COMMENTARY

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

ANSI/TIA-222-H

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This Commentary is not a part of the ANSI/TIA-222-H Standard nor is it an official document of the Telecommunications Industry Association.

The purpose of this Commentary is to provide information regarding the background and intent of the provisions of the Standard. This Commentary does not duplicate the commentaries of the reference standards listed in Annex U.

The Commentary is based on the Author's participation on committees during the development of the Standard since 1984. Tower Numerics Inc. and the Author encourage interested parties to provide comments, clarifications, corrections, and proposed revisions to continuously improve and enhance this document. The goal is to create a starting point for establishing a consensus document to be published as a TIA approved commentary in future revisions of the ANSI/TIA-222 Standard.

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C OBJECTIVE

Standards for communication structures have developed as an Industry Standard intended to facilitate the design, manufacturing and procurement of communication structures starting in 1949 with the Radio-Electronics-Television Manufacturers Association publication of TR-116 for structures supporting radio transmitting antennas. Since the first publication, the standard has evolved with the advancement of technology for communications. In 1957, RS-194 was published for microwave relay systems by the same manufacturers association and eventually was combined with the TR-116 standard in 1959 for the publication of RS-222 by the Electronic Industries Association (EIA). This standard became an internationally recognized standard and was continually updated with revisions A (1966) through H (2017). ANSI accreditation was obtained for Revision D (1986) facilitating the inclusion into the US national building codes as recognized literature. In 1988 the Telecommunications Technologies Group of EIA merged with the United States Telecommunications Suppliers Association (USTSA) to form the Telecommunications Industry Association (TIA). The EIA designation eventually was removed from the designation of the standard. Because the TIA-222 standard was often used by the small wind turbine industry, an annex was added in Revision H which included fatigue design criteria critical for structures supporting small wind turbines (SWT).

Throughout the development of the Standard, the objective has been to provide a uniform straight forward approach to procurement, design and manufacturing to result in similar products between manufacturers and to provide guidance to owners for purchasing and maintaining communication structures. As the Industry grew, providing uniform methods of evaluating existing structures became an important part of the Standard.

Communication and SWT supporting structures have been considered unique structures and not adequately covered in building code standards. Research funds were not readily available for communication structures compared to funds for building, bridges and other structures. Consequently, the standards for communication structures were based on the performance of structures obtained from owners, manufactures and consultants active in the deployment of communication structure. National standards for wind, ice and earthquake design criteria were reviewed and adopted into the standards. Additional criteria were adopted by consensus of the committee membership as required due to the unique characteristics of communication structures. National design standards such as the American Institute of Steel construction (AISC), The American Welding Society (AWS) and the American Concrete Institute (ACI) were also used when possible and modified and supplemented by consensus. Portions of international standards such as the British standard BS8100 were also adopted.

Throughout the development of the Standard, it was recognized and understood by the committee that the design of communication and SWT supporting structures could not be considered as an exact science and that relying on past performance was essential. The committee's objective was to create an environment where sharing of past performance was encouraged in committee meetings with the objective of providing recognized literature that could be referenced by national and international building codes which would also facilitate the economical deployment of structures to accommodate the rapidly changing needs of the Industry. This philosophy has resulted in a continuous process of improvement based on new research in combination with the vast experience obtained by members of the committee with acceptable and unacceptable performances of existing structures.

Safety issues have been a major consideration for the standard as the industry grew. An unacceptable number of accidents were occurring as the Industry rapidly expanded. Safety criteria from OSHA and other international standards were reviewed and adopted, modified and supplemented by the consensus process for the unique aspects of communication structures.

As the Industry grew, criteria for mounting systems became essential for safety and for providing a uniform approach for procurement, design, manufacturing and the evaluation of exiting mounting systems.

C SCOPE

The Standard starting with Revision G is based on a limit states design (load and resistance factor design or LRFD). Prior revisions of the standard were based on allowable stress design (ASD) criteria. This decision was made considering that most recent structural engineering research was based on limit states design but more importantly because many structures covered by the Standard are flexible structures (e.g., guyed masts and self-supporting poles). The allowable stress method was not believed to result in consistent reliability for flexible structure due to a flexible structure's non-linear response to loading.

As an example, for a guyed mast, a design could appear to be adequate using working strength level wind loads, when in reality, a minor increase in loading could result in instability or other types of failure due to the non-linear response characteristics of flexible structures to loading.

Section 3.0 of the standard outlines the required methods of analysis for use with the limit states design approach. Another advantage of LRFD is that using the analysis methods specified in Section 3.0 of the Standard avoids the need to investigate elastic buckling for self-supporting pole structures as well as cantilever and intermediate spans of guyed masts.

Although the Standard is primarily intended to apply to steel structures, other material have been proposed for communication structures such a fiberglass, concrete, wood, etc. The scope of the standard is not to provide comprehensive the design and Maintenance criteria for other than steel structures, but to require an equivalent level of expected reliability.

The need for structures to support communication equipment has rapidly increased and has resulted in the use of structures primarily intended for other uses such as water towers, utility structures, sign structures, bridges, etc. In order to avoid confusion among different industries and their associated standards, the scope of the Standard clarifies that the standard developed for the primary use of a supporting structure should be used for structural evaluation to support communication equipment. The required reliability of other types of structures may vary from the reliability requirements for communication structures. In cases where the reliability requirements are lower for the supporting structure than specified in the Standard for steel structures (e.g., wood utility poles), the supporting structure may need to be reinforced.

Serviceability requirements are important for the proper performance of many compunction structures; therefore, the serviceability requirements of the standard (e.g., limitations for the movement of the supporting structure) are required to be satisfies. In addition, the Standard requires that the methods used to determine effective projected area from the Standard apply as most standards for other supporting structure do not adequately cover the determination of effective projected areas of most communication equipment and appurtenances.

Structural requirements during construction and associate means and method for the structures covered by the standard are unique compared to other industries. Other standards (ANSI/TIA-322 and ANSI/ASSE A10.48) have been developed for these purposes and are excluded from the scope of the Standard.

C1.0 GENERAL

The limit states for strength and serviceability are intended to be defined in accordance with traditional LRFD criteria.

Serviceability limits states are often critical for structures supporting microwave antennas, radar installations, directions antennas, etc.

The methods of structural analysis to be used with the limit states are defined in Section 3.0 of the Standard and are critical to ensure elastic stability for flexible structures.

C2.0 LOADS

C2.2 Risk Categorization of Structures

The risk categorization of structures covered by the Standard is based on the services provided, the consequences of delays in returning the services and the risk to human life and property. The categorizations are based on the input of users throughout the Industry and are intended to correlate with the risk categorizations of ASCE 7 for buildings and other structures.

C2.2.1 Definitions

Hardened networks were introduced in Rev H. The need for hardened networks became painfully obvious during hurricane events in Louisiana and Florida. During these events, although some structures failed, the majority of issues with continuing communication operations during emergency conditions were related to the damage of equipment that supported the communications such as power issues, vandalism, etc. as opposed to failures of structures. These incidences heightened the need for awareness of the importance of hardened sites in addition to the reliability requirements for the supporting structures.

C2.2.2 General

Table 2-1 outlines the descriptions of four risk categories starting from the lowest to the highest reliability requirements (Risk Categories I to IV). The loading requirements as well as other requirements throughout the standard become more stringent as the risk category increases. As an example, Risk Category III communication structures that support non-redundant services are expected to be hardened as defined in Section 2.2.1.

Many structures covered by the Standard are in remote locations and provide optional services or services that are available from other means where a delay in replacing a structure would be acceptable. These structures are intended to be classified as Risk Category I and are not required to satisfy earthquake or ice loading conditions per the exceptions listed in Section 2.3.2. There is a vast number of existing and planned structures in this category and the cost to the public requiring these structures to meet extreme ice and earthquake loading could not be justified given the low risks involved. This is especially the case for lightweight guyed masts. Similar examples in other industries would be wood utility poles in remote areas, certain temporary and agricultural structures, etc.

Many communication structures are located adjacent to critical structures with higher risk categories than the communication structures. Many of these communication structures are of a very low mass compared to the adjacent structures or are located adjacent to barriers that would prevent the higher risk category structure from being physically impacted by the failure of the communication structure. In these cases, the risk category of the communication structure applies to determine the design criteria and other requirements for the communication structure.

C2.2.2.1 Multiple Services

Multiple types of services are commonly supported on commercial communication structures, from simple 2-way radio communications to emergency communications. Many times, the emergency services are not required to be hardened, for example when other communication methods are available to a police or fire department. Under these circumstances, the risk category is allowed to be based on the risk category associated with the non-emergency commercial communication services supported on the structure.

C2.3 Combination of Loads

The loads governing the design of structures covered by the Standard are dead loads, ice loads, wind loads, earthquake loads and temperature change effects. A load factor of 1.0 is applied to these loads as they are considered ultimate loads. With the exception of the loads due to temperature effects, the loads are based on return periods adopted from ASCE 7 that correspond to the risk category of the structure.

Guy tensions create a significant load on guyed masts but are not considered as an external load with a load factor applied and therefore are not included in the loading combinations. Guy tensions under loading conditions depend on the following: their initial tension under a no-load condition, the temperature change from the no-load condition to the extreme loading condition and the forces acting directly on the guys from the extreme loading condition.

The loading combinations presented are unique to the structures covered by the Standard and are different from than the loading combinations for building and other structures presented in ASCE 7. The loading combinations specified in the Standard were established by the committee based on decades of experience in combination with research and data compiled for buildings and other structures. It was recognized that it is not feasible to model the exact loads from extreme wind, ice and earthquake loads which are extremely complex for the structures covered by the Standard. From the committee's experience, structures that satisfy the specified loading combinations will be robust and perform with the reliability intended for each risk category.

Additional loading combinations may be justified to consider when there are loads, that when removed, may result in significantly higher strength requirements. For example, for a guyed mast, if there is a significant load in both the top cantilever and the span immediately below the cantilever, removal of one of the loads (e.g., a future loading condition) may govern the strength requirements in the span below the cantilever. This occurs because the mast behaves in a similar manner as a continuous beam over multiple supports. Although Section 3 of the Standard does not address the removal of loads, the requirements for pattern loading for guyed masts are based on this characteristic of guyed masts.

The wind and earthquake loading combinations for self-supporting structures require two loading combinations, one for maximum dead loads (1.2 load factor) and one for minimum dead loads (0.9 load factor). Dead loads for the structure and appurtenances are difficult to determine with a high degree of accuracy due to the thickness tolerances of commonly utilized structural members, variations in galvanizing thicknesses, unknown appurtenance data, etc. The weight of guy assemblies is known with a higher accuracy compared to the dead load of structural members and appurtenances and is the justification for the 1.0 load factor applied to the weight of guy assemblies. The weight of guy assemblies is also included in the loading combinations that exclusively apply to self-supporting structures due to the occasional use of guy assemblies as tension-only diagonal bracing members.

Both the maximum and minimum dead load conditions (1.2 and 0.90 load factors) are required for wind and earthquake loading for self-supporting structures since vertical loads can significantly increase or decrease strength requirements. For example, the legs of a self-supporting latticed tower under compression from overturning loads would be governed by the maximum dead load condition whereas the minimum dead load condition would govern the legs under tension from overturning loads. A similar situation occurs with mat foundations supporting self-supporting structures. The maximum dead load condition may govern due to a limiting soil bearing pressure whereas the minimum dead load condition may govern the required weight of the foundation and soil overburden to prevent overturning. Guyed masts resist overturning and vertical loads in a different manner than self-supporting structures and are only required to satisfy maximum dead load conditions.

In determining earthquake loading, a load factor is not applied to dead loads since earthquake loads are based on a maximum design ground acceleration and therefore the earthquake loads from deal loads are considered ultimate loads. Applying a load factor greater than 1.0 to dead loads for determining earthquake loads would overestimate earthquake loading and a load factor less than 1.0 would underestimate earthquake loading. The dead loads applied to the structure directly for the earthquake loading combinations do have the 1.2 and 0.9 load factors applied as for the extreme wind loading combinations.

The ice loading combination is intended to represent a maximum vertical load condition (1.2 dead load factor) which must be satisfied for ensuring overall stability for a structure and adequate member strengths. A minimum dead load condition is not required to be considered as it would be contrary to the objective of investigating a maximum vertical load condition. Additionally, the variance in dead loads is considered to be insignificant compared to the approximate method of determining the weight of ice for an extreme ice condition.

Guyed masts are especially sensitive to extreme ice loading. This occurs due to the combined effect of ice weight and wind pressure directly applied to the guys which increases the guy tension and in turn imposes a large downward force on the mast. For this reason, the extreme ice loading combination often governs stability and strength requirements for guyed mast.

There are 2 ice loading combinations that generally govern: one for a maximum ice thickness condition with a corresponding wind speed and one with a lower ice thickness occurring with a higher wind speed. The maximum ice thickness condition results in the maximum vertical load; however, the lower ice thickness with a higher wind speed may govern the maximum lateral load. The ASCE 7 ice maps were established using ice accretion models correlated with historical ice storm data. The ASCE 7 ice map inherently covers both the maximum vertical and the maximum lateral loading conditions but only indicates one ice thickness and one wind speed for a given location. This was accomplished by specifying the maximum ice thickness with an equivalent calculated wind speed. A wind speed was determined such that when applied to the maximum ice thickness, the governing lateral load would be obtained.

Foundation designs require additional considerations for the load factors apply to dead loads. The weight of soil directly supported by the foundation (i.e., the soil directly above a spread footing) and the weight of the foundation are both considered as dead loads for the loading combination under consideration (i.e., a 1.2 or 0.9 load factor). The weight of soil outside the perimeter of the foundation that is considered to resist uplift or overturning reactions is considered as a nominal soil strength with a 0.75 resistance factor applied as specified in Section 9.4. For example, for a foundation design resisting uplift that includes the consideration of an equivalent uplift cone of soil to resist uplift, a resistance factor of 0.75 is required to be applied to the weight of soil in the uplift cone. Refer to the Section C9.0 for additional commentary.

A unique situation exists for guy anchorage foundation because only a maximum dead load combination with a 1.2 dead load factor is required for guyed masts. The anchorage reactions from each loading combination are to be considered for the design of a guy anchorage; however, the weight of soil directly above the foundation and the weight of the foundation are to be multiplied by 0.9 when determining the strength of the foundation to resist uplift. As with other foundation types, the soil outside the perimeter of the foundation considered to resist uplift (e.g., an equivalent uplift cone) is considered as a nominal soil strength and multiplied by a 0.75 resistance factor in accordance with Section 9.4.

C2.4 Temperature Effects

Temperature effects from a change in temperature between the time of installation and the occurrence of an extreme wind or earthquake loading event are considered insignificant. This is accomplished by defining the design initial tension as the tension of the guys at their anchorage at an ambient temperature of 60 degrees F. The extreme wind and earthquake loading are then assumed to occur at a temperature of 60 degrees F. The effects of actual temperature changes under an extreme wind or earthquake condition are considered to be insignificant.

When the ambient temperature at the site is not 60 degree F at the time of installation, the initial tension must be adjusted in accordance with Section 13.3.2 such that the design initial tension would be expected to be present when the ambient temperature becomes 60 degrees F. The installation guy tensions would need to be lower than the specified design initial tension for ambient temperatures greater than 60 degree F at the time of installation or under no load conditions and conversely higher for colder temperatures. The design initial tension temperature of 60 degrees F was chosen as it was believed to reasonably represent the condition of the guys appropriate for the guy damper vibration considerations specified in Section 7.6.

For the extreme ice loading condition, temperature effects are required to be considered for guyed masts given the sensitivity of the mast response to changes in guy tensions and the magnitude of the drop in temperature that often occurs with an extreme ice condition. Additionally, the drop in temperature effects the lateral stiffness of the guys and also increases the download forces applied to the mast.

C2.6.3 General

The TIA-222 Standard is intended to provide recognized literature for communication and small wind turbine support structures as these structures are not adequately covered in other design standards. Loading criteria for buildings and other structures from national standards such as

ASCE 7 and AASHTO as well as other international standards were used where appropriate. Supplemental requirements were included in the Standards when required. Many of these requirements are discussed in the commentary for the sections of the Standard where supplementary, alternate or special requirements have been specified.

The wind loading criteria in Section 2.0 is intended to take into consideration the load magnification effects caused by along wind gusts. Cross wind effects are addressed in Annex M.

C2.6.4 Basic Wind Speed and Design Ice Thickness

Annex B of the Standard provides wind speeds and design ice thicknesses from ASCE 7. These design parameters are most easily obtained from the ASCE online Hazard tool.

Basic wind speeds for use with the Standard are ultimate 3-second gust wind speeds at 33 feet [10 meters] above ground level in open flat terrain based on mean recurrence intervals between 350 and 3,000 years depending on risk category. Wind speeds must be converted to equivalent 3-second gust wind speeds as defined above for use with the Standard.

Annex L tabulates wind speed conversions for other averaging periods (e.g., fastest-mile, 1-minute average and hourly mean). ASCE 7 Figure C26.5-1 may be used to convert wind speeds based on other averaging periods to hourly mean wind speeds which may be converted to 3-second gust wind speeds using Annex L. ASCE 7 Table C26.5-2 provides the relationship between 3-second gust wind speeds and Saffir-Simpson hurricane wind speeds. ASCE 7 Appendix CC provides 3-second gust wind speed maps based on mean recurrence intervals of 10, 25, 50 100 years which may be required to investigate special serviceability requirement for the structure or for a foundation.

Design ice thicknesses for use with the Standard are based on freezing rain with a density of 56 pounds per cubic foot and are based on a 500-year mean recurrence interval. Importance factors are applied to the design ice thickness to result in 1,000 and 1,400-year mean recurrence intervals for use with Risk Category III and IV structures respectively. Importance factors for other mean recurrence intervals are provided in ASCE 7 Table C10.4-1.

Ultimate ice thicknesses less than 0.5 inches are not required to be considered. This provision is a carryover from TIA-222-G where 50-year mean recurrence interval ice thicknesses less than or equal to 0.25 inches were not required to be considered.

Ice maps for 250, 500, 1,000 and 1,400 mean recurrence intervals are planned to be included in the next revision of ASCE 7 which will eliminate the need for the use of importance factors for ice thickness as done for wind speed in the current version of ASCE 7.

The committee has traditionally allowed the use of the most recent ASCE 7 wind and ice maps for use with the Standard as the revisions of each standard are inevitably published on different time schedules.

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

The parameters essential for determining an acceptable degree of certainty for wind speeds or ice thickness are specified for areas where wind speeds or ice thicknesses are not specified by ASCE 7 such as international site locations, locations where wind or ice is known to vary significantly or in a region prone to in-cloud icing. The ASCE 7 ice thicknesses are based on glaze ice from freezing rain and do not include ice thicknesses for in-cloud icing which can result in significantly greater ice thicknesses. In-cloud ice loading conditions must be based on regional climatic data.

C2.6.5.1.2 Exposure Categories

Limits to the upwind surface roughness length required to define exposure categories are specified. The limits were not obtained from ASCE 7 but were derived for use with tall structures (e.g., guyed masts) with heights equal to or greater that the nominal height of the atmospheric boundary layer associated with the upwind surface roughness (also referred to as the gradient wind speed height). The gradient wind speed height is the height above ground level above which the wind speed is assumed to be constant and no longer effected by the friction of the earth’s surface. The wind speed at ground level increases with height for each exposure category up to the gradient wind speed height where the wind speed becomes constant and is equal for all exposure categories (i.e., the gradient wind speed). The gradient wind speed heights are 1,200, 900 and 700 ft for exposures B, C and D respectively. In other words, rougher surfaces impact wind speeds up to higher elevations. Also, for rougher terrains (e.g., Exposure B), the friction of the earth reduces the gradient wind speed at a faster rate which results in a lower design wind speed at the earth’s surface compared to smoother terrains (e.g., Exposure D).

A common misconception is that Surface Roughness B cannot apply to a tall structure assuming that shielding of the terrain would not have an impact on the structure. The exposure category is defined by the roughness of the upwind terrain over a much larger area compared to a structure's footprint. It is the friction of the ground surface that reduces the design wind speed for Exposure B as opposed to shielding. For the same reason, the presence of just one tall narrow structure 6,000 ft upwind of a structure under investigation would not be considered in defining the surface roughness or exposure category.

ASCE 7 requires the extent of a given upwind surface roughness to meet a minimum distance in order to justify the use of Exposure B or D. This distance is equal to the greater of 2,600 ft. or 20 times the height of the structure for Exposure B and 5,000 ft or twenty times the height of the structure for Exposure D.

The committee added limits to the distance requirements based on structure height for tall structures. The logic for the limits involves structures with a height greater than or equal to the gradient wind speed height for a ground surface roughness. The distance required need not be greater than the distance that results in the full development of the wind profile up to the gradient wind speed height, above which the wind speed is considered constant and unaffected by ground surface roughness. This distance occurs when the height of the structure is equal to the gradient wind height and the extent of the upwind surface roughness is twenty times the height of the structure (i.e., twenty times the gradient wind speed height). A greater extent of upwind surface roughness would not be expected to change the wind profile. For Surface Roughness B the limit of required surface roughness is equal to 20(1,200) or 24,000 ft and for Surface Roughness D the limit is 20(700) or 14,000 ft.

As an example for Surface Roughness B terrain: for a 1,200 ft. structure (i.e., the gradient wind speed height for Surface Roughness B), a fully developed wind profile for Exposure B is considered to exist up to the top of the structure when there is 20 times 1,200 or 24,000 ft of upwind Surface Roughness B terrain. The wind speed is considered to be equal to the gradient wind speed at the top of structure and all elevations above the top. It follows that a taller structure 1,500 ft. in height at the same location would be subjected to the same wind profile and additional upwind Surface Roughness B would not be expected to change the wind profile for the taller structure. The Standard does not require the 1,500 ft. structure to use the more stringent Exposure C wind profile for design or analysis or to require an additional 20(1,500 - 1,200) or 6,000 ft. of additional upwind Surface Roughness B terrain in order to justify the use of the Exposure B wind profile.

The 20 times the height of the structure requirement becomes more of an issue as the structure height increases up to 2,000 ft. which is not uncommon for guyed mast broadcast structures. The 20 times the structure height requirement without a limit results in overly conservative requirements for the use of Exposure B in the lower portion of the structure below the gradient wind speed height. For Exposure D, the requirement without a limit results in a greater required distance of Surface Roughness D before the higher Exposure D wind profile must be used which is considered unconservative.

When a separation of Surface Roughness B or C exists between an Exposure D area and a site, ASCE 7 and the Standard require the use of Exposure D for a minimum distance from the Exposure D area equal to the greater of 600 ft. or 20 times the height of the structure. The Standard specifies a limit to the 20 times the height of the structure distance equal to 20 times the gradient wind height for Surface Roughness C (20 times 900 equal to 18,000 ft.) as this is the distance of Surface Roughness C that would result in the full development of the wind profile for Exposure C as explained above for Surface Roughness B and D. In lieu of a similar limit for Surface Roughness B conditions separating a site from an Exposure D area, the Standard allows the use of Exposure C when the separation consists of at least 10,000 ft. of Surface Roughness B which is considered an adequate distance by the committee for a relatively rough terrain to justify the use of Exposure C vs. Exposure D for a structures of any height.

The Standard allows the use of site-specific exposures using site-specific investigations which can be cost justified for existing structures. The 0.70 value of K_{zmin} is specified to not permit wind speeds less than the minimum for Exposure Category B.

C2.6.6.1 Wind Speed-Up Over Hills, Ridges and Escarpments

The Standard adopted the provision of ASCE 7; however, the slope of the topographic feature is defined as the ratio of height to upwind length. ASCE 7 defines the slope as the ratio of height to the horizontal distance from the crest to the location on the feature where the elevation is equal to half the height of the crest. The definition used by the Standard was chosen because it was considered to be less confusing when only general information about the topographic features in an area was available for a proposed structure and detailed information regarding the site would not be known until a later date in the planning process.

C2.6.6.2 Topographic Procedures

Wind speed must increase when wind is obstructed by a topographic feature as any fluid does when moving through a constricted channel. Wind speed-up results in higher wind velocities at the top of the feature but the effect of the topographic feature reduces as the elevation above the top increases. Eventually the wind speed becomes equal to the wind speed from a wind profile upwind of the topographic feature. The elevation that this occurs is dependent on the geometry of the feature.

The Standard allows 3 methods for determine wind speed-up criteria. Determining the magnitudes and profiles of wind speeds at topographic features is not an exact science. Engineering judgement is required.

Method 1 is a conservative simplified method useful when detailed topographic information is not available for new structures or when an owner wishes to compare quotations for structures based on a consistent simple to define design criteria or for purchasing a standard design that can be used at multiple locations. Although this method is conservative, it has an advantage for new structures by allowing future capacity for additional or alternative equipment loading by using the more detailed Methods 2 or 3 for the analysis of the structure as an existing structure.

C2.6.6.2.1 Simplified Topographic Factor Procedure (Method 1)

Method 1 is based on the ASCE 7 method with the conservative assumption that the structure is located at the crest of the topographic feature and the feature has a slope equal to the ASCE 7 upper bound slope. For simplification, the ratios of the ASCE 7 Exposure Category B and D K_1 values to the Exposure Category C K_1 value were determined for each topographic feature. The average of the ratios for each Exposure Category are presented in Table 2-4 as K_c along with other exposure category dependent variables used in the Standard.

C2.6.6.2.2 Rigorous Topographic Factor Procedure (Method 2)

Method 2 is based on the document SEAW RSM-03 referenced in Annex U. This method allows a more detailed approach to determining wind speed-up values. The method is closely aligned with the ASCE 7 method but also includes provisions for flat top hills and ridges that result in significant reductions in wind speed-up values compared to the ASCE 7 values for hills and ridges. Use of this method, as with the ASCE 7 method, requires the use of engineering judgment as topographic features often have complex geometries that vary significantly in different directions.

C2.6.6.2.3 Site-Specific Topographic Procedure (Method 3)

The use of a site-specific procedure is allowed by the Standard given the complexities of many topographic features and the significant increase in wind speed-up that may be determined using Methods 1 or 2.

C2.6.7 Rooftop Wind Speed-Up Factor

Depending on the geometry of a building and the surroundings around the building, wind speed-up can be significant due to the obstruction of the building to wind flow. It is recognized that buildings in cites may be surrounded by similar sized buildings. In these cases, wind flow would be disrupted by the surrounding buildings and wind speed-up is not required to be considered. The surrounding buildings must be relatively close to the building supporting the structure in order to ignore wind speed-up. Different wind directions may have different obstructions requiring different wind speed-up consideration for different wind directions.

As with topographic features, wind speed-up determination for rooftops is not an exact science and engineering judgement is required. The criteria presented in the Standard is not intended to represent exact wind speed-up conditions but is intended to result in robust designs that will provide the desired level of reliability based on the required risk category for a structure. A maximum wind speed-up factor of 1.3 is specified which may conservatively be used over the entire height of the structure vs. the linear increase indicated in Figure 2-2.

Similar to wind speed-up for topographic features, the wind speed-up for rooftops is assumed insignificant at a height above the rooftop equal to the width or the height of the building whichever is less. Due to the complexities involved, the Standard allows the use of wind tunnel test data that may be available from the design criteria used for the supporting building or other available site-specific information.

Rooftop wind speed-up is conservatively not included in the velocity coefficients (C) for consideration of transitional or supercritical flow force coefficients. The effect of turbulence from parapets and equipment typically located on rooftops is not well understood and together with the conservative method of determining rooftop wind speed-up values, it is a trade-off to not consider the higher wind speeds in the calculation of the velocity coefficients for structures or appurtenances.

C2.6.8 Ground Elevation Factor

The ground elevation factor accounts for the change in air density with increased elevations and was adopted from ASCE 7.

C2.6.9 Gust Effect Factor

The Standard is based on a static wind loading approach to design and analysis. The gust effect factor is intended to account for the varying nature of wind and the dynamic responses of structures. Self-supporting or bracketed latticed towers, pole structures and guyed masts all have different responses to dynamic wind loading. The Standard specifies methods for each type of structure.

There are abundant theories based on research and development regarding the nature of wind and the complex variables that effect the responses of structures to wind. The variables are not easily determined and often can vary under different environmental conditions such that it was not considered feasible to base the Standard on these variables. The objective of the committee was not to quantify the exact varying nature of wind or the exact dynamic responses of structures to wind but to specify prescriptive methods that have proved effective over decades of experience with thousands of structures covered by the Standard.

Fortunately, the structural design of members and connections presented in Section 4.0 are relatively straight forward based on extensive structural engineering research. The approach of the committee was to provide a path to move from the complexity of wind loading and structural responses to well defined structural engineering design criteria. The approach is intended to provide conservative simplified methods based on the past performance of each type of structure covered by the Standard.

The gust effect factors specified are applicable only to 3-second gust design wind speeds. Wind speeds based on other averaging periods would require different gust effect factors to account for the magnitude of gusts that would occur over the averaging period. Annex L provides equivalent 3-second gust wind speeds that may be used with the Standard when wind speeds are specified over different averaging periods.

For rigid structures, the committee adopted the 0.85 rigid structurer gust effect factor from ASCE 7 The 0.85 gust effect factor was considered justified for rigid structures covered by the Standard because a 3-second gust design wind speed was not considered capable of enveloping entire structures. A wind speed averaged over a longer period would be needed for design and analysis. Using the Durst curve from ASCE 7 Table C26.5-1, a gust effect factor of 0.85 would be equivalent to using a design wind speed averaged over a 15 second period. This was considered a reasonable averaging period to consider for the rigid structures covered by the Standard.

Many structures covered by the Standard are flexible which increases the complexity of determining an appropriate gust effect factor. The ASCE 7 gust response factor for flexible structures (ASCE 7 equation 26.11-10) was not adopted by the committee. The ASCE 7 gust effect requires the fundamental natural frequency and the ratio of damping to critical damping in order to determine the gust effect factor. The magnitude of damping ratios is not well known and a small change in magnitude can result in a large difference in the gust effect factor using the ASCE 7 equation. The fundamental natural frequency is known to vary under iced conditions and for frozen soil conditions. For these reasons the committee specifies what is believed to be conservative higher gust effect factors for structures considered to be flexible by the committee.

A single higher gust factor to apply to the entire structure was not considered appropriate for guyed masts as it was known that local gusts occur in different mast spans at different elevations which often results in governing loading conditions. A higher gust effect factor applied to the entire structure would not capture the alternate span loading effects. Pattern loadings were adopted in lieu of a higher single gust factor to account for the dynamic responses of guyed masts with large mast spans.

