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48.	Page 236	Appendix 1 – Remove note 3 on Figure A1-2d
49.	Page 237	Appendix 1 – Remove note 3 on Figure A1-2e

5. A topographic category and topographic factor, K_{zt}, shall be determined in accordance with 2.6.6.

6. A gust effect factor, G_h, shall be determined in accordance with 2.6.7.

7. The design ice thickness shall be escalated with height in accordance with 2.6.8.

8. The design wind force shall be determined in accordance with 2.6.9.

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. Wind shall be considered to come from any horizontal direction. Ice shall be considered to be glaze ice.

Ice may be ignored for structures located in regions where the design ice thickness is less than or equal to 0.25 inches (6 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 Categories

2.6.5.1 General

An exposure category that adequately reflects the characteristics of ground surface irregularities at the site shall be determined. Account shall be taken of variations in ground surface roughness that arise from natural topography and vegetation as well as from constructed features. The exposure category for a structure shall be assessed as being one of the following:

1. **Exposure B:** Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger. Use of this exposure shall be limited to those areas for which terrain representative of Exposure B surrounds the structure in all directions for a distance of at least 2,600 ft [800 m] or twenty times the height of the structure, whichever is greater.

2. **Exposure C:** Open terrain with scattered obstructions having heights generally less than 30 ft [9.1 m]. This category includes flat, open country, grasslands and shorelines in hurricane prone regions. Sites located in Exposure B terrain that are located further than two miles but less than twenty times the height of the structure from an Exposure D terrain.

3. **Exposure D:** Flat, unobstructed shorelines exposed to wind flowing over open water (excluding shorelines in hurricane prone regions) for a distance of at least 1 mile [1.61 km]. Shorelines in Exposure D include inland waterways, lakes and non-hurricane coastal areas. Exposure D extends inland a distance of 660 ft [200 m] or twenty times the height of the structure, whichever is greater. Smooth mud flats, salt flats and other similar terrain shall be considered as Exposure D.

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 2-3, 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 2.6.9.2.1, refer to 2.6.9.4)
= 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 specified above shall not be applied in addition to the applicable K_a values specifed in 2.6.9.2.2 through 2.6.9.2.5)

θ = relative angle between the azimuth associated with the normal face of the appurtenance and the wind direction (refer to Figure 2-4)

$(EPA)_N$ = 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

The larger value of $(EPA)_N$ or $(EPA)_T$ may be conservatively used 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 2-8

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.

The total $(EPA)_A$ for a wireless carrier shall be determined in accordance with Annex C when specific antenna and mounting information is not available.

2.6.9.2.1 Antenna Mounting Pipes

The effective projected area of a mounting pipe above and below the shielded portion of the mounting pipe 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 Ca determined from Table 2-8.

2.6.9.2.2 Effective Projected Area for Mounting Frames (Figure 2-5)

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 2-5), C_{as} shall be equal to C_f in accordance with 2.6.9.1.1.

$(EPA)_{FN}$ = 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:

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

supporting structure. A drag factor of 2.0 for flat members and a drag factor of 1.2 for round members shall be applied to the projected areas of all members. The total effective projected area shall be multiplied by factor equal to 0.75 for square platform 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.

2.6.9.2.5 Effective Projected Area for Symmetrical Circular Ring Platforms

The effective projected area, $(EPA)_A$, of symmetrical circular ring platforms (refer to Fig 2-9) 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 and a drag factor of 1.2 for round members shall be applied to the projected areas of the supporting members and the ring members. A 0.50 factor shall be applied to total effective projected area of the supporting members and a 1.75 factor shall be applied to the total effective projected area of the ring members. The resulting total 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.

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 effective projected area.

2.6.9.3 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 2-10

C_d = 1.2, drag factor for guy

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

L_G = length of guy

G_h = gust effect factor from 2.6.7.2

q_z = velocity pressure at mid-height of guy from 2.6.9.6

θ_g = true angle of wind incidence to the guy chord

Note: A higher drag factor, 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 z for a guy segment shall not be less than zero.

2.6.9.4 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 2-11.

Except as provided in 2.6.9.2.1, shielding from an appurtenance shall not be considered when a value of K_a less than 1.0 per 2.6.9.2 is used to determine the design wind force on the appurtenance.

Note: Shielding considerations will vary with wind direction.

2.6.9.5 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 2-12. 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 2.6.9.4.

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 2-12.

2.6.9.6 Velocity Pressure

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

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

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

I_S = moment of inertia of a section of a structure;
i = number designating the level of the 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_g = equivalent stiffness of guys;
K_m = simplified natural frequency conversion factor for guyed masts;
K_S = coefficient used to determine fundamental frequencies of a structure;
k_e = seismic force distribution exponent;
L = height of pole structure;
L_{gi} = average chord length of guys at elevation i;
L_S = length of a section of a structure;
M_S = total mass of a section of a structure;
m = subscript denoting quantities in the m^{th} mode;
N = standard penetration resistance of a soil;
N_i = number of guys at guy elevation i;
n = number designating the uppermost level of the structure or number of guy levels;
PI = plastic index of a soil;
R = response modification coefficient;
S_1 = maximum considered earthquake spectral response acceleration at 1 second;
S_A = design spectral response acceleration;
S_{am}= design spectral response acceleration at period T_m;
S_{az}= acceleration coefficient at height z;
S_{D1} = design spectral response acceleration at a period of 1.0 second;
S_{DS} = design spectral response acceleration at short periods;
S_s = spectral response acceleration at short period;
S_U = undrained shear strength of a soil;
T_m = period for mode m;
T_O = period used to define the design spectral response;
T_S = period used to define the design spectral response;
V_s = total seismic shear;
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 guys;
W_I = weight used to determine fundamental frequencies of a structure;
W_L = weight of structure excluding appurtenances;
W_m= effective modal gravity load;
W_t = total weight of structure including appurtenances and guys;
W_u = weight of discrete appurtenances in the top third of structure;
W_2 = weight of structure and appurtenances within top 5% of structure height;
w_a = average face width of structure;
w_i = portion of total gravity load assigned to level i;
w_o = face width at base of structure;
w_z = portion of total gravity load assigned to level under consideration;
z = number designating the level under consideration.

2.7.3 General

Antennas and antenna supporting structures require special considerations of their response characteristics in regions of high seismicity. The provisions of this Standard provide design

2.7.5 Maximum Considered Earthquake Spectral Response Accelerations

The maximum considered earthquake spectral response accelerations at short periods (S_s) and at 1 second (S_1) shall be as given in Annex B and Appendix 1 except as provided by 2.7.5.1.

2.7.5.1 Site-Specific Procedures for Determining Ground Motion Accelerations

For structures located in regions not included in Annex B, the maximum accelerations S_s and S_1 shall be based on regional seismicity and geology and shall be expressed as a ratio to the acceleration due to gravity. The maximum considered earthquake ground motion shall be taken as the motion represented by assuming 5% of critical damping having a 2% probability of exceedance within a 50 year period.

