of anchor rod force;

υ = governing angle defining effective base plate width resisting bending;

υ_1 = half-angle between radial lines extending from pole centerline through midpoints between adjacent anchor rods;

υ_2 = angle defining limiting effective base plate width based on plate thickness;

υ_3 = angle defining limiting effective base plate width based on distance between anchor rod bolt circle and effective pole outside diameter;

ϕ_b = strength resistance factor for base plate yielding due to bending;

ϕ_v = strength resistance factor for base plate yielding due to shear.

Q3.0 General

The method provided in this Annex is based on rigid base plate behavior, without the use of stiffeners, that results in insignificant anchor rod bending and insignificant secondary pole wall bending stresses under factored reactions from limit state strength loading conditions. Other rational methods developed in accordance with the load and resistance factors specified in this Standard may be used in place of this Annex. This Annex is intended to apply to base plates conforming to all of the following:

1. Tubular steel poles with round cross sections or polygonal cross sections with 6 or more sides.
2. Socketed and butt welded base plate connections.
3. Round uniform thickness base plates.
4. Base plates without stiffeners.
5. Base plates with round center openings with galvanizing drainage or vent holes or slots.
6. Base plates supported on leveling nuts.
7. Base plates with or without the use of grout.
8. Anchor rods equally spaced about an external or internal bolt circle.
9. A minimum of 8 anchor rods.
10. Holes for anchor rods fabricated without slots to a free edge.

Although this Annex is intended for tubular pole base plates, the provisions for determining plate thickness requirements for bending and shear may be extended to flange plates used for flanged tubular pole splices and top plates and for solid round and tubular leg flanges for latticed structures. Other provisions in this Annex are intended to only apply to tubular pole base plates.

Q4.0 Design Criteria

Base plates conforming to the requirements of this Annex shall be considered rigid. The design and analysis criteria for non-rigid base plates shall be based on rational design methods that account for the use of stiffeners or for the additional anchor rod stresses and secondary pole wall stresses resulting from base plate bending.

Notes:

1. The method for determining base plate bending strength is based on yield line theory assuming both transverse and radial yield lines and assumes the use of ductile steel capable of yielding and redistributing stress to form yield lines under a limit state strength loading condition. Rigidity is addressed by restricting the geometry of the base plate and anchor rods, by limiting the effective widths of the transverse and radial yield lines and by limiting the design yield strength of the base plate. Base plate shear strength is required to match the design tensile bending strength of the pole wall.
2. The required base plate bending strength is based on the required anchor rod forces to resist the applied reactions from the pole. Anchor rod forces are determined by assuming an equivalent steel ring with a diameter equal to the anchor rod bolt circle in conjunction with a correction factor based on the number of anchor rods. Anchor rod forces may be determined based on a plastic distribution of forces when the anchor rods are fully developed into the foundation.

Q4.1 Anchor Rods

The distance between the anchor rod bolt circle and the effective pole diameter shall not exceed three times the base plate thickness for rigid base plate behavior. The limitation on anchor rod bolt circle diameter may be expressed by the following equations:

For external base plates: $D_{BC} \leq D_T + w_1 + 6(t_{TP})$

For internal base plates: $D_{BC} \geq D_T - 2(t_T) - w_2 - 6(t_{TP})$

Anchor rods shall be on a symmetrical circular pattern. A minimum of 8 anchor rods shall be utilized. The minimum anchor rod nominal diameter shall be 0.75 in. [19 mm]. The spacing between anchor rods measured circumferentially along the bolt circle shall not exceed 6 times the base plate thickness but in no case shall the spacing exceed 15 in. [381 mm].

Q4.2 Base Plate Thickness

Base plates shall satisfy the thickness requirements for plate bending and shear outlined in Sections Q6.0 and Q7.0.

Base plate thickness shall not be less than the anchor rod diameter for anchor rods with required minimum guaranteed yield strengths exceeding 75 ksi [520 MPa] and for lower strength anchor rods, not less than the anchor rod diameter minus 0.25 in. [6 mm].

Q4.3 Yield Strength

The design yield strength of the base plate, F_{yf}, shall equal the published guaranteed minimum yield strength of the base plate material but shall not exceed 60 ksi [415 MPa].