C2.6.9.1 Self-Supporting or Bracketed Latticed Structures

The committee has traditionally accounted for the dynamic responses of taller latticed structures by increasing the gust effect factor above that used for rigid structures.

Self-supporting and bracketed latticed structures 450 feet or less in height are considered to be rigid. A 0.85 gust effect factor is specified as load magnification effects are not considered to be significant. For taller structures, although the gust wind speed is not considered to envelope the entire structure, load magnification effects due to dynamic responses are expected to be significant. For this reason, the gust effect factor is linearly increased from 0.85 to 1.0 for structures 600 feet or more in height.

Localized gusts are considered to have a significant effect on self-supporting latticed towers with straight sections that extend above the apex defined by the projection of inclined legs. Pattern loadings are specified in Section 3.0 for this condition.

C2.6.9.2 Guyed Masts

Guyed masts with typical guy anchorage locations are considered to behave as rigid structures and therefore a 0.85 gust effect factor is specified. The dynamic response acting as a flexible structure is considered insignificant. Guyed masts are, however, considered to have a nonlinear response to wind loading due to the nonlinear supports provided by the guys supporting the mast and significant P-delta effects from relatively large lateral displacements. For example, the down pull of guys supporting the mast can result in significant moments in the mast as the mast translates laterally. The effects of nonlinear behavior are accounted for by using the analysis requirements for guyed mast specified in Section 3.0.

A guyed mast behaves as a continuous beam over nonlinear elastic supports. Localized gusts occurring on a guyed mast have the same effect as alternate span loading on a continuous beam over multiple supports. A single gust effect factor applied to the entire structure cannot capture this behavior. For this reason, pattern loadings are required per Section 3.0 for guyed masts with large mast spans between guy levels.

C2.6.9.3 Pole Structures

An increase from a 0.85 gust effect factor for a rigid structure to a 1.10 gust effect factor is specified for pole structures to account for their dynamic responses to wind loading. Based on the experience of the committee, the 1.10 gust effect factor was believed to be conservative for Telecom poles with multiple lines installed inside the pole providing significant damping to dynamic wind loads.

Pole structures are considered flexible nonlinear structures. The dynamic response to wind loading for a flexible pole structure is accounted for by increasing the 0.85 gust effect factor for a rigid structure to 1.10. The nonlinear response of pole structures is due to the significant P-delta effects resulting from relatively large lateral displacements. Nonlinear effects are accounted for by using the analysis method requirements for pole structures specified in Section 3.0. Pole structures are considered as self-supporting structures and therefore not governed by localized gusts.

Traditionally a 1.69 gust effect factor has been used with fastest-mile design wind speeds. The 1.69 value was adopted from previous version of the AASHTO standard referenced in Annex U. A fastest-mile wind speed is the average wind speed over the time period for 1 mile of wind to pass the weather station. Part of the 1.69 gust effect factor is intended to account for gusts that could occur over the averaging time period. For example, a 60 mph fastest-mile design wind speed would represent a wind speed averaged over a 1 minute period. A 120 mph fastest-mile design wind speed would represent a wind speed averaged over a 30 second period. The 3-second gust wind speed is considered as a peak wind gust and would be expected to be higher than a fastest-mile wind speed for the same extreme wind loading event. Because each fastest-mile wind speed is averaged over a different time period, there is not a single conversion factor from fastest-mile to 3-second gust wind speeds. Although it is not an exact science, it is generally accepted that a fastest-mile wind speed is approximately 80% of a 3-second gust wind speed. Note that the ratio of fastest-mile wind speeds to 3-second gust wind speeds from Annex X varies from 0.78 to 0.89. With the 80% approximation, the equivalent gust effect factor for a 3-second gust wind speed would be equal to 1.69 times the square of 0.80 for a 1.08 value. Although the committee had established the 1.10 gust effect factor based on correlation with existing pole designs, the value closely matches the 1.08 value.

C2.6.9.4 Spines and Pole Structures Supported on Flexible Structures

The responses of flexible spines and pole structures mounted on guyed masts or self-supporting latticed structures can be amplified due to the response of the supporting structure. This may occur when the supporting structure oscillates under dynamic wind loading and there is a discontinuity or irregularity in stiffness between the cantilever and the supporting structure. The same amplified response can occur when the supporting structure is a flexible building. Flexible buildings are defined as buildings with height to width ratios greater than 5.

The amplification is accounted for by specifying higher gust effect factors for cantilevered structures with a fundamental frequency less than 1 which are considered flexible and sensitive to dynamic effects. Latticed spines are considered less susceptible to amplified responses compared to poles and consequently a lower gust response factor is specified. The higher gust effect factor only applies to the strength design of the spine or pole and the connection to the supporting structure. The connection includes structural members and connections used to transfer the reactions of the cantilevered spine or pole to the supporting structure. For example, the increased gust response factor would apply to a channel frame supporting a top plate connected to the flange of a cantilevered pole as well as the connection of the channel frame to the supporting structure.

The overall response of the supporting structure is not required to be amplified. For this reason, the reactions from the cantilevered spine or pole applied to the supporting structure for analysis of the supporting structure may be based on the gust response factor for the supporting structure. When a cantilevered spine or pole is included in the structural model for the supporting structure, the gust effect factor for the supporting structure may be applied to the cantilevered spine or pole to determine strength requirements for the supporting structure. A separate analysis using the higher gust effect factor could be performed to determine strength requirements for the cantilevered spine or pole and the connection to the supporting structure. The height of the supporting structure is required to include the height of the cantilevered spine or pole. This results in a higher gust response factor for self-supporting latticed towers when the combined height is greater than 450 feet.

C2.6.10 Design Ice Thickness

The design ice thickness requirements for freezing rain were adopted from ASCE 7. The accumulation of ice is known to be a function of wind speed as more water droplets impact and freeze on the structure as the wind speed increases. The wind speed-up factor for buildings is not considered for the escalation of ice thickness as the wind speed up values are considered to be localized compared to a much larger topographic feature where K_{zt} is increased to the 0.35 power.

The ASCE 7 ice thickness maps are based on a 500-year mean recurrence interval for Risk Category II structures. Table 2-3 provides importance factors to be applied to the design ice thickness for higher risk categories. Ice is not a consideration for Risk Category I structures and an importance factor is not provided. The importance factors for Risk Categories III and IV result in ice thicknesses associated with 1,000 and 1,400-year mean recurrence intervals respectively.

As the ice thickness increases, wind loads increase due to the additional projected area of ice. All importance factors for wind loads on ice are therefore equal to 1.0 for all risk categories.

Rime ice and in-cloud icing are not included in the Standard and require site-specific data for locations subjected to such conditions.

The methods for determining the weight and the effective projected area of ice are illustrated in Figure 2-3. The characteristic dimensions for calculating the weight of ice were adopted from ASCE 7.

C2.6.11 Design Wind Load

The design wind load criteria of the Standard are based on determining the design wind forces on the structure without appurtenances and adding the design wind forces from appurtenances and supporting guys. The determination of the effective projected area of a structure with appurtenances such as feed lines, antennas, mounting frames, platforms, climbing facilities, etc. was considered by practicing engineers as the most significant determination impacting the design and analysis of the structures covered by the Standard. Effective projected area determinations were also considered as the most variable and undefined in previous revisions of the Standard. The committee considered the most straight forward, consistent and reasonable approach was to establish a standard procedure based on first considering the design wind forces for the bare structure and then consider reduction factors for determining the design wind forces from appurtenances that account for shielding and wake interference considerations for the structure and the attached appurtenances.

The effects of shielding and wake interference could alternately be considered when determining the effective projected area of the supporting structure assuming the wind direction was such that a mounted appurtenance was on the windward side of the structure. However, for wind in the opposite direction, it would be reasonable to consider shielding and interference effects when determining the effective projected area of the appurtenance. In reality, wake interference effects occur due to upwind and downwind obstructions. The committee chose the simplified approach outlined above to standardize a method that could be consistently applied by practicing engineers. The intent is to improve the prescriptive methods as more research and development becomes available and experience is gained with new types of appurtenances as the Industry evolves with changing technologies.

The design wind forces are conservatively assumed to be in the direction of the wind except for microwave antennas and supporting guys. This was done for simplification and believed to be a conservative approach to a very complex issue. All appurtenances, regardless of their strength or wind capacity, are required to be considered to maintain their shape and remain attached to the structure.

Microwave antennas have unusual shapes and have well documented aerodynamic characteristics that vary with wind direction which results in forces that vary from the direction of the wind (refer to Annex C). The design wind force on guys is considered to act normal to the chord of the guy in the plane defined by a vector defining the wind direction and the guy chord (refer to Figure 2-12). Guy forces, depending on the wind direction may result in both uplift and download forces applied to the guys. For example, windward guys would be subjected to downward forces and leeward guys would be subjected to upward forces.

The wind directions for latticed structures with triangular cross sections are required to be considered normal to each face (0 degrees), each parallel direction to each face (+/- 90 degrees) and into each apex (60 degrees) for a total of 12 directions. For square cross sections the required wind directions are normal to each face (0 degrees) and into each corner (45 degrees) for a total of 8 wind directions. The wind directions resulting in the maximum responses for pole structures depend on the type and orientation of the appurtenances supported by the structure.

For the strength design of the supporting structure, the gust effect factor for the structure is used to determine the design wind forces from appurtenances and guys. For the strength design of appurtenances, a gust effect factor of 1.0 is required to be considered as the 3-second gust is assumed to be capable of enveloping discrete appurtenances and linear appurtenances between their levels of support from the supporting structure. Also, a directionality factor (refer to Section 2.6.11.6) equal to 0.95 is required for the strength design of appurtenances regardless of the directionality factor used for determining the design wind forces to be applied to the supporting structure. This requirement is based on the conservative assumption that the strength requirement for an appurtenance is not significantly dependent on the wind direction.

Shielding considerations are also not allowed for the strength design of appurtenances because it is conservatively assumed that the appurtenance will be subjected to unobstructed wind flow. Shielding considerations are allowed when determining the design wind forces from an appurtenance for the design or analysis of the supporting structure depending on the location of the appurtenance (refer to Section 2.6.11.2 and 2.6.11.4).

The directionality factors specified in Table 2-2 account for the reduced probability that the design wind speed will occur from a direction that results in the maximum structural response. For example, for a triangular cross section latticed structure, the maximum leg compression occurs when the wind direction occurs normal to one face. In addition, depending on the appurtenances supported on the structure, one of the three wind directions normal to a face would be expected to govern. The directionality factor of 0.85 accounts for the reduced probability that the return period wind speed will occur from that specific direction. For cross sections with more than 4 sides, the probability increases that the wind direction will occur from a governing direction and a 0.95 directionality factor is specified.

For pole structures without appurtenances, the maximum response would be expected no matter what direction the return period wind speed occurred from. The directionality factor for this condition is therefore specified to be 1.0. The directionality factor also accounts for the reduced probability that the design wind speed would occur from the direction that results in the maximum force coefficient for appurtenances. For this reason, a 0.95 directionality factor is specified for a pole structure that support appurtenances as well as for the strength design of an appurtenance. The directionality factor appropriate for the supporting structure is intended to be used for determining the design wind forces for appurtenances supported by the structure and applied to the structure for the design or analysis of the supporting structure. The directionality factor for the strength design of an appurtenance may be higher than the directionality factor for the supporting structure.

For the condition of a cantilevered tubular or latticed spine, pole or similar structure supported on another structure, the directionality factor appropriate for the cantilever structure is required to be used to determine the design wind force for the strength design of the cantilever. For the design of the supporting structure, however, the directionality factor appropriate for the supporting structure is intended to be used for determining the design wind force from the cantilever and applied to the supporting structure for the design or analysis of the supporting structure. As with appurtenances, the directionality factor for the strength design of the cantilever may be higher than the directionality factor for the supporting structure.

The maximum design wind force for a latticed structure is limited to the design wind force considering a face solidity ratio of 1.0. This may occur when there are numerous appurtenances attached to the structure and the combination of effective projected areas and force coefficients result in an effective projected area greater than a solid faced structure. Appurtenances supported outside the normal projected area of the structure in the direction of the wind under consideration must be considered in addition to the effective projected area of the solid faced structure. Sections of structures with shrouds may be considered as an appurtenance based on the aspect ratio of the shroud itself (refer to Table 2-9) resulting in a smaller force coefficient compared to a solid faced structure.

2.6.11.1.1 Effective Projected Area of Latticed Structures

The effective projected area equations were adopted from the IASS reference in Annex U. The effective projected areas for latticed towers are for a wind direction normal to a face. The projected areas are adjusted for other wind directions by using wind direction factors. The IASS wind direction factors for flat and round members were not adopted by the committee. The IASS wind direction factors varied with the ratio of round and flat members to the gross area. A simplified approach using wind direction factors believed to be conservative based on the performance of structures covered by the Standard was adopted to account for the effect of wind direction (refer to Table 2-7).

The effective projected area of a square cross section is known to increase as the wind direction varies from a direction normal to a face to a direction along the diagonal of the cross section (i.e., 45 degrees). The increase is dependent on the solidity ratio of a face with a maximum of a 20% increase. This increase accounts for the additional width of the square cross section exposed for the diagonal wind direction and applies to both round and flat structural components.

For triangular cross sections, there is not a significant increase in the width of the cross section for different wind directions compared to a square cross section and no adjustment is required for round structural components. For flat structural components, wind loading is considered to reduce as the wind direction varies from the wind normal direction. This occurs because of the effects of the wind incidence angle on flat structural components combined with the shielding effect provided by flat structural components.

The reduction factor for round structural elements was derived from the IASS equations by dividing the IASS effective projected area equations for cross sections consisting of all round elements by the equation for cross sections with all flat elements. A best fit equation based on the face solidity ratio was used to establish the equation for the reduction factor for round elements specified in the Standard. This was done for both subcritical and supercritical flow conditions.

The IASS equations conservatively assume the presence of internal plan and hip bracing and therefore the projected area of these members is not required to be included in the solidity ratio or projected areas for determining the effective projected area of a latticed structure. The IASS equations also account for the projection of bracing members in adjacent faces. Connection plates for the connections of structural members in a face are required to be included in the projected area for determining the design wind force for a structure.

U shaped or channel members were not addressed in IASS and are required to be considered as flat structural members by the Standard. Although the wind loading on a windward face could be less severe than a flat member, the wind load for a wind direction towards the open side of a U-shaped member would be more severe than a flat member. Considering U-shaped and channel members as flat members is believed to be a rational method to account for these shapes.

The Standard allows the consideration of lower force coefficients associated with supercritical flow conditions for larger diameter round members for no-ice conditions. The irregular formation of ice is considered to prevent the occurrence of transitional or supercritical flow for loading conditions with ice. The concise shapes assumed for determining the projected area and weight of ice are equivalent ice models and not intended to represent the actual shape of ice formation on structural members.

The start of transitional flow from subcritical to supercritical is conservatively assumed to occur at a Reynolds number equal to $3.6x10^5$ (C = 39) with full supercritical flow conditions occurring at a Reynolds number of $7.2x10^5$ (C =78). These Reynolds numbers were adopted from the AASHTO standard referenced in Annex U. (Refer to the additional commentary for Section 2.6.11.1.2).

Attachments to round structural members create roughness which increases the force coefficient and interferes with the formation of supercritical flow. The Standard uses the ratio of the projected area of attachments to the projected are of the round member (R_a) to quantify roughness. As the ratio increases above 0.1, the force coefficient for a round member is linearly increased up to 30% for a ratio of 0.2. For higher ratios, the attachments are required to be considered independently and subcritical flow conditions must be assumed for determining the reduction factor for the round member. For iced conditions, the thickness of ice need not be considered in the determination of the ratio R_a as the criteria for accounting for attachment to round members is prescriptive and not intended to represent actual physical conditions.

Although roughness is not a factor for the determination of force coefficients for flat elements, attachments to flat structural elements are required to be considered independently from the members when the ratio of the projected area of the attachments to the projected area of the member exceeds 0.1. As for round members, when the ratio is 0.1 or less, the projected area of the attachments may be ignored.

2.6.11.1.1.1 Effective Projected Area of Latticed Leg Structures

The effective projected area equations for latticed structures assume either round or square structural elements. Latticed legs made up of smaller individual members are treated as an equivalent round member. The latticed leg is considered as an individual latticed section of a structure in accordance with Section 2.6.11.1.1. The effective projected area of the latticed leg is determined based on the projected area and solidity ratio of one face of the latticed leg. The effective projected area for the direction normal to one face is used along with subcritical values for round elements. Subcritical values are assumed because of the typical size of the structural elements of latticed legs but also because the narrow face widths of latticed legs are assumed to prevent the formation of transitional or supercritical wind flow. The diameter of the equivalent round element is determined by dividing the effective area of the latticed leg calculated as an individual latticed section by its length and by a force coefficient of 1.2. The actual width of the latticed legs is used for determining the solidity ratio of one face of the structure as opposed to the diameter of the equivalent leg. The effective projected area of the structure is determined using the equivalent round member without a reduction for supercritical flow regardless of the diameter of the equivalent round latticed leg.

The effective projected area and weight of ice for a latticed leg is determined in the same manner as a latticed structure. The thickness of ice is therefore not added to the equivalent round member diameter as the effect of ice is included in the calculation of the equivalent diameter.

2.6.11.1.2 Effective Projected Area of Tubular Structures

The effective projected area of tubular structures is a function of Reynold's number, surface roughness, attached irregularities and the inscribed angle of each side and outside corner radius for polygonal cross sections.

The force coefficients specified in the Standard were established based on the ASHTO standard referenced in Annex U together with the combined experience of the committee. Because of the interaction of the variables involved, engineering judgement was required for the generation of Tables 2-8a and 2-8b. The force coefficients specified are based on the research presented in the AASHTO standard referenced combined with the experience of the committee with the unique attached appurtenances typically utilized with the tubular structures covered by the Standard.

Reynolds number is a dimensionless number but is often expressed as the product of 9,200 times the wind speed in mph and the member width in feet. The committee adopted the AASHTO Reynolds numbers of 39 mph-ft and 78 mph-ft for defining the transitional zone between subcritical and supercritical flow conditions. These values are based on using a 3-second gust wind speed for the determination of Reynolds number and have been increased from previous AASHTO specifications that were based on fastest-mile wind speeds.

The Standard is based on ultimate (or factored) 3-second gust wind speeds. Prior to the use of ultimate wind speeds, a load factor of 1.6 was applied to wind loads. A comparatively lower unfactored wind speed was therefore used to determine Reynolds number which could result in higher force coefficients. The consensus of the committee was to use the same Reynolds numbers to define the transitional zone for all 3-second gust wind velocities, whether ultimate for strength design or lower unfactored values for investigating serviceability requirements. Although it was considered unconservative to apply the entire load factor to wind velocity (i.e., ASCE 7 ultimate wind speed approach), the Reynolds numbers used to define the transitional zone were also considered conservative and requiring a reduced wind speed from the ultimate design wind speed to define the transitional zone was not necessary. For fatigue wind loading considerations, the wind speed specified in Section 17.12.1 represents a range of cyclic wind speeds as opposed to an absolute wind velocity. Cyclic loading is considered to occur under relatively low wind speed conditions and for this reason, subcritical flow conditions are required to be considered for fatigue investigations.

Surface roughness and irregularities are accounted for by providing two tables, one for structures with linear attachments (Table 2-8a) and one for structures without linear attachments (Table 2-8b). The force coefficients in Table 2-8a include the effects of step bolts and a defined set of other attachments common for communication structures. The force coefficients in Table 2-8b may also be used for tubular structures with step bolts as well as a safety cable; however, unlike Table 2-8a, the step bolts and safety cable must be considered separately with appropriate appurtenance force coefficients.

Attachments are accounted for depending on their location and their projected areas compared to the projected area of the bare structure. Windward, lateral and leeward zones are defined in Figure 2-5 for the purpose of defining attachment locations.

The sum of the projected areas of flat and round attachments in both of the lateral zones are used to establish appropriate force coefficients for the use of Table 2-8a. Subcritical flow force coefficients are required to be used for the structure when the projected areas exceed 20% of the projected area of the bare structure and additionally the attachments must be considered separately with appropriate appurtenance force coefficients. When attachments are between 10% and 20% of the projected area of the structure, the attachments need not be considered separately but the force coefficient for the structure based on the value of the velocity coefficient (C) must be linearly increased up to 30% but need not be greater than the subcritical force coefficient. When attachments do not exceed 10% of the projected area of the bare structure, the attachments may be ignored and no modification to the force coefficients in Table 2-8a are required.

Flat members in the windward zone can disrupt the formation of supercritical flow and can create a blunt windward shape requiring higher force coefficients for the structure. Flat elements in the windward zone exceeding 20% of the projected area of the structure require the use of subcritical force coefficients linearly increased up to 25% resulting in a maximum 1.5 force coefficient for the structure. Flat plates attached within the flat width of multi-sided sections (e.g., structural reinforcing plates) are not considered to disrupt the formation of supercritical flow or create a blunt windward shape.

Attachments in the windward or leeward zones may be ignored except for flat attachment in the windward zone as explained above which can disrupt the formation of supercritical flow and create a blunt windward shape requiring higher force coefficients for the structure. All attachments or portions of attachment outside any of the zones must be considered separately with appropriate appurtenance force coefficients.

As with the reduction factor for round elements used for latticed structures, the thickness of ice is not required to be considered when determining the percentage of projected areas of attachments to the bare structure. Also, for all iced conditions, subcritical force coefficients are required to be used for both Table 2-8a and 2-8b; however, an increase may be required as explained above when significant flat members are attached in a windward face.

It is always acceptable to conservatively consider all attachments separately with appropriate appurtenance force coefficient and use subcritical force coefficients for the structure increased when required due to flat elements attached to the windward face.

Tables 2-8a and 2-8b include different shape factors for polygonal cross sections with different sides which define the inscribed angle for each side. The force coefficients are based on a nominal corner radius assumed appropriate for communication and SWT tubular structures. When the corner radius is known for the sections of a structure, a reduction in the force coefficients listed in Table 2-8a and 2-8b may be possible by using Table 2-8c. All adjustments for attachments specified for the use of Table 2-8a and 2-8b still apply. Depending on the corner radius, the tabulated force coefficients for polygonal cross sections may be reduced to as low as the force coefficient for a round cross section.

Section 2.6.11.1.2 provides criteria for determining the minimum cross-sectional area required for the placement of lines inside a tubular structure. Many times, the bend radius of lines is not known and using unconservative assumptions may lead to installation issues when multiple lines are intended to be placed in the interior of the structure. Placing lines on the outside of a structure may result in overstresses due to increased force coefficients.

2.6.11.1.3 Uniform Wind and Ice Applied to Structure

Based on the experience of the committee, uniform wind and ice may be considered over the lengths specified. The assumptions required to be made for the design criteria specified by the Standard do not warrant more accurate modeling of wind and ice loading.

2.6.11.2 Design Wind Force on Appurtenances

The general equation for determining the effective projected area of appurtenances was adopted from the IASS reference in Annex U. The equation is prescriptive and not intended to represent the exact loading from an appurtenance. It is intended for noncomplex shapes and is a simplification that has proven to provide reasonable estimates of wind forces that can be assumed to occur in the direction of the wind. The equation assumes that the contribution to the total design wind force from each exposed side of an appurtenance, $(EPA)_N$ and $(EPA)_T$ (refer to Figure 2-6), is proportional to the square of the wind velocity component normal to each side. In reality, for complex shapes such as MW antennas presented in Annex C, a wind tunnel test would indicate that the normal and transverse wind forces from an appurtenance would be a function of the wind incidence angle and that the resultant of the normal and transverse wind forces would not necessarily align with the wind direction. For this reason, the

Standard allows the use of more accurate data when available to determine design wind forces for appurtenances.

A shielding or wake interference factor (K_a) is applied to effective projected area of appurtenances as explained in Section C2.6.11. The concept is based on using a simplified method for determining effective projected areas for structures with appurtenances. For example, for latticed structures, the effective projected area of the bare structure is calculated without regard to attachments. The effective projected areas calculated for appurtenances are adjusted with a reduction factor based on shielding and wake interference considerations. Multiple reductions are not allowed. For example, when supercritical or transitional flow force coefficients are used to determine the effective projected area of an appurtenance, further reductions due wake interference or shielding are not allowed.

Wake interference and shielding considerations depend on the location of an appurtenance and the type of structure supporting the appurtenance. Figure 2-5 defines face zones for this purpose. A reduction may be applied to the effective projected area of appurtenances located entirely inside the cross section of a latticed tower or outside the cross of a structure when located entirely within a face zone as defined in Figure 2-5. The straight-line reduction based on the face solidity ratio of a latticed section is a simplification of the quadratic equation provided in the IASS reference in Annex U. The provision for not requiring a shielding factor greater than 0.6 was also adopted from IASS. The boundaries of the face zones in Figure 2-5 were established by consensus based on the engineering judgement of the committee.

Wake interference and shielding considerations are also provided for mounting configurations. The use of the 0.8 factor specified accounts for shielding of the structure from the mounting configuration and the shielding of the components of the mounting configuration from adjacent mounting members. No additional reductions are allowed when the 0.8 factor is used to determine effective projected areas.

The typical wireless carrier antenna loading EPA values specified in Annex C of Revision G of the Standard were removed for Revision H because of the ongoing rapid changes in antenna geometry and equipment configurations. Many carriers developed their own effective projected areas values desired for design. The EPA values specified in Rev G were intended to account for the mount and all mounted antennas and equipment at an elevation. The total EPA values were intended to be considered the same for all wind directions with a K_a value equal to 1.0.

The Standard provides criteria to utilize appurtenance data that were established based on Revision C of the Standard. Revision C used a wind pressure equation that included a force coefficient for flat member. Accordingly, Revision C specified equivalent flat plate areas based on a 1.0 force coefficient for flat elements and 0.67 for round elements. The 2.0 conversion factor specified for equivalent flat plate areas conservatively assumes that the appurtenance consists of flat elements unless it is known that the appurtenance is made up of all round members, in which case a conversion factor of 1.8 is specified. The use of the 1.8 conversion factor for round elements results in a force coefficient equal to 1.8(0.67) equal to 1.2.

Table 2-9 provides force coefficients for appurtenances to be used in the absence of more accurate data. The table conservatively assumes that an appurtenance is made up of noncomplex elements. The force coefficient is dependent on the aspect ratio of the appurtenance. Because wind flow can occur more freely around all sides of a short appurtenance with a small aspect ratio, lower fore coefficients are specified compared to appurtenances with higher aspect ratios. The aspect ratio is required to be based on the actual dimensions of the appurtenance and not the distance between the supports for the appurtenance nor the section considered to have a uniform wind or ice load.

The force coefficients for HSS members were adopted from the AASHTO standard reference in Annex U. The force coefficients for flat and round elements were established based the engineering judgement of the committee along with criteria from the ASCE 7 and IASS references in Annex U.

Larger diameter round appurtenances under high wind speeds may develop transitional or supercritical flow conditions resulting in reduced force coefficients. Attached irregularities may disrupt this formation. The force coefficients specified in Table 2-9 assume a minimal level of attached irregularities.

For cylindrical appurtenances, irregularities may be ignored when their projected areas do not exceed 10% of the bare projected area of the appurtenance. When their projected areas exceed 10% but are not more than 20%, the irregularities may still be ignored but the tabulated force coefficients for the appurtenance must be linearly increased up to 30% but not greater than the tabulated subcritical force coefficients. For higher percentages, the attachments must be considered separately with appropriate force coefficients and subcritical force coefficients must be used for the appurtenance.

For flat appurtenances, when the projected areas of irregularities exceed 10% of the bare projected area of the appurtenance, the irregularities must be considered separately with appropriate force coefficients. For lower percentages, the irregularities may be ignored.

Subcritical flow is required to be considered for all ice loading conditions; however, the thickness of ice, as with Table 2-8a does not require to be considered when comparing the projected areas of irregularities to the supporting appurtenance.

Section 2.6.11.2 is intended to provide a standardized method for calculating effective projected areas for appurtenances mounted on structures for the purpose of determining the design wind forces to be applied to the supporting structure. Different methods are appropriate for determining the effective projected areas and design wind forces for appurtenances themselves (refer to Section 2.6.11) and for the structural design or analysis of mounting systems (refer to Section 16.0).

C2.6.11.2.1 Antenna Mounting Pipes

The portions of a mounting pipe above and below the shielded portion from an antenna are required to be included in the determination of the effective projected area for the windward antenna normal face. A force coefficient of 1.0 is specified considering the typical lengths exposed. The full length of the mounting pipe with the appropriate appurtenance force coefficient must be included in the determination of the effective projected area for the windward antenna side face as there is no shielding from the antenna for this wind direction.

C2.6.11.2.2 Effective Projected Area for Mounting Frames

The effective projected area equations for the windward normal face of a single mounting frame were adopted from the IASS reference in Annex U. Only subcritical force coefficients are provided for round members because the diameters of round members typically used for mounting frames would not result in Reynolds numbers high enough to justify transitional or supercritical flow conditions. The IASS reference provides a graphical representation of the effective projected area equations. The equations provided in the Standard were adopted from the Eurocode prEN 1993-3-1 Standard for Towers, Masts and Chimneys. The equations for latticed towers from Section 26.11.1.1 are used for determining the effective projected areas for the windward normal faces of square or triangular truss mounting frames.

A prescriptive approach is specified for determining the effective projected area for members supporting a mounting frame and for the windward side faces of the mounting frame. The equations are based on shielding provided by members of the mounting frame system and therefore a shielding factor equal to 1.0 is specified for a single mounting frame configuration.

When 3 or more mounting frames are mounted at the same relative elevation a shielding factor of 0.80 may be used for calculating the effective projected area of each mounting frame in accordance with the general equation for appurtenances specified in Section 2.6.11.2. The 0.80 shielding factor is intended to account for the shielding and wake interference effects from the group of mounting frames on the mounting frames themselves as well as on the supporting structure (refer to Section C2.6.11).

When the arrangement of 3 or more mounting frames is in an arrangement that results in direct shielding of the structure as illustrated in Figure 2-8, a shielding factor of 0.75 may be used as additional shielding and wake interference effects would be expected for the mounting frames as well as the supporting structure.

Because shielding effects are accounted for using the shielding factor applied to the effective projected area of the mounting frames, no shielding is allowed for the determination of the projected area of the supporting structure.

Mounting frames are treated in a similar fashion as a supporting structure with regard to antennas or equipment supported by the mounting frames. The shielding and wake interference effects from mounting pipes, antennas or other supported equipment are accounted for by using a shielding factor applied to the effective projected areas of the supported appurtenances.