A site-specific geotechnical investigation and a dynamic site response analysis shall be used to determine S_s and S_1 for structures in all Site Class F locations (see Tables 2-12 and 2-13).

2.7.6 Design Spectral Response Accelerations

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

$$S_{DS} = 2/3\ F_a\ S_s$$

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

where:

F_a = acceleration-based site coefficient based on site class and spectral response acceleration at short periods from Table 2-12.

F_v = velocity-based site coefficient based on site class and spectral response acceleration at 1 second from Table 2-13

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

2.7.7 Equivalent Lateral Force Procedure (Method 1)

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 total seismic shear (V_s) in accordance with 2.7.7.1.

3. Distribute the total seismic shear in accordance with 2.7.7.2.

4. Analyze the structure statically using the seismic forces as external loads.

2.7.7.1 Total Seismic Shear

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

$$V_s = \frac{S_{DS} \; W \cdot I}{R}$$

Alternatively, for ground-supported structures, the total seismic shear, V_s, need not be greater than:

$$V_s = \frac{f_1 S_{D1} W \cdot I}{R}$$

When the alternate equation for V_s is used, V_s shall not be less than 0.044 S_{DS} W I and for sites where S_1 equals or exceeds 0.75, V_s using the alternate equation shall also not be less than:

$$V_s = \frac{0.5 S_1 W \cdot I}{R}$$

where:

S_{DS} = design spectral response acceleration at short periods from 2.7.6

S_{D1} = design spectral response acceleration at a period of 1.0 second from 2.7.6

S_1 = maximum considered earthquake spectral response acceleration at 1 second from 2.7.5

f_1 = fundamental frequency of the structure in accordance with 2.7.11

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 structures, 2.5 for latticed guyed masts and 1.5 for tubular pole structures

2.7.7.2 Vertical Distribution of Seismic Forces

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

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

Table 2-1
Classification of Structures

Description of Structure	Class
Structures that due to height, use or location represent a low hazard to human life and damage to property in the event of failure and/or used for services that are optional and/or where a delay in returning the services would be acceptable.	I
Structures that due to height, use or location represent a significant hazard to human life and/or damage to property in the event of failure and/or used for services that may be provided by other means.	II
Structures that due to height, use or location represent a substantial hazard to human life and/or damage to property in the event of failure and/or used primarily for essential communications.	III

Table 2-2
Wind Direction Probability Factor

Structure Type	Wind Direction Probability Factor, Kd
Latticed structures with triangular, square or rectangular cross sections including appurtenances	0.85
Tubular pole structures, latticed structures with other than triangular, square or rectangular cross sections, strength design of appurtenances	0.95

Table 2-3
Importance Factors

Structure Class	Wind Load Without Ice	Wind Load With Ice	Ice Thickness	Earthquake
I	0.87	N/A	N/A	N/A
II	1.00	1.00	1.00	1.00
III	1.15	1.00	1.25	1.50
Note: Ice and earthquake loads do not apply to Class I structures				

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

For the purposes of this Standard, the following definitions apply.

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 Notation

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

The minimum acceptable models of analysis are as follows:

(a) 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.

(b) Self-Supporting Pole Structures

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].

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 noted above.

(c) Guyed Masts

1. An elastic three-dimensional beam-column where the mast is modeled as equivalent three-dimensional beam-column members supported by cables represented either as non-linear elastic supports or cable elements. 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. The cables are represented as cable elements.

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. The cables are represented as cable elements.

3.4.1 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). 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 five. 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 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 Figure 3-2 and 3-3):

3.6.1 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 2.6.9.6) 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.

3.6.2 Guyed Masts

For guyed masts with three or more spans and with at least one mast span greater than 80 feet [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 feet [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 2.6.9.6) 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 loadings 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.

K = effective length factor;
KL/r = effective slenderness ratio;
$\left(\frac{KL}{r}\right)_o$ = effective slenderness ratio of built-up member acting as a unit;
$\left(\frac{KL}{r}\right)_m$ = modified effective slenderness ratio of built up member;
L = laterally unbraced length of member;
L_B = lateral unbraced length;
L_T = torsional unbraced length;
L/r = slenderness ratio of member;
L_c = clear distance for design bearing strength;
M_n = nominal flexural strength;
M_{nx} = nominal flexural strength about the x-axis;
M_{ny} = nominal flexural strength about the y-axis;
M_{nw} = nominal flexural strength about the major principal axis;
M_{nz} = nominal flexural strength about the minor principal axis;
M_u = flexural moment due to factored loads;
M_{uw} = flexural moment about the major principal axis due to factored loads;
M_{uz} = flexural moment about the minor principal axis due to factored loads;
N_t = flexural to torsional unbraced length ratio;
n = number of threads per inch;
p = pitch of threads;
P_e = elastic Euler buckling load;
P_l = nominal strength of a link plate;
P_n = nominal axial strength;
P_r = resistance required at a panel point within a face of a latticed structure;
P_s = minimum bracing resistance normal to the supported member;
P_u = axial compressive force due to factored loads;
r = governing radius of gyration about the axis of buckling;
r_i = minimum radius of gyration of an individual component of a built-up member;
r_{ib} = radius of gyration of individual component about its centroidal axis parallel to the axis of buckling under consideration for the built-up member;
r_x = governing radius of gyration about the x axis of buckling;
r_y = governing radius of gyration about the y axis of buckling;
r_z = governing radius of gyration about the z axis of buckling;
R_n = nominal bearing strength at bolt or attachment holes;
R_{np} = nominal strength of connecting element;
R_{nt} = nominal tensile strength of bolt or anchor rod;
R_{nv} = nominal shear strength of bolt or anchor rod;
S = minimum elastic section modulus;
S_w = elastic section modulus to the leg tip about the major principal axis;
S_z = elastic section modulus to the leg tip about the minor principal axis;
s = longitudinal center to center spacing (pitch) of any two consecutive holes;
T_n = nominal torsional strength;
T_u = torsional moments due to factored loads;
T_{ub} = bolt tensile force due to factored loads;
t = thickness of the member or of the critical connected part;
U = reduction factor for effective net area calculation;
U_{bs} = reduction coefficient for block shear rupture;
V_n = nominal shear strength;
V_{ub} = bolt shear force due to factored loads;
V_u = transverse shear force due to factored loads;
W_n = net width of the part;

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:

(a) One of the diagonal members is continuous and one of the diagonal members is subjected to tension.
(b) Triangulated horizontal plan bracing (Fig. 4-2) is provided at the intersection point with sufficient resistance as defined in 4.4.1.
(c) A continuous horizontal member meeting the following criteria is connected at the crossover point:
   i. The continuous horizontal has sufficient strength to provide resistance to the leg as defined in 4.4.1.
   ii. 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:

(d) Triangulated horizontal plan bracing with sufficient resistance as defined in 4.4.1 is provided at the crossover point.
(e) A continuous horizontal with sufficient strength as defined in (c) 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).