Q4.4 Center Opening for Butt Welded Base Plates

The minimum center opening diameter shall be equal to 30% of the pole diameter for proper galvanizing. Additional holes or slots may be utilized for drainage or venting. Additional holes or slots may be ignored for determining the bending strength of external base plates but shall be considered for internal base plates in accordance with section Q6.2.

Q4.4.1 External Base Plates

The maximum center opening diameter shall be equal to 75% of the pole outside diameter. The perimeter of the center opening shall not be closer than 2 times the base plate thickness to the inside pole wall.

Q4.4.2 Internal Base Plates

The maximum center opening diameter shall be equal to 75% of the anchor rod bolt circle diameter. The perimeter of the center opening shall not be closer than 2 times the base plate thickness to the anchor rod bolt circle diameter.

Q4.5 Strength Resistance Factors

The strength resistance factor for base plate yielding due to bending, ϕ_b, shall be equal to 0.90.

The strength resistance factor for base plate yielding due to shear, ϕ_v, shall be equal to 1.00.

Q5.0 Anchor Rod Force

The maximum anchor rod force due to factored pole reactions shall be used to investigate base plate bending and to investigate anchor rod strength in accordance with 4.9.9. The maximum anchor rod force shall be calculated in accordance with the following equation:

$$P_u = \left[\frac{(n_c)(\pi)(M_u)}{n(D_{BC})} + \frac{R_u}{n}\right]$$

where:

n_c = anchor rod force correction factor from Table Q-1

Notes:

1. For use of Table Q-1, anchor rods may be considered fully developed into a foundation when their anchorage capacity for both tension and compression is governed by steel strength or concrete pullout (bearing) strength as opposed to being governed by concrete breakout or side-face blowout strength.
2. For pole base plates, the resultant overturning moment reaction shall not be considered less than 50% of the pole bending strength.

Q6.0 Base Plate Bending

The minimum base plate thickness required for bending shall be calculated in accordance with the following equation:

$$t_{TP} \geq \sqrt{\frac{4(P_u)(X)}{\phi_b(F_{yf})(B_{eff})}}$$

The variables in the equation shall be calculated in accordance with sections Q6.1 and Q6.2 (refer to Figures Q-1 and Q-2).

Q6.1 External Base Plates

$D_e = D_T + w_1$

$D_{oe} = D_{OD}$ but $\leq D_{BC} + 6(t_{TP})$

θ = minimum of υ_1, υ_2 and υ_3

$\theta_1 = \frac{\pi}{n}$ radians

$\theta_2 = \sin^{-1}\left[\frac{12(t_{TP})}{D_{BC}}\right]$ radians, when $12(t_{TP}) \geq D_{BC}$, $\upsilon_2 = \upsilon_1$

$\theta_3 = \cos^{-1}\left[\frac{(D_{BC} + D_e)}{2(D_{BC})}\right]$ radians

$x = 0.50(D_{BC} - D_e)$

$B_{et} = D_{BC}(\sin\theta)$

$B_{er} = (D_{oe} - D_e)(\sin\theta)$

$B_{eff} = B_{et} + B_{er}$

Q6.2 Internal Base Plates

$D_i = D_T - 2(t_T) - w_2$

$D_{ce} = D_{OP}$ but $\geq D_{BC} - 6(t_{TP})$

θ = minimum of υ_1 and υ_2:

$\theta_1 = \frac{\pi}{n}$ radians

$\theta_2 = \sin^{-1}\left[\frac{12(t_{TP})}{D_{BC}}\right]$ radians, when $12(t_{TP}) \geq D_{BC}$, $\upsilon_2 = \upsilon_1$

$x = 0.50(D_i - D_{BC})$

$B_{et} = D_{BC}(\sin\upsilon)$

$B_{er} = [D_i - D_{ce} - 2(d_s)](\sin\upsilon)$ but ≥ 0.0

$B_{eff} = B_{et} + B_{er}$

Q7.0 Base Plate Shear

The minimum base plate thickness required for shear shall be calculated in accordance with sections Q7.1 and Q7.2.