C2.6.11.2.3 Effective Projected Area for Symmetric Frame/Truss Platforms

A prescriptive approach is presented for continuous mounting frame platforms due to their unique shielding and wake interference characteristics. Platforms that are not continuous are intended to be considered as mounting frames in Section 2.6.11.2.2.

The equations from Section 2.6.11.1 account for the geometry of a platform based on solidity ratios. The method for accounting for the projected areas of the members supporting a platform include provision for shielding and wake interference effects; therefore, a shielding factor equal to 1.0 is specified for all wind directions. Shielding and wake interference effects from appurtenances mounted on the platform are accounted for by using a shielding factor equal to 0.75 when determining the effective projected areas of the appurtenances mounted on the platform.

C2.6.11.2.4 Effective Projected Area for Low Profile Platforms

A prescriptive approach is presented for continuous low-profile platforms due to their unique shielding and wake interference characteristics. Platforms that are not continuous are intended to be considered as mounting frames in Section 2.6.11.2.2.

A square low-profile platform is considered to have a lesser shielding and wake interference effect compared to a triangular platform. Shielding and wake interference effects from appurtenances mounted on a low-profile platform are considered to be less compared to frame/truss platforms and are accounted for by using a shielding factor equal to 0.80 when determining the effective projected areas of the appurtenances mounted on the platform.

C2.6.11.2.5 Effective Projected Area for Circular Ring Platforms

A prescriptive approach is presented for circular ring platforms due to their unique shielding and wake interference characteristics. Factors are applied to the components of the platform according to their degree of shielding and wake interference effects; therefore, a shielding factor equal to 1.0 is specified for all wind directions.

Shielding and wake interference effects from appurtenances mounted on circular ring platforms are considered to be identical to low profile platforms and are accounted for by using a shielding factor equal to 0.80 when determining the effective projected areas of the appurtenances mounted on the platform.

Notes for Sections 2.6.11.2.2 through 2.6.11.2.5

The Standard does not allow multiple reductions in effective projected areas as explained in section C2.6.11.2; therefore, for all platforms, regardless of type, the shielding factor (K_a) for determining the effective projected area of appurtenances mounted on a platform must be set equal to 1.0 when transitional or subcritical force coefficients are used to determine the effective projected area of the mounted appurtenances.

The prescribed methods for determining the effective projected areas for platforms are considered conservative and do not require additional effective projected areas to be considered for grating or other working surfaces.

C2.6.11.3 Design Wind Force on Guys

The design wind force on guys is based on applying a force coefficient of 1.2 to a guy element. The design wind force is considered to act normal to the guy chord in a plane defined by the guy chord and a vector representing the wind direction (refer to Figure 2-12). The parallel design wind force is considered negligible. The design wind force is considered to be a function of the square of the wind velocity component normal to the guy chord and is the basis for the squared term in the equation for the design wind force. The length of guy element may be considered to be equal to the guy chord as the difference is considered insignificant based on the guy tensions associated with the structures covered by the Standard.

The velocity pressure and ice thickness may be determined based on the mid-height of a guy element. For ground supported applications where a guy anchorage is located at an elevation below the elevation of the mast base, the velocity pressure and design ice thickness must not be taken less than the values determined at the elevation of the mast base (i.e., the height above grade not considered less than zero).

C2.6.11.4 Shielding

The Standard allows for shielding of elements depending on their dimensions and the separation between elements except when a shielding factor (K_a) less than 1.0 is considered in the determination of effective projected areas. The shielding factors specified in the Standard are intended to represent the full effect of shielding and additional shielding considerations based on spacing between elements is not allowed. An exception exists for the shielding of mounting pipes specified in Section 2.6.11.2.1 where the effective projected area calculated in accordance with this section can be used with the shielding factors specified in other sections of the Standard.

The Standard requires the unshielded element to be considered as a flat element unless it is known that both elements involved are round. This is due to the fact that when one element is flat, there is a wind direction normal to the flat member that justifies the use of a flat element force coefficient. It is considered impractical to use different force coefficients for the different wind directions.

Shielding considerations may significantly vary with wind direction and when shielding is considered, the magnitude of shielding must be determined for each wind direction considered.

2.6.11.5 Round or Elliptical Transmission Lines Mounted in Clusters or Blocks

It is very common for transmission or feed lines to be mounted in clusters or blocks. The Standard provides a prescriptive means for determining effective projected areas for this type of mounting. Gaps between lines may not align with gaps in other layers and also a slight change in wind direction may result in no gaps between lines for a slightly different wind direction. For these reasons, a cluster or block of lines is required to be considered as an equivalent solid block exposed to the wind unless each individual line in each layer is considered exposed to the wind and considered independently from each other. In most cases, considering lines independently would result in higher effective projected areas compared to considering the group of lines as an equivalent block as specified in this section.

The solid block is intended to be considered as a single appurtenance for the purposes of calculating effective projected area based on the windward normal and side faces of the equivalent block. Shielding considerations may be based on the dimensions of the equivalent block. The weight of ice is to be based on the thickness of ice being applied to each line except that the total cross section of ice need not be considered greater than the cross section of ice illustrated in Figure 2-14.

The validity for the consideration of an equivalent solid block to represent a group of lines is limited by the spacing between the lines. For example, an equivalent block or cluster is not allowed to determine effective projected areas or the weight of ice when the spacing between lines within a single row of lines or between the layers of multiple rows of lines exceeds 3 times the larger line diameter in the cluster. The small degree of shielding that occurs when there is wide spacing between lines does not justify an equivalent block approach. This limitation is particularly important to recognize when determining the effective projected area for the windward side face of a single row of lines.

A group of lines may be considered as an equivalent block to determine effective projected areas for specific wind directions and as individual lines for other wind directions. As an example, a single row of lines may be considered as individual lines for a wind normal condition and as a group of lines for a wind direction parallel to the row of lines.

2.6.11.6 Velocity Pressure

The velocity pressure equation was adopted from ASCE 7.

C2.7 Seismic Load Effects

The strength requirements to withstand earthquakes are based on ASCE 7 and modified to incorporate special considerations for the structures covered by the Standard. Special steel detailing requirements are not required due to the low values of the response modification coefficients specified in Section 2.7.7.1.1. Provisions to ensure adequate ductility and post-elastic energy dissipation are specified in Sections 2.7.9 for anchorages and 9.0 for foundations.

C2.7.7.1 Equivalent Lateral Force Procedure

The equivalent lateral force procedure may be used for any structure. Traditional methods for determining fundamental periods for the structures covered by the Standard are presented which are required to determine the maximum total seismic shear force for a given structure type.

C2.7.7.2 Modal Analysis Procedure

The modal procedure is only applicable to structures that may be considered to have linear responses to seismic forces. Due to the modeling complexities and possible nonlinear responses, structures that are not supported by the ground do not qualify. Guyed masts are not considered to have linear responses and also do not qualify. The procedure assumes a symmetrical structure and therefore only applies to poles and triangular or square latticed structures with individual sections panels that have the same bracing pattern, bracing members and leg members in each face.

The requirement for the minimum effective modal gravity load was determined by consensus of the committee based on the analysis of actual structures identifying the limit where the consideration of additional higher modes would not contribute significantly to the total seismic response.

C2.7.8 Structures Supported on Buildings or Other Supporting Structures

Structures with a significant weight compared to the supporting structure require a more complex approach to determining seismic forces in accordance with ASCE 7 which includes modeling the supporting structure. Typical structures covered by the Standard have a minimal weight compared to supporting structures and qualify for a simplified approach using amplification factors applied to the results of an equivalent lateral force analysis for the structure as if it were ground mounted. Other rational methods are allowed as long as the calculated seismic load effects are not less than 85% of the seismic load effects determined from the equivalent lateral force procedure. Although the equivalent lateral force analysis method is considered conservative, results less than 85% are not considered credible.

C2.7.9 Anchorage Design Strengths

An overstrength factor is specified for anchorages (anchor rods and guy anchor shafts) to ensure anchorages do not fail as the structure deforms plastically during a seismic event. The plastic deformation is required due to the response modification coefficients specified in the

Section 2.7.7.1.1 which are based on the degree of energy absorption expected for a given structure type.

When the actual material specification used to fabricate a pole is known, anchor rod strength need not exceed the expected upper bound nominal bending strength of the pole. The upper bound nominal bending strength is determined using the minimum yield strength for the pole material specification times the values tabulated in Table 2-13. The tabulated values are intended to represent the ratio of the actual yield strength to the minimum specified yield strength for the pole material A pole must be capable of reaching its actual yield strength and deforming plastically in order to absorb the energy assumed for the seismic analysis of the pole. Providing additional anchorage capacity would not serve a purpose as the pole would be capable of absorbing the assumed energy without a premature failure of the anchorage.

C2.8 Serviceability Requirements

The structures covered by the Standard often have rotation and lateral displacement limits for the proper operation of equipment. Limit state deformations are specified based on a 60 mph basic wind speed. A direction probability factor equal to 0.85 is specified as the limiting deformations are most often required in a specific direction (e.g., microwave azimuths).

The 60 mph basic wind speed was selected as a minimum requirement which has served the communication industry well for most systems. More stringent requirements may be specified for unique site-specific applications.

C 3.0 ANALYSIS

C3.1 Scope

The unique characteristic of structures covered by the Standard require special analysis models and techniques.

C3.4 Analysis Models

Holes in structural shapes and openings in tubular poles may be ignored when creating structural models for the global analysis of structures. All models may be based on the gross cross-sectional properties of the structural elements.

C3.4.1 Self-Supporting Latticed Towers

Self-supporting latticed towers must be analyzed as three-dimensional structures. Members may be modeled as elastic straight truss members with pin connection regardless of the type of end connections utilized. This includes leg members that are continuous in the actual structure because the moments generated in fixed joints in three-dimensional latticed models are small and have an insignificant effect on the axial forces in the truss elements. It is acceptable; however, to model members that are continuous across a joint, (e.g., leg members) as beam elements which generate moments in the members. The combined stresses from axial load and moment in beam elements must be considered in accordance with Section 4.0.

The models outlined above each will capture P-delta effects and detect instability by non-convergence of the model when subjected to loading.

C3.4.2 Self-supporting Pole Structures

An elastic three-dimensional beam-column model is required for tubular pole structures. The model must be capable of capturing P-delta effects and detecting instability due to combined axial load and moment. This is most often accomplished by requiring a minimum number of elements within each section with a limit on the maximum length of an element. Because of the presence of lateral load, such a model will not converge if the stiffness of the structure is not adequate for stability under combined axial load and overturning moment. For this reason, effective length factors (K) are not required to determine limit state strengths under combined axial load and moment in Section 4.0. Structural elements that consider second order effects within each beam-column element also properly capture P-delta effects and detect instability by non-convergence.

Structural elements that consider second order effects within each beam element of the model also capture P-delta effects and detect instability by non-convergence of the structural model.

The strength requirements specified in Section 4.0 are intended for use with the stresses from a beam-column structural model. The methodology required for accurate structural models using plate or shell elements is beyond the scope of the Standard. A wide range of results may occur from one structural model to another. When used, the stresses shall not be considered less than the stresses from a beam-column model meeting the requirements of the Standard.

Slip joints and flange connections are commonly used for tubular pole structures. Slip joints may be modeled with element cross sectional properties that linearly vary between the properties of the sections above and below the joint. Flanged connections need not be included in the model when the design of the flange is based on rigid plate behavior, otherwise the additional flexibility of the plates must be considered to properly investigate stability and P-delta effects.

C3.4.3 Guyed Masts

Guyed masts may be modeled using three-dimensional beam column elements for the mast and guy elements for the guys. Unless the analysis model considers second-order effects within each beam-column element, a minimum of 5 elements is required in any span or cantilever. This is required in order to detect instability and properly capture P-delta effects. Effective length factors (K) are not required to determine limit state strengths under combined mast axial loads and moments in Section 4.0. Structural elements used for the mast that consider second order effects within each beam-column element capture P-delta effects and detect instability by non-convergence. Horizontal bracing members at guy elevations resisting the horizontal components of guy forces must be considered independently from the model as individual bracing members would not be included in the beam column model of the mast. Special modeling considerations are also required for beam-column mast models for candelabras and for extensions from the mast for guy connections to minimize twisting of the mast (often called star mounts or torque arms).

It is also acceptable to model latticed masts with the models specified for latticed self-supporting structures (refer to C3.4.1).

The base of a guyed mast must be properly modeled as the response of the structure will be dependent on the degree of fixity to overturning and torsional moments. A pinned base will allow the mast to rotate under lateral loading to minimize moments in the mast. Pinned bases may be detailed to prevent a torsional reaction or friction may be considered to resist torsion depending on the configuration of the base connection. Overestimating the frictional resistance of the base connection to resist torsion may result in improper mast and foundation strength requirements (e.g., the magnitude of torsional moment) or underestimate mast twisting rotation which may be significant when investigating twist limitations for microwave antennas.

The standard requires guys to be modeled as cable elements or as non-linear supports at guy elevations. The use of non-linear supports requires consideration of the eccentricity of the vertical components of the guys with the mast, a method to determine the wind loading on the guys as the guys move under wind loading and a method to account for axial deformations and catenary effects.

C3.4.4 Application of Wind Forces to Structural Models

Wind forces for three-dimensional truss or frame-truss models are intended to be distributed equally to the cross section at each leg panel point as opposed to applying a higher percentage of the total force to the windward side of the model or to apply uniform loads to individual members. This distribution is specified to eliminate the unnecessary complication of varying the applied forces for a given wind loading condition for each wind direction. The assumptions required for determining the total wind force do not justify the complication of distributing the total forces to each face of the structure depending on its orientation to the wind direction.

Local bending is only considered significant when supported appurtenances are supported in the middle half of a member. For the typical communication structure, many appurtenances are supported outside of the middle half of the member and do not require the consideration of bending. For bracing members, the critical wind direction for investigating bending is normal to the member where the axial load in the member would be minimal.

C3.5 Displacement Effects

Displacement effects for guyed masts have a pronounced effect on mast strength requirements due to the downward forces from the vertical components of guy forces.

Displacement effects are considered insignificant for self-supporting latticed structures with large base widths.

Due to the flexibility of pole structures, displacement effects are significant and are accounted for using the analysis models required in accordance with section 3.4.2.

C3.6 Global Stability Considerations

Geometric imperfections increase displacements and therefore increase P-Δ effects. Geometric imperfections are generally small for the structures covered by the Standard and therefore are only a consideration for strength investigations for loading conditions with a basic wind speed less than 30 mph.

C3.7 Wind Loading Patterns

The static wind escalation model used to determine wind loads is a simplistic model used with a gust effect factor to account for the effect of wind gust. The approach of using a gust effect factor is considered adequate for typical self-supporting latticed towers and pole structures.

Using a simple gust effect factor can significantly underestimate member forces for guyed masts and latticed self-supporting structures with a significant number of straight sections supported above tapered sections. For these structures, removing load from the structure can actually increase member forces. For a guyed mast, removing load from the cantilever can increase the load in the top guy span and each alternate guy span due to continuous beam effects. This occurs because the load in the cantilever introduces displacements in these spans in the opposite direction compared to the displacements from the wind forces applied in these spans. A similar effect occurs when removing a load in any guy span.

The same effect may occur for latticed self-supporting structures with a large number of straight sections supported by tapered sections (refer to Figure 3-1). When the apex point defined by the extension of the tapered legs occurs above the height of the structure, all lateral loads applied to the straight sections contribute to the bracing forces in the tapered sections. When the apex point occurs below the top of the structure, lateral loads applied above the apex point produce bracing forces in the opposite direction compared to the direction resulting from lateral loads applied below the apex point. For this reason, overestimating forces above the apex point by assuming the gust wind speed occurs over the entire height of the structure can result in inadequate bracing forces in the tapered sections. Similarly, if large lateral loads are present at the top of the structure above the apex point, higher bracing forces may occur in the tapered sections if the assumed gust does not occur in the lower portion of the structure.

For the above reasons, a simple gust effect factor is not adequate to determine strength requirements. Pattern loading is intended to account for these effects when gusts occur over only a portion of the structure as opposed to occurring over the entire height of the structure.

Pattern loading is only required for the extreme wind loading condition. The design wind pressure is assumed to equal the pressure due to a 3-second gust wind speed with a 0.85 gust effect factor. Wind pressures are reduced for pattern loading according to Sections 3.7.1 for latticed self-supporting towers and 3.7.2 for guyed masts. The reduced pattern loading is equal to the 3-second gust wind pressure multiplied by a mean wind conversion factor (m) specified in Table 3-1.

The mean wind conversion factors were determined by comparing the ratio of the escalation of an hourly wind speed to the escalation of a 3-second gust wind speed for each exposure category. The escalation of the wind speeds was determined from the equations presented in ASCE 7. Pattern loading was considered by the committee as an empirical approach to address a complex issue. A simplified approach to pattern loading was selected by the committee. The analysis of guyed masts using the simplified pattern loading criteria were compared to other international standards which indicated the simplified method yielded reasonable results considering the additional response criteria specified in Section 3.8.

C3.7.1 Latticed Self-Supporting Towers

Self-supporting latticed towers with multiple changes in leg slope as represented in Figure 3-2 require the consideration of multiple pattern loadings as the location of the apex point for each tapered section defines the location where the consideration of non-uniform gusts may impact the bracing in each tapered section

C3.7.2 Guyed Masts

The effects of pattern loading (i.e., local gusts) are only considered significant for masts with 3 or more spans and with at least one span greater than 80 ft. in the top one-third height of the structure.

Pattern loading is not applied to the guys supporting the mast as the guys are considered to be a significant distance from the mast and not subjected to the same gusts impacting the mast itself. In addition, the effect of pattern loading on guys is considered minimal compared to the mast and does not justify the complications in analysis for modeling pattern loading on guys.

The pattern loading is required to be considered for the top three spans or additional spans as required to represent at least one-third the height of the mast (refer to Figure 3-3). This is because the effects of pattern loading on the lower spans is considered insignificant. For tall masts over 450 ft in height, the gust wind speed is not required to be considered over the entire height of the mast when pattern loading is considered. This is because the pattern loading for taller masts are considered a better representation of wind loading compared to the conservative assumption of the gust wind speed occurring over the entire height of the mast.

When guy elevations occur at a close distance to each other (e.g., when double guying is provided for broken guy loading considerations), the midpoint between the guy elevations are used to define an equivalent guy elevation for the purposes of determining the extent of pattern loading.

C3.8 Mast Shear and Torsion Responses for Guyed Masts

The pattern loading criteria specified in Section 3.7 does not fully account for the variation in shear and torsion that can occur within a guy span. For a typical span, the shear and torsion vary between positive and negative values (i.e., opposite directions) resulting in a point of zero shear or torsion. Optimizing bracing designs to match the shear and torsion from an analysis would not be justifiable because the location of the point of zero shear or torsion would be expected to vary with different gust patterns. The minimum values of shear and torsion are intended to prevent designs with under designed bracing without having to run a multitude of different pattern loadings to capture adequate strength requirements for all the diagonals in a guy span.

C 4.0 DESIGN STRENGTH OF STRUCTURAL STEEL

C4.1 Scope

Design strengths for the most commonly used structural shapes used for the structures covered by the Standard are addressed in Section 4.0. The design strengths are based on the AISC 360, ASCE 10, ASCE 48 and AASHTO LRFDLTS references listed in Annex U and modified as required for applications specific to latticed towers, poles and guyed masts for this Standard

C4.4.1 Minimum Bracing Resistance

The lateral resistance at a node or panel point (P_s), required to consider a reduction in the unbraced length of a member is specified in this section. The lateral resistance is based on the axial design compressive force in the supported member and the effective slenderness ratio of the supported member in the plane of buckling under consideration. Bracing or secondary members must provide the required minimum lateral resistance to prevent buckling in the plane of buckling on either side the longitudinal axis of the supported member, (e.g., bracing or secondary members would be subjected to both tension and compression). The minimum resistance to provide lateral support to a member is not required to be combined with the required strength from any loading combination.

The equation for the minimum resistance was obtained by comparing the requirements of other international standards for latticed structures to the past performance of tower installations designed to previous revisions of the TIA-222 Standard. As a supported member becomes more flexible (i.e., KL/r increases above 60), the required resistance increases from 1.5% to a maximum of 2.5% of the axial design compressive force in the supported member.

When either the axial design compressive force or slenderness ratio for a supported member varies on each side of a node or panel point considered to reduce the unbraced length of the supported member, P_s must be determined for each side and the maximum value must be used for determining the required minimum resistance at the node or panel point.

The bracing or secondary members at a node or panel point must provide the minimum resistance normal to the supported member in the direction of buckling under consideration. When a member providing lateral support to a member is not normal to the supported member and/or does not lie within the plane of buckling under consideration, only the component of the resistance in the direction of buckling is considered to contribute to P_s. Per the note for Table 4-2, the required bracing resistance in a face may be considered to be in a horizontal plane when supporting legs with slopes 15 degrees or less.

The minimum resistance for bracing and secondary members must be based on all supported members. For example, a secondary member may be used to reduce the unbraced length of a diagonal and a leg. The minimum resistance would be equal to the larger value calculated considering the axial design compressive force and effective slenderness ratio of each supported member as well the orientation of the secondary member to each supported member.

Leg members have two planes of resistance, one for each face. Table 4-3 illustrates the buckling directions to consider for determining the maximum effective slenderness ratio to be used for determining the compression strength of a leg member and the bracing resistance required to provide lateral support to the leg (refer to Table 4-3, Note 2).

Table 4-1 illustrates the minimum required bracing resistance in each face for angle and round cross section legs. Table 4-2 illustrates the strength requirements of individual bracing and secondary members in a face at a panel point in order to provide the required bracing resistance.

For square or triangular tower cross sections with angle legs and symmetrical bracing patterns (e.g., cross bracing with lateral support in each face at a panel point), weak-axis buckling always governs P_s. The resistance in each face contributes to the resistance required to provide lateral support for the leg. The component of resistance in each face in the direction of buckling is equal to cosine(45°) and cosine(30°) for square and triangular tower cross sections respectively. The bracing resistance required in each face (P_r) would therefore equal 0.707 P_s and 0.577 P_s respectively for square and triangular tower cross sections respectively as illustrated in Table 4-1.

For square tower cross sections with staggered bracing patterns, the bracing resistance required when in-plane or out-of-plane buckling governs, is simply equal to P_s. Because the resistances are identical, the bracing resistance required for out-of-plane buckling is not included in Table 4-1 for square tower cross sections.

For triangular tower cross sections with staggered bracing patterns, the bracing resistance required when out-of-plane buckling governs is greater than P_s because the adjacent face is not normal to the direction of buckling and only a component of the bracing resistance is effective in providing lateral support. The component of the adjacent face bracing resistance in the direction of buckling is equal to P_r times cosine(30°) and the required brace resistance for the face is therefore equal to P_s/0.866 or 1.15 P_s. Because the required bracing resistance is always greater for the out-of-plane buckling direction compared to the in-plane buckling direction (i.e., 1.0 P_s), the bracing resistance required for the in-plane buckling direction is not included in Table 4-1 for triangular tower cross sections.

Weak-axis buckling of angle legs with staggered bracing are a special case due to staggered bracing subjecting an angle leg to twisting as leg buckling occurs about the weak axis. In-plane, out-of-plane or weak-axis buckling may govern the maximum effective slenderness ratio for legs with staggered bracing. The effective slenderness ratio for weak-axis buckling of an angle leg used with a staggered bracing pattern in Table 4-3 is increased to result in an equivalent effective slenderness ratio intended to be used to determine the required bracing resistance in accordance with Table 4-1 when weak-axis buckling governs. As the value of N in the equivalent effective slenderness ratio equation approaches 1.0, the equivalent effective slenderness ratio approaches the weak-axis effective slenderness ratio of a symmetrical bracing pattern. The equation for the equivalent slenderness ratio for the weak axis was first introduced in Rev F of the Standard and by consensus, based on the performance of installed towers has been adopted for Rev H of the Standard.

The equation for P_s assumes that the design axial compression force in the supported member is limited by the effective slenderness ratio of the supported member for the direction of buckling under consideration. For other buckling directions with lower effective slenderness ratios, the supported member is not considered to be on the verge of buckling. For example, when weak-axis buckling governs the effective slenderness ratio for an angle leg in a square tower cross section, the required bracing resistance in a leg equal to 0.707 P_s from Table 4-1 is considered adequate for providing lateral support for the in-plane buckling direction. The leg would be expected to buckle in the weak-axis direction if additional loading were applied to the leg. Providing a higher bracing resistance (e.g., $P_r = P_s$ for a 90 degree angle leg when in-plane buckling governs) in each face would not prevent buckling about the weak axis. Engineering judgement is required when the effective slenderness ratio for either the in-plane or out-of-plane buckling direction approaches the effective slenderness ratio for the weak-axis buckling direction for angle legs with staggered bracing patterns. The required bracing resistance in a face can be governed by the in-plane or out-of-plane buckling directions despite having a lower effective slenderness ratio and value of P_s compared to the weak-axis direction because only one face provides resistance to buckling. For simplicity, it is conservative to alternately determine the required bracing resistance for any leg or bracing pattern using the worst-case effective slenderness ratio to determine P_s and multiplying the result by 1.15 for triangular tower cross sections and 1.00 for square tower cross sections (refer to Table 4-1, Note 1).

The required bracing resistance for members supporting a diagonal must provide the full lateral support in the direction having the maximum effective slenderness ratio, unlike legs where there are two planes providing lateral support. For example, the resistance of secondary members in the face of a tower supporting a single 90 degree angle diagonal would not be fully effective for resisting weak-axis buckling of the diagonal when weak-axis buckling governs. The resistance of the secondary members normal to the diagonal would need to equal or exceed P_s/cosine(45°) in order to provide the required resistance in the weak-axis direction of buckling for the diagonal.

As illustrated in Table 4-2, secondary members in latticed towers considered to reduce unbraced lengths, must be connected to a joint that is part of a triangulated bracing pattern. The required resistance, however, is not required to be distributed beyond the members indicated in Table 4-2 nor is more than one panel point location for the application of P_r required to be considered to occur at a time.

Table 4-2 illustrates the strength requirements of individual bracing and secondary members in a face at a panel point for commonly used bracing patterns. When a secondary diagonal member is connected to a horizontal secondary member, the diagonal member force is equal to one-half of P_r divided by the cosine of the angle between the members. These criteria are based on the assumption that one-half of P_r is distributed as a shear force on each side of P_r to the reactions for the triangulated bracing pattern providing support to the horizontal member. It was the consensus of the committee that the complexity of a more rigorous approach was not justified given the assumptions involved determining minimum resistances to prevent buckling; however, a more rigorous approach is allowed by the Standard.

C4.4.2 Slenderness Ratios

Preferred maximum slenderness ratios are presented based on experience with the installation and performance of latticed towers. Long slender members are difficult to install and can result in vibration issues after installation. The values are not absolute values where issues are known to certainly occur when exceeded.

C4.4.3 Design Values

The Standard requires the minimum nominal values of yield and ultimate tensile strength to be used for the determination of design strengths. It is not considered appropriate to use values from a material mill certification that may indicate higher values.

Electric-resistance welded (ERW) HSS and pipe shapes are manufactured from plate or coil material where modern manufacturing techniques have resulted in nearly all material thicknesses being produced to the minimum thicknesses based on the tolerance allowed by the corresponding material specification. For this reason, the AISC Manual of Steel Construction publishes design wall thickness for HSS and pipe shapes equal to 93% of the nominal wall thickness. The reduction is not required for shapes conforming to ASTM A1085 because of the more stringent wall thickness tolerance for this material specification.

The consensus of the committee, based on the performance of structures covered by the Standard and the HSS and pipe sizes commonly utilized, was not to significantly reduce member strengths for these shapes from Revision G of the Standard. Only ERW shapes with specified minimum yield strengths above 52 ksi required a reduced wall thickness to be used for design. The justification for not reducing the design wall thickness for shapes with lower specified minimum yield strengths, in addition to the performance of these shapes in existing structures, was that materials with relatively low specified minimum yield strengths typically are produced to a higher yield strength, which compensates for the shapes being produced to their minimum specified wall thickness. The 52 ksi yield strength was selected because of the performance of structures covered by the Standard commonly using HSS shapes conforming to API 5L Grade X52 with a 52 ksi minimum specified yield strength.

Because the AISC Steel Construction Manual tabulates design wall thicknesses equal to 93% of nominal wall thicknesses for HSS and pipe shapes, in order to avoid confusion in the Industry, the design wall thickness in the Standard is also specified to be taken as 93% of the nominal wall thickness (i.e., the AISC tabulated value) except for HSS sections conforming to ASTM 1085. In order not to significantly reduce member strengths from Revision G of the Standard, the resistance factors in Section 4.0 for ERW HSS and pipe shapes with specified minimum yield strengths not greater than 52 ksi are increased from the resistance factors specified in Revision G of the Standard by dividing by 0.93 which equates to 0.97 and 0.81 for the Revision G resistance factors of 0.90 and 0.75 respectively. The resistance factors for ASTM 1085 HSS shapes and ERW HSS and pipe shapes with F_y > 52 ksi are unchanged from Revision G of the Standard.

The intent of adjusting resistance factors is to enable the use of AISC tabulated design wall thicknesses for all sections, excluding ASTM 1085 sections, without significantly reducing or increasing member strengths compared to Revision G of the Standard. When a reduced nominal wall thickness is used as the design wall thickness, resistance factors are increased from Revision G of the Standard except for shapes with specified minimum yield strengths greater than 52 ksi. When a nominal wall thickness is used as the design wall thickness (i.e., ASTM 1085 HSS shapes), or when a reduced nominal wall thickness is used as the design wall thickness for ERW HSS or pipe shapes with F_y > 52 ksi, resistance factors are not increased from Revision G of the Standard.

C4.4.4 Normal Framing Eccentricities

Connections of members in latticed structures typically involve eccentricities due to the types of members involved, detailing limitations or other circumstances. The Standard allows for eccentricities within the limitations specified for leg and bracing members. The provisions for accounting for eccentricities were established based on the performance of existing structures covered by the Standard, results of full-scale tests and various international structural steel standards.

C4.4.4.1 Leg Members

When normal eccentricities are exceeded for leg members, Section 4.8.1.1 provides an interaction equation for investigating combined axial forces and moments based on the eccentricity that exceeds normal eccentricities.