4.5.3 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.

A minimum of two bolts shall be used at each intermediate connector point when the connected width (i.e. connected leg width of a double angle) of a compression member exceeds 4 in. [102 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 legs for double angles), the effective slenderness ratio shall be modified by the following equations:

(a) 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 $0.75(KL/r)_o$:

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

(b) 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 $0.75(KL/r)_o$:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + 0.82\frac{\alpha_i^2}{\left(1+\alpha_i^2\right)}\left(\frac{a_i}{r_{ib}}\right)^2} \leq \frac{KL}{r_{ib}}$$

(c) For all other conditions:

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

where:

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

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

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

$\frac{a_i}{r_{ib}}$ = slenderness ratio of individual components relative to its centroidal axis parallel to the axis of buckling under consideration for the built-up member

a_i = distance between connectors

r_i = minimum radius of gyration of individual component

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

α_i = separation ratio = $h/2r_{ib}$

h = distance between centroids of individual components perpendicular to the axis of buckling under consideration for the built-up member

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 4.8. 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

4.5.4.1 Effective Yield Stress

For 60° and 90° angle members, the effective yield stress for axial compression, F'_y, shall be determined as follows:

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

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

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

The width to thickness ratio (w/t) shall not exceed 25 for angle members (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 400. The effective yield stress, F'_y, shall be determined as follows:

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

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

$0.448\,E/F_y < D/t \le 400$ $\qquad F'_y = \dfrac{0.337E}{(D/t)}$

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 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 shall be detailed with draw 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:

$\phi_t = 0.80$ for guy anchor shafts
$\phi_t = 0.90$ for other members

$$P_n = F_y A_g$$

For tension rupture in the effective net section:

$\phi_t = 0.65$ for guy anchor shafts
$\phi_t = 0.75$ for other members

$$P_n = F_u A_{en}$$

For block shear rupture:

$\phi_t = 0.65$ for guy anchor shafts
$\phi_t = 0.75$ for other members

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

$U_{bs} = 1.0$ for latticed tower connections

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

where:

Ag = gross area

A_{en} = effective net area

A_{gv} = gross area subject to shear

A_{nv} = net area subject to shear

A_{nt} = net area subject to tension

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 $s^2 / 4g$ in accordance with the following:

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

where:

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

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 = $1\text{-}x/L_c$ where $0.75 \le U \le 0.9$ unless otherwise noted herein

x = connection eccentricity (distance from the outer face of the connected element to the centroid of the member for eccentric connections and the distance from the centroid of the cross section to the centroid of the half-cross section for concentric connections)

L_c = length of the connection in the direction of loading (center-to-center of outside holes or the length of weld in the direction of loading).

Notes:
1) For single bolted angle members U shall be equal to 0.75.
2) Alternatively, when the outstanding leg of a member is ignored in calculating A_n, U need not be less than 1.0.
3) For HSS bracing members with single concentric gusset plates U need not be less than 0.75.
4) For solid round bracing members with single concentric gusset plates U shall be equal to 1.0.
5) For bracing members with single eccentric gusset plates, flattened ends or other eccentric connections U need not be less than 0.70.
6) The effective net area reduction accounts for the effects of eccentric axial brace loads.
7) Leg members connected on one side with sleeves shall satisfy the interaction equation for joint eccentricities in 4.8. Leg members connected with more than one sleeve shall be considered concentric with U equal to 1.0.

4.7 Flexural Members

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

$\phi_f = 0.9$
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' = effective yield stress as determined from 4.5.4.1

Z = plastic section modulus

4.7.2 Tubular Round Members

For tubular round members, the diameter to thickness ratio (D/t) shall not exceed 400. M_n shall be determined as follows:

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

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

$$0.309\frac{E}{F_Y} < \frac{D}{t} \leq 400 \qquad M_n = \left(\frac{0.330\ E}{(D/t)}\right)S$$

where:

D = outer diameter of tubular member

t = wall thickness of tubular member

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

S = elastic section modulus

Z = Plastic 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 Single Equal Leg Angle Members

For 60^O and 90^O single angle members, bending shall be considered about the major and minor principal axes of the member.

4.7.4.1 Effective Yield Stress

Effective yield stress for flexural loading, F'_{yf}, based on local buckling shall be determined in accordance with the following:

$$\frac{b}{t} \leq 0.54\sqrt{\frac{E}{Fy}} \qquad F'_{yf} = 1.5\ Fy$$

$$0.54\sqrt{\frac{E}{Fy}} < \frac{b}{t} \leq .91\sqrt{\frac{E}{Fy}} \qquad F'_{yf} = \left[2.43 - 1.72\left(\frac{b}{t}\right)\sqrt{\frac{Fy}{E}}\right]Fy$$

$$\frac{b}{t} > 0.91\sqrt{\frac{E}{Fy}} \qquad F'_{yf} = \frac{0.71E}{\left(\frac{b}{t}\right)^2}$$

where:

b = width of angle leg

t = thickness of angle

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

F_y = steel yield strength, ksi [MPa]

When investigating joint eccentricities that exceed normal framing eccentricities as defined in 4.4.4, the following interaction equation shall be satisfied:

$$\left|\frac{P_u}{\phi_n F_y A}\right| + \left|\frac{M_u}{\phi_f M_n}\right| \leq 1.0$$

where:

P_u = axial force due to factored loads

P_n = nominal axial strength

M_u = resultant flexural moment due to factored loads based on the eccentricity exceeding normal framing eccentricities

M_n = nominal flexural strength

ϕ_a = 0.90 = resistance factor for axial compression
= 0.90 = resistance factor for axial tension

ϕ_n = 0.90 = resistance factor for yielding under normal stress

ϕ_f = 0.90 = resistance factor for flexure

4.8.1.2 Single Equal Leg Angle Members

Single Angle members subjected to combined bending and axial force shall satisfy the following interaction equations:

(a) when $\dfrac{P_u}{\phi_a P_n} \geq 0.2$

$$\frac{8}{9}\left(\left|\frac{P_u}{\phi_a P_n}\right| + \left|\frac{B_1 M_{uw}}{\phi_f M_{nw}}\right| + \left|\frac{B_1 M_{uz}}{\phi_f M_{nz}}\right|\right) \leq 1.0$$

but $\dfrac{P_u}{\phi_a P_n}$ shall not be greater than 1.0

(b) when $\dfrac{P_u}{\phi_a P_n} < 0.2$

$$\left|\frac{P_u}{2\varphi_a P_n}\right| + \left|\frac{B_1 M_{uw}}{\varphi_f M_{nw}}\right| + \left|\frac{B_1 M_{uz}}{\varphi_f M_{nz}}\right| \leq 1.0$$

4.9.6.2 Design Bearing Strength

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

$$R_n = 1.2(L_c+d/4)tF_u \leq 2.4dtF_u$$

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

$$R_n = 1.0L_ct\ F_u \leq 2.0dtF_u$$

where:

ϕ = 0.80

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]

F_u = specified minimum tensile strength of the critical connected part

d = nominal bolt diameter

t = thickness 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

(a) When threads are excluded from the shear plane:

$$R_{nv} = 0.55\ F_{ub}\ A_b$$

(b) When threads are included in the shear plane:

$$R_{nv} = 0.45\ F_{ub}\ A_b$$

where:

F_{ub} = Specified minimum tensile strength of bolt

A_b = nominal unthreaded area of bolt

Slotted holes must be perpendicular to the line of force. 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:

$\phi = 0.75$

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

R_{nv} = Nominal shear strength of bolt

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

R_{nt} = Nominal 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.