Q7.1 External Base Plates

$$t_{TP} \geq \frac{(\phi_b)(t_T)(F_{yp})}{(\phi_v)(0.60)(F_{yf})}$$

Q7.2 Internal Base Plates

$$t_{TP} \geq \left[\frac{(\phi_b)(t_T)(F_{yp})}{(\phi_v)(0.60)(F_{yf})}\right]\left(\frac{D_I}{D_{BC}}\right)$$

where:

$$D_i = D_T - 2(t_T) - w_2$$

Q8.0 Socketed Connections

Socketed connections shall be limited to external base plates. Socketed connections shall be connected with inner and outer fillet-welds and shall be limited to poles with diameters 24 in. [610 mm] or less.

The minimum insertion into a base plate shall be 0.75 in. [19 mm]. The inner fillet-weld size shall not be less than the pole wall thickness minus 0.06 in. [2 mm]. The combined strength per unit length of the inner and outer fillet-welds shall not be less than 0.90(F_{yp}) times the pole wall thickness. The strength of inner fillet-welds less than 0.19 in. [5 mm] shall be ignored. No increase in weld strength for fillet-welds loaded transverse to the longitudinal weld axis shall be considered unless the inner fillet-weld strength is ignored.

The design strength per unit length of the outer fillet-weld shall not be less than the design strength per unit length of the inner fillet-weld. The outer fillet-weld shall be an unequal leg weld with the long leg on the pole wall terminating at an approximately 30 degree angle.

The gap between the pole wall and the base plate shall not exceed 0.19 in. [5 mm]. When the width of the gap exceeds 0.06 in. [2 mm], the inner and outer fillet-weld sizes required for strength shall be increased by the width of the gap.

Q9.0 Butt Welded Connections

Butt welded connections shall be used for all external base plates for tubular poles greater than 24 in. [610 mm] in diameter and for all internal base plates. Butt welded connections shall be full-penetration groove-welds with reinforcing fillet-welds.

Reinforcing fillet-welds shall be unequal leg welds with the long leg on the pole wall terminating at an approximately 30 degree angle. The size of reinforcing fillet-welds shall result in a through-thickness stress no greater than 36 ksi [250 MPa] assuming a pole wall stress equal to F_{yp}.

Q10.0 Base Plate Anchor Rod Holes

Oversized holes may be utilized for anchor rod holes to facilitate construction. The design bearing strength requirements of 4.9.6.2 shall be satisfied for edge distance and hole spacing. The requirements of 4.9.4 do not apply to base plates; however, anchor rod holes shall be detailed such that anchor rod nuts or washers (when utilized) cannot extend over the edge of the base plate.