C 4.4.4.2 Bracing Members

The effective slenderness ratio formulas specified in Section 4.5.2 account for normal framing eccentricities for bracing members. A reduction factor is specified to be applied to angle bracing members with leg sizes less than 3 inches when normal framing eccentricities are exceeded. The formulas specified in Section 4.5.2 are considered adequate to account for eccentric end conditions for angle sizes with legs greater than 3 inches due to their greater resistance to bending moment.

C4.4.5 Member Continuity

When the analysis model for a structure includes continuous members or fixed connections, both axial, shear and moments are required to be considered as combined effects when determining conformance to the Standard (refer to Section 4.8).

C4.5.1 Leg Members

Table 4-3 illustrates effective slenderness ratios for leg members for determining axial compression strengths in accordance with Section 4.5.4 and the required bracing resistance in accordance with Section 4.4.1.

For staggered bracing patterns with angle legs, the effective slenderness ratios to consider depends on the leg cross section. For 90 degree angles, the governing effective slenderness ratio could be governed by buckling about the X, Y or Z axis. For 60 degree angles, only buckling about the X or Z axis is considered. The Y axis for a 60 degree angle is not included in Table 4-3 as buckling about the Y-axis would not govern over buckling about the Z-axis due to the larger value of ry compared to rz. Refer to the commentary for Section 4.4.1 for additional commentary for Table 4-3.

Regardless of whether leg members are modeled as truss elements or continuous beam elements for analysis, the effective length factor considered is required to be equal to 1.0. This is required because leg members can buckle in an S shape changing the direction of buckling at each panel point which results in an effective buckling length equal to the panel spacing. Because of the buckling mode shape of a leg, the length L is required to be considered to be the distance between panel points as opposed to the distance between the ends clips or gussets attached to the leg for the connection of bracing or secondary members.

C4.5.2 Bracing Members

The effective length considerations for bracing members are more complex compared to those for leg members. The criteria for bracing members specified are dependent on the bracing pattern, the forces in the bracing, the strength of connected secondary members, member end restraints and framing eccentricities. For slender members, the degree of end restraint becomes a critical factor whereas for less flexible members, the magnitude of framing eccentricities becomes a critical factor.

Table 4-4 illustrates the effective slenderness ratios for bracing member except for round bracing members welded directly to legs which are illustrated in Table 4-5.

The formulas in Table 4-4 were adopted from the ASCE 10 Standard referenced in Annex U. Bracing members with slenderness ratios (L/r) less than 120 are considered as less flexible members where framing eccentricities are a critical factor. Bracing members with greater slenderness rations are considered as slender members where the degree of end restraint is the critical factor. The conditions defining the degree of framing eccentricity and end restraint are defined in the table. The conditions at each end of a member can be different. As one example, for a double bracing system with single angle bracing with slenderness ratios above 120, a brace may have multiple bolts at the connection to a leg and only a single bolt at the crossover point. For this condition, Formula 5 in the table would apply to the determination of the effective slenderness ratio. For the same bracing pattern with braces having a slenderness ratio less than 120, a single angle connected on one side to an angle leg or gusset plate is considered as eccentric, but the crossover point is considered to be concentric. For this condition, Formula 2 in the table would apply to the determination of the effective slenderness ratio.

A multiple bolt or welded connection of a brace connected only to a gusset plate is only considered to provide in-plane restraint to a bracing member. The gusset plate is not considered to have adequate strength about its weak axis to provide adequate out-of-plane restraint against rotation unless the bracing member is also attached directly to the leg (e.g., an angle leg). Restraint can be considered for in-plane and weak axis buckling. For this reasons and others, different buckling directions can require different formulas for determining effective slenderness ratios. Each critical direction of buckling must be considered to determine the maximum effective slenderness ratio to use for design.

The formulas in Table 4-5 were derived from input from manufactures and consultants with experience with bracing welded directly to tower legs. The value of K for slenderness ratios less than 80 reflect the degree of eccentricity of each joint type. The values for slenderness ratios greater than 120 reflect the degree of end restraint of each joint type. The equations for intermediate slenderness ratios represent a linear transition between the slenderness ratios of 80 and 120.

The value of L in Table 4-5 is based on the panel spacing and the clear distance between the legs. For double lacing patterns, the value of L is defined as the distance from the face of the leg to the crossover intersection point. This definition of L assumes support at the crossover point which requires both bracing members to be continuous across the intersection point, the braces to be connected at the intersection point and that at least one brace be subjected to tension. When these parameters are not met, the intersection point is not considered to provide out-of-plane buckling resistance and the bracing members are to be considered as single bracing members.

Unbraced lengths for other bracing patterns are illustrated in Tables 4-6 and 4-7. For bolted connections, the unbraced length is defined as the distance between the centroids of connection patterns. The unbraced lengths depend on whether resistance to out-of-plane buckling is provided at the crossover point for X bracing patterns and at the apex of K-bracing patterns as defined in Sections 4.5.2.1. through 4.5.2.4.

C4.5.2.1 Cross Bracing

The diagonals must be connected at the crossover point to be considered as providing resistance to out-of-plane buckling. When connected, resistance to out-of-plane buckling (i.e., lateral support) may be assumed for determining unbraced lengths when the requirements specified in this section are satisfied.

Lateral support is allowed to be considered at the crossover point when at least one member is continuous and at least one diagonal is subjected to tension. A continuous member is assumed to provide support when the bracing pattern starts to buckle in the out of plane direction. The continuous member is assumed to provide support as long as one member is in tension. The member under tension does not need to be continuous and provides support to the continuous member by undergoing increasing tension as the bracing system first initiates buckling.

Triangulated plan bracing is considered to provide lateral at an intersection point. The plan bracing must provide the required bracing resistance specified in Section 4.4.1. Plan bracing that is not triangulated is not considered to provide lateral support (refer to Figure 4-2).

When a continuous horizontal is provided at a crossover point, lateral support is considered to be provided when the horizontal has adequate strength to provide lateral support to the leg. Otherwise, the horizontal may buckle prematurely and fail to provide lateral support to the bracing members. The strength of the horizontal must be determined using a length ignoring out-of-plane buckling resistance of the bracing at the crossover point. This requirement is needed for a condition when the bracing members and both legs are subjected to compression and lateral support for the horizontal would not exist at the crossover point.

Cross bracing patterns can become unstable when there are no continuous members at the crossover point. For this condition, the crossover point must provide support resisting out-of-plane buckling using either triangulate horizontal plan bracing or a continuous horizontal.

C4.5.2.2 K-Type or Portal Bracing

For stability considerations, when the horizontal brace is not continuous across the apex point, triangulated horizontal plan bracing is required at the apex point with sufficient strength to provide lateral support to the horizontal brace.

When the horizontal brace is continuous across the apex point, horizontal plan bracing is not required. For stability considerations, however, the unbraced length of the horizontal is required to be equal to 75% of the full length of the horizontal (i.e., 75% of the full face width). This is because the apex point is not considered to be fully effective in providing out-of-plane buckling resistance to the horizontal at the apex point. In addition, the horizontal must have sufficient strength to provide lateral support to the legs based on the full length of the horizontal (i.e., 100% of the full face width). This is required for the stability of the legs when there is minimal axial force in the diagonals.

C4.5.2.3 Cranked K-Type or Portal Bracing

For stability considerations, triangulated hip bracing (refer to Figure 4-2) is required at the main diagonal bend locations (refer to Figure 4-1). The strength of the hip bracing must be capable of providing support to the diagonals. The effective slenderness ratios for diagonals with staggered hip bracing patterns are to be considered in the same manner as staggered bracing members supporting leg members illustrated in Table 4-3. Because hip bracing consists of a triangular cross section, the resistance of the hip bracing to support the diagonals is required to be determined in the same manner as the out-of-plane bracing resistance required for triangular cross section towers in Table 4-1 (i.e., $P_r = 1.15\ P_s$).

C4.5.2.4 Tension-Only Bracing

Tension-only diagonals must be capable of providing support to secondary members providing lateral support of leg members. The bracing resistance required in a face (P_r) is considered to be provided by 2 diagonals in tension at the crossover point. This may be a governing loading condition especially for the lower sections of a guyed mast where the leg load can be relatively large and diagonal forces relatively low. For stability considerations, when no member (i.e., diagonals or the horizontal member) is continuous over the crossover point, the strength of the horizontal is required to be based on the full face width. This is because the legs may be under significant axial compression and the tension forces in the diagonals could be insignificant and not capable of providing lateral support to the horizontal member.

4.5.3 Built-Up Members

The spacing of intermediate connectors for built-up members is required to result in a maximum slenderness ratio of the individual members forming the built-up member to not exceed the governing effective slenderness ratio of the built-up member. This is to prevent the individual members from buckling prior to the built-up member. A minimum of two intermediate fasteners across the width of angles members with legs greater than 5 inches are required to minimum twisting of the individual angle members.

An additional requirement for the spacing of intermediate connectors is specified for built-up members designed to resist buckling as a composite section (i.e., where intermediate connections are provided to resist shear forces between members required for two members to act compositely under flexural buckling). In this case, the spacing of intermediate connectors must result in a maximum slenderness ratio of the individual members forming the built-up member to not exceed 75% of the effective slenderness ratio of the built-up member for the buckling direction where the individual members are assumed to act compositely.

The more stringent intermediate connector spacing requirement is necessary to ensure composite action. The modified effective slenderness ratio for the direction of buckling where composite action is assumed is a function of the magnitude of relative deformation anticipated between the individual members. The more deformation that occurs, the less the individual members act compositely. In the worst case, the members would simply act as individual members which would require the use of the radius of gyration of the individual members vs. a significantly higher radius of gyration if the members were to act compositely.

The use of snug tight bolts is expected to result in higher relative deformations between the individual members compared to when fully tensioned high-strength bolts are used and therefore the modified effective slenderness ratio equations result in higher ratios when snug tight connectors are used compared to when fully tensioned high-strength bolts are used. When the criteria for the use of either equation is not met, the members are required to be considered as individual members without composite action.

The equations are based on the AISC Specification. An exception to the AISC requirements was adopted regarding the end connection for built-up members utilizing snug-tight bolted connections. The AISC requirement for end connections to be fully tensioned high-strength bolts in order to minimize slip at the end connections was not adopted by the committee. For the double angle members commonly used in the structures covered by the Standard, the performance of double angles acting compositely with snug-tight bolts was considered by the committee to be adequate to resist slip when a minimum of two intermediate fasteners were used over the length of the member.

The modified effective slenderness ratios determined by the equations specified need not be less than the effective slenderness ratio of the individual components for the direction of buckling (i.e., equivalent to ignoring any increase in strength due to composite action).

Criteria for built up members connected by lacing are specified. The criteria are the same as for bracing in a latticed tower section. When the lacing is not triangulated, the built-up member must be treated as a Vierendeel truss accounting for the combined bending and axial force for design.

4.5.4 Design Compression Strength

The criteria for determining design compression strengths were modeled after the AISC Specification and the ASCE 10 and ASCE 48 Standards.

The effective slenderness equations specified in Tables 4-4 and 4-5 account for the effects of end eccentricities and end restraint for bracing members (refer to commentary for Section 4.5.2).

The effective yield stress equations specified in Section 4.5.4.1 in combination with the effective slenderness equations from Table 4-4 account for torsional-flexural buckling of individual angle members. For this reason, the torsional-flexural buckling strength of single angle members are considered equal to their flexural buckling strength (refer to Section 4.5.4.3). The criteria specified was adopted from the ASCE 10 Standard. Additional torsional-flexural buckling criteria in Section 4.5.3 for double-angle members was adopted from the ASIC Specification

C4.5.4.1 Effective Yield Stress

The Standard uses effective yield stresses to account for local buckling and also for single angle members, flexural-torsional buckling. The effective yield stress is used for the strength equations presented in Section 4.0.

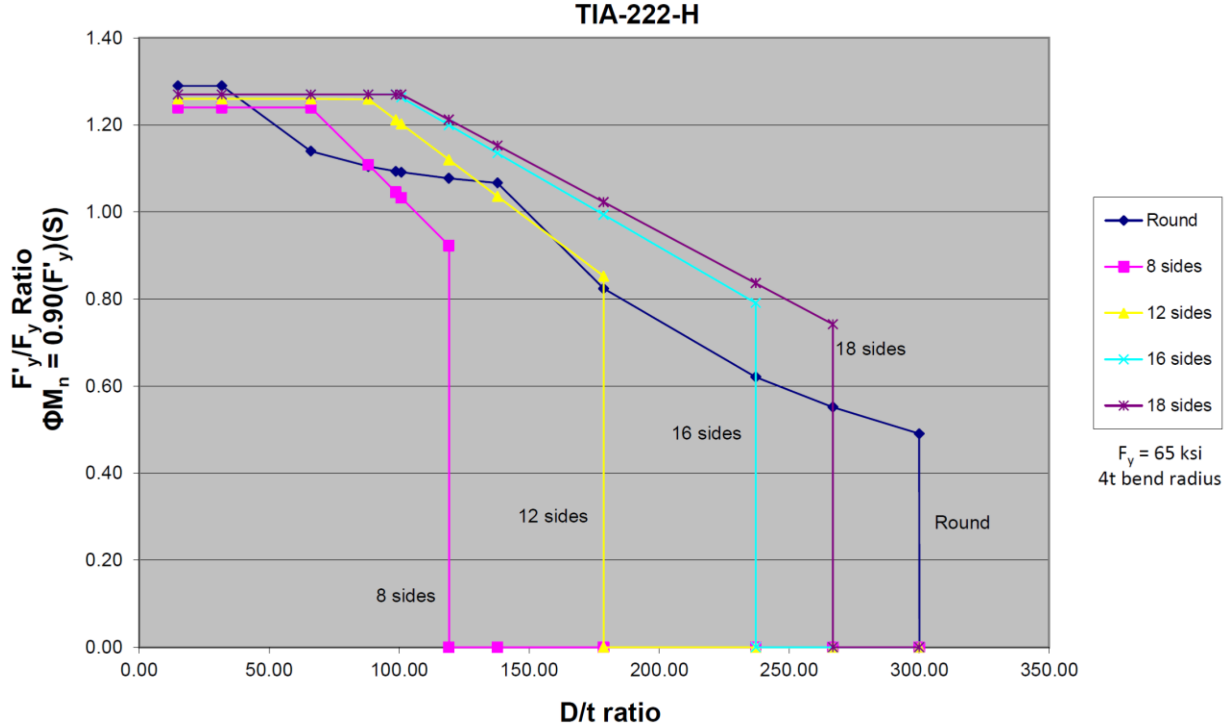
The effective yield stress equations for angle members are based on the local buckling criteria from the ASCE 10 Standard in order to be compatible with the effective slenderness ratio formulas from ASCE 10 specified in Table 4-4. The width-to-thickness ratio from ASCE 10 is based on the flat width of an angle member (refer to Figure 4-3) as opposed to the full width used for AISC local buckling criteria.

Round solid members are not subjected to local buckling or flexural-torsional buckling and the effective yield stress is equal to the specified minimum yield stress of the material.

The effective width equations for round members were adopted from the AISC Specification. The equation for F'_y when the D/t ratio exceeds $0.448E/F_y$ was derived from the AISC equations. Tubular round members are commonly used for pole structures where sway or lateral deflection limitations or minimum diameters for installing lines result in large diameter sections. In many of these cases, standard material is used which have a higher yield stress than required for strength. In these cases, the AISC limiting value of D/t based on F_y of the standard material would unjustifiably not allow the use of the material. In these cases, the consensus of the committee was to allow the use a lower F_y value that would satisfy the D/t limitation when the lower F_y value would satisfy strength requirements and the D/t ratio did not exceed 300.

The maximum D/t ratio of 300 was determined by consensus to be a reasonable limitation based on manufacturing, handling and installation and was in close agreement to the maximum D/t ratios for polygonal members based on the maximum w/t ratios specified in Table 4-8 (refer to Figure C4-1).

Figure C4-1
Effective Yield Stresses
For Flexure
Tubular Pole Members
TIA-222-H



The equation for F'_y when the D/t limit of 0.448 E/F_y is exceeded was obtained by setting the equation 0.448 E/F_y equal to the D/t for a member and solving for F_y which results in a value of F_y equal to 0.448 E/(D/t). This value of F_y satisfies the AISC maximum D/t ratio limitation. Substituting this equation for F_y into the AISC equation for F'_y results in the TIA equation for F'_y when the AISC maximum D/t ratio is exceeded using the higher actual specified minimum yield strength of the material. The equation is simply the AISC F'_y equation using the maximum F_y that for a given D/t ratio, satisfies the AISC maximum D/t ratio. This approach allows the use of higher yield strength materials up to a D/t ratio of 300 when a lower strength material satisfies the AISC maximum D/t ratio and all strength requirements.

Using the above approach, the highest yield strength that would satisfy the AISC D/t limitation and be used to determine strengths can be determined from the equation 0.448 E/(D/t). For a member with the upper bound D/t ratio limit of 300, the highest yield strength would be 43 ksi with an effective yield stress F_y' equal to 33 ksi. For example, when a member with a D/t ratio of 300 is used, and the specified minimum yield strength of the material is 50 ksi, only a yield strength of 43 ksi is allowed to be used for determining the effective yield stress and member strengths. The Standard does not require the calculation of the maximum yield stress and instead directly provides the value of F'_y to be used for determining strengths in accordance with Section 4.0.

Providing an equation for the elastic range of buckling that is independent of F_y and only a function of E and D/t is consistent with the determination of F'_y for other shapes in the elastic buckling range.

The effective yield stresses for polygonal tubular members are presented in Table 4-8. The effective yield stresses were obtained from the ASCE 48 Standard except for 18-sided shapes which were extrapolated from the ASCE 48 equations. Shapes with more than 18 sides are to be considered as round shapes with a diameter equal to the distance across flats of the polygonal shape The limiting maximum w/t ratio was adopted from the AASHTO Specification which also compares well with the test data provided in the commentary for ASCE 48. The variable E was incorporated into the ASCE 48 equations to make the equations dimensionless.

The effective yield stress for polygonal tubular members is used for both axial compression and flexural strength determinations. The exception for using the equations presented in the commentary for ASCE 48 for 8 or fewer sides when the axial stress exceeds 1 ksi was not adopted by the committee considering axial stresses are generally very low for the tubular pole members covered by the Standard.

The determination of the w/t ratio is a critical step in determining strength of polygonal tubular pole members. In order to determine the w/t ratio, the inside bend radius of a pole member must be determined. Table 4-8 specifies that when the bend radius is unknown, it is required to use an inside bend radius equal to 1.5 times the member thickness. This bend radius was established by consensus of the committee to be a practical minimum bend radius to result in a conservative estimate of flat widths and the w/t ratio. In accordance with the ASCE 48 Standard, in no case is an inside bend radius allowed to be taken greater then 4t because the effective flat width for the investigation of local buckling extends into the corner radius.

The ASCE 48 equations were established based on full-scale tests using full-scale manufactured poles which by consensus of the committee justified the use of a 1.0 resistance factor for determining strengths as adopted by the ASCE 48 Standard. Because a 0.9 resistance factor is used in Section 4.7 and 4.8.2 for design compression and flexural strengths, the ASCE 48 equations were divided by 0.9 to result in an effective resistance factor of 1.0 for determining design strengths.

In accordance with Section 3.4.2, pole structures are required to be analyzed using three-dimensional beam elements; therefore, flexural buckling is not required to be considered in determining design compressive stresses from axial loads. The effective yield stresses in Table 4-8 are used for determining both nominal axial compressive strengths and nominal flexural strengths in accordance with Section 4.8.2. The effective yield stress for determining the nominal compressive strength per Section 4.8.2 is limited to F_y as the effective yield stresses from Table 4-8 increase above F_y to account for the flexural strength for compact sections (refer to explained below).

Nominal flexural strengths from Section 4.7.3 are based on an effective yield stress applied to the elastic section modulus for polygonal tubular members. The effective yield stresses in Table 4-8 for compact sections are therefore equal to Z/S (F_y). The value of Z/S is dependent on the member shape and is equal to 1.27 for 18 and 16-sided shapes, 1.26 for 12-sided shapes and 1.24 for 8-sided shapes.

The ASCE 48 Standard equations for the effective yield stress, limits the effective yield stress equations at the w/t ratio resulting in the yield strength of the member as the occurrence of first yield is considered the limit state for transmission pole structures due to the unique loading conditions applicable to transmission line pole structures. The test data used to establish the ASCE 48 equations indicate that the effective stress increases for compact sections having lower w/t ratios. This would be expected to occur until the effective yield stress increased to a value equal to the yield stress times Z/S. Therefore, the ASCE 48 effective yield stress equations were extended in the lower w/t range to the w/t ratio where the effective yield stress was equal to F_y (Z/S). The w/t limits in Table 4-8 for compact sections represent the w/t ratios where the TIA equations for F'_y equal F_y(Z/S) as opposed to the lower ASCE 48 w/t limits which represent the w/t ratios where the ASCE 48 equations for F'_y equal F_y.

Refer to Figure C4-1 for an illustration of effective yield stresses for round and polygonal tubular pole members.

C4.5.4.2 Flexural Buckling Compression Strength

The flexural buckling criteria for angle and U-shaped members was adopted from the ASCE 10 Standard. The ASCE 10 criteria was based on full-scale manufactured latticed tower tests which justifies the use of a 1.0 resistance factor as adopted by the ASCE 10 Standard.

The equations specified are identical to the design compressive stress equations specified in the ASCE 10 Standard but are presented in the AISC LRFD format for flexural buckling by substituting the ASCE 10 equation for C_c into the ASCE 10 equation and then incorporating the ASIC equation for F_e.

The use of the ASCE 10 design compressive stress equations is necessary as they are correlated with the effective length formulas in Table 4-4. The effective length formulas in Table 4-4 account for the effects of eccentricity and flexural-torsional buckling in the lower slenderness range and the effects of end restraint in the higher slenderness range for angle members.

The AISC LRFD nominal flexural buckling equations account for factors effecting buckling in the lower and higher slenderness ranges that when used with a 1.67 constant factor of safety, closely match the allowable stresses from previous ASD versions of AISC which used a variable factor of safety to account for factors effecting buckling. Using the AISC LRFD nominal flexural buckling equations with the ASCE 10 effective slenderness ratio formulas, established specifically for angle shapes, would be double accounting for the factors effecting buckling.

The flexural buckling equations for solid and tubular members are adopted from the AISC Specification.

C4.5.4.3 Flexural-Torsional Buckling Compressive Strength

For 60 degree and 90 degree single angle members and formed 60 degree U-shaped members, using the effective slenderness formulas from Table 4-4 account for flexural-torsional buckling strengths.

Double angle members require the investigation of flexural-torsional buckling by modifying the flexural buckling strengths determined from Section 4.5.4.2 by substituting the equation specified for the value of F_e. The equation for F_e was adopted from the AISC Specification. The equations are based on using St. Venant torsional constant for a single angle which is included in the AISC Steel Construction Manual. For this reason, the term for F_{ez} includes a factor of 2 to account for the St. Venant torsional constant for a double angle.

Flexural-torsional buckling is not a consideration for solid and tubular round shapes.

C4.6.1 Built-up Members

The preferred spacing of longitudinal connectors between built-up tension members (e.g., back-to-back channels for guyed mast anchorages) is based on the slenderness ratio for tension members in Section 4.4.2.

C4.6.2 Tension-Only Bracing Members

The requirement for welded end tabs for tension-only bracing members for developing the yield strength of the member in tension is based on reducing the stress level at the welded connection. Welded end tabs represent a significant stress concentration. They are prone to cracking under repeated cycles of stress which may occur due to the stress reversals inherent with tension-only bracing members.

The detailing requirement to ensure members are in tension after installation is intended to avoid loose connections due to hole tolerances which can lead to excessive deflection of the entire structure as the member cycles between a no load condition to full tension under wind loading or when cycling between changes in wind direction.

C4.6.3 Design Tensile Strength

The criteria for design strength were adopted from the AISC Specification. The 0.80 and 0.65 resistance factors for guy anchor shaft tension yielding and rupture respectively, account for bending under variable loading conditions that result in unequal changes in guy tensions (i.e., unbalanced guy tensions) causing eccentric loading on the anchorage and changes in the direction of the resultant anchorage tension force.

The reduction coefficients U_{bs} for block shear were based on the commentary for the AISC Specification.

C4.6.3.2 Effective Net Area

The reduction factors specified for effective net area calculations were adopted by consensus of the committee based on the unique connections (e.g., single bolt connections) commonly used for the structures covered by the Standard. The reduction values presented account for the effects of unequal distribution of tensile stress due to eccentric loading and when the tension force is not uniformly applied across the cross section of the tension member. AISC reduction factors may be used as an alternative to the values presented (e.g., for round HSS or pipe shapes with a single gusset plate through slots in the member where U is based on the length of the connection and the diameter of the shape).

C4.7 Flexural Members

Flexure for bracing members connected with normal farming eccentricities is accounted for by using effective length factors in accordance with Section 4.5 and are not considered as flexural members.

Flexural strength criteria for solid round and tubular shapes were adopted from the AISC Specification.

The AISC equation for tubular round shapes was extended for higher D/t ratios in a similar matter as presented in C4.5.4.1 for the effective yield stress for tubular round shapes to allow the use of higher yield strength materials up to a D/t ratio of 300 when a lower strength material satisfies the AISC maximum D/t ratio and all strength requirements.

The criteria for polygonal tubular members were adopted from the ASCE 48 Standard. Refer to C4.5.4.1 for commentary regarding the effective stress for polygonal tubular members.

Refer to Figure C4-1 for an illustration of effective yield stresses for round and polygonal tubular pole members.

C4.8.1 Latticed Structures

Members of latticed structures subjected to moments from applied loads between the ends of the members when the member is subjected to axial compression are required to be amplified to account for displacements unless the analysis model for the member considers second order effects within the member. This is required because compression forces increase the magnitude of internal moments as the member undergoes displacement. The moment magnification factors specified were adopted from Appendix 8 of the AISC Specification.

C4.8.1.1 Leg Members

Leg members in latticed structures are allowed to be modeled as either truss or beam elements in accordance with Section 3.4.1. Both axial tension and compression conditions must be investigated in accordance with the interaction equations specified. Shear and torsion are considered negligible for leg members.

Leg members modelled as a truss or beam element have approximately the same axial force. Secondary moments in beam elements due to member continuity, although also generally small, must be included in the interaction equations. Beam elements are commonly used for staggered bracing pattern models because a leg modeled as a truss element is unsupported in a direction normal to a face which results in an unstable analysis.

The interaction equations are adopted from the AISC Specification with an exception provided for solid and tubular round members with secondary moments due to member continuity that occur when a leg is modelled as a beam element. The combined axial and bending interaction equation is multiplied by an 8/9th reduction factor. This is justified due to the more accurate analysis using beam elements. Without a reduction factor, the interaction equation for a beam element may exceed 1.0 but be less than 1.0 for the same leg modeled as a truss element. Regardless of how a leg is modeled, the axial load must not exceed the axial strength of the member without a reduction factor.

The interaction equation specified for the investigation of joint eccentricities is considered a simplified method as opposed to modelling the eccentricities in an analysis model for the structure. The interaction equation for combined axial load and the bending moment that exceed normal framing eccentricities as defined in Section 4.4.4, is based on the limit states of yielding or rupture. Buckling is not a consideration as the moment from an eccentric bracing connection or from an eccentric leg splice, is considered a local moment applied at or near a panel point. The maximum moment is considered to occur at the point of application with an inflection point (i.e., zero moment from the eccentricity) near the midpoint of the panel. The limit state of yielding or rupture at the point of application is therefore considered to govern over buckling.

C4.8.2 Tubular Structures

The interaction equation specified was adopted from the AISC Specification for combined axial load, flexure, shear and torsion for tubular structures. The nominal axial compressive stress is limited to F_y as the effective yield stresses in Table 4-8 have been increased for use with compact sections in flexure as explained in C4.5.4.1. The increases in the effective yield stresses above the yield strength for compact sections subjected to flexure would not be applicable to compact sections subjected to compression.

The resistance factor for torsion has been increased to 0.95 from the AISC Specification and was adopted from the AASHTO Specification. The AISC commentary for Section H3 explains that an approximately 10% increase in strength was ignored for edges fixed at the end which would justify increasing the 0.90 AISC resistance factor for torsion to 0.95.

The reinforcing criteria for entry and exit ports in tubular sections is a simplified approach to ensure that the strength of a reinforced opening will not be less than the strength of the tubular section without the opening. The intent of using this approach is to be able to ignore a reinforced opening in the structural model for a tubular section. The area of the reinforcing steel times it yield strength is equated to the section material removed for the opening times its yield strength. In addition, the material removed from the section contributing to the plastic section modulus of the section times its yield strength is equated to the area of the reinforcing contributing to the plastic section modulus of the section times its yield strength. For simplicity, the center of the section without the opening is used to determine the distance from the centroid of the section to the centroid of the reinforcing. Also, for simplicity, the calculations are permitted to be based on dimensions at the centerline elevation of the opening. The yield strength of the reinforcing is not allowed to exceed the yield strength of the section material due to strain compatibility between the section and the reinforcing. For lower reinforcing yield strengths, the reinforcing material is assumed to yield as the strain exceeds the strain associated with the yield strength of the reinforcing.

C4.8.2.1 Round Tubular Sections

The shear and torsional strength criteria for round tubular section was adopted from the AISC Specification. The variable Lp is defined by AISC as the distance from the point of maximum shear to the point of zero shear. For simplicity the Standard conservatively uses the height of the pole or when guyed, the distance between the guy elevations (or between the base and the first guy elevation) or the cantilevered height for a guyed mast.

C4.8.2.2 Polygonal Tubular Sections

The shear and torsion moment strength criteria for polygonal tubular sections was adopted from the AASHTO Specification.

For simplification, the effective area for resisting shear is set equal to 50% of the gross area for all polygonal shapes. Theoretically the shear area percentage of the gross area varies from approximately 48% for 8-sided shapes to 50% for round shapes.

C4.9.1 Bolts

The provision for allowing the use of galvanized A325 bolts after they have been tensioned to no greater than 40% of their ultimate capacity is because at a 40% level of pretension, the bolt is assumed to not have yielded nor undergone permanent deformation. In these cases, it should be possible to freely run the nut up the bolt threads by hand. Galvanized bolts that have undergone permanent deformation should not be reused due to the potential cracking that may occur.

C4.9.2 Nut-Locking Devices

The structures covered by the Standard are subjected to low amplitude, high frequency vibrations from laminar wind loading and other conditions. Nut-locking devices are therefore required for bolts installed to a snug-tight condition. Pretensioned bolts have proved not to require nut locking devices.