$\phi_p = 0.90$ for yielding

$\phi_p = 0.75$ for rupture

$\phi_p = 0.75$ for block shear

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

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

(c) For shear yielding:
$R_{np} = 0.60 F_y A_{gv}$

(d) For shear rupture:
$R_{np} = 0.60 F_u A_{nv}$

(e) For block shear:
Refer to 4.6.3.

where:

A_{gv} = gross area subject to shear

A_{gt} = gross area subject to tension

A_{nv} = net area subject to shear

A_{nt} = net area subject to tension

Notes:
1. Refer to 4.6.3.1 for the determination of net area.
2. The net area of a connection plate shall not be considered larger than 85% of the gross area.

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 towers, 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 [2200 kN].

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

4.9.7.1 Tubular Pole Structures

The design length of a slip type splice 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.

4.9.8 Guy Assembly Link Plates

The design strength of a link plate, ϕP_l, shall be taken as the lowest value of:

(a) Tension on the effective area:

$\phi = 0.75$ $\quad\quad P_l = 2\ t\ b_{eff}\ F_u$

(b) Shear on the effective area:

$\phi = 0.75$ $\quad\quad P_l = 0.6\ A_{sf}\ F_u$

(c) Bearing on the projected area at the pin:

$\phi = 0.90$ $\quad\quad P_l = 1.8\ A_{pb}\ F_y$

(d) Yielding on the gross area:

$\phi = 0.90$ $\quad\quad P_l = A_g\ F_y$

where:

a = the shortest distance from the edge of the pin hole to the edge of the member measured parallel to the direction of the force

A_{pb} = projected bearing area

A_{sf} = 2 t (a + d/2)

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.

d = the pin diameter

t = the thickness of the plate

Table 4-8: Effective Yield Stress for Polygonal Tubular 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, ksi [MPa]
t = wall thickness, inches [mm]
w = flat side dimension calculated using an inside bend radius equal to 4t
E = modulus of elasticity, ksi [MPa]

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.

$$\mathrm{CVN} = \frac{F_y\ t}{5.54\ c}\ \mathrm{ft-lb} \quad \text{or} \quad \mathrm{CVN} = \frac{F_y\ t}{710\ \ c}\ \mathrm{Joules}$$

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.

Unless design wall thicknesses are based on actual wall thickness measurements, the design wall thickness for HSS shapes with design yield strengths above 52 ksi [360 MPa] shall not exceed 0.93 times the nominal wall thickness.

5.5 Fabrication

Fabrication shall be in accordance with AISC-LRFD-99 section M2.

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 then 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 D1.1-04 Structural Welding Code-Steel.

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.

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

The 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 1200 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^{o} F [16^{o} C] shall be within upper and lower limits of 15 and 7 percent, 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.

7.7.2 Pre-stressing

Pre-stressing shall be required for guy assemblies (excluding insulators) with factory installed end fittings at both ends. The pre-stressing force for a cable shall be equal to 45 percent 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-stressing.

7.7.4 Striping

When pre-stressing 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 (examples are provided in 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
= 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 percent 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 anchors for structures designed in accordance with this Standard.

9.2 Definitions

Foundation or anchorage: Substructures designed to transmit reactions to the underlying soil or rock or supporting structure.

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.

Rock: a natural aggregate of mineral grains connected by strong and permanent cohesive forces.

9.3 Site Investigation

In the absence of a geotechnical report, presumptive soil parameters are provided in Annex F.

A site investigation shall be required for a Class III structure and is preferred for Class I and II structures. Recommendations for geotechnical investigations are contained in Annex G.

9.4 Design Strength

The nominal resistance multiplied by the appropriate resistance factor specified herein shall be equal to or greater than the reactions from all factored load combinations defined in 2.3.2.

(Note: When geotechnical recommendations are based on allowable resistances, the nominal resistance of soil or rock shall be determined by multiplying the allowable resistances by the corresponding factor of safety reported in the geotechnical recommendations. When specific geotechnical parameters or the factor of safety is not reported in the geotechnical recommendations, a factor of safety equal to 2.0 shall be used).

9.4.1 Design Strength of Soil or Rock

The design strength of soil or rock shall be equal to $\phi_s R_s$.

where:

ϕ_s = 0.60 for bearing on rock or soil 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.

(Note: for foundation analyses which model the lateral stiffness of the soil, factored reactions for the analysis shall be divided by ϕ_s. 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).

R_s = nominal soil resistance

9.4.2 Design Strength of SubStructure

The design strength of concrete and steel foundations, and anchorages shall be in accordance with ACI 318-05 and AISC-LRFD-99, respectively, or the appropriate material specification for other materials. (Note: The strength reduction factors specified in ACI 318-05 Appendix C apply for the load combinations defined in 2.3.2.)

9.5 Displacements

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. For these conditions, displacements may only be ignored when the lateral displacement at grade level is less than or equal to 0.75 inch [20 mm] for the serviceability limit state condition specified in 2.8.

9.6 Seismic Considerations

When a self-supporting latticed structure is supported by independent foundations and is located in a region where the earthquake spectral response acceleration at short period, S_s, from 2.7.5 is greater than 1.00, 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 seismic shear as calculated in 2.7.3 in compression and in tension.

Exception: Other approved methods may be used where it can be demonstrated that equivalent restraint can be provided.

9.7 Frost Depth Considerations

When a structure is supported on soil, which displays significant ice lens development during freezing, the minimum foundation base depth shall be at or below the frost depth listed in Annex B. Alternatively, the provisions of SEI/ASCE 32-01 “Design and Construction of Frost-Protected Shallow Foundations” may be satisfied.

9.8 Submerged Conditions

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.

For cable support systems, a 3/8 in. [10 mm] diameter cable shall be considered as standard in order to minimize safety sleeve size requirements.