Q11.0 Grouted Base Plates

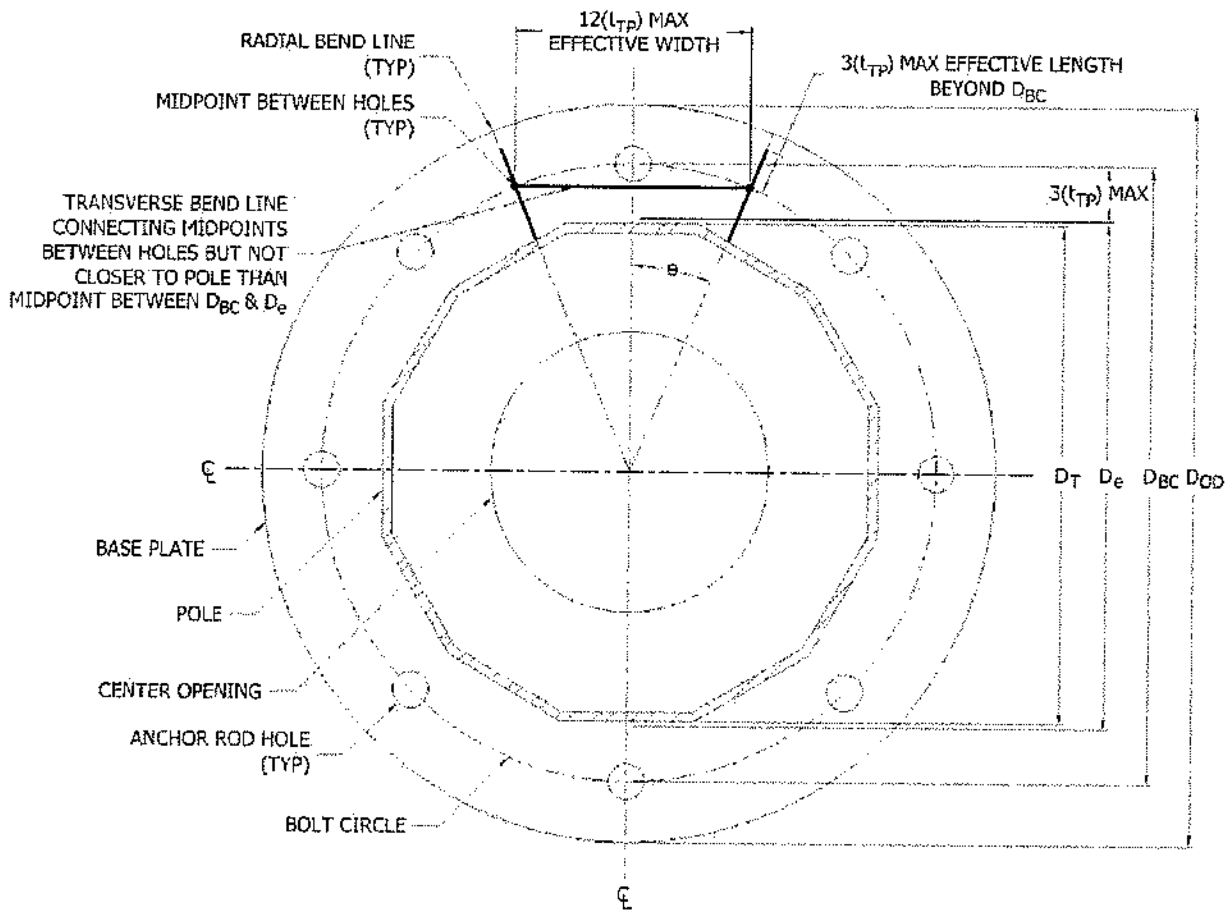
For installations with grouted base plates, the effect of grout shall be ignored for determining anchor rod forces and base plate thickness requirements.

Note: The use of grout for pole structures is not recommended due to the difficult installation circumstances associated with relatively large base plates with center openings. For non-deformed anchor rods, an additional embedded bearing nut may be required at the top of shallow foundations to transfer compression in order to prevent a punching shear failure at the base of the foundation.

Table Q-1: Anchor Rod Force Correction Factors, n_c

Number of Anchor Rods	Anchor Rods Fully Developed Into Foundation	Anchor Rods Not Fully Developed Into Foundation
8 - 9	1.05	1.27
10 - 11	1.04	1.27
12 - 16	1.02	1.27
>16	1.00	1.27