Lock washers have a history of cracking upon installation, especially when overtightened and the lock washers are hot-dip galvanized vs. mechanically galvanized. The use of lock washers is limited to structures no greater than 1,200 feet in height, which by consensus of the committee is the height where most issues have occurred due to the level of vibrations common for structures at these heights. Cracking of lock washers for a structure of any height can lead to the lock washer falling out within the grip of the bolt assembly, which can lead to loose connections and consequently multiple structural issues.

C4.9.3 Pretensioned Bolts

Connections subjected to tension forces (e.g., flange plate connections for latticed tower leg splices) are required to use pretensioned bolts when the connected parts subjected to bending are not designed as rigid elements reducing prying action to insignificance. The requirement is based on the high level of stress reversals that can occur due to prying action under variable wind loading which can result in a fatigue failure of the bolt assembly. Annex Q provides a method for designing flange plates based on rigid plate material (refer to Annex Q Section Q3.0).

A325 and A490 bolt specifications require rotation capacity and other quality control measures for bolt assemblies such as tests for excessively over-tapped nuts, insufficient material ductility, efficiency of lubricants, etc. These tests are required to assure bolt assemblies will function together as a unit to achieve the required pretensioning. When other grades of high strength bolt assemblies are utilized (i.e., A354 bolts for large diameter bolts not available as A325 bolts), similar tests would be appropriate to specify as part of the procurement specification.

C4.9.4 Edge Distances

Edge distances for sheared edges are required to be 1.5 times the bolt diameter which exceeds the AISC requirement for all bolt diameters over 1/2 inch. Single bolt connections are commonly used for the structures covered by the Standard, and based on manufacturing tolerances for sheared edges, the Standard has traditionally used 1.5d as a minimum requirement for sheared edges. The edge distance is not critical for flange or base plates with minimal shear forces acting toward the edge of a plate. Edge distances for flange or base plates are therefore only required to prevent the nut or bolt head from extending over the edge of the plate in order to provide the full bearing surface area to resist bolt or anchor rod tension forces. Annex Q Section Q10.0 requires the edges distance be adequate to prevent washers, when used, to also not extend over the edge of the plate.

C4.9.5 Bearing Type Connections

For connections primarily loaded in shear, bearing connections used with oversized or slotted holes in the direction of the shear force would be expected to slip under loading and are not permitted. Pretensioned bolts are required for these applications.

C4.9.6.1 Design Tensile Strength

The nominal tensile strength of a bolt (or threaded part) is defined as the specified minimum tensile strength of the bolt times the net area for the threaded portion. Different equations for the net area apply to ANSI inch series threads and ISO metric series threads. The nominal strength is different from the nominal strength specified by the AISC Specification. AISC approximates the net area to be equal to 75% of the gross area of the bolt or threaded part. The AISC approach becomes less accurate and more conservative as the diameter of a bolt increases. For example, AISC assumes the net area is equal to 75% of the gross area, but using the equations specified, the net area based on ANSI inch series threads for a 3/4 inch diameter bolt is equal to 74% of the gross area. For a 1-1/2 inch diameter bolt, the ANSI inch series equation results in a net area equal to 80% of the gross area.

The 0.75 resistance factor was adopted from the AISC Specification.

C4.9.6.2 Design Bearing Strength

The AISC design bearing strength equation was modified by consensus of the committee based on the performance of single-bolt connections for structures covered by the Standard, test results and comparisons to other international standards for latticed towers. Because test results indicate that significant deformation occurs prior to tear out when the edge distance is not less than 1.5d as required by Section 4.9.4, a resistance factor of 0.80 was adopted vs. the AISC 0.75 resistance factor. In addition, the failure plane defined by AISC for tear out was considered too conservative for single-bolt connections. Using only the clear distance to the edge of the hole would make an unreasonable amount of existing single-bolt latticed tower connections appear to be overstressed. Using a tear-out length equal to the clear distance plus 25% of the bolt diameter with a 0.80 resistance factor results in good correlation of design bearing strengths relative to design shear strengths based on previous editions of the Standard.

The AISC maximum bearing stress equal to $2.4F_u$ was adopted as deformation at bolt holes is a consideration considering stress reversals from varying wind loading conditions.

The Standard allows the summation of the bearing resistances of individual bolts for multiple-bolt connections as deformations at bolt holes would be expected to redistribute forces should an unequal distribution of bolt forces occur.

C4.9.6.3 Design Shear Strength

The design shear strength when threads are excluded from the shear plane was adopted from the AISC Specification using connection length reduction factors determined from the AISC commentary for Section J3.6. The 0.75 resistance factor and the connection length reduction factors account for the effects of bending due to the deformation of the connected parts and for multiple bolt connections, the effects of differential strain. For single-bolt connections and for multiple-bolt connections up to 16 inches in length, these effects are negligible. Because single bolts are commonly used for structures covered by the Standard, the consensus of the committee was to introduce a connection length reduction factor vs. including a 0.90 reduction factor in the nominal strength for shear per AISC Table J3.2.

The nominal shear strength for when threads are included in the shear plane is equal to 80% of the nominal shear strength for when threads are excluded from the shear plane in accordance with the AISC commentary for Section J3.6. The reduction in strength accounts for the reduced shear area of the threaded portion a bolt or threaded fastener.

C4.9.6.4 Combined Shear and Tension

The interaction ratio for combined shear and tension was adopted from the AISC Specification using the elliptical relationship for shear and tension per the AISC commentary for Section J3.7.

C4.9.6.5 Connecting Elements

Criteria for connecting elements (e.g., gusset plates) were adopted from the AISC Specification.

The net area limitation of 85% of the gross area of a connection plate was obtained from the ASCE 48 Standard and is intended to provide a simplified method for determining effective widths compared to the use of the Whitmore section referenced in Part 9 of the AISC Steel Construction Manual.

C4.9.7 Splices

The leg member splice (e.g., flange plates) minimum tension strength requirement for guyed masts is a requirement when a guyed mast is not designed for a guy rupture condition in accordance with Annex E. Designs considering guy rupture in accordance with Annex E are considered to have adequate leg splice strengths.

Leg splices of guyed masts in non-cantilevered spans are often governed by compression with very little or no tension forces from the loading combinations in Section 2.0. During installation of guyed masts, guy slippage has been known to occur subjecting the mast to sudden large bending moments occurring with a relatively low downward force which can result in a significant tension force at a leg splice. The minimum tension strength requirement is intended to minimize the potential for collapse of the mast under this condition.

A minimum tension strength requirement is often needed for installation of a guyed mast when gin poles are utilized for erection which may subject a leg splice to significant tension due to the application of overturning moment reactions from the gin pole with relatively low downward forces. Significant tension forces may also be present for gin pole construction and other types of construction methods when cantilevered sections exist in the mast prior to the connection of the guys for the upper guy level of a span.

The TIA 322 Standard referenced in Section 18.0 includes loading combinations related to installation, alteration and maintenance associated with rigging plans prepared in accordance with the ASSE 10.48 Standard also referenced in Section 18.0. The results of an analysis in accordance with TIA-322 may indicate tension forces in leg members of a guyed mast; however, the minimum tension strength of TIA-222 are intended to be satisfied regardless of the results of an analysis per TIA-322.

The loading combinations from Section 2.0 are considered to govern the overall design of guyed masts; however, because of localized gust patterns and other varying loading conditions, tension forces, although not large enough to govern the design of leg members, may subject leg splices to tension forces not considered in design. For example, the minimum tension strength requirement may govern the number of splice bolts required at a leg splice. Also, splices of large solid round legs are often detailed for direct bearing with minimal weld strength between the legs and the flange plates. The minimum tension strength requirement can govern the size of the welds as well as the number of flange bolts required in these types of joints.

The minimum design tensile strength specified was determined by consensus of the committee to provide practical minimum strength for guyed mast leg splices. The limit of 500 kips was specified because the 33% requirement was considered excessive and not practical for large diameter solid round legs often used for tall broadcast guyed masts.

Self-supporting latticed structure legs are subjected to significant tension and compression forces due to the loading combination of Section 2.0 and therefore a minimum tension strength requirement is only specified for guyed masts.

The minimum strength requirement for pole structures was adopted from the ASCE 48 Standard which was considered by consensus of the committee to be a practical minimum requirement.

C4.9.7.1 Tubular Pole Structures

The slip splice criteria were adopted from the ASCE 48 Standard. The Standard assumes there are no tension forces at a slip spice. For special conditions where tension forces are expected at a slip joint, a locking device would be required (e.g., for an installation condition in accordance with a rigging plan where spliced sections are lifted into place).

The length of a slip splice varies with tolerances in manufacturing and other factors which prevents an accurate determination of the height of the structure. The length specified for determining heights is intended to provide a consistent and practical value to be used when creating structural models.

Provisions for applying jacking forces are required in order to conform to the jacking force requirements of Section 13.3.5.

C4.9.8 Guy Assembly Link Plates

The strength and dimension requirements for link plates were adopted from the AISC Specification for pin connected members with the exception of the resistance factor for bearing strength. By consensus of the committee, the AISC 0.75 for bearing resistance factor is not considered applicable to guy assembly link plates where the governing tension force is from an extreme wind, ice or earthquake loading condition. Based on the performance of guyed mast link plates, a 0.90 resistance factor for yield strength bearing with factored tension forces is considered adequate to limit material deformations and satisfy joint rotation requirements.

Link plates are not considered eyebars as defined by AISC and the 1/32 inch oversize pin hole diameter for an eyebar and other associated dimensional limitations specified in ASCE Section D6.2 do not apply to guy assembly link plates.

C4.9.9 Anchor Rods

Leveling nuts are required for anchor rods to allow leveling adjustments for installations. Unless otherwise specified, it is not considered practical to rely on a level supporting surface when only downward reactions occur from the loading combinations in Section 2.0.

The use of grout below a base plate is not allowed to be considered when determining anchor rod axial forces. Reactions must be fully resisted by the anchor rods. The use of grout has resulted in numerous cases of anchor rod corrosion due to improper grout installation (e.g., lack of drainage), improper grout material and the lack of maintenance (refer to Annex Q Section Q11.0).

The anchor rod installation requirements were adopted from the AASHTO Specification. The tightening requirements are intended to result in pretensioned anchor rods to prevent nut loosening without the use of a nut-locking device. When oversized holes are utilized, appropriate washers are required for both bottom and top nuts. The loosening of top or bottom nuts can result in an unequal distribution of forces within an anchor rod group and can result in premature anchor bolt failures, cracking in the base plate or weld at the interface with the structure and other issues. It is important to tighten all top and bottom nuts using the procedure specified.

Anchor rod interaction equations are provided based on the anchor rod projection from the supporting surface to the bottom of the leveling nut. The interaction equations involving tension are based on an elliptical representation to correspond with the combined shear and tension interaction equation specified for bolts in Section 4.9.6.4. The consensus of the committee was to use the more conservative squared representation for the compression interaction equations.

Bending is ignored for projections up to a maximum of one anchor rod diameter in accordance with the AASHTO Specification. For projections up to 4 anchor rod diameters, the effective slenderness ratio is small, and the compression strength of the anchor rod can be considered equal to the anchor rod yield strength. When the projection of an anchor rod is greater than 4 anchor rod diameters, the anchor rod buckling strength is based on the critical compression stress based on a 1.2 effective length factor for the condition of fixity at both ends (refer to commentary for AISC Appendix 7).

The moment in the anchor rods when the projection is greater than one anchor rod diameter is conservatively based on an inflection point at 65% of the anchor rod projection above the supporting surface to account for anchor rods embedded in concrete where the point of fixity may be slightly below the surface.

The equations assume that the lateral displacement of the anchor rods is insignificant and that secondary moments due to axial forces can be ignored. Engineering judgement is required with large projections considering that as the anchor rod projection increases, secondary effects may increase significantly. The actual distribution of forces becomes complex based on the interaction with the base plate distributing shear forces with the onset of yielding of an anchor rod.

Tensile strength is based on the tensile strength of threaded fasteners with a 0.75 resistance factor. Compression yield strength is based on the gross area of an anchor rod with a 0.90 resistance factor. Compression buckling strength is based on the critical buckling stress on the gross area of an anchor rod with a 0.90 resistance factor.

Shear strength in combination with tension is based on the shear strength for fasteners with threads included in the shear plane with a 0.75 resistance factor. Shear strength in combination with compression is based on yielding on an effective shear area equal to 75% of the gross area and a resistance factor equal to 0.90.

Flexural strength is based on yielding using the plastic section modulus based on the nominal diameter of an anchor rod with a 0.90 resistance factor.

When the projection exceeds one anchor rod diameter, the proper use and type of grout may be used to eliminate considering bending (i.e., equivalent to assuming a projection not greater than one anchor rod diameter). The grout is considered to act as a shim to resist moment; however, because the uniform and complete placement of grout below a base plate is not considered to be consistently achieved due to the typical geometry of base plates, grout is not allowed to be considered in the determination of anchor rod compression or tension forces. The 3 inch limitation on the thickness of grout is due to the tendency of grout with greater thicknesses to crack after placement and become ineffective in acting as a shim.

C4.9.10.1 Tubular Pole Structures

Longitudinal seam welds are subjected to low stress levels and are parallel to the bending stresses in the pole wall. For this reason, partial penetration welds are acceptable except for the locations discussed below. The requirement of a 60% minimum penetration was adopted from the AASHTO Specification.

Transverse seams are normal to bending stresses and are required to be complete penetration or full fusion through the full wall thickness. This requirement was also adopted from the AASHTO Specification.

Because of the hoop stresses in the outer section at slip splices, longitudinal seam welds are required to be complete penetration or full fusion through the wall thickness for a length equal to the slip splice length plus 6 inches. This requires the pole manufacturer to anticipate what the maximum splice length will be after installation based on the tolerances effecting the lap length. Longitudinal seam welds are also required to be complete penetration or full fusion within 6 inches of circumferential welds and flange or base plates. These requirements were adopted from the AASHTO Specification.

Base plate to pole shaft welds are required to be complete penetration welds. This requirement was adopted from the ASCE 48 Standard. An exception is made for pole diameters not greater than 24 inches where socketed connections are permitted (refer to Annex Q Section 8.0). Pole structures can be subjected to wind induced oscillations (i.e., fatigue loading), especially for large diameter poles (refer to Annex M). Socketed connections are known to be more susceptible to fatigue cracks compared to full penetration butt joints. Socketed joints can be a more economical base plate connection compared to a full penetration butt joint; however, the difficulties associated with manufacturing small diameter poles with full penetration welds (e.g., internal access through a small center circle) are minimal for pole diameters greater than 24 inches. It was therefore the consensus of the committee to limit the use of socketed joints for pole diameters up to 24 inches.

C4.9.11 U-Bolt Connections

U-bolt connections are commonly used for the attachment of appurtenances. The provision specified are based on the performance of U-bolted connection for the structures covered by the Standard.

C4.9.11.3 U-Bolt Strength

U-bolt connections are not allowed to transfer torsion to a round supporting member for strength or stability purposes except for connections of appurtenances where slippage of the connection may be corrected after a significant loading event.

U-bolts are considered as threaded fasteners and the strength requirements of Section 4.9.6 apply. An additional requirement is specified to limit the design axial tensile strength for each leg of the U-bolt to yielding on the gross cross section with a 0.85 resistance factor. This requirement was established by the committee to limit stresses to below the yield stress. The shear and torsional strength of a U-bolt assembly would be expected to diminish rapidly under tension loads in excess of yielding due to the loss of clamping forces as the legs of the U-bolt elongated. Reduced stresses are also desired to account for the curvature of round U-bolts, the corner radius of square U-bolts and to limit the crushing stresses imposed on attached members.

An interaction equation is provided for combined sliding and torsion. An elliptical representation was selected by the committee as the directions of slippage are normal to each other. Torsion would only be applicable for the attachments of appurtenances.

The nominal sliding resistance is based on a 0.30 coefficient of friction between the U-bolt and the supporting bracket. When the legs of a U-bolt are tightened to a given pretension at installation, there will be a normal force on the U-bolt side of the member and an equal and opposite force on the opposite side of the member in contact with the supporting bracket. The force at each location will be equal to two times the pretension force in each leg of the U-bolt. As the supported member imposes a tension force on the U-bolt assembly, the supported member will remain in contact with the supporting bracket until the applied tension equals two times the installed pretension in each leg of the U-bolt. At this point, the supported member will lose contact with the supporting member with any additional tension applied. As the applied tension increases, the force on the supported member from the U-bolts will increase and remain equal to the applied tension force. Although there will be a frictional resistance on the U-bolt side of the supported member, excessive displacement would be expected with any applied sliding force. The nominal sliding strength of a U-bolt assembly is therefore limited to the frictional resistance between the supported member and the supporting bracket. Because the reserve frictional resistance is ignored on the U-bolt side of the supported member, a 1.0 resistance factor is used to determine the design shear strength.

Torsional resistance is conservatively limited to the sliding resistance at the contact surface of the supporting bracket with the supported member. Although there would be frictional resistance on the U-bolt side of the supported member, the capacity of the U-bolt assembly to resist torsion through a frictional force applied at this location would be minimal. As with sliding, the potential additional torsional resistance is justification for a 1.0 resistance factor.

For most installations it is not practical to measure the installed tension of U-bolts. By consensus of the committee, a 20 ksi stress would represent a reasonable installation stress that can be used to determine sliding and torsional strengths. An exception would be if the supporting bracket or supported member cannot resist that level of pretension without bending the bracket or crushing the supporting member.

The U-bolt criteria presented is based on conservative estimates of strength and the use of documented tests for specific U-bolt connections are acceptable to be used as an alternative.

C5.0 MANUFACTURING

C5.4.1 General

The materials listed in Table 5-1 represent the most commonly used structural steels used for the structures covered by the Standard which have performed well with supplemental specifications for maximum tensile strength and Charpy V-notch requirements as specified in Section 5.4.1.

C5.4.1 General

Supplemental specifications commonly considered for galvanized structural steels that are not included in Table 5-1 include but are not limited to minimum yield strength, minimum tensile strength, minimum elongation, maximum tensile strength and maximum silicon, phosphorous and manganese content for galvanizing considerations. For welded applications, common supplemental specifications include but are not limited to a maximum carbon equivalency or an AWS D1.1 composition factor used to determine minimum preheat and interpass temperatures in order to avoid brittle heat effected zones.

Polygonal tubular poles involve cold bending of higher strength materials (e.g., 65 ksi minimum yield strength), welding of relatively thick base or flange plates compared to the pole material, acid pickling and hot-dip galvanizing. All these factors can combine to create conditions susceptible to cracking. The Standard specifies a minimum Charpy V-notch value for pole material as a means to obtain a degree of toughness considering the potential of cracking. Other manufacturing variables also affect the potential for cracking such as the die opening and nose radius used for forming and the temperature of the steel at the time of forming. The minimum Charpy V-notch requirement applies to both the formed shape and butt-welded base plates. The minimum Charpy V-notch requirement was adopted from the ASCE 48 Standard.

Tubular shapes do not have a minimum Charpy V-notch requirement as they are not subjected to cold working. Socketed flange plate material also does not have a minimum Charpy V-notch requirement because of the limitation of the pole diameter allowed for socketed connection specified in Section 4.9.10.1 and the anticipated flange plate to pole wall thickness ratios.

The maximum tensile strength for polygonal pole material is specified to minimize the potential for cracking during galvanizing considering the strain hardening expected to occur due to cold forming of the polygonal shapes.

C5.4.2 Non-Pre-Qualified Steel

The minimum Carbon Equivalent and minimum elongation values were adopted by consensus of the committee as minimum requirements for the proper performance of structures designed in accordance with the Standard. Many other considerations may be appropriate based on manufacturing methods and the site-specific application of structures. The Carbon Equivalent equation was adopted from AWS D1.1 and varies from the ASTM A6 Carbon Equivalent equation by including Si with Mn in the equation. The AWS D1.1 equation was adopted to correspond with the determination of minimum preheat temperatures in accordance with AWS D1.1 Annex H.

The minimum elongation value was adopted by consensus of the committee as a minimum level of ductility. The length for the elongation test specimen was not specified as the test length varies between different material specification and the minimum elongation is an empirical value and was considered to be applicable regardless of the test specimen length used in accordance with the ASTM 6 Standard.

The location of the tension test specimen for solid round shapes was adopted by consensus of the committee as the most representative location for the test specimen when full size samples were not used for tension tests. The location applies to hot-rolled and cold-finished bars and supersedes the requirements of ASTM A370.

The consensus of the committee was to require minimum Charpy V-notch (CVN) values for large diameter solid round members (i.e., greater than 5 inch diameters) with yield strengths of 50 ksi or higher based on the performance of structures designed in accordance with previous revisions of the Standard. The alternate CVN equation was adopted from the CSA S-37 Standard. The equation results in higher CVN values than the 15 ft-lbs minimum specified value. The higher values may be useful for determining conformance to the Standard for locations with lowest monthly mean temperatures greater than 0 degrees Fahrenheit.

The CVN value varies with the location of the test specimen obtained from a solid round member. The consensus of the committee was to establish a location to result in conformity in the Industry and was believed to adequately represent the CVN value for solid round members.

C5.5 Fabrication

By consensus of the committee, the requirement for AISC fabricator certification was justified based on the past performance of structures supplied in accordance with previous revisions of the Standard.

The tolerance for straightness was established by the committee based on reasonably obtainable tolerances using standard fabrication practices and the tolerances known to facilitate installation.

Complete contact of compression members in direst bearing (e.g., solid round leg members in guyed masts) are not required to be in 100% contract. The 75% requirement was established by the committee as a reasonable obtainable value using standard fabrication practices and would satisfy the design intent of the Standard.

The CVN requirement for weld metal and the heat affected zone was adopted from AWS D1.1 to result in a consistent CVN value for the pole material, the flange plate and the weld metal.

Cracking at complete penetration welds at flange and base plate locations occurring during hot-dip galvanizing has been reported by many fabricators. The requirement of 100% testing was adopted from the ASCE 48 Standard.

C5.6. Corrosion Control

Galvanized steel has proven to be provide the preferred method of corrosion control for structures designed in accordance with the Standard. The minimum requirements of this section were established by consensus of the committee. based on the performance of structures designed in accordance with previous revisions of the Standard.

C6.0 OTHER STRUCTURAL MATERIAL

C 6.1 Scope

The Standard primarily addresses steel structures; however, the criteria for use of alternative materials is addressed in Section 6.0.

C6.2 General

Designs must be based on limit states design standards for the material utilized. The level of reliability must be equivalent to the reliability for structural steel using the resistance factors specified in Section 4.0. Refer to Section 17.13 for additional requirements appropriate for the use of other structural material.

C7.0 GUY ASSEMBLIES

The requirements of Section 7.0 are based on the performance of guyed masts designed in accordance with previous revisions of the Standard.

C 7.3.3 Wire Rope

The use of wire rope is limited to special applications requiring higher flexibility compared to strand. The stretch associated with wire rope results in a loss of pretension under loading and is not considered appropriate for use as a structural guy element without considering the loss of pretension and the special end connections required for wire rope.

C7.4.1 Thimbles

The strength of a formed guy grip can be significantly reduced using a thimble with an inadequate bend radius resulting in the end connection becoming the weak link in the guy assembly.

C7.4.2 Formed Guy Grips

Many different types of guy strands are utilized as guy elements. Using a compatible formed guy grip is essential to avoid slippage, excessive corrosion and a reduction in the strength of a guy assembly.

Once a formed guy grip is in service and removed, the grip is considered to be compromised and not allowed to be reused.

C7.4.3 Clips

The tolerance of 1/16 inch was established by consensus of the committee based on the performance of structures designed in accordance with previous revisions of the Standard.

C7.4.4 Sockets

As with formed guy grips, using a compatible socket is critical for the performance of a guy assembly. Sockets for non-steel guy assemblies (i.e., nonconductive guys used to eliminate interference with RF transmission) must meet the performance characteristics for steel guys.

C7.4.5 Shackles,

The steel grades and heat treatment requirement specified for guy assembly connection components were established by consensus of the committee based on the performance of galvanized guy assemblies designed in accordance with previous revisions of the Standard. Higher strength materials are considered unacceptable for galvanizing. The residual stresses from forged components without heat treatment are considered unacceptable for use in guy assemblies for the structures covered by the Standard.

C7.4.6.1 Turnbuckles

Refer to C7.4.5.

C7.4.6.2 Bridge Sockets

Refer to C7.4.5.

C7.5 Guy Dampers

Low frequency high amplitude galloping can result in overstresses and fatigue failures in guy assemblies or in the structure (i.e., guy lugs or structural members). High frequency low amplitude Aeolian vibrations can lead to fatigue failures in guy strands at end terminations. Guy strands with rigid end termination (e.g., bridge sockets) are more susceptible to fatigue damage compared to guy strands with formed guy grip terminations. The requirements for dampers were established by consensus of the committee based on the performance of structures designed in accordance with previous revisions of the Standard.

The initial guy tension in a guy assembly is a significant factor influencing vibration in guy assemblies (refer to Section 7.6.1).

C7.6.1 Initial Tension

The tendency of a guy assembly to vibrate is related to the initial tension under no-load conditions. Based on the experience of the committee with guyed masts, it was the consensus to establish an acceptable range of initial tensions that would minimize the occurrence of excessive oscillations. The initial tension in a guy assembly will vary with temperature and the location along the guy span. The initial tension at the anchorage at 60 degrees F was determined to best define the desired range of initial tensions. The 60 degree F temperature is intended to be used as a default in the absence of site-specific data.

C7.6.2 Design Strength

Two resistance factors are applied to the ultimate breaking strength of a guy assembly. Because of the variables associated with guy assemblies, the resistance factor for strength is lower than the 0.75 resistance factor specified for tension rupture in Section 4. Non-metallic cables are considered to have additional factors effecting strength and therefore have a lower resistance factor. An additional reissuance factor based on end fitting strength efficiency is applied to both metallic and non-metallic cables.

C7.6.2.1 Ultimate Breaking Strength

The strength of an end fitting or take-up device is not required to meet or exceed the strength of the guy and may govern the design strength of the guy assembly.

C7.6.2.2 End Fittings Strength Efficiency Factor

The type of end fitting used for a given cable may not prevent slippage or damage to the cable under a tension equal to the breaking strength of the cable. Many formed guy grips do prevent slippage and do not damage to the cable when properly sized and therefore have a 1.0 strength efficiency factor. Other types of end fittings have strength efficiency factors less than or equal to 1.0. Wrap-around end terminations (e.g., around a leg of a guyed mast) are assumed to significantly damage a guy strand due to their rigidity and are not allowed. Refer to Section 15.7 for exemptions for steel strand wrap-around end terminations for existing structures.

C7.6.3 Modulus of Elasticity

Stranded cables inherently stretch under loading due to their construction. The modulus of elasticity is therefore not constant or well defined. A pre-stretched strand is expected to have a higher modulus of elasticity as a portion of the pre-stretch remains permanent after the initial stretch. The consensus of the committee was to define default modulus of elasticity values to be used in the absence of more accurate data from the cable manufacturer.

C7.6.4 Articulation

Guy assemblies are subjected to significant movements during wind and other loading events resulting in a change in the direction of the tension forces at both ends of the assembly. When rotations are not free to occur, bending stresses occur in the guy which can lead to premature failure under extreme loading conditions or to a progressive fatigue failure of the individual guy strands. Low frequency vibrations (i.e., galloping) can subject a guy assembly to extreme stresses when the ends of a guy assembly are not free to rotate. Non-metallic guys are considered to be more prone to damage compared to steel strands. The 10 degree articulation value was established as a minimum requirement by consensus of the committee based on the performance of guyed masts designed in accordance with previous revisions of the Standard.

C7.7 Manufacture

Non-metallic guy assembly components (e.g., insulators or guys) can be prone to damage from UV exposure and require their expected life to be provided by the manufacturer.

C7.7.1 Proof Loading of Assemblies

Factory installed end sockets are required to be proof tested due to issues which may occur during the manufacturing process that are not visible after manufacturing. The proof testing criteria was established by consensus based on the experience of the committee with guyed masts designed in accordance with previous revisions of the Standard.

C7.7.2 Pre-Stretching

When factory installed end fittings (e.g., zinc-poured sockets) are utilized at each end of a guy assembly, the accuracy of the length of the guy assembly becomes critical compared to guy assemblies with a formed guy grip at one or both ends. Pre-stretching minimizes the stretch that otherwise could consume a significant portion of the adjustment length of a take-up device used in the guy assembly to provide the desired initial tension. The pre-stretching force specified was established by consensus based on the experience of the committee with guyed masts designed in accordance with previous revisions of the Standard.

C7.7.3 Length Measurements

Guy strand will twist as tension is applied due to the guy strand manufacturing process. When formed guy grips are utilized, the twisting of a guy assembly can be accommodated during the installation of the guy assembly as the initial tension is applied. Twisting cannot be accommodated when both ends utilize factory installed end fittings (e.g., zinc-poured sockets).

Length measurements are required after pre-stretching at the initial tension and temperature specified for the design of the structure to ensure the take-up device of the assembly will have an adequate adjustment length.

C7.7.4 Striping

Striping is required in order for the installer to align the strip in a straight line to ensure twisting will not occur as the initial tension is applied to the guy assembly.

C7.8 Installation

Guyed masts subjected to wind loading or galloping, results in slack-taut conditions. Guy strand will twist under varying tension conditions which can lead to the loosening of turnbuckles.

Devices are required to prevent the disengagement of the turnbuckle under repeated twisting events.

C8.0 INSULATORS

C8.1 Scope

The requirements of Section 8.0 are based on the performance of structures designed in accordance with previous revisions of the Standard.

C 8.2 Design

The steel grades and heat treatment requirement specified for steel end fittings were established by consensus of the committee based on the performance of galvanized end fitting components. Higher strength materials are considered unacceptable for galvanizing. The residual stresses from forged components without heat treatment are considered unacceptable for use with the structures covered by the Standard.

Because of the variables associated with non-metallic insulators, the resistance factors for strength are lower than the 0.75 resistance factor specified for tension rupture in Section 4. The resistance factor for fail-safe insulators is higher than other insulators due to their reduced potential for failure.

C8.3 Manufacturer

Insulators are required to be proof tested due to issues which may occur during the manufacturing process that are not visible after manufacturing. The proof testing criteria was established by consensus based on the experience of the committee with insulators designed in accordance with previous revisions of the Standard.

The proof testing value is lower than the proof testing value specified in Section 7.0 for guy assemblies due to the lower design strength to ultimate strength ratio specified for insulators.

The expected life of insulators is required to be provided by manufacturers in order for owners to establish appropriate maintenance programs.