Notes:

1. When a safety climb device is not continuous over the entire height, climber attachment anchorages shall be available at a maximum spacing of 4 ft [1.2 m] over the height not equipped with a safety climb device.
2. A safety climb device is not required for each climbing facility when multiple climbing facilities are provided. The safety climb device shall be provided for the climbing facility that is continuous over the height of the structure.
3. Ladder cages and hoops are not recommended for communication structures due to the need to service the structure at various locations. If provided, a separate safety climb device is required for structures over 30 ft. [9 m] in height.
4. Climbing and safety climb devices need not be installed over the entire height of a structure when their installation would adversely affect the performance of an antenna. In such case, the structure shall be equipped with a warning sign or climber attachment anchorages shall be provided in accordance with the requirements of Note 1.
5. Structures not designed for nor equipped with a climbing facility over their entire height (i.e. structures not intended to be climbed that are maintained by other access means), need not have warning signs.

12.4 Strength Requirements

A load factor, α_L = 1.5, shall be applied to the nominal loads specified herein:

The minimum nominal load on individual rungs or steps shall be equal to a normal concentrated load of 250 lbs [1.1 kN] applied at the worst-case location and direction.

The minimum nominal load on ladders shall be 500 lbs [2.2 kN] vertical and 100 lbs [445 N] horizontal applied simultaneously, concentrated at the worst-case location between consecutive attachment points to the structure.

The nominal load on rest platforms or antenna mounts designed for access shall be equal to 250 lbs [1.1 kN].

The minimum uniform nominal live load, in addition to dead loads, on working platforms shall be equal to 25 pounds per square foot [1.2 kPa] over the entire platform working area but not less than 500 lbs [2.2 kN] nominal load.

The minimum nominal concentrated live load on a handrail shall be equal to 150 lbs [670 N] and applied in any direction. The minimum nominal uniform live load on a handrail shall be equal to 40 lb/ft [580 N/m] and applied in any direction (not simultaneous with concentrated load).

Safety climb devices shall meet the requirements of ANSI A14.3-1992 Section 7.0. The anchor points of cable-type safety climb devices shall be designed for a nominal vertical load of 2,700 lbs [12 kN]. For rail-type safety climb devices that are attached to the ladder, the ladder supports shall be designed for a nominal vertical load of 1,400 lbs [6 kN] for each 20 ft [6 m] length.

The minimum vertical nominal load on a climbing attachment anchorage shall be 3300 lbs [14.7 kN].

13.3.5 Slip Splice

The slip splice length tolerance shall not exceed + 20% and – 0% of the design slip splice length. Splices shall be pulled together to ensure firm contact. When installed slip splices are in firm contact but do not satisfy the design splice length of 4.9.7.1, the installed joint shall be evaluated by reducing the effective yield stress for each section at the splice linearly from F'_y to 0.50 F'_y as the splice length reduces from 100% to 67% of the design splice length. Installed slip splices with lengths less than 67% of their design splice lengths shall require reinforcement.

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 10 mph [4.5 m/s] at ground level and with no ice on the structure or the guys and with no solar distortion effects.

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:

a) 6 in. [152 mm] for guys with nominal diameter of 0.5 in. [13 mm] or less;
b) 10 in. [254 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 0.50 in. [13 mm].

14.0 MAINTENANCE AND CONDITION ASSESSMENT

14.1 Scope

This section addresses the maintenance and condition assessment of structures.

14.2 Maximum Intervals

Maintenance and condition assessment recommendations are as follows:

a) Three-year intervals for guyed masts and five-year intervals for self-supporting structures.
b) After severe wind and /or ice storms or other extreme conditions.
c) Shorter inspection intervals may be required for Class III structures and structures in coastal regions, in corrosive environments, and in areas subject to frequent vandalism.

Maintenance and condition assessment guidelines are provided in Annex J.

15.0 EXISTING STRUCTURES

15.1 Scope

This section addresses the evaluation and modification of existing structures.

15.2 Definitions

Design documents: Documents indicating the proposed design and related details for the modification of an existing structure including the reinforcement or replacement of existing members and/or their connections.

Existing structure: An erected structure.

Fabrication drawings: Drawings required for the fabrication of components for a proposed modification of an existing structure including member cut length, hole sizes, edge distance, tolerances, weld details and other related fabrication details.

Feasibility structural analysis: A preliminary review to determine the overall stability and the adequacy of the main structural members of an existing structure to accommodate a proposed changed condition in accordance with this Standard.

Installation documents: Documents indicating the implementation procedures for a proposed changed condition and/or modification of an existing structure including rigging methods, temporary support requirements and other related construction considerations to provide for the safety and stability of the existing structure during construction.

Rigorous structural analysis: A comprehensive structural analysis to determine the overall stability and the adequacy of structural members, foundations and connections of an existing structure to accommodate a changed condition in accordance with this Standard.

Rigging: Equipment and techniques used during the installation and modification of the structure and/or its appurtenances.

15.3 Classification

The classification of an existing structure shall be determined in accordance with Table 2-1, considering the reliability requirement of the structure based on the land use surrounding the structure and the performance requirements of the services provided.

15.4 Changed Conditions Requiring a Structural Analysis

As a minimum, existing structures shall be analyzed in accordance with this Standard, regardless of the standard used for the design of the original structure, under any of the following conditions:

a) a significant change in type, size, or number of appurtenances such as antennas, transmission lines, platforms, ladders, etc.
b) a structural modification, excepting maintenance, is made to the structure
c) a change in serviceability requirements
d) a change in the classification of the structure to a higher class in accordance with Table 2-1.

Note: Existing structures need not be re-analyzed for each revision of this Standard unless there are changed conditions as outlined above.

Evaluation of appurtenance changes for existing Class I and II structures originally designed in accordance with a previous revision of this Standard may be based on Revision F of this Standard. The appurtenance change shall be considered significant when strength requirements increase by more than 5% for any structural component, in which case the required modifications and/or final acceptance shall be determined in accordance with this Standard.

15.5 Structural Analysis

15.5.1 Feasibility Structural Analysis

A feasibility structural analysis is used as a preliminary review to identify the impact of proposed changed conditions. This type of analysis determines the overall stability and the adequacy of the main structural members to support a proposed changed condition. Acceptance of changed conditions shall be based upon a rigorous structural analysis in accordance with 15.5.2. A feasibility structural analysis does not include the evaluation of connections and may consider that the structure has been properly installed and maintained.

The reactions from a feasibility structural analysis may be compared to the original design reactions to identify the impact on foundations due to proposed changed conditions. When the original design reactions are based upon an Allowable Stress Design procedure, the original reactions shall be multiplied by a 1.35 factor for comparison to the reactions determined in accordance with this Standard.

15.5.2 Rigorous Structural Analysis

A rigorous structural analysis is used to determine the final acceptance of proposed changed conditions and/or required modifications. This type of analysis determines the overall stability and the adequacy of structural members, foundations and connection details. A rigorous structural analysis may consider that the structure has been properly installed and maintained.

For a rigorous analysis of a foundation, site-specific geotechnical and foundation data are required.

Note: Certain foundation details and connection details (such as inside weld sizes of flanged leg connections) cannot be determined without dismantling the structure or extensive field non-destructive testing. The assumptions regarding these types of details shall be documented along with the results of the rigorous structural analysis.