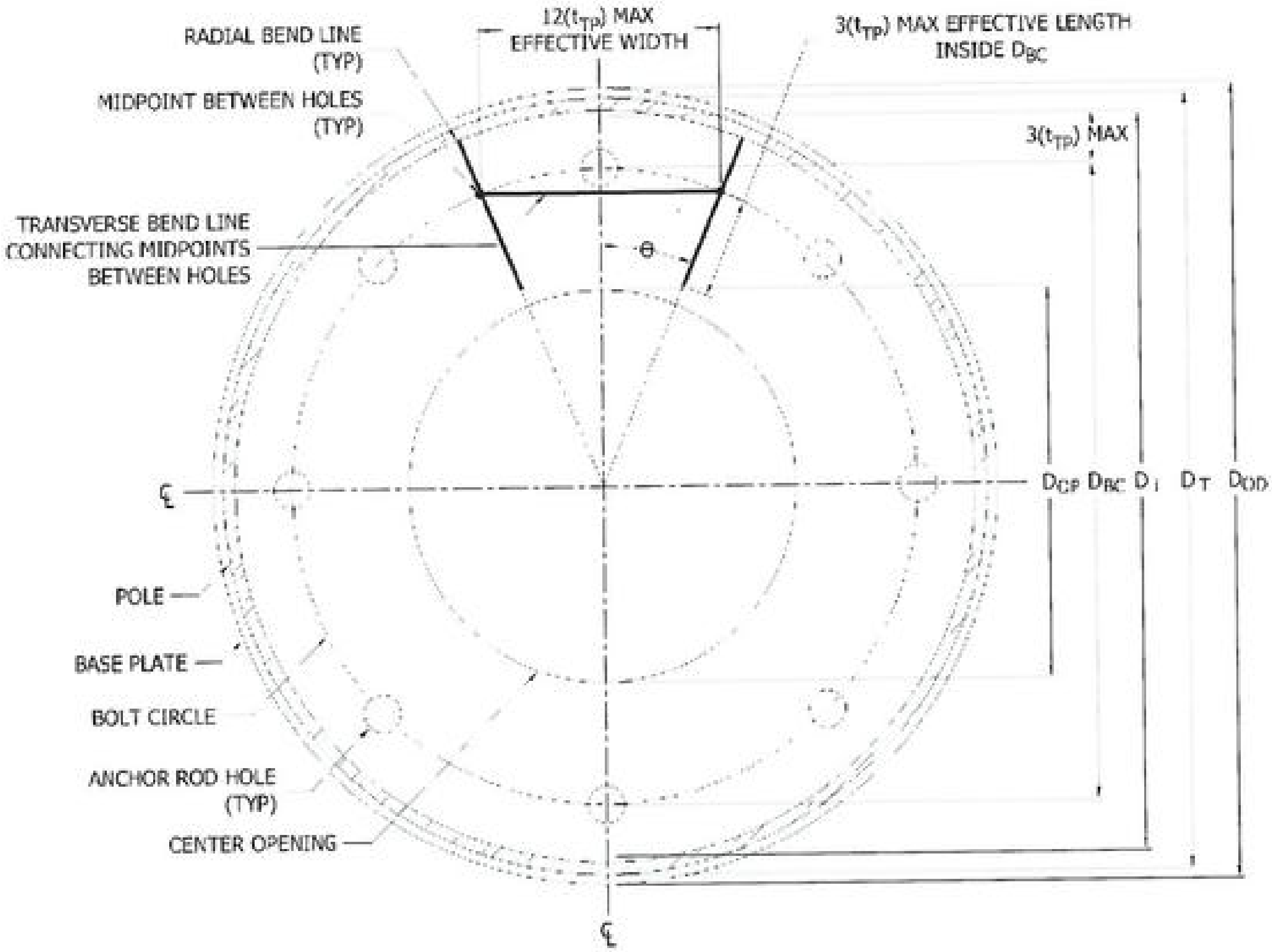
Figure Q-1: Base Plate Yield Lines for External Base Plates



Notes:

1. Drainage/Vent holes or slots are not shown for clarity.
2. Yield lines apply to socketed and butt welded connections.

Figure Q-2: Base Plate Yield Lines for Internal Base Plates



Note: Drainage/Vent holes or slots are not shown for clarity.

ANNEX R: ASSUMED MATERIAL STANDARDS (Informative)

This Annex provides suggested material standards, in the absence of other information, based on member shape and/or age of structure and typical standard practices in the United States of America. Material standards commonly used in other countries may be substituted as applicable.

Solid Steel Shapes: ASTM A7 for structures originally installed prior to 1962 and ASTM A36 for structures installed thereafter.

Pipe: ASTM A53 Grade B.

Tubular Shapes: ASTM A500 Grade B.

Polygonal Cross-section Poles: ASTM A572 Grade 50.

Bolts: ASTM A307 for structures originally installed prior to 1975, A325 for structures installed thereafter.

Anchor Rods (Bolts): ASTM A36 for structures originally installed prior to 1985, ASTM A615 Grade 75 for deformed anchor rods and ASTM A572 Grade 50 for smooth anchor rods installed thereafter.

Welds: matching weld electrode per AWS D1.1.

Concrete: 3,000 psi [21 MPa] specified compressive strength.