Manufacturers are also required to provide shipping, handling and inspection procedures due to the susceptibility of insulators to damage.

9.0 FOUNDATIONS AND ANCHORAGES

The criteria specified in this section, unless otherwise indicated, were established by consensus of the committee based on experience with foundations and anchorages designed in accordance with previous revisions of the Standard.

C9.3 Geotechnical Investigation

A geotechnical investigation is not required for Risk Category I and II structures; however, verification of assumed foundation design parameters is required prior to installation. A geotechnical report prior to foundation design is preferred to avoid costly delays when assumed design parameters are determined not to be applicable for a site when verification is performed immediately before installation.

Direct embed tubular poles are required to be designed to prevent upheaving when the base of the pole is sealed with a bearing plate.

The frost depths indicated in Annex B are considered minimum values. Greater frost depths may be desired based on local data or for high-risk category structures based on the return period desired for the determination of the frost depth.

The development of ice lenses below a foundation is required to result in frost heave. For coarse granular soils where ice lenses are not expected to occur, the depth of a foundation is not required to be below the frost depth. In some cases, it is practical to remove soil susceptible to the formulation of ice lenses to a depth below the anticipated frost depth and replace with a coarse granular backfill.

The tension force from expansive soil is considered as a pretension force applied to the foundation. When an external tension force is applied, the force from the expansive soil is assumed to gradually reduce until the magnitude of the external force exceeds the force from the expansive soil. Upon further increase in the external tension force, the force in the foundation would be equal to the external force. The dead load factor of 0.9 is specified as dead load is resisting the tension force from the expansive soil. The load factor for the expansive soil tension force is equal to the load factor used for the dead weight of soil when acting as a load as opposed to a resistance (refer to Section 2.0).

C9.4 Foundation Design

The weight of the foundation and the weight of soil or other material directly above the foundation are considered as dead loads as specified in Section 2.0. The load factor applied should correspond to the load factor used for dead load for the loading combination. Different loading combinations may govern the foundation compared to the supported structure. For example, for a spread footing subjected to overturning, the loading condition with a dead load factor of 1.2 may govern soil bearing strength requirements whereas the loading combination with a load factor of 0.9 may govern the consideration of the maximum eccentricity specified in Section 9.4.1. For foundation design, it is not required to consider a dead load factor different from the dead load factor for the loading combination except for guy anchorages as explained below.

The weight of soil outside the foundation perimeter of a foundation used to resist uplift or overturning (e.g., an equivalent uplift cone of soil) is required to be considered as a nominal soil strength with a 0.75 resistance factor applied.

A unique situation exists for guy anchorage foundations because only a maximum dead load combination with a 1.2 dead load factor is required for guyed masts. The anchorage reactions from each loading combination are to be considered for the design of a guy anchorage; however, the weight of soil directly above the foundation and the weight of the foundation are resisting the anchorage tension reaction and are required to be multiplied by 0.9 when determining the strength of the foundation to resist uplift for all loading combinations.

Foundation displacements are not required to be considered for the analysis of the supported structure except when the lateral displacement at grade exceeds 0.75 inches for the serviceability limit state condition for displacement sensitive soils or for structures supported on a single caisson foundation (e.g., a tubular pole structure). The 0.75 inch value was determined by consensus of the committee based on the performance of structures designed in accordance with previous revisions of the Standard.

C9.4.1 Design Strength

A triangular or rectangular soil distribution is considered acceptable based on limit states design criteria using factored reactions. The maximum eccentricity is intended to prevent excessive foundation displacements subjected to overturning and is analogous to the 1.5 factor of safety against overturning utilized for ASD foundation design criteria specified in previous revisions of the Standard.

Maximum moments may occur for the parallel or diagonal directions of overturning. For self-supporting latticed tower mat foundations, maximum foundation moments and shears may occur in the interior or external portion of footprint of the tower (e.g., positive moments for the interior and cantilever moments for the exterior).

Structures supported by a single caisson or drilled foundation (e.g., a pole) require the use of a flexible analysis method for deeper foundations that consider the flexibility of the foundation and the displacement of the soil. The shear and moment distribution in a deep caisson or drilled shaft using a rigid analysis method (i.e., a simplified method that considers the foundation as a rigid element) is considered to grossly misrepresent the actual distribution of shear and moments.

The minimum longitudinal reinforcement ratio for piers, caissons or drilled shafts is based on the ACI 318 requirement for columns that are not governed by compression. Longitudinal reinforcement in piers supported on spread foundations or supporting a spread foundation are required to be fully developed in tension, regardless of the design stress level in the reinforcement in order to insure adequate flexibility during an earthquake. Longitudinal reinforcement in piers subjected to compression are only required to be developed for the level of compressive stress in the reinforcement considered for design. For clarity, the development length for compression for a hooked bar is defined as the depth to the outer surface the hook (i.e., the embedment length considered for the development of the hook under tension).

Piers, caissons and drilled piers are considered a flexural members and closed stirrup criteria (as opposed to column tie criteria) is considered acceptable.

C9.4.2 Transfer of Pier Forces

When a pier is transferring overturning moment to a slab, the strength of the slab in flexure and shear must be adequate for the transfer. The requirements for the transfer are in addition to the requirements for the development of pier longitudinal reinforcing bars or anchor rods into the slab to keep them from pulling out of the slab. The criteria for the transfer of pier forces to a slab was adopted from ACI 318.

The potential failure modes for the transfer of pier forces to a slab include: local slab failure due to the concentration of flexural stresses in the slab at the pier/slab interface due to the concentrated pier moment being transferred to the slab, punching shear through the slab due to the combination of a pier moment and an axial force being transferred to the slab, slip or pull out of pier reinforcing inadequately developed above and below the pier/slab interface or pull out of inadequately developed anchor rods into the slab.

For a pole foundation with the pier in the center of the slab, the pier moment transferred by flexure is resisted by two slab moments. Depending on the direction of the applied overturning moment, one slab moment is supplied by the top layer reinforcement on one side of the pier and by the bottom layer on the other side (1/2 moment on each side, similar to a simply supported beam with a concentrated moment applied at center span). The pier moment being transferred to the slab by flexure is assumed to be distributed over a limited width, hence the 1.5(slab) width limitation in ACI 318. The slab moment due to the transfer of a pier moment by flexure is a local connection consideration and is not intended to be additive to the internal slab moment determined due to resisting soil pressure. The larger of the calculated moment per unit of width is intended to be used for determining horizontal reinforcement requirements for the slab. In the same manner, punching shear stresses due to the transfer of pier moment and axial force are not combined with the shear stresses determined due to resisting soil pressure (e.g., one-way shear).

For a latticed tower on a mat, the slab is not continuous on one side of the pier for a pier moment applied in a direction normal to the edge of the slab. For this condition, the slab moment conservatively can be assumed to be resisted on only one side. For many latticed tower foundations, the magnitude of the local slab moment from the transfer of pier moment by flexure is small and does not govern over the slab moment due to resisting soil pressure; however, the slab horizontal reinforcing bars must be adequately developed beyond the inner face of the pier towards the free edge of the slab to resist the local slab moment from the transfer of pier moment by flexure.

For the investigation of punching shear in a latticed tower supported on a slab, the shear strength for an edge or corner condition may govern even when the critical section towards the free edge of the slab lies between the face of the pier and the free edges. This may occur due to the reduction in strength specified by ACI 318 for an edge or corner condition when the critical section is extended to a free edge for an edge condition or adjacent free edges for a corner condition.

For uplift conditions, a pier is considered as a cracked concrete section and the geometry of the pier reinforcing or anchor rods developed into the slab is used for determining effective sections for investigating punching shear.

C 9.4.3 Direct Embed Foundations

The moment in the shaft of a direct embed pole increases below grade until the point of zero shear occurs which depends on the lateral soil resistance distribution along the embedment depth.

Precast embedded foundations that utilize a slip splice for the upper tubular steel pole are subjected to crushing forces at the surface of the concrete at the top and bottom of the slip splice length and shear forces from the internal couple due to the transfer of the pole overturning moment to the precast concrete section. The overstrength factor specified in Section 2.7.9 applies to the portion of the foundation over the length of the slip splice. As required for longitudinal reinforcing bars used to develop anchor rods as specified in Section 9.6, the longitudinal reinforcement in a precast embedded foundation is required to be fully developed in tension within the slip splice length. The slip splice length is generally governed by concrete shear, flexure and crushing strengths and the minimum slip splice length specified for steel poles in Section 4.9.7.1 does not apply.

C9.4.3.1 Effective Foundation Diameter

The effective width of a direct embed foundation for the purposes of determining embedment depths and internal foundation shear and moments are a function of the rigidity of the backfill. Concrete backfill is considered rigid and treated as a drilled pier with a diameter equal to the diameter of the concrete annulus surrounding the embedded pole. The concrete backfill is not effective in adding strength to the foundation. Gravel backfill is assumed to act as a shim between the embedded pole and the undisturbed soil with a reduced effective diameter compared to when concrete backfill is utilized. Backfill is not considered to contribute to the effective foundation diameter when soil or other material is used as backfill. The mid-depth diameter of the embedded pole is used for simplicity when determining effective foundation diameters. The criteria specified is considered as best practice based on the experience of the committee with direct embed poles designed in accordance with previous revisions of the Standard.

C9.4.3.2 Corrosion Control

Direct embed steel poles require additional corrosion control provisions for the installations as specified in Section 9.4.3.2.1. The protection is required to extend above grade to provide a tolerance for the actual embedment depth and to provide corrosion control for vegetation. The coating is required to have a feathered edge at the termination of the coating as opposed to an abrupt termination of thickness. Abrupt coating terminations have proven to lead to coating failure due to rain draining down from the upper pole surfaces gradually leading to debonding of the coating.

Coatings that are not UV protected will deteriorate with extended exposure to sunlight. In these cases, an additional exterior coating is required that provides UV protection. Some coatings only discolor under extended exposure to sunlight and the physical properties of the coating do not degrade. In this case, an additional exterior coating is not required unless the color or appearance of the coating was selected for landscaping or other aesthetic purposes.

C9.4.3.2.1 Direct Embed Steel Sections

Galvanized steel poles are considered to provide adequate corrosion control except for the conditions specified. When concrete backfill is extended above grade level, temperature steel is required in the concrete to prevent spalling and cracking of the concrete which leads to moisture entrapment and the premature corrosion of the pole.

Ground sleeves are required to be continuously welded at the top and bottom of the ground sleeve to prevent moisture and other corrosive substances from being entrapped between the ground sleeve and the pole wall. Although ground sleeves are not intended to be considered as contributing to the strength of the pole, the longitudinal seam weld requirement for the pole is also required for the ground sleeve to prevent cracking of the seam and becoming a potential entry point for moisture and other corrosive substances. Venting of ground sleeves is required to prevent the rupture of the ground sleeve during the hot-dip galvanizing process due to entrapped moisture between the sleeve and the pole wall converting to steam when dipped into the galvanizing kettle.

C9.4.3.2.2 Direct Embed Precast Concrete Sections

Precast concrete foundations are also subject to damage due to corrosion and are required to be in conformance with this section.

C9.6 Anchorages

Anchor rods are considered as compact elements and a plastic analysis is allowed when their design strength is not governed by concrete strength.

Anchor rods with minimum specified tensile strengths higher than 120 ksi should not be utilized due to possible embrittlement resulting from galvanizing.

High strength anchor rods (i.e., with specified minimum yield strengths greater than 55 ksi) are required to meet minimum Charpy V-Notch requirements to result in adequate toughness based on anticipated installation methods and to provide adequate energy absorption under earthquake loading. The testing criteria was adopted from the referenced ASTM specifications.

Longitudinal reinforcing bars in piers or caissons used to develop anchor rods in tension are required to be fully developed within the specified potential breakout surface. This was specified to provide adequate energy absorption under earthquake loading. The maximum distance from the anchor rods to the reinforcing bar arrangement was specified to avoid a breakout surface forming without engaging the longitudinal reinforcing bars.

Shrinkage and temperature steel is required in large diameter piers or caissons to avoid concrete cracking between the anchor rods and the perimeter of the foundation.

C9.6.2 Deformed Anchor Rods

The embedment lengths specified for deformed anchor rods under tension were adopted from the referenced ASCE 48 Standard.

C9.6.3 Headed Anchor Rods

The side-face blowout and pullout strengths from the ACI 318 Specification are considered appropriate concrete strength limitations for anchor rods connected to a properly developed embedment plate. Other anchor rod arrangements are required to also conform to the other provisions of ACI 318 Chapter 17.

Smooth anchor rods with a headed end (e.g., a heavy hex nut) placed near the bottom of a slab have very limited strength under compression when leveling nuts are utilized for the base plate connection of the structure. Embedded nuts or plates located at the upper ends of the anchor rods may be required to provide adequate strength for compression and to avoid a punching shear failure at the base of the slab.

Concrete pryout is not considered a potential failure mode for anchor with embedment depths greater than 25 times their nominal diameter.

C9.7 Design Strength of Soil or Rock

The design strength of soil or rock is based on limit states design criteria. When allowable strengths are specified in a geotechnical report, a 2.0 factor of safety is considered a conservative value to determine nominal strengths when the geotechnical report does not report the factor of safety used to determine the reported allowable strengths.

A 0.75 resistance factor is specified for soil or rock except for determining design strength for the following where lower resistance factors are required: bases of guyed masts, foundations with only a single anchoring device resisting tension and for foundations utilizing non-battered piles with tapered cross sections. The bases of guyed masts are subjected to long term loading from the weight of the mast and the down pull of the guys justifying a more conservative, lower resistance factor compared to foundations subjected to short term loading. A more conservative, lower resistance factor for foundations utilizing a single anchorage device to resist tension is justified due to the probability of complete collapse in the event of a single component in the foundation. Non-battered tapered piles resisting tension are more susceptible to pull out compared to other pile types and is justification for a more conservative, lower resistance factor.

The lateral stiffness of soil is a significant parameter in determining the embedment depth of deep foundations such as drilled piers and caissons subjected to lateral loading. The stiffness of soil is considered as a strength with considerable variability due to the nature of soil. Using a resistance factor applied to soil stiffness was not considered best practice by the committee as it would be analogous to using a resistance factor applied to the modulus of elasticity for the structure. As an alternative to using a resistance factor applied to stiffness, factored reactions are increased by dividing the reactions by a resistance factor of 0.75 and used for the lateral load analysis. Internal foundation shears and moments from the analysis are then multiplied by 0.75 and used for the strength design of the foundation.

The weight of overburden soil that determines the nominal strength or stiffness of soil are also not multiplied by a resistance factor. The appropriate resistance factor is applied to the calculated nominal strength from the overburden weight to determine the design strength.

C9.8 Seismic Considerations

Special considerations are required in high seismic areas which are considered to be areas where the earthquake spectral response at short periods is greater than 1.0.

C9.8.1 Independent Foundations

Latticed self-supporting towers supported on independent foundations can experience significant increases in member forces due to differential lateral movements during an earthquake. Grade beams are required to minimize differential displacements between foundations in high seismic areas except for the alternatives specified that can be considered to provide equivalent lateral resistance to differential displacements.

C9.8.2 Longitudinal Reinforcement

The requirements for fully developing longitudinal pier reinforcement are intended to provide ductility and the energy absorption capability assumed for earthquake design.

The orientation of the free ends of hooked pier reinforcing bars is not considered critical for providing the assumed ductility for the seismic design of structures in accordance with Section 2.0.

Grouted reinforcing bars are required to be tested to 125% of their yield strength as grouted bars are considered rigid elements and subjected to stresses above their yield strength during an earthquake as the structure displaces and absorbs energy during an earthquake.

C9.8.3 Transverse Reinforcement

Stirrup splices are required to be staggered in order to provide the assumed ductility and energy absorption assumed for the seismic design of structures in accordance with Section 2.0.

Piles, piers or caissons supporting a pile cap or mat are subjected to significant lateral forces during an earthquake due to the significant discontinuity in stiffness at their interface with the pile cap or mat and are required to meet the more stringent ACI 318 requirements specified.

C9.8.4 Batter Piles

Batter piles, subjected to lateral loading, are considered to be rigid elements compared to non-battered piles and may be subjected to stresses above their calculated pile reactions from the loading combinations specified in Section 2.0. Designing the piles for their strength as a short column minimizes the potential for premature pile failure during an earthquake.

C9.8.5 Precast Concrete Foundations

Precast concrete foundations for poles are expected to be subjected to the flexural yield strength of the supported pole as the pole absorbs energy during an earthquake.

C10.0 PROTECTIVE GROUNDING

C 10.1 Scope

The requirements of Section 10.0 are intended to provide a minimum level of protection for the structure and foundation when site-specific grounding is not specified in a procurement specification. The adequacy of all grounding systems should be verified based on site-specific conditions at the time of installation. Additional protection may be required for site-specific conditions or for equipment supported on the structure.

Site-specific grounding specifications are required for AM tower installations as the structure is energized. Other site conditions such as rock at or near the ground surface also require site-specific protective grounding.

C10.3 General

The maximum 10 ohm resistance requirement is based on providing protection for personnel coming in contact with the structure. Site-specific conditions or local code requirements may require a different resistance requirement.

C10.4.1 Materials

The materials specified were determined by consensus of the committee based on the performance of grounding systems provided based on previous revisions of the Standard. The requirements are intended to meet most local building code requirements and to minimize galvanic corrosion due to dissimilar metals in the grounding system and the structure.

C10.4.2 Grounding System Configuration

The ground rod configurations specified is intended to meet the 10 ohm maximum ground resistance and reasonably reduce the grounding impedance to earth due to power surges such as lightning. Additional grounding components may be required based on site-specific conditions.

The requirement for no portion of the grounding system passing through a concrete foundation is based on the experience of the committee with damage to foundations attributed to lighting. Site-specific grounding specifications may require connections to the foundation reinforcement, which with sufficient connections, can provide an efficient path for the discharge of electrical energy to the earth.

C11.0 OBSTRUCTIVE MARKING

The consensus of the committee was to provide a reasonable tolerance for the termination of color bands for obstruction marking. Using panel points of a latticed structure is a convenient and cost-effective location for the termination of a color band as bracing members would be colored with the same color along their entire length.

C12.0 CLIMBING FACILITIES

C12.1 Scope

The structures covered in the scope of the Standard have unique climbing facility requirements. The criteria presented in Section 12.0, unless otherwise indicated, were adopted by consensus of the committee based on experience with structures installed in accordance with previous revisions of the Standard. and with reference to fall protection standards for other similar industries (e.g., IEEE standard 1307 for utility structures).

Fall protection anchorages are often required at the location of appurtenances mounted on a structure or for installation or maintenance purposes and are required to be included in the procurement specification for a structure. Fall protection anchorages are required at obstructions to a climbing facility unless a warning sign is installed. The strength requirements in limits states design format are specified in Section 12.4. Although minimum strength requirements for the top anchorage of safety climb systems are specified in Section 12.4, the safety climb systems are considered as an appurtenance and strength requirements for the components of the safety climb system are not within the scope of the Standard. More stringent strength requirements for the top anchorage may be required for site-specific climbing facilities.

C12.3 General

Cable safety lines are available in many different types that can vary slightly in diameter and construction (e.g., wire rope and strand with different numbers and types of individual strands making up the safety line). The cable safety line material can also vary (e.g., galvanized steel, stainless steel, etc.). Safety sleeves are tested with specific safety lines and may not perform properly or may inadvertently detach from the safety line when used with a different type of safety line. To help ensure proper performance, safety sleeves are required to be marked to identify compatible safety line sizes and types. A metal identification tag is required to be affixed to the base of a structure indicating the size and type of the cable safety line installed to allow climbers to verify the safety line will be compatible with their safety sleeve.

The initial tension in a cable safety line can be critical for the proper performance of a cable safety climb system and is required to be specified by the safety climb system manufacturer.

Step bolts are not intended for use as a fall protection anchorage which have more stringent design loading requirements in accordance with Section 12.4.

Maintenance work is often performed at many elevations of a structure by skilled climbers and the requirement of installing toe boards is not considered justified unless required by an owner of a structure (e.g., due to repetitive use of a platform by unskilled climbers).

Climbing facilities are subjected to damage due to shipping, handling, installation, extreme loading events, maintenance, etc. and are required to be inspected prior to use in accordance with the ANSI/ASSP 10.48 standard referenced in Section 12.1.

C12.4 Strength Requirements

Step bolts installed in a step bolt clip are required to installed with the end of the step bolt contacting the supporting member in accordance with Section 12.6. This requirement is intended to minimize the moment applied to the clip due to a moment reducing couple created by the step bolt contact with the supporting structure. The magnitude of the couple is not well defined due to variables involved (e.g., the friction coefficient between the end of the step bolt and the supporting structure). Due to these variables, the full moment from the design force applied to the step bolt in accordance with Section 12.4 is conservatively required to be resisted by the step bolt clip.

The design loads on the side rails of a climbing facility (e.g., a ladder) are not considered to occur simultaneously with the design loads on the individual rungs or steps intermittently attached to the side rails.

Horizontal members around the sides or ends of a platform are not intended to be used as handrails and are most often provided to support antennas or other appurtenances. A minimum load is specified for installations with minimal attachments to a support rail. Site-specific installations may require more stringent strength requirements.

Strength requirements for the top anchorage of safety climb systems are minimum requirements. The requirements from an owner for specific safety climb systems may exceed the minimum requirements specified and must be included in the procurement specification for a structure.

The strength requirement for fall protection anchorages is intended to be used by a qualified licensed professional engineer to design or certify the strength of a fall protection anchorage and as such considered as an engineered or certified anchorage which are terms commonly used in the Industry.

C12.5 Dimensional Requirements

More stringent dimensional requirements are specified for climbing and working facilities used by authorized climbers (i.e., Class A vs. Class B climbing and working facilities that are limited for use with higher skilled competent climbers per Table 12-1).

The minimum and maximum dimensions of rungs are intended to minimize climber fatigue during climbing.

Providing minimum clear spaces and clearances at rungs, steps and step bolts is onerous for many climbing facilities for the structures covered by the Standard and is not considered justified for facilities exclusively used by competent climbers.

C12.5.1 Step Bolts

The minimum step bolt diameter is specified for the purpose of minimizing climber fatigue. The clear width requirement is intended to accommodate the width of a climber’s boot. The maximum horizontal spread between the attachment points is considered the maximum practical spread for repetitive climbing on a large diameter supporting structures (e.g., a tubular pole).

Step bolt and clip material specifications are provided to result in uniformity in the Industry that will provide a minimum level of toughness considering the abuse step bolts can be subjected to during and after installation.

Step bolts may be used as fall protection anchorages when the strength requirements from Section 12.4 for fall protection anchorages are satisfied. Using plates attached between the step bolt clip and the outer step bolt double nut connection to the clip is commonly used as a fall protection anchorage when the strength of the plate, clip and step bolt meet the required strength requirements.

C12.5.1.1 Latticed Structures

The criteria specified was established based on the best practice obtained from professional climbers.

C12.5.1.2 Pole Structures

The criteria specified was established based on the best practice obtained from professional climbers.

C12.6 Step Bolt Installation Requirements

Refer to the commentary for Section 12.4 regarding the basis of the requirement for the end of a step bolt making contact with the supporting structure. The outer nut pretensioning is intended to prevent the step bolt connection from loosening after installation.

Step bolts may be reused if not tightened beyond their yield strength. This criterion is allowed to be determined when a step bolt nut can be freely run up and down the entire length of the step bolt threads by hand and an inspection is performed by a competent climber.

Welding is not allowed due to concerns over embrittlement of the step bolt.

C12.7 Climber Attachment Anchorages

It was not considered practical to require an engineering analyses of attachment anchorages commonly used by climbers. Annex I is intended to provide examples of climber attachment anchorages that do not require engineering analyses prior to use when approved by a competent climber after inspection.

C13.0 PLANS, ASSEMBLY TOLERANCES AND MARKING

C13.2 Plans

The information required in Section 13.0 is not only necessary for the proper installation of a structure but also for the building permit process and subsequent engineering analysis of structures for changed conditions (e.g., for the addition of appurtenances).

C13.3 Tolerances

The tolerances for the initial installation are based on the consensus of the committee for tolerances obtainable using best installation practices that would not significantly impact the strength or functionality of structures designed in accordance with the Standard. The tolerances are not intended to apply to a structure after being exposed to loading or during solar distortions (e.g., tubular pole displacements due to expansion of one side compared to the opposite side due to solar exposure).

C13.3.5 Slip Splice

The strength of a slip splice depends on the pole sections at a splice being in firm contact regardless of the slip splice length obtained when installing the upper section over the lower section. Slip splices, due to manufacturing tolerances, may result in an upper pole section meeting the minimum slip splice length during initial fit up without jacking; however, the strength of the splice will be compromised if jacking of the joint is not performed to obtain firm contact throughout the slip splice length.

The slip splice length after jacking is required to meet or exceed the minimum slip splice length specified in Section 4.9.7.1 without a tolerance for the minimum length. No tolerance is given for the maximum slip splice length; however, longer slip splice lengths will reduce the height of the pole and will impact the overall height tolerance specified in Section 13.3.1. Excessive slip splice lengths may also result in the upper section coming into contact with a weldment or other attachment on the lower section preventing the top section from coming into firm contact with the lower section during the jacking process. In this case, the obstruction must be moved, and the jacking procedure repeated.

When the minimum slip splice length is not obtained after jacking into firm contact, the strength of the sections at the splice are assumed to linearly decrease in strength down to 50% of the design strength for a slip splice length ratio equal to 1.0. Slip splice length ratios less than 1.0 are considered excessively sensitive to variations in the geometry of the pole sections which may result in unpredictable reductions in strength (e.g., due to local buckling at the ends of either section at the joint). Local buckling may occur due to the concentrated lateral forces from the couple required to resist the overturning moment at the splice.

C13.3.8 Take-Up Devices

The take-up adjustment lengths specified are intended to provide adequate adjustment for guy initial tensions throughout the life of the structure for structures installed meeting the tolerance in Section 13.0.

C13.4 Marking

The minimum height of markings was considered as best practice by consensus of the committee to enable reading the part numbers from various positions during installation and maintenance and after multiple years of exposure to weather.

C14.0 MAINTENANCE AND CONDITION ASSESSMENT

C14.3 Intervals

The intervals recommended represent the consensus of the committee based on the performance of structures designed in accordance with previous revisions of the Standard. Site-specific management plans for structure may require more or less frequent intervals.

C14.4 Guy Anchor Shafts

Steel anchor shafts in direct contact with soil require a corrosion management plan established by the owner of a structure based on site-specific corrosion conditions. Refer to Section 5.6.6 and Annex H for additional corrosion control methods, conditions prone to accelerated corrosion and requirements for Risk Category II, III and IV structures.

C15.0 EXISTING STRUCTURES

C15.1 Scope

Section 15.0 pertains to structures as opposed to appurtenances supported on structures. Refer to Section 16.0 for appurtenance mounting systems.

Changed conditions may require modifications to a structure. Minor changes do not require modifications to a structure in order to be in conformance with the current Standard. Section 15.3 defines changed conditions which require an evaluation of the structure in accordance with the current Standard. Based on the evaluation, Section 15.8 defines the minimum strength requirement for modifications when modifications are required. All evaluations and modifications must conform to the current Standard regardless of the revision of the Standard used for the design of the original structure or the last modification.

Existing structures without changed conditions, designed in conformance with a previous revision of the Standard, are not required to be evaluated for conformance to the current Standard.

The criteria specified in Section 15.5 assume that a structure, including modifications, have been properly designed, installed and maintained in accordance with the current Standard or a previous revision of the Standard in effect at the time of construction.

The evaluation of construction loads for modifications and related means and methods are not within the scope of the Standard. The scope of Section 15.0 is limited to the evaluation of an existing structure for changed conditions and to the design of modifications when modifications are required.

C15.3 Changed Conditions Requiring an Evaluation

A feasibility study that investigates the overall stability of a structure and the strength requirements for the main load carrying members is permitted to be used for the evaluation of changed conditions. A comprehensive structural analysis, that further evaluates strength requirements for connections, anchorages and foundations is required to identify and design the modifications required for a changed condition. All evaluations are required to be performed in accordance with the current Standard regardless of the revision of the Standard used for the design of the original structure or subsequent modifications.

C15.4 Risk Category`

The risk category must be determined in accordance with the current Standard regardless of the risk category considered for the original design of the structure or for subsequent modifications. The risk category may change compared to the original design or latest modification considering the current use of the structure regardless of nature of the proposed changes.

C15.5 Evaluation of Changed Conditions

The evaluation of a changed condition is based on the increase in strength requirements from a baseline appurtenance loading condition. When any strength requirement increases by more than 5%, the change is considered significant, and the structure and foundation must conform to the current Standard requiring modifications where necessary for conformance. Risk Category IV structures may be subjected to a service level agreement for a hardened network which may specify lower increases in demand-capacity ratios as a threshold for requiring modifications.

The baseline appurtenance loading is **not** defined as the current appurtenance loading on the existing structure. The baseline appurtenance loading is defined as the appurtenance loading used for the latest analysis of the structure that indicated conformance to the revision of the Standard in effect at the time of the analysis. The baseline appurtenance loading, regardless of the current loading configuration, would be either the appurtenance loading considered for the original design of the structure, or the appurtenance loading considered for the latest modification. For the purpose of determining the increase in strength requirements for a changed condition, the baseline loading is required to be analyzed in accordance with the current Standard, regardless of the revision of the Standard used for the original design of the structure or for the latest modification.

The proposed appurtenance loading for the evaluation of a changed condition includes all existing appurtenances supported on the structure intended to remain and all proposed additional appurtenances The analysis for the proposed appurtenance loading is required to be analyzed in accordance with the current Standard, regardless of the revision of the Standard used for the original design of the structure or for the latest modification.

The above approach is intended to prevent unjustified incremental approvals of changed conditions. For example, consider one appurtenance is proposed as a changed condition and found not to result in a 5% demand-capacity ratio increase from the existing appurtenance loading. At a future date, another appurtenance could be proposed as a changed condition and found not to result in a 5% strength increase from the existing appurtenance loading (which included the appurtenance for the first changed condition). Using this method, there could be unlimited revisions of appurtenances when considered incrementally.

It is important to recognize that although demand-capacity ratios from the analysis of a proposed appurtenance loading may exceed 1.0, modifications are not required unless a demand-capacity ratio for the proposed appurtenance loading exceeds the demand-capacity ratio for the baseline condition by more than 5% (i.e., the proposed change is significant). The 5% threshold value was determined to be a reasonable value by consensus of the committee based on the multiple assumptions involved with determining both loading and member strengths and that the construction risks associated with modifying a structure could be significantly greater than the risk of structural damage or collapse attributed to the additional appurtenances.

When a changed condition is considered significant or when documentation is not available for establishing a baseline appurtenance loading, structures are required to meet the current Standard using the proposed appurtenance loading with modification as required to result in all demand-capacity ratios less than or equal to 1.05 based on a comprehensive structural analysis.

Annex S presents an acceptable alternative for the analysis of a changed condition based on target reliabilities (refer to the commentary for Annex S). Use of Annex S may result in less modifications for Risk Category II structures, especially when documentation is not available for establishing a base line appurtenance loading.

C15.6.2 Foundation Analysis

Significant risk may be involved in determining the details of a foundation such as dimensions, concrete reinforcing, material strengths, etc. The consensus of the committee was to allow the use of documented design reactions used for the design of an existing foundation to determine if an existing foundation would be adequate for a proposed appurtenance loading. The justification was based on the low risk of foundation damage or failure due to the magnitude of the changed condition (i.e., less than a 5% increase in reactions) and the multiple conservative assumptions involved with conventional foundation designs.

The ASD conversion factor of 1.35 was determined by consensus of the committee considering the nominal safety factors used in allowable stress design compared to the nominal load and resistance factors used in limit states design. A lower bound conversion factor that could be used for any loading combination was desired in order to result in a conservative comparison to limit state reactions. A load factor of 1.6 for wind loading multiplied by a directionality factor of

0.85, used for limit state designs before the publication of the ASCE 7 ultimate wind speed maps, rounded off to 1.35 was selected to represent a reasonable conversion factor.

C15.6.3 Structural Analysis Report

Assumptions are often necessary in order to perform a structural analysis within a reasonable time frame or to allow feasibility and cost studies prior implementing proposed changed conditions or modifications. It is required to include all assumptions in the structural analyses report and identify critical assumptions that must be verified prior to implementation.

In accordance with Section 15.5, feasibility studies are limited to evaluating if a proposed change is significant and therefore requires the structure and foundation to conform to the current Standard. When a feasibility analysis report indicates such a condition, the report must state the requirement of completing a comprehensive structural analysis prior to implementation of the changed condition.

C15.7 Exemptions

The exemptions specified are based on the consensus of the committee where requiring conformance to the current Standard is either not possible (e.g., requirements involving actions prior to installation) or would represent an unjustified risk for implementation on existing structures.

15.8.1 Design

In accordance with Section 15.5, modifications to a structure are not required for a changed condition when demand-capacity ratios from an analysis of the changed condition using the current Standard do not increase by more than 5% above the demand-capacity ratios from an analysis of the baseline appurtenance loading using the current Standard. Although the increase in demand-capacity ratios may not exceed 5%, there is no limit on the magnitude of the demand-capacity ratios for the changed conditions; however, when the increase is greater than 5%, modifications are required to result in magnitudes of demand-capacity ratios no greater than 1.05. It is important to recognize that the criteria for determining if modifications are required is based on the percent increase in demand-capacity ratios (i.e., demand-capacity ratio threshold) whereas the criteria for the design of modifications when modifications are required is based on the magnitude of the demand-capacity ratios for the changed condition using the current Standard.

C16.0 APPURTENANCE MOUNTING SYSTEMS

C16.1 Scope

Appurtenance mounting systems are considered as separate structures from the supporting structure. Unique criteria pertaining to appurtenance mounting systems that are in addition to the criteria specified for supporting structures are presented in Section 16.0.

C16.4 Strength Limit State Load Combinations

Mounting frames are governed by download and lateral loads and therefore the loading combinations from Section 2.0 pertaining to minimum dead load conditions are not required to be considered.

C16.4.1 Sector Mounts and Integral Mounting Systems

Sector mounts and integral mounting systems are more sensitive to download loads compared to the supporting structure. The dead load only loading combination from ASCE 7 with a 1.4 load factor was adopted as an appropriate loading combination. The 1.2 dead load factor from Section 2.0 was adopted for dead load in combination with other loads.

Maintenance vertical downloads may be a significant loading for sector mounts and integral mounting systems. The 1.5 load factor was adopted from the previous revision of the Standard (Rev G) for climbing facilities. Maintenance loads are considered at each mounting point to provide a minimum level of strength for the installation of antennas or other appurtenances. A nominal wind load is assumed to occur simultaneously with the loads applied during construction or maintenance.

Vertical loads are required to be applied at the center of horizontal members and at the ends of horizontal cantilevered members to provide a minimum level of strength to support climbers during construction.

C16.4.2 Side Arms and Standoffs

Side arms and standoffs vary from very lightweight mounting systems to robust mounting systems. Many lightweight systems are not intended for use in maintenance or climbing activities and requiring a minimum level of strength for maintenance or climbing is not justified by the committee. When used for such purposes, side arms and standoffs must be investigated based on a rigging plan in accordance with the TIA-322 and ASSP A10.48 Standards.

C16.5 Analysis Models

Members in mounting systems are often subjected to bending and require the use of three-dimensional beam elements for analysis.

The strength of mounting systems can be significantly impacted by the type of connections to the supporting structure as well as the strength of the members of the supporting structure

C16.5.1 Application of Forces to Structural Models

The transfer of forces from appurtenances to a mounting system can be complex and can significantly impact the strength requirements of the components supporting the appurtenances.

C16.6 Wind and Ice Loads

The response of mounting systems to gusts is different than the response of supporting structures. The impact of direction and wake interference is also different compared to supporting structures. The design values specified were determined by consensus of the committee as best practice for mounting systems based on the performance of mounting systems designed in accordance with previous revisions of the Standard.

C16.6.1.2 Mounting Systems

The wind loads for mounting systems are dependent on a complex set of variables. A simplified method was adopted by the committee with the intent of providing a uniform, practical approach based on the performance of mounting systems designed in accordance with previous revisions of the Standard.

The considerations for determining the design wind loads on mounting systems differ from the criteria for determining the design wind loads on the supporting structure (e.g., wake interference effects for an individual side arm compared to wake interference effects for the supporting structure from multiple sectors mounted on the supporting structure at the same relative elevation).

C16.8 Design Strength of Members

Horizontal members supporting mounting pipes or appurtenances are subjected to torsion. Due to the minimal torsional strength of open cross sections, such members are required to be investigated in accordance with the AISC Specification.

C16.9 Existing Appurtenance Mounting Systems

The maintenance loads specified in Section 16.4.1 are intended for new structures. Existing mounting systems may have a limited capacity to support maintenance loads and must be investigated on a site-specific basis if used as part of a rigging plan.

The load modification factors for wind load from Annex S are based on target reliabilities for supporting structures and are not applicable to mounting systems.

C16.11 Assembly Documents

Information pertaining to the determination of effective projected areas of mounting frames, and other information related to loading, are required by users, other than the manufacturer, to select appropriate mounting frames for specific applications and for the strength investigation of the supporting structure.

Mounting system are utilized with a wide variety of supporting structures. Using appropriate attachment hardware is critical for the proper performance of the mounting system.

Mounting systems that utilize struts or tie-backs require information regarding the acceptable range of strut or tie back angles to the members of the mounting system and to the supporting structure. Angles outside the range may significantly reduce the strength of the mounting system.

C17.0 SMALL WIND TURBINE SUPPORT STRUCTURES

C17.1 Objective

The criteria defining small wind turbines was adopted from the IEC Standard referenced in Annex U.

The dynamic interaction between an operating wind turbine, wind fluctuations and the response of the supporting structure and foundation are complex. The wind turbine system consists of the turbine, the supporting structure and the foundation. Section 17.0 establishes design methods for the supporting structure based on criteria provided by the turbine manufacturer. Many supporting structures to date have been designed using design criteria from a variety of standards and building codes, most of which were not originally intended to cover the complex design issues involved with small wind turbine support structures. Prescriptive design criteria are included in Section 17.0 based on the consensus of the committee. The Standard is anticipated to further develop and evolve through an iterative process as more research and knowledge is gained through the continued growth in the SWT industry.

Due to the wide range of operating conditions of different wind turbines from different wind turbine manufactures and the wide range of structure types (i.e. latticed self-supporting towers, pole structures and guyed masts), the design methods used for large wind turbine support structures involving extensive modeling, testing and third-party certification are not feasible for SWT support structures. Section 17.0, in combination with other sections of the Standard, are intended to provide a straight forward, unified method for the design and analysis of SWT support structures based on information provided by the turbine manufacturer. Due to the complexities involved, more stringent maintenance and condition assessments are specified compared to antenna supporting structures.

Many small wind turbine manufacturers rely on manufactures of antenna supporting structures for structures to support their wind turbines. Fatigue damage has proven to be the most significant issue with existing SWT support structures and was the justification for the development of a standard for SWT support structures and including it in the TIA-222 Standard.

C17.3.1 Design Criteria

The design criteria is based on a limit state approach for fatigue loading and extreme wind, ice and earthquake events. Criteria for the investigation of operating loading conditions, other than tip displacements and natural frequency modes, are not considered to govern the supporting structure requirements unless otherwise specified by the turbine manufacturer.

Small wind turbines less than 22 square feet of rotor swept area are commonly placed on antenna supporting structures. Wind turbines of this size are not considered large enough to present significant cyclic loading. Due to their small size, a 1.0 wake interference factor is specified. The 22 square feet rotor swept area is also used by the IEC Standard referenced in Annex U as the size of turbines that are not required to be designed as a system consisting of the turbine, the supporting structure and the foundation.

The masts supporting any size turbine is required to conform to Section 17.0 as fatigue may govern the design of the mast due to the relatively small strength requirements for the mast due to extreme wind, ice and earthquake loads.

C17.3.2 Turbine Model

For the extreme wind, ice and earthquake loading conditions, a turbine is modeled as a simple appurtenance, similar to an omni-directional antenna for an antenna supporting structure. The turbine is considered to be in a parked condition for extreme wind, ice and earthquake loading conditions. Unless otherwise specified by the turbine manufacturer, the vertical and lateral loads from the turbine may be applied at the hub height of the turbine without offsets from the vertical centerline of the turbine base connection. When a horizontal offset of the turbine mass is specified by the turbine manufacture, the resulting overturning moment from the weight of the turbine is conservatively required to be considered additive to the overturning moment from the applied lateral loads.

For fatigue loading, an equivalent constant-range fatigue turbine load is used with an equivalent constant-range fatigue wind load to determine equivalent constant-range fatigue stresses in the members and components of the supporting structure to be compared with their fatigue thresholds.

C17.4 Turbine Manufacturer Data

The minimum information required for the design or analysis of a SWT supporting structure is specified. Additional information may be required for specific turbines.

C17.6 Extreme Wind Condition

The wind turbine is considered to be in a parked position during an extreme wind loading event due to the safety features of the turbine shutting down its operation under high wind speeds.

The minimum 140 mph ultimate wind speed is intended to result in adequate sizes of members and components of the supporting structure and foundation that will result in cyclic stresses below their fatigue thresholds for all but the components designated as either Category 1 or 2 components in Section 17.12.4.1. The use of a minimum basic wind speed for investigating the extreme wind condition is justification for limiting the requirement of a fatigue analyses to only Category 1 and 2 components that have relatively low fatigue thresholds. The equivalent constant-range fatigue stresses in other members and components (e.g. connection plates and fasteners) are assumed to be less than their fatigue thresholds.

The minimum basic wind speed becomes overly conservative as the effective projected area of the turbine becomes a smaller percentage of the total effective projected area supported by the structure. The minimum basic wind speed may be reduced to 114 mph when the effective projected area of the turbine is less than 10% of the total effective projected area supported by the structure.

The 140 mph wind speed was adopted based on the performance of existing SWT supporting structures that were designed for a 50-year return period basic wind speed equal to 110 mph in order to minimize fatigue damage. The 114 mph lower bound wind speed was adopted by consensus of the committee as best practice based on a 90 mph 50-year return period basic wind speed.

The IEC Standard classifies small wind turbines according to design wind speeds for the turbine. The IEC wind speeds specified for a given classification of a wind turbine is intended to represent wind speeds for many different sites as opposed to a specific site and are based on the reliability requirements for the turbine as opposed to the supporting structure.

C17.7 Extreme Ice Condition

As for the extreme wind loading condition, the wind turbine is considered to be in a parked position for the extreme ice loading condition due to the magitude of wind speeds asssumed to occur simultaneoulsy with ice.

Ice loading on a turbine is a function of a complex set of variables and is far from an exact science. The consensus of the committee was to provde a simplified consevative method for determining the weight and projected area of ice accumlation for the extreme ice loading combination.

C17.8 Extreme Earthquake Condition

Although a earthquake may occur during normal operating conditions, the operational loads are considered insiginficant compare to the loads from the extreme earhtquake loading condition.

C17.9 Critical Turbine Moments

Critical turbine moments, when specified by a turbine manufacture, are required to be considered as a limit sate loading condition simultaneous with a 40 mph basic wind speed unless otherwise specified by the manufacturer. The 40 mph basic wind speed was considered by the committee to be a reasonable upper bound wind speed to occur simultaneously with a critical turbine moment.

The consensus of the committee was to consider critical moments as live loads that vary based on the Risk Category of the structure. It was the consensus of the committee to use the effective importance factors in Annex L used for the conversion of 50-year return period wind speeds to wind speeds based on risk categories. The 1.6 load factor used in conjunction with the specified importance factors result in the same conversion ratios used for the wind speed conversion table listed in Note 1 of Annex L (e.g. for Risk Category III, the square root of the quantity 1.6(1.15) = 1.36).

For torsional moments, a counterclockwise rotation in plan view was chosen by the committee to result in a unified approach for analyses and design of the supporting structure.

C17.10 Stiffness Requirements for Top Mounted Turbines

Maximum lateral displacement and torsional rotation values are provided for top mounted turbines based on the experience of the committee. Larger displacements or rotations may increase fatigue loading due to additional secondary stresses in the supporting structurer. In addition, larger displacements or rotations may be noticeable by the general public resulting in calls of concerns to the owner.

The minimum offset of the turbine represents a minimum eccentricity for the turbine weight which may be the significant contributing factor to lateral displacements considering the relatively low wind speed considered for the service load condition.

Other displacement or rotation limitations may be specified by the turbine manufacture (e.g. for turbine blade clearances).

Displacement and rotation limitations must be provided by the turbine manufacture for side mounted turbines.

C17.11 Dynamic Requirements

The first three natural frequency modes are considered modes that may resonate with a turbine. Due to the importance of avoiding resonance, simplified methods of determining mode frequencies are not permitted. Modes determined from models that do not include the foundation are required to be adjusted. The +/- 0.10 Hertz adjustment value was determined by consensus of the committee as a reasonable estimate of the impact the properties of the foundation may have on modal frequencies. When frequency ranges are provided by the turbine manufacture, it is assumed that the ranges are based on a variety of anticipated foundation conditions for multiple site installations.

Torsional mode frequencies are assumed to be higher than the third mode. Proper modeling of the locations of the turbine components is critical when torsional modes require investigation.

The wind turbine system according to the IEC Standard referenced in Annex U consists of the turbine, the supporting structure and the foundation. The proper design of the wind turbine is the responsibility of the turbine manufacture. Requirements for the structure necessary for the proper operation of the system must be specified by the turbine manufacturer.

C17.12 Design for Fatigue

The investigation for fatigue is based on analyzing a structure under an equivalent constant-range wind speed and turbine loading condition. The objective of the investigation is to determine if the stresses based on an elastic analysis exceed the fatigue threshold values for the components of the structure.

The equivalent constant-range loading is intended to represent the complex variable amplitude loading experienced by the structure over its lifetime. The fatigue loading does not represent an actual magnitude of load expected and the results of the fatigue analysis do not represent the magnitude of stresses expected in the components of the structure for a given loading condition. The stress range has no sign (e.g. neither tension or compression for a latticed tower leg). The stresses from the fatigue analysis define a range of stress (i.e. from a minimum to a maximum stress, that when considered to occur for an infinite number of cycles can be used for the purposes of investigating fatigue (i.e. equal to a dynamic time history fatigue analysis using the actual variable loading applied to the structure over its lifetime).

The equivalent constant-range loading may appear to be high and unrealistic to occur for an infinite number of cycles; however, the equivalent constant-range load must account for high stress cycles that occasionally occur over the lifetime of the structure which disproportionally decrease the fatigue life of the structure compared to normal operating loads. These higher stress cycles significantly reduce the fatigue life of the structure and are accounted for by using a higher constant-range loading compared to normal operating conditions.

Because the range of stresses from a fatigue analysis are considered to be applied for an infinite number of cycles, the stresses can be compared to the range of stress (i.e. fatigue threshold) determined by modeling and testing, that can be applied to a given component for an infinite number of cycles without forming or propagating a fatigue crack. An infinite number of cycles is assumed to be possible when an increase in cycles does not reduce the magnitude of the cyclic stress range that results in a fatigue crack.

The nominal stress in a component determined from an elastic analysis using the equivalent constant-range loading specified in Section 17.12.1 and 17.12.2 are intended to be compared to the fatigue thresholds specified in Section 17.12.4. Stress concentration factors, unless otherwise specified, are not required to be applied to nominal stresses determined from a fatigue analysis as fatigue thresholds represent the nominal stress range below which fatigue cracks are not expected to form or propagate.

Only components listed in Section 17.12.4.1 require investigation. Components not listed in Section 17.12.4.1 are considered to have sufficiently high fatigue thresholds when their design strengths meet or exceed the strength requirements for the extreme wind loading condition specified in Section 17.6.

The turbine and the structure act together as a system. The turbine manufacture is required to provide more stringent requirements when required for the proper performance of the system.

C17.12.1 Equivalent Constant-Range Fatigue Wind Loading on Supporting Structure

Variable wind speeds occur in combination with variable turbine loading over the life of the supporting structure. The equivalent constant-range fatigue turbine loads specified in Section 17.12.2 were derived from the simplified design equations presented in Annex G of the IEC reference listed in Annex U. The IEC turbine loads are based on a wind speed range equal to 1.4 times the annual average wind speed based on the IEC SWT turbine class. The highest IEC average annual wind speed for a SWT class is equal to 10 m/s. Using the 1.4 factor and converting to mph, results in a wind speed range equal to 31 mph. The consensus of the committee was to conservatively specify a 30 mph wind speed range for determining constant-range fatigue loads for both the supporting structure and the turbine.

Because the 30 mph wind speed represents a range in wind speed, all adjustment factors, other than wind directionality and importance factors, do not apply and are set equal to 1.0. The wind directionality factor is justified because the maximum stress range in a component of the structure does not occur for all wind speed directions. For example, for a pole structure, the location of the maximum stress range at a base plate rotates around the pole circumference as the wind direction changes. For a triangular cross section latticed tower leg at a flange plate, the maximum stress range generally only occurs for two wind directions normal to the opposite face of the structure (one towards the tower and one away from the tower).

The equivalent constant-range fatigue loading is an empirical value and the importance factor is intended to increase the reliability of the structure based on its risk category.

The importance factors for fatigue turbine loads in Table 17-1 were derived from the average annual wind speed ratios for the four IEC SWT classes (i.e. 22,19,17 and 13 mph). The consensus of the committee was to use the ratios between the wind speeds for the SWT classes for the ratios between importance factors with an exception that a 0.90 importance factor be used for Risk Category I structures. This required using an annual average wind speed of 14.4 mph instead of 17 mph as the denominator for determining the ratio of importance factors (i.e. the ratios of the IEC average annual wind speeds to 14.4 mph were used as the importance factors specified in Table 17-1 rounded off in order to not imply a higher degree of accuracy).

The wind speeds occurring under turbine operational conditions generating fatigue loading are generally low justifying the use of subcritical flow force coefficients to determine wind forces for the fatigue loading condition.

C17.12.2 Equivalent Constant-Range Fatigue Turbine Loads

The equivalent constant-range fatigue loads for horizontal axis turbines were derived from the simplified design equations presented in Annex G of the IEC reference listed in Annex U and are intended to be used for designs meeting the strength requirements for the extreme wind condition specified in Section 17.6. The derivations of the equations for fatigue loads are presented below. Refer to Annex G of the IEC reference for commentary on the source of the IEC criteria for fatigue loads.

The simplified equations are based on the assumption that a turbine cycles between 50% and 150% of its AWEA electrical power rating during operating conditions. This assumption conveniently allows the range of loading to be set equal to the turbine loading at its power rating (i.e. 150% -50% = 100%). The consensus of the committee was to conservatively assume this range of loading occurs for an infinite number of cycles and be considered as the equivalent constant-range fatigue turbine loading to occur in combination with the equivalent constant-range fatigue wind loading on the supporting structure.

The equivalent constant-range fatigue turbine horizontal force (parallel to the x-axis illustrated in Figure 17-1, F_{xt}) is based on a force coefficient (i.e. drag factor) equal to 1.5 times the power coefficient of a turbine under operating conditions. A 0.35 power factor is assumed resulting in a 0.53 force coefficient. The force coefficient is applied to the swept area based on the rotor diameter (D_r). For a 30 mph uniform wind representing the power generation operating range, the term C_{fxt} was derived as follows:

$$C_{fxt} = 1.5(0.35)(0.00256)(30)^2(0.7854) = 0.95 \text{ rounded up to } 1.0$$

A 0.85 wind directionality factor and the importance factor from Table 17-1 are used to determine the equivalent constant-range fatigue turbine horizontal force (F_{xt}). Refer to Section C17.12.1 regarding the basis of the 30 mph wind speed range and the use of wind directionality and importance factors.

The effective projected area of a turbine determined in accordance with Section 17.5 is based on the horizontal thrust under an extreme wind loading condition with the turbine in a parked stationary position and is not used to determine equivalent constant-range fatigue loadings.

The equivalent constant-range fatigue turbine overturning moment (i.e. pitch moment about the y-axis illustrated in Figure 17-1, M_{ty}) is based on the eccentric weight of the rotor and the eccentric lateral thrust from the rotors. The overturning moment from each of these components is conservatively assumed to occur in the same direction.

The overturning moment load range due to the eccentric rotor weight, conservatively assuming a wind direction range of 180 degrees, is equal two times the static overturning moment determined using the horizontal distance from the vertical centerline of the supporting structure to the center of the rotor mass.

The overturning moment load range due to the eccentric lateral trust from the rotors is assumed to equal the static moment determined using the magnitude of the constant-range fatigue turbine horizontal force times an eccentricity equal to 1/12 times the rotor diameter. A 0.85 directionality factor is applied only to the overturning moment load range due to the eccentric rotor weight as a directionality factor is included in the equation for the constant-range fatigue turbine horizontal force (F_{xt}). Importance factors are applicable only to loads dependent on the wind velocity and is included in the equation for F_{xt}.

The equivalent constant-range fatigue turbine rotor shaft torsion (i.e. roll moment about the x-axis illustrated in Figure 17-1, M_{tx}) is based on rotor torque and the rotating eccentric rotor weight. The torsion from each of these components is conservatively assumed to occur in the same direction.

The rotor shaft torsion due to rotor toque is equal to the turbine electrical power generation divided by the rotational rotor speed. The turbine power generation range can be determined from the following equation for a 30 mph wind speed representing the power generation operating range:

$$\text{Power} = (0.35 \text{ power factor})(0.00375)(30)^3(0.7854)(D_r)^2 = 27.8(D_r)^2 \text{ ft-lb/sec}$$

The rotational rotor speed (N_r) provided by the turbine manufacturer expressed in RPM must be converted to radians per second for determining the rotor shaft torsion range as follows:

$$\text{Rotational rotor speed, radians/sec} = N_r(2\pi)/60$$

The rotor shaft torsion range due to rotor torque can be expressed as follow:

$$\text{Rotor torque} = [27.8(D_r)^2](60)/[N_r(2\pi)] = 265(D_r)^2/N_r \text{ rounded up to } 275(D_r)^2/N_r \text{ ft-lbs}$$

In order to accommodate SI units, the term C_{mtx} is used as the multiplier applied to the square of the rotor diameter.

The rotor shaft torsion range due to the rotating eccentricity rotor weight is equal to two times the static moment determined using the weight of the rotor times an eccentricity equal to 0.0025 times the rotor diameter.

A 0.85 wind directionality factor is applied to both terms of the rotor shaft torsion range. The importance factor from Table 17-1 is only applied to rotor torque as it is a function of wind speed.

The unit direction vector for shaft torsion is specified to result in consistency in fatigue analyses. The shaft torsion M_{tx} represents a cyclic range of loading and as with all turbine fatigue loadings, does not represent a load from a specific direction.

Yaw moments (about the z-axis illustrated in Figure 17-1) are considered to result in low stresses and occur with a limited number cycles and are therefore not required to be considered for investigating fatigue strength unless otherwise specified by the turbine manufacturer.

The weight of a turbine is considered as a static load and an eccentricity to the turbine base is not required to be considered for a fatigue analysis.

Simplified equations for determine fatigue loading are not feasible for vertical axis turbines and must be provided by the turbine manufacturer.

C17.12.3 Fatigue Analysis

Fatigue loads are considered as wind loads and must be applied in the directions that result in the maximum responses in accordance with Section 2.6.11.

C17.12.3.1 Self-Supporting or Bracketed Structures

Static loads do not contribute to fatigue damage and therefore a load factor of zero is specified for dead loads. Fatigue loads are considered as operating condition loads occurring over an infinite number of cycles and therefore a 1.0 load factor is specified for all fatigue loads. Because all fatigue loads represent a range of loading (i.e. cyclic loads occurring in alternating directions), the stresses in all components are to be considered as absolute values to be compared to the fatigue thresholds specified in Section 17.12.4.

C17.12.3.2 Cantilever Portions of Guyed Masts

The cantilever portion of a guyed mast is considered as a self-supporting structure for the investigation of fatigue strength.

C17.12.3.3 Guyed Masts bellow the Cantilever

Guyed masts require multiple analyses due to the effect of dead loads and initial guy tensions subjecting members below the cantilever to compression. A fatigue analysis must consider dead loads and guy tensions in order to properly model the response of a mast to fatigue loading. The stress in members under the initial tension condition must be known in order to determine the total change in stress in a member due to fatigue loads. Members subjected to cyclic loads, but remain in compression, are not required to be investigated for fatigue strength.

C17.12.3.3.1 Latticed Masts

The stress in leg members due to dead loads and initial guy tensions acting alone must be known in order to determine the change in stress in a leg member due to fatigue loads. Leg members subjected to cyclic loads, but remain in compression, are not required to be investigated for fatigue strength. This can be assumed to occur in leg members that remain in compression for all fatigue loading directions.

When a leg member is subjected to tension for any fatigue loading direction, dead loads and initial guy tensions are not adequate to prevent the leg member from being subjected to tension. The total change in stress in the leg is required for comparison to the fatigue threshold of the leg member, regardless of whether a portion of the change in stress occurred while the member was subjected to compression. This requires the magnitude of the compression stress in the leg member due to the initial tension condition to be added to the tensile stress in the member from the fatigue analysis.

The stress in bracing members under the initial tension condition is considered negligible and the bracing forces from the fatigue analysis can be used to determine the required fatigue strengths.

The wind directionality factor applied to fatigue loads in Sections 17.12.1 and 17.12.2 accounts for the fact that the magnitude of the change in stress for a given leg member due to fatigue loads varies for each wind direction.

C17.12.3.3.2 Tubular Pole Masts

The stress in tubular mast members due to dead loads and initial guy tensions acting alone must be known in order to determine the change in stress in a pole section due to fatigue loads. Pole sections subjected to cyclic loads but remain in compression over the entire cross section for all fatigue loading conditions, are not required to be investigated for fatigue strength.

When a pole section is subjected to tensile stress for any fatigue loading direction, dead loads and initial guy tensions are not adequate to prevent the pole section from being subjected to tensile stresses. The total change in stress in the pole section is required for comparison to the fatigue threshold of the pole section, regardless of whether a portion of the change in stress occurred while the pole section was subjected to compression stress. This requires the magnitude of the compression stress in the pole section due to the initial tension condition to be added to the tension stress in the pole section from the fatigue analysis.

The wind directionality factor applied to fatigue loads in Sections 17.12.1 and 17.12.2 accounts for the fact that the magnitude of the change in stress at a given location due to fatigue loads varies for each wind direction.

C17.12.4 Fatigue Thresholds (ΔF_{TH})

When the strength requirements for the extreme wind condition are satisfied for a structure, only the main load carrying members defined as Category 1 or 2 Components specified in

Sections 17.12.4.1 and 17.12.4.2 are required to be investigated for fatigue strength. Fatigue strength investigations are also required at locations of weldments or openings on poles sections in accordance with Sections 17.12.4.3 through 17.12.4.6 according to the severity of the stress concentration or notch sensitivity of the detail. Fatigue strength investigations are required for anchor rods in accordance with Section 17.12.4.7.

The fatigue threshold values specified are based on the AASHTO reference in Annex U and other international standards and modified by consensus of the committee for application to the structures covered within the scope of the Standard. Fatigue thresholds represent the range of stress reversals that may be applied an infinite number of times without generating a fatigue crack in a member or propagating a fatigue crack from a weld defect that meets the quality requirements of AWS D1.1 for members subjected to cyclic loading.

The consensus of the committee was not to require a fatigue strength investigation for details with fatigue thresholds of 10 ksi and above when the strength requirements for the extreme wind condition specified in Section 17.6 are satisfied.

The nominal stress is permitted to be evaluated at the nominal elevation of the detail defining the fatigue threshold to avoid creating excessive nodes with small distances between nodes on the structural model.

Welded attachments to latticed tower legs not exceeding 2 inches in length measured along the longitudinal axis of the leg are considered to have a fatigue threshold equal to or greater than 10 ksi.

Internal flange plates are considered to result in a higher stress concentration compared to external flange plate and are required to be considered as a Category 2 Component. The higher stress concentration occurs due to the reduced bolt circle and reduced stiffness resulting from little or no extension of the flange plate beyond the exterior of the pole wall. Fatigue thresholds for exterior flange plates are specified in Section 17.12.4.6.

Bracing members with effective slenderness ratios less than 60 have relatively high buckling strengths required to satisfy the extreme wind loading condition and may also have correspondingly higher stresses from the fatigue loading condition compared to bracing members with higher effective slenderness ratios.

C17.12.4.3 Welded Attachments to Tubular Pole Structures

Welded attachments less than 2 inches in length along the longitudinal axis of a tubular pole section are considered to have a fatigue threshold equal to or greater than 10 ksi. The fatigue threshold values in Table 17-3 were adopted from the AASHTO Specification referenced in Annex U with the exception of the 2.6 ksi fatigue strength for attachments greater than 1 inch in thickness which was based on the consensus of the committee.

C17.12.4.4 Reinforced Hand-holes, Cutouts and Ports in Tubular Pole Structures

Fatigue strengths are required to be investigated directly above and below an opening as well as at the centerline elevation of the opening. In each case, the nominal stress in the pole wall, as opposed to the reinforcing, is used to determine nominal stresses for the fatigue investigation. Stress concentrations are applied to the nominal stresses in the pole wall in order to account for the size of the opening.

Elastic stresses are required for all fatigue strength investigations. The elastic section modulus for two orthogonal axes are required to determine the minimum elastic section modulus of the cross section with the opening. One axis passing through the opening (i.e. where the sides of the reinforcing are subjected to opposite signs of stress under bending) and an orthogonal axis (i.e. where both sides of the reinforcing are subjected to the same sign of stress under bending).

The roughness profile limitation of 1,000 microinches was adopted from the AISC Specification for fabrication requirement for components subject to cyclic loading. It was the consensus of the committee to consider the requirement satisfied for thermally cut holes that are ground smooth.

The geometry, location and minimum weld size limitations for reinforced openings were adopted from the AASHTO reference in Annex U.

C17.12.4.5 Unreinforced Hand-holes, Cutouts and Ports in Tubular Pole Structures

Fatigue strengths are required to be investigated at the centerline elevations of openings. The nominal stress in the pole wall is required to be based on the elastic section modulus of the cross section with the opening. The minimum section modulus occurs for the axis parallel to the width of the opening Stress concentrations are applied to the nominal stresses in the pole wall in order to account for the size of the opening.

The roughness profile limitation of 1,000 microinches was adopted from the AISC Specification for fabrication requirement for components subjected to cyclic loading. It was the consensus of the committee to consider the requirement satisfied for thermally cut holes that are ground smooth.

The geometry and location limitations for openings were adopted from the AASHTO reference in Annex U.

C17.12.4.6 Tubular Pole Structure Exterior Flange Plates

Fatigue thresholds are a function of the geometry of a flanged connection and the associated stress concentrations. Unless otherwise specified, fatigue thresholds apply to the nominal stress in the pole wall as opposed to the stress in the flange.

Equations for determining geometry coefficients are provided in Table 17-4. Geometry coefficients are used to determine stress concentration factors in accordance with Table 17-5. Stress concentration factor are then used to establish fatigue thresholds in accordance with Table 17-6. Generally, stress concentrations increase with the flexibility of a flanged connection and result in lower fatigue thresholds. Flanged connections meeting the limitations specified in Table 17-7 are permitted to have higher fatigue thresholds compared to other flanged connections that do not meet the limitations.

The limitation specified in Table 17-7 and the stress concentration ranges specified in Table 17-6 are based on AASHTO validity ranges applicable to the AASHTO fatigue thresholds and were adopted by the committee with some exceptions as noted herein.

For the determination of geometry coefficients in Table 17-4, when a value exceeds an AASHTO validity range, the AASHTO limit is required to be used in the geometry coefficient equations (i.e. values outside AASHTO validity ranges are not permitted to reduce geometry coefficients and result in lower stress concentration factors).

The minimum cope requirement was based on the consensus of the committee to avoid a triple point intersection of welds which can result in excessive stresses and cracking due to weld shrinkage from three orthogonal directions. Copes also serve as drainage slots for hot dip galvanizing.

The TIA committee limited the use of socketed flange plate connections to pole diameters 24 inches or less in diameter. The AASHTO Specification provides fatigue thresholds for socketed flange plate connections up to poles 50 inches in diameter.

Socketed connections provide a significant manufacturing cost saving for small diameter poles where access to the interior of the pole is limited for full penetration butt welded flange plate connections (e.g. access for backgouging or for the placement of backer bars). The majority of fatigue failures reported by members of the committee occurred for larger diameter poles with socketed flange plate connections. It was the consensus of the committee that the higher flexibility of socketed flange plate connections compared to full penetration butt welded flange plate connections was a significant contributing factor for the failures. The stiffness of a flange plate connection due to radial moments in the flange plate reduces as the pole diameter increases (refer to Annex Q). In addition, the flange plate extension inside the pole for butt welded flange plate connections adds to the stiffness of the joint and reduces the secondary stresses in the pole wall. In order to provide a minimum level of stiffness, a maximum center opening diameter is specified in Section 17.12.5.6 along with a requirement that the perimeter of the center opening not be closer than two times the flange thickness from the inside of the pole wall.

For poles greater than 24 inches in diameter, access to the interior of the pole is available and the lower cost savings of socketed connections was not considered justification by the committee to allow their use considering the increased risk of fatigue failure. The limitation of 24 inches for the maximum pole diameter for socketed joints is therefore less than the AASHTO maximum 50 inch diameter. It should be noted that the ASCE 48 Standard requires full penetration butt welded flange plate connections for all pole diameters.

The AASHTO Specification minimum diameter limitation for socketed joints with stiffeners was 24 inches. The TIA committee conservatively adopted the AASHTO fatigue thresholds for joints with stiffeners when full penetration butt welds were used vs. socketed connections, however, a minimum fatigue strength of 4.5 ksi was adopted as the minimum fatigue threshold for a butt welded joint.

The AASHTO Specification provides three stress concentration ranges for socketed joints with stiffeners with maximum values of 4.0, 6.5 and 7.7 associated with fatigue threshold values of 7.0. 4.5 and 2.6 ksi respectively. Because of the minimum 4.5 ksi fatigue threshold adopted by TIA for butt welded joints with stiffeners, only two stress concentration ranges are included in Table 17-6 with maximum values of 4.0 and 7.7 associated with fatigue threshold values of 7.0 and 4.5 ksi respectively.

The fatigue threshold values in Table 17-6 for joints that do not satisfy the limitation specified in Table 17-7 were established by the consensus of the committee.

The default inside corner bend radius for a polygonal section was adopted from the AASHTO default bend radius used or determining width-thickness ratios for the calculation of nominal bending strengths.

The treatment of drainage holes or slots in flange plates for galvanizing drainage or venting was established by consensus of the committee based on the performance of structures conforming to previous revisions of the Standard.

C17.12.4.6.1 Longitudinal Stiffener Interface with Flange Plates

The fatigue threshold criteria for flange plates with stiffeners was adopted from the AASHTO reference in Annex U. The fatigue threshold for stiffeners and their connection to the flange are applicable only to stiffeners greater than 0.5 inches in thickness. Thinner stiffeners are considered to have fatigue thresholds greater than or equal to 10 ksi.

C17.12.4.7 Anchor Rods

The stresses that occur during cyclic loading conditions are considered to occur in the elastic range. It was the consensus of the committee to ignore moment from shear forces for gaps less than the diameter of the anchor rod. For larger gaps, an inflection point is assumed at 0.65 times the gap dimension as assumed in Section 4.9.9 (refer to C4.9.9 for additional commentary). The fatigue strength of 7.0 ksi on the anchor rod tensile root area was adopted from the AASHTO Specification referenced in Annex U. The equations for determining the stresses in anchor rods are based on assuming an equivalent ring of steel and assuming that shear and moment conservatively occur from the worst case direction for a given arrangement of anchor rods in a symmetrical circular pattern.

C17.12.5.1 Connection Bolts for Turbine Bases

Bolted connections are required to be pre-tensioned due to the cyclic loading from the wind turbine.

C17.12.5.2 Anchor Rods

High strength anchor rods are required to meet the Charpy V-notch impact strengths specified in Section 9.6. A minimum degree of toughness is desired for anchor rods resisting cyclic loads to avoid a sudden brittle failure in the event a fatigue crack should form. Lower strength anchor rods are assumed to have adequate toughness (refer to C9.6 for additional commentary).

The fatigue threshold for anchor rods is based on installations with misalignments no greater than 1:40. The misalignment tolerance was adopted from the AASHTO Specification referenced in Annex U. When anchor rods are not installed in a vertical position, not only are additional bending stresses introduced into the anchor rods under tension loading but tightening of the top and bottom nuts introduce additional bending stresses, further reducing the fatigue strength of the anchor rods. Beveled washers minimize the additional stresses from the tightening of the top and bottom nuts.

C17.12.5.3 Latticed Structures

The requirements of Table 17-2 are based on the consensus of the committee based on experience with structures supporting small wind turbines. Bracing members with high slenderness ratios have been reported to resonate with the frequency of small wind turbines. Thin gusset plates have been reported to develop fatigue cracks due to out-of-plane deformations as bracing members are subjected to compression.

C17.12.5.4 Guyed Mast Guy Anchorages

The magnitudes of the guy forces attached to a guy anchorage vary independently from each other. Although the guy forces may be balanced about the shaft of the anchorage at the time of installation, changes in the guy forces during operating conditions will cause a cyclic rotational moment to occur at the connection of the guy connection plate to the anchorage shaft. Additional cyclic rotational moments are present due to the change in the magnitude and direction of the resultant guy forces under operating conditions. Fatigue failures have been reported at this connection for guyed masts supporting small wind turbines. The use of a pinned connection eliminates the rotational moment (other than due to friction in the pinned connection).

C17.12.5.5 Tubular Pole Structures

The minimum requirement of 8 sides for polygonal cross sections was adopted from the AASHTO Specification referenced in Annex U.

C17.12.5.6 Tubular Pole Structure Flange Plates

The minimum requirement of 8 connection bolts was adopted from the AASHTO Specification referenced in Annex U.

The requirements for a symmetrical pattern, the minimum nominal flange bolt or anchor rod diameter and the spacing limitations were based on the consensus of the committee.

Unsymmetrical arrangements and a large spacing between connections result in stress concentrations in the welded connection to a pole due to the transfer of stress from the perimeter of a pole section to the flange bolt or anchor rod. The spacing limitation of six times the thickness of the flange plate is intended to result in rigid plate behavior (refer to commentary below for the maximum distance from the center of the pole wall weld reinforcing to the bolt circle equal to three times the flange plate thickness). A radial bend line is assumed midway between connectors. A limitation of three times the flange plate thickness is used on each side of the yield line. Further research into rigid plate behavior may lead to relaxed spacing limitations. The not to exceed value of 15 inches was considered best practice by the committee. The minimum nominal diameter of flange bolts or anchor rods was considered best practice considering the minimum flange plate thicknesses specified in this section.

The use of pretensioned bolts to reduce cyclic stresses in the bolts to insignificance is recommended for flange-to-flange connections.

The minimum flange plate thicknesses specified based on pole diameter were adopted from the AASHTO Specification. The minimum thickness based on flange bolt or anchor rod diameter and their yield strength was considered best practice by the committee.

Partial penetration butt joints are not allowed as the gap between the end of the pole wall and the flange plate is normal to the tensile stresses in the pole wall and therefore may act as a significant stress concentration leading to cracking in the weld.

The maximum distance from the center of the pole wall weld reinforcing to the bolt circle equal to three times the flange plate thickness was based on the review of test data indicating when rigid plate behavior could be expected. Larger distance may result in flexible plate behavior. Bending of the flange plate introduces significant secondary stresses into the pole wall exceeding the fatigue threshold of the joint. For flange-to-flange connections, prying action may occur and result in significant increases in the connections bolt forces which may exceed their fatigue threshold. For base plate connections, the top and bottom nuts will be forced to rotate with the flange and introduce significant bending stresses in the anchor rods which may exceed their fatigue threshold. This limitation was adopted in lieu of the AASHTO validity range (refer to C17.12.4.6) for the maximum bolt circle-to-pole diameter ratio (equal to 2.5 per the AASHTO Specification).

The center opening diameter of a flange reduces the stiffness of the flange. In order to result in rigid plate behavior, the limitations specified were considered best practice by the committee. The limitations were not considered to interfere with the proper and efficient fabrication of butt welded joints including access for welding, inspection and hot dip galvanizing drainage and venting requirements. The limitations are more stringent than the AASHTO validity range specified in Table 17-7 for the maximum inside center hole diameter-to-pole diameter ratio equal to 0.90. Socketed connections are allowed for smaller diameter poles (refer to Section 17.12.5.6.2).

The minimum 3 times the wall thickness inside corner bend radius requirement was adopted from the recommendation from the American Galvanizers Association for avoiding cracking during the hot-dip galvanizing process due to strain hardening occurring from cold bending. The use of a larger bend radius in accordance with Table 17-7 may result in a higher fatigue threshold. The requirement for a uniform radius throughout the arc of the bend is intended to prevent the use of a smaller nose radius in a press to obtain the required bend angle based on the number of sides of a polygonal section.

For flanges with stiffeners, anchor rod and flange plate bolt forces result in both normal and parallel pole wall stresses. The design of the stiffeners must be based on a rational method to prevent buckling or tear-out in the pole wall and local buckling or rupture of the stiffeners.

The gap between a stiffener and the pole wall for fillet or partial penetration welds is parallel to the bending stress in the pole wall and is not considered to significantly reduce the fatigue threshold of the joint. A fillet weld must be continuous for hot-dip galvanizing. Using a transition radius on a stiffener at the pole wall or grinding of the wrap-around fillet weld are not allowed. This requirement was adopted from the AASHTO Specification to avoid reducing the fatigue threshold of the joint.

The backing ring criteria was adopted from the AASHTO Specification.

C17.12.5.6.1 Butt Welded Flange Plate Connections

Full penetration butt welds are required to obtain a minimum fatigue threshold as a lack of penetration would be normal to the stress in the pole wall representing a significant stress concentration. Socketed joints often have fatigue thresholds less than butt welded joints due to the inherent rigidity of butt joints compared to socketed joints. Fabrication cost saving benefit for using socketed joints for pole diameters greater than 24 inches were considered by the committee to not override the risk for the development of fatigue cracking. The majority of reported fatigue failures for pole structure base plates involved socketed joints. The rigidity of internal flanges is similar to exterior socketed connections. Refer to Section C17.12.4.6 for additional commentary.

Reinforcing fillet welds are required to reduce the stress concentration of the joint and increase the fatigue threshold. The use of unequal reinforcing fillet welds with a 30 degree termination angle with the pole wall may increase the fatigue threshold of a joint compared to the use of equal angle reinforcing fillet welds (refer to Table 17-7).

C17.12.5.6.2 Socketed Flange Plate Connections

Refer to Section C17.12.5.6.1.

C17.13.1 Extreme Loading Conditions

The resistance factors in Table 17-8 were derived from the IEC 614000-1 Standard, “Wind turbines-Design requirements”, Third edition, 2005-08 rounding off to the values tabulated.

C17.13.2 Fatigue Loading Condition

The survival probability, confidence levels and resistance factors in Table 17-9 were derived from the IEC 614000-1 Standard, “Wind turbines-Design requirements”, Third edition, 2005-08 rounding off to the values tabulated.

A limiting stress range based on 5 million cycles was adopted by the committee as best practice for materials that may not develop a constant fatigue threshold after a given number of cycles.

C17.14 Foundations

The requirement for full compressive soil bearing pressure over the full plan dimension of the foundation under operating conditions is intended to avoid reduction of the soil bearing capacity under repetitive soil pounding.

C17.15 Maintenance and Condition Assessments

Many factors can affect the performance of SWT supporting structures due to cyclic loads, turbine malfunctions, etc. that warrant shorter interval periods compared to antenna supporting structures. As experience is gained using the Standard and improvements made in futures revisions based on performance, it is anticipated that the recommended interval period may be increased.

C18.0 INSTALLATION

The erector of a structure is responsible for establishing a rigging plan for the installation of a structure and providing the necessary equipment and additional structural components for the proposed construction method.

The ASSP 10.48 Standard specifies the requirements for establishing a rigging plan and other means and methods criteria for a proposed construction or maintenance activity. The TIA-322 Standard specifies engineering criteria addressing strength and stability requirements based on the information provided in a rigging plan.

Appurtenances are often routinely added to an existing supporting structure. For safety reasons it is imperative that additions to the structure do not impair or damage a climbing facility.

C ANNEX A: USER AND PROCUREMENT GUIDELINES

The default values presented are believed to represent the parameters appropriate for the majority of structures within the scope of the Standard.

CA.2.0 Loads

The Standard represents a minimum standard and is not intended to address more stringent requirements that may exist in local building codes or standards or for unique site-specific locations.

CA.2.2 Risk Category of Structures

Previous revisions of the Standard up to Revision G were based on ASD criteria using a 50-year MRI for wind loading which corresponds with the default Risk Category II for limit states design.

CA.2.3.2 Strength Limit State Load Combinations

The minimum basic wind speed of 85 mph was derived from the minimum basic wind speed of 75 mph specified in Revision G of the Standard. The square root of the product of the Rev G load factor (1.6) and directionality factor (.85) is equal to 1.17. Applying this value to a 75 mph unfactored wind speed equates to 87 mph. This value was rounded down to 85 mph in order to not imply a higher degree of accuracy.

CA.2.4 Temperature Effects

The default reduction of 50 degrees F for loading conditions that include ice was adopted from Revision G of the Standard.

CA.2.6.4 Basic Wind Speed and Design Ice Thickness

Wind speeds specified in international standards are often based on averaging periods other than 3 seconds and must be converted for use with the Standard. Annex L provides conversion factors for the most common averaging periods used in other international standards. Ice thicknesses for use with the Standard must be based on a 500-yr MRI for use with the Standard.

The Standard does not provide design criteria for rime and in-cloud ice loading as conditions vary significantly in regions prone to this type of ice loading.

CA.2.6.5 Exposure Categories

The default Exposure C category was adopted from previous revisions of the Standard.

CA.2.6.6 Topographic Effects

Method 1 is useful when only the general location of a proposed structure is known. The height of the feature is required to determine topographic effects when the procurement specification requires consideration of terrains other than flat or rolling terrains. When the specific location is known for a structure such as for an existing structure, using Methods 2 or 3 can result in lower wind speed-up wind loading compared to Method 1.

Differences in the ground anchor supports for a guyed mast can significantly affect the forces in the guys and the members of the mast and are required to be addressed in the procurement specification.

CA.2.6.7 Rooftop Wind Speed-Up Factor

Structures supported on rooftops are subjected to higher wind speeds compared to ground supported structures. Specific information is required to be included in the procurement specification in order to properly model the wind escalation with height for the structure.

CA.2.6.8 Ground Elevation Factor

The default ground elevation factor of 1.0 is intended to correlate with the default design parameters from Revision G of the Standard. The ground elevation factor is intended to account for the change in air density at different elevations above sea level which affects the conversion of wind speeds into wind pressures.

CA.2.6.11.5 Transmission Lines Mounted in Clusters or Blocks

The intent of the Standard is to allow the designer of a structure to assume the most economical location of transmission lines unless specific criteria is included in the procurement specification.

CA.2.7.3 Seismic Load Effect Parameters

The default Site Class D was adopted from Revision G of the Standard.

CA.2.7.8 Structures Supported on Buildings or Other Structures

The default weight percentage specified is believed to represent the vast majority of structures within the scope of the Standard. For higher weight percentages, the determination of an appropriate amplification factor becomes complex and is a function of the combined structural characteristics of the supported and the supporting structure.

CA.2.8 Serviceability Requirements

The serviceability requirements specified are considered minimums for the proper performance of communication structures. More stringent serviceability requirements may be required depending on the type of antennas or equipment supported on a structure. The default requirements specified were adopted from Revision G of the Standard. Stiffness requirements for SWT support structures are specified in Section 17.10.

CA.5.6.6 Guy Anchorages (Corrosion Control)

The default condition of non-corrosive soil was adopted from Revision G of the Standard. Corrosion control requirements are dependent upon subsurface conditions and other factors such as the expected life of a structure and site-specific management plans for inspection and maintenance. Justification of the default condition by the committee was based on the consensus that additional corrosion control measures may be retrofitted when necessary by the owner in consultation with a corrosion control expert.

CA.5.6.7 Ground Embedded Poles (Corrosion Control)

The default condition of non-corrosion soil for guy anchorages was adopted for ground embedded poles. Refer to CA.5.6.6 for additional commentary.

CA.7.5 Guy Dampers

Refer to Annex M for additional information regarding wind induced oscillations.

CA.9.0 Foundations and Anchorages

The default clay soil type and frost depth were adopted from Revision G of the Standard.

The proper development of anchorages is a function of the type of foundation, concrete strength, edges distances, etc. and are best determined by the foundation engineer. Due to the many variables involved with roof-mounted structures, pile caps, etc. specific requirements for anchorages to be provided with the procurement of a structure must be included in the procurement specification.

CA.10.0 Protective Grounding

Grounding requirements for equipment supported on a structure may require site-specific designs considering the soil conditions at the site. Grounding systems may be provided by the owner or included in the procurement specification for the structure.

CA.11.0 Obstruction Markings

The Standard assumes that obstruction marking, when required, will be provided by the owner unless the requirements are included in the procurement specification for the structure.

CA.12.0 Climbing Facilities

The default climbing facility classification was adopted from Revision G of the Standard assuming that only competent climbers will be climbing the structure.

CA.14.4 Guy Anchor Shafts

A corrosion management plan established by the owner is required as the potential for corrosion is highly dependent on site-specific subsurface conditions and may not become visible for years after installation.

CA.15.0 Existing Structures

The default risk category specified is the same as the default category for new structures.

Annex S provides load modification factors less than 1.00 for Risk Category II structures when a site-specific management plan for inspections is implemented by an owner. The use of Annex S must be addressed in the procurement specification for a structure.

CA.17.0 SWT Support Structures

The sections from Section 17.0 of the Standard that require information from the owner or the owner's representative are listed for developing a procurement specification for SWT support structures.

C ANNEX B: WIND, ICE, EARTHQUAKE AND FROST DEPTH MAPS

County listings from Revision G of the Standard have been eliminated for Revision H due to the development of the ASCE 7 online Hazard Tool specified in Section 2.6.4.

Use of the ASCE 7 online Hazard Tool allows access to the latest wind, ice and earthquake design criteria as more data is available to update the maps.

C ANNEX C: DESIGN WIND FORCE ON TYPICAL MICROWAVE ANTENNAS

Microwave antennas subject the supporting structure to local axial loads, side forces and twisting moments. Although these values are dependent on the geometry of the antenna, Annex C is required to be used in the absence of more accurate information per Section 2.6.11.2.

C ANNEX D: TWIST AND SWAY LIMITATIONS FOR MICROWAVE ANTENNAS

Due to the distance between microwave transmitting and receiving antennas, a signal can degrade significantly due to angular rotations of the structure (i.e., both twist and sway). Annex D provides minimum requirements for limiting angular rotations based on signal degradation as a function of dish diameter and frequency. The limitations are intended to be used in conjunction with the serviceability requirements of Section 2.8.

C ANNEX E: GUY RUPTURE

Annex E is provided for guyed masts when a procurement specification for a new structure or for the analysis of an existing structure includes the requirement to consider an additional loading condition involving a guy rupture. The criteria were developed by the committee based the review of other international standards. It is based on a simplified conservation of energy method intended to be used as an alternative to a more complex dynamic analysis.

C ANNEX F: PRESUMPTIVE SOIL PARAMETERS

Presumptive soil parameters were established by consensus of the committee based on the performance of foundations designed in accordance with previous revisions of the Standard. The parameters are allowed to be used in the absence of a site-specific geotechnical report for Risk Category I and II structures per Section 9.3. The design parameters are required to be verified prior to installation.

C ANNEX G: GEOTECHNICAL INVESTIGATIONS

Site-specific geotechnical investigations are required for Risk Category III and IV structures and is preferred for Risk Category I and II structures in accordance with Section 9.3. Annex G lists information that should be included in a geotechnical report for all structures. Additional information may be needed for site-specific conditions based on the geotechnical engineer's review of the proposed foundation type, location and reactions of the structure.

C ANNEX H: ADDITIONAL CORROSION CONTROL

The criteria presented in Annex H was established by consensus of the committee based on the performance of structures designed in accordance with previous revisions of the Standard. Additional corrosion control is required by the Standard in accordance with Sections 5.6.6 and 9.4.3.2.1. A corrosion management plan is required for guyed masts with guy anchor shafts in direct contract with soil in accordance with Section 14.4. In addition, Annex J requires these shafts to be assessed prior to climbing the structure in accordance with the corrosion management plan for the site prior to a maintenance and condition assessment. The use of Annex S requires a site-specific management plan which would include a corrosion management plan for guyed masts with anchor shafts in direct contract with soil.

C ANNEX I: CLIMBER ATTACHMENT ANCHORAGES

The examples of climber attachment points presented were established by consensus of the committee.

C ANNEX J: MAINTENANCE AND CONDITION ASSESSMENT

CJ.1 Maintenance and Condition Assessment

The assessment outline contained in Annex J were established by the committee of the committee based on the performance of structures designed in conformance with previous revisions of the Standard. The assessments listed are guidelines and may not be applicable to all site-specific structures. Other assessments may also be required for site-specific structures or locations.

CJ.2 Field Mapping

The requirements for field mapping are intended to provide the necessary information for completing an analysis of an existing structure or mounting system. Connection details are required in order to complete a comprehensive structural analysis in accordance with Section 15.0.

CJ.2.3 Tolerances

The tolerances specified were established by consensus of the committee based on the degree of accuracy required for a meaningful structural analysis and have proven to be reasonably obtainable using common practices available for the inspection and mapping of structures.

C ANNEX K: MEASURING GUY TENSIONS

The methods presented in Annex K can be assumed to result in the guy tension tolerance specified in Annex J.

The direct method using a dynamometer requires periodic calibration of the dynamometer to result in the necessary tolerance for guy tension measurements specified in Annex J. The pulse method and the tangent intercept indirect methods are mathematical solutions based the physical properties of a guy under tension. The shunt indirect method is based on calibrations to specific guy sizes and types and requires periodic inspections in order to result in the necessary tolerance for guy tension measurements specified in Annex J.

C ANNEX L: WIND SPEED CONVERSIONS

Wind speeds for use with the Standard must be based on a 3-second gust. Annex L may be used to convert 50-year return period wind speeds that are based on other averaging periods to equivalent ultimate 3-second gust wind speeds for use with the Standard for Risk Categories I through IV. The return periods listed with each risk category are the mean recurrence intervals used for the generation of the ASCE 7 basic wind speed maps adopted for use with the Standard.

The conversions assume a constant ratio of the 50-year 3-second gust wind speed to the 50-year wind speed for a given averaging period as illustrated below. The ratios were obtained from the Durst gust duration ratio graph presented in Figure C26.5-1 in the ASCE 7 commentary. The non-hurricane Durst ratios were conservatively used as opposed to using the ESDU hurricane ratios. When available, site-specific ratios may be used for the conversions.

To generate the Annex L table, the tabulated 3-second gust wind speeds were first converted to hourly mean wind speeds using a ratio equal to 1.51 per the Durst curve. For example, for a 100 mph 3-second gust wind speed, the equivalent hourly mean wind speed is equal to 100/1.51 or 66 mph.

The Durst curve can then be further used to convert the tabulated hourly mean wind speeds to equivalent wind speeds for other averaging periods. For example, per the Durst curve, the ratio of wind speeds for a conversion from an hourly averaging period to a 10-minute (600 seconds) averaging period is equal to 1.05. For the 66 mph hourly mean wind speed calculated above, the equivalent wind speed for a 10-minute averaging period would be equal to 66(1.05) or 69 mph.

Conversion from an hourly mean wind speed to a fastest-mile wind speed requires the averaging period of the equivalent fastest-mile wind speed to equal to averaging period assumed for selecting the conversion ratio from the Durst curve. An iterative approach was used for the generation of the tabulated fastest-mile wind speeds. For example, for the 66 mph hourly mean wind speed calculated above, the correct averaging period was found to be equal to 42 seconds. Per the Durst curve, the ratio of wind speeds for a conversion from an hourly averaging period to a 42-second averaging period is equal to 1.29. For the 66 mph hourly mean wind speed, the equivalent wind speed for a 42-second averaging period is equal to 66(1.29) or 85 mph. The averaging period for a fastest-mile wind speed is equal to the time for 1 mile of wind to pass an anemometer. For an 85 mph fastest-mile wind speed, the averaging period would be equal to 3,600/85 or 42 seconds which matches the assumption made to determine the 1.29 conversion ratio and the equivalent fastest-mile wind speed of 85 mph. The tabulated values of equivalent 3-second gust and fastest-mile wind speeds for a 50-year return match the values tabulated in Table C26.5-7 in the ASCE 7 commentary.

The equivalent ultimate 3-second gust wind speeds for use with the Standard for each risk category were obtained by multiplying the 50-year return period 3-second gust wind speeds by 1.18, 1.26, 1.36 and 1.41 for Risk Categories I, II, III and IV respectively. The conversion factors were derived using a 1.6 load factor and importance factors equal to 0.87, 1.00, 1.15 and 1.24 for risk categories I, II, III and IV respectively. The importance factors for Risk Categories I, II and III were adopted from Revision G of the Standard. The 1.24 importance factor for Risk Category IV was derived from Equation C26.5-4 from the ASCE7-10 commentary based on a 3,000-year return period (note that Equation C26.5-4 was not included in ASCE 7-16 as load factors are no longer applied to wind loading due to the publication of the wind speed maps based on risk category). The conversion factor for the equivalent 3-second gust wind speed for a given risk category is equal to the square root of the importance factor times a 1.6 load factor. For example, for Risk Category IV, the conversion factor is equal to the square root of 1.24(1.6) or 1.41.

C ANNEX M: WIND-INDUCED STRUCTURAL OSCILLATIONS

The structures within the scope of the Standard can be subjected wind-induced structural oscillations which can significantly reduce the life of a structure. Annex M is provided in order to explain this type of structural behavior while more research is completed to allow the inclusion of design criteria addressing this behavior in future revisions of the Standard. The information in Annex M is based on the consensus of the committee based on the performance of structures design in accordance with prevision revisions of the Standard and on the research available to the committee.

C ANNEX N: INITIAL CONSTRUCTION INSPECTION

Inspections during and after the initial construction process are critical to ensure the proper performance of structures designed in accordance with the Standard. The inspections included in Annex N represent the consensus of the committee based on the performance of structures designed in accordance with previous revisions of the Standard.

C ANNEX O: EXISTING STRUCTURES MODIFICATION INSPECTION

Inspections before, during and after the construction process are critical to ensure the proper performance of structures designed in accordance with the Standard. The inspections included in Annex O represent the consensus of the committee based on the performance of structures designed in accordance with previous revisions of the Standard.

C ANNEX P: TUBULAR POLE STRUCTURE WELD TOE CRACK EVALUATION

The criteria presented in Annex P is based on the consensus of the committee based on the performance of structures designed in accordance with