15.5.3 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:

a. Previous structural analysis
b. Installation, material lists and fabrication drawings
c. Geotechnical reports
d. As-built drawings of the original installation and/or subsequent modifications
e. Field mapping, measurements and/or material testing (refer to Annex J)
f. Existing and proposed appurtenances listing.

15.5.4 Structural Analysis Report

The structural analysis report shall specify the type of analysis (feasibility or rigorous). A feasibility report shall state that final acceptance of changed conditions shall be based upon a rigorous structural analysis.

15.6 Exemptions

Existing structures originally designed in accordance with a previous revision of this Standard are exempt from the provisions of this Standard pertaining to manufacturing and installation when investigating changed conditions.

Existing structures originally designed in accordance with a previous revision of this Standard shall also be exempt from the following provisions of this Standard pertaining to strength requirements when investigating changed conditions:

a. Section 3.7 Mast shear and torsion: 40% minimum requirement need not apply
b. Section 4.4.1 Minimum bracing resistance: P_r = 1.5% F_s may be used
c. Section 4.6.2 Tension-only bracing members
d. Section 4.9.2 Lock washer restriction for structures > 1200 ft [366m]
e. Section 4.9.7 Minimum leg tension splice capacity
f. Section 7.6.4 Guy articulation
g. Section 8.3 Insulators: proof loading need not apply to existing insulators
h. Section 10.0 Protective grounding
i. Section 12.0 Climbing facility requirements

15.7 Modification of Existing Structures

15.7.1 Design

Modifications to existing structures shall be based on a rigorous structural analysis. A design document shall be prepared indicating the proposed reinforcement of existing members and/or connections and all proposed additional members.

Prior to implementation of the changed conditions and/or modifications, the data designated on the design document requiring verification shall be resolved.

16.0 INSTALLATION

Rigging and temporary supports such as temporary guys, braces, false work, cribbing or other elements required for the 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.

ANNEX A: PROCUREMENT AND USER GUIDELINES (Normative)

This Annex is intended to assist in the procurement of antenna supporting structures and antennas designed in accordance with the ANSI/TIA-222-G Standard. Sections referenced in this annex correspond to sections in the 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.

A.2.2 Classification of Structures

The Standard establishes three classifications of structures based on reliability criteria.

The default Structure Classification is Class II.

The following descriptions indicate the appropriate Classification for a new structure based on the type of service to be provided:

Class I: Structures used for services that are optional or where a delay in returning the services would be acceptable such as: residential wireless and conventional 2-way radio communications; television, radio and scanner reception; wireless cable; amateur and CB radio communications.

Class II: Structures used for services that may be provided by other means such as: commercial wireless communications; television and radio broadcasting; cellular, PCS, CATV, and microwave communications.

Class III: Structures used primarily for essential communications such as: civil or national defense; emergency, rescue or disaster operations; military and navigation facilities.

A.2.3.2 Strength Limit State Load Combinations

The Standard is based on limit states design and specifies appropriate load factors to be applied to the nominal loads defined in the Standard. When supplementary loading requirements for “withstand” or “survival” conditions are specified, load factors, importance factors, gust factors, limit state conversion factors for ice, and directionality factors equal to a minimum of 1.0 shall be used. Unless otherwise specified in the procurement specifications, structures shall also be designed for the load factors and nominal loads contained in the Standard. For locations not included in Annex B, the minimum basic wind speed considered for nominal loads shall be 75 mph [34 m/s] 3-sec gust or as determined in accordance with Section 2.6.4.1 for the site location.

State	County	Min. Basic Wind Speed V (mph)	Max. Basic Wind Speed V (mph)	Min. Basic Wind Speed with Ice V_i (mph)	Max. Basic Wind Speed with Ice V_i (mph)	Min. Design Ice Thickness t_i (in.)	Max. Design Ice Thickness t_i (in.)	Design Frost Depth (in.)	Min. S_S	Max. S_S	Notes
TUTUILA	TUTUILA (AMERICAN SAMOA)	125	125	0.00	0.00	0.00	0.00	0	1.00	1.00	-
VI	VIRGIN ISLANDS	145	145	0.00	0.00	0.00	0.00	0	0.00	0.60	-

Notes:

For a site location not designated as a county, refer to the design criteria maps in Appendix 1 to determine the design values for the site.

The tabulated wind speeds represent the minimum and maximum basic wind speeds associated with a county when multiple basic wind speed contours traverse a county. For these counties, refer to the design criteria maps in Appendix 1 to determine the basic wind speed for a specific site location. When a site is located between two contours, the basic wind speed shall be determined by linear interpolation and rounded off to the nearest 5 mph increment.

The tabulated design values represent the minimum and maximum design values associated with a county when multiple ice and/or wind zones traverse a county. For these counties, refer to the design criteria maps in Appendix 1 to determine the basic wind speed with ice and the design ice thickness for a specific site location.

Earthquake effects may be ignored per this Standard for site locations where S_s does not exceed 1.0 (refer to 2.7.3). Maximum and minimum values for S_s are tabulated to facilitate identifying those counties where earthquake effects may be ignored. For a specific site location where the maximum value of S_s exceeds 1.0 for the county, or for any location where earthquake effects are desired to be determined, refer to the design criteria maps in Appendix 1 to determine the values of S_s and S_1 for the specific site location. When a site is located between two contours, linear interpolation may be used. Values for S_s and S_1 shown on the design criteria maps are expressed as a percent of gravity and therefore must be divided by 100 to be used with this Standard. (The tabulated S_s values were provided by Ken Rukstales, US Geological Survey, National Seismic Hazard Mapping Program.)

For metric units, refer to Annex M for appropriate conversion factors.

The following notes correspond to the notes listed in the county listings of design criteria:

1. Special wind regions exist within the county. Refer to the design criteria maps in Appendix 1. The authority having jurisdiction may require higher basic wind speeds for a specific site location in these regions to account for local wind conditions.

2. Special ice regions exist within the county. Refer to the design criteria maps in Appendix 1. The authority having jurisdiction may require higher basic wind speeds with ice and/or higher design ice thicknesses for a specific site location in these regions to account for local wind on ice conditions.

3. The design frost depth shall be based on regional climatic data and knowledge of local conditions in accordance with 2.6.4.1.

ANNEX J: MAINTAINANCE AND CONDITION ASSESSMENT (Normative)

This annex provides checklists for: (a) maintenance and condition assessment, and (b) field mapping of structures and appurtenances.
Note: This annex does not provide means and methods for RF protection.