Concrete Reinforcing Steel Yield Strength: 40 ksi [280 MPa] except 60 ksi [420 MPa] for bar sizes larger than #4 [13] in foundations installed after 1980.

Guy Cables: ASTM A475 Extra High Strength Grade (EHS) for cables up to and including 1 in. [25 mm] diameter, and ASTM A586, Grade 1, Class A Coating Throughout (Bridge Strand) for cables greater than 1 in. [25 mm] diameter.

Annex S: Analysis of Existing Antenna Supporting Structures Based on Target Reliabilities (Informative)

S.1 Scope

This Annex addresses the evaluation of existing antenna supporting structures based on target reliabilities considering demand-capacity ratios and reduced reference periods for extreme loading events based on the historical performance of antenna supporting structures.

All provision of Section 15.0 also apply to this Annex unless otherwise specified in this Annex.

S.2 Definitions

Existing structure load modification factor: a reduction factor applied to a limit state load to result in a structural reliability appropriate for an existing antenna supporting structure.

S.3 Loading Combinations

It shall be permissible to apply the existing structure load modification factors, K_{es}, from Table S-1 to the loads determined in accordance with Section 2.0, with the exception of service loads for the evaluation of serviceability requirements in accordance with section 2.8 where K_{es} shall be equal to 1.0.

S.4 Evaluation of Changed Conditions

A proposed changed condition shall require the supporting structure and foundation to conform to this Standard when applying this Annex. The maximum demand-capacity ratio of any component shall not exceed 1.05 based on a comprehensive structural analysis.

When strengthening is required, modifications to the structure or foundation when applying this Annex shall be in conformance with this Standard based on a comprehensive structural analysis to limit the maximum demand-capacity ratio for the changed condition to 1.05.

S.5 Structural Analysis Report

In addition to the requirements of Section 15.0, a structural analysis report shall indicate that the evaluation has been performed in accordance with this Annex.

Table S-1: Existing Antenna Supporting Structure Load Modification Factors

Risk Category of Existing Structure	Existing Structure Load Modification Factors, K_{es}		
	Wind Load (F_w)	Design Ice Thickness (t_i)	Seismic Loads Effects (E_v and E_h)
I	1.00	N/A	N/A
II	0.95	0.85	1.00
III & IV	1.00	1.00	1.00

Notes:

1. Application of the existing structure load modification factors, K_{es}, to the loads specified in Section 2.0 in combination with a 1.05 maximum demand-capacity ratio result in probabilities of exceedance of the factored loads considered appropriate for existing antenna supporting structures.

2. When load modification factors for Risk Category II are used for the comprehensive structural analysis used to confirm conformance to this Annex, periodic inspection evaluations in accordance with a site-specific management plan for the structure shall be performed (in addition to the condition assessments as per Section 14.0). Damages to structural components discovered shall be reviewed and repaired or replaced as needed. Local conditions may warrant further evaluation of the existing structure under more stringent loading compared to the minimum loading specified by this Annex.

ANNEX T: SI CONVERSION FACTORS (Normative)

This annex provides conversions commonly required for the International System of Units (SI).

To Convert From	To	Multiply By
inches (in)	millimeters [mm]	25.40
feet (ft)	meters [m]	0.3048
square feet (ft^2)	square meters [m^2]	0.0929
cubic feet (ft^3)	cubic meters [m^3]	0.0283
gravity, g, (32.1 ft/s^2)	gravity, g, [9.81 m/s^2]	0.3048
pounds [force] (lb)	newtons [N]	4.4482
pounds [mass] (lb)	kilograms [kg]	0.4536
pounds per cubic feet (lb/ft^3)	kilonewtons per cubic meter [kN/m^3]	0.1571
pounds per square foot (lb/ft^2)	Pascals [Pa]	47.88
kips per square inch (ksi)	megapascals [MPa]	6.8948
miles per hour (mph)	meters per second [m/s]	0.4470
miles per hour (mph)	kilometers per hour [km/hr]	0.6214

ANNEX U: REFERENCES (Informative)

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NAVFAC, "Soil Mechanics", NAVFAC DM 7.01, Naval Facilities Engineering Command, VA, 1996.

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