J.1 Maintenance and Condition Assessment

A) Structure Condition
   1) Damaged members (legs and bracing)
   2) Loose members
   3) Missing members
   4) Climbing facilities, platforms, catwalks – all secure
   5) Loose and/or missing bolts and/or nut locking devices
   6) Visible cracks in welded connections
   7) 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)
   8) 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
   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 (functioning)
      a) Flasher
      b) Photo control
      c) Alarms

D) Grounding
   1) Connections
   2) Corrosion
   3) Lightning protection (secured to structure)

E) Antennas and Lines
   1) Antenna condition
   2) Mount and/or ice shield condition (bent, loose, and/or missing members)
   3) Feed line condition (flanges, seals, dents, jacket damage, grounding, etc.)
   4) Hanger condition (snap-ins, bolt on, kellum grips, etc.)
   5) Secured to structure

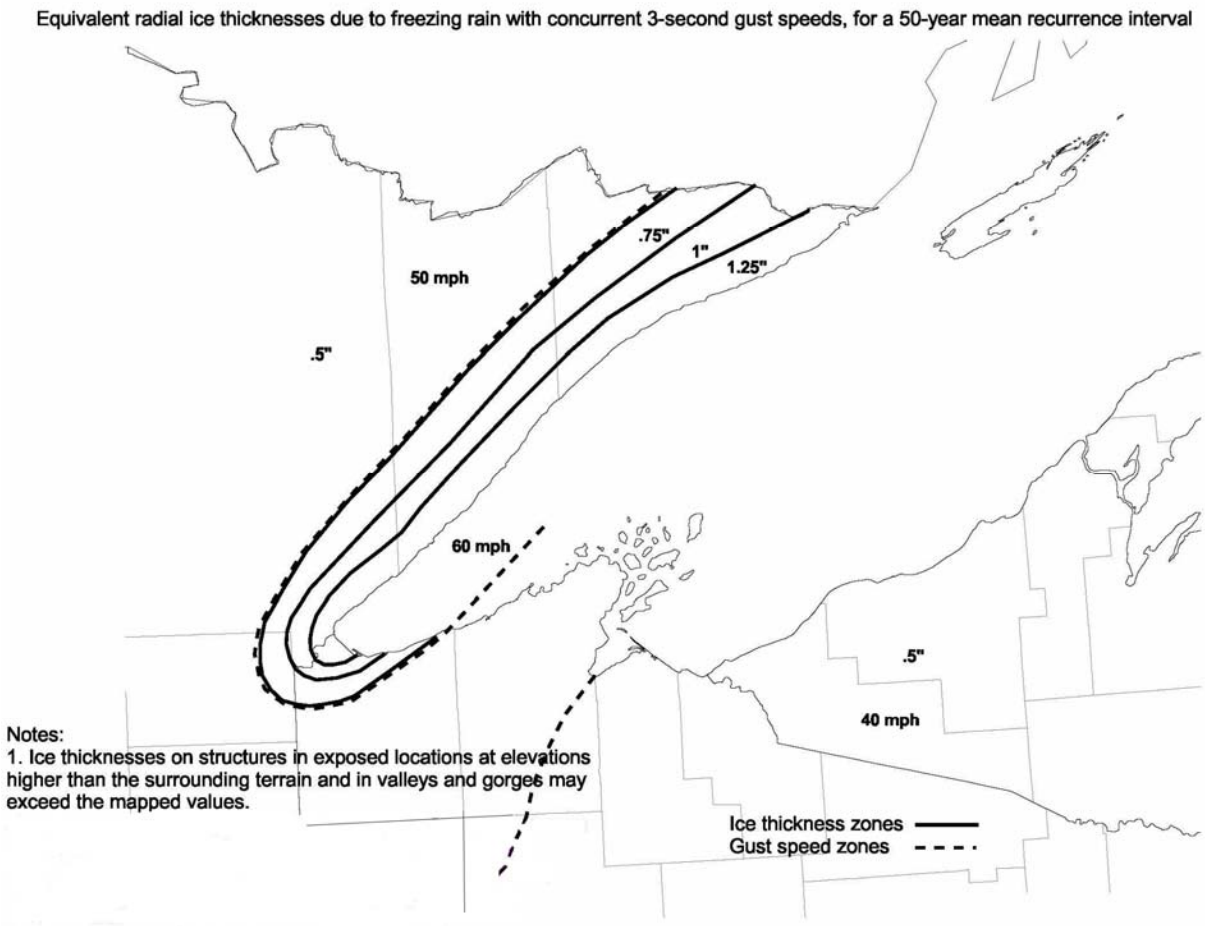
F) Other appurtenances (walkways, platforms, sensors, floodlights, etc.)
   1) Condition
   2) Secured to structure

ANNEX L: WIND SPEED CONVERSIONS (Normative)

This annex provides conversion of wind speeds based on various averaging periods to the 3-sec gust wind speed. Wind data based on other averaging periods are to be converted to a 3-sec gust wind speed for use with the Standard.

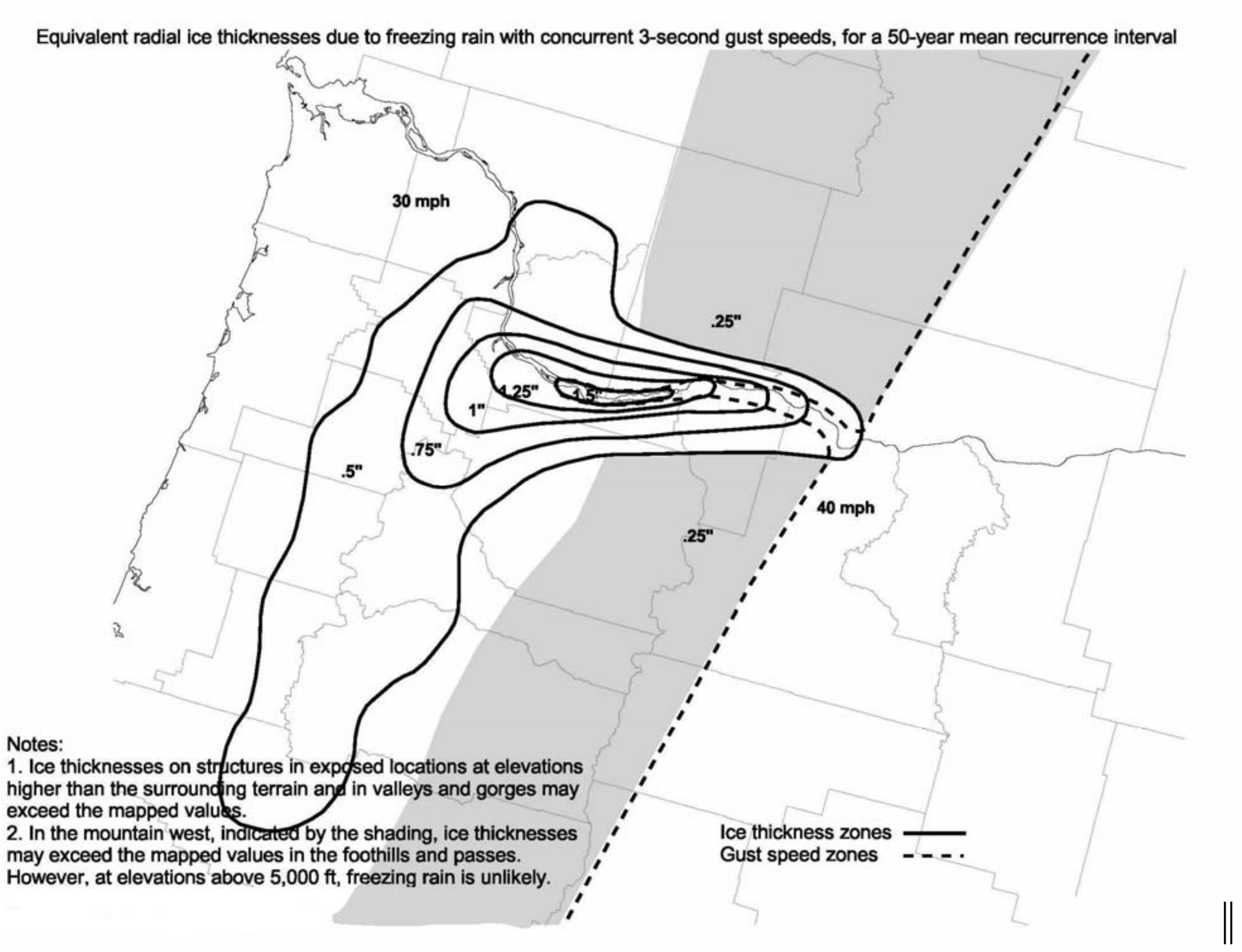
3-sec gust (mph)	Fastest-mile		10-min avg. (mph)	Hourly mean (mph)
	Wind speed (mph)	Averaging period (sec)		
60	47	77	42	40
70	57	63	49	46
80	66	55	56	53
85	71	51	59	56
90	76	47	62	60
95	80	45	66	63
100	85	42	69	66
105	90	40	73	70
110	95	38	76	73
115	100	36	80	76
120	104	35	83	79
125	109	33	87	83
130	114	32	90	86
135	119	30	94	89
140	123	29	97	93
145	128	28	101	96
150	133	27	104	99
155	138	26	108	103
160	142	25	111	106
165	147	24	115	109
170	152	24	118	113

Notes: 1. For conversion to [m/s] multiply the above values by 0.447.
2. Linear interpolation may be used between the values shown.



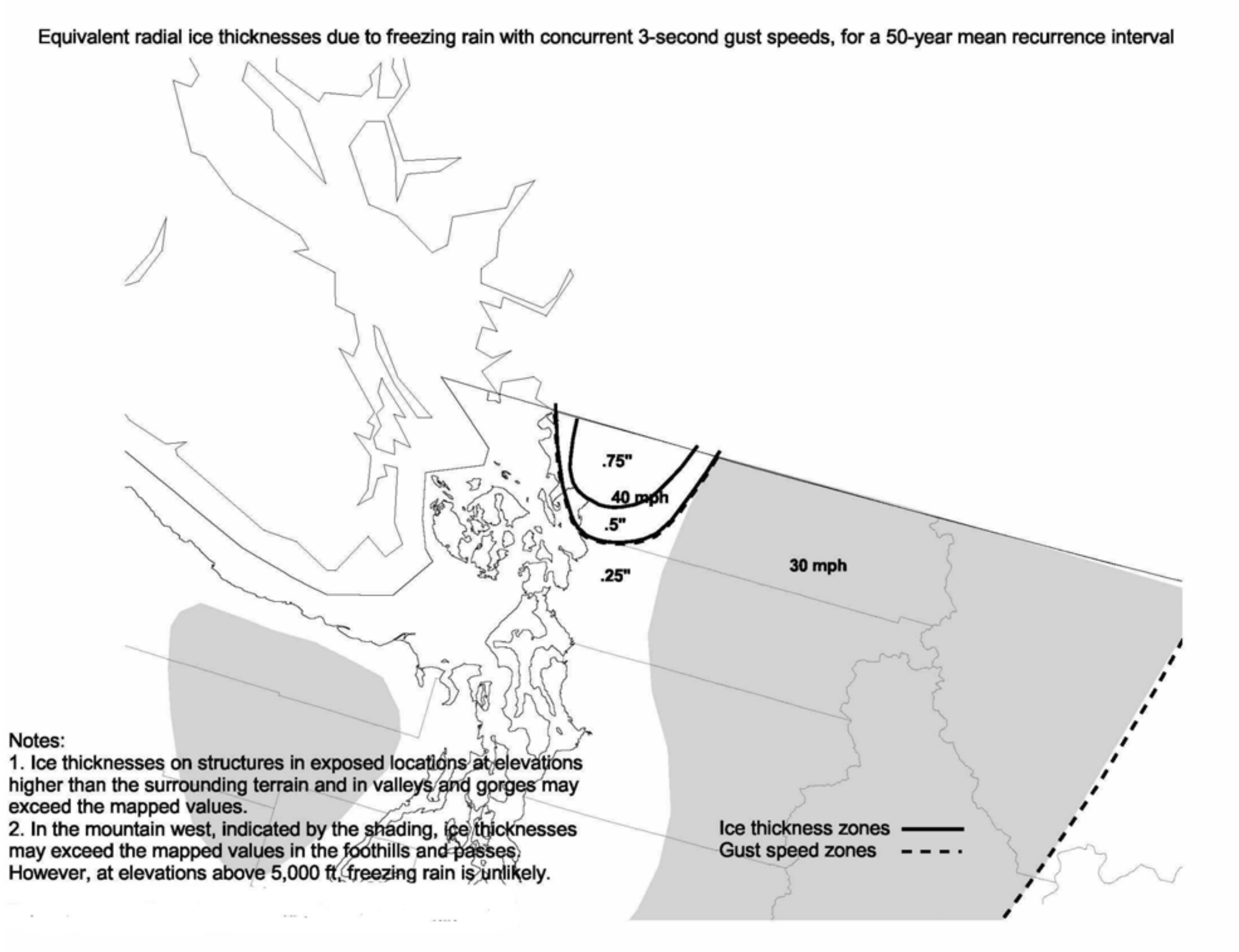
Note: For purposes of this Standard, ice may be ignored in the ¼" thickness zones

FIGURE A1-2c LAKE SUPERIOR REGION BASIC WIND SPEED WITH ICE V_i mph and DESIGN ICE THICKNESS t_i inch



Note: For purposes of this Standard, ice may be ignored in the ¼" thickness zones

FIGURE A1-2d COLUMBIA RIVER BASIN BASIC WIND SPEED WITH ICE V_i mph and DESIGN ICE THICKNESS t_i inch



Note: For purposes of this Standard, ice may be ignored in the ¼" thickness zones

FIGURE A1-2e NORTHWEST REGION BASIC WIND SPEED WITH ICE V_i mph and DESIGN ICE THICKNESS t_i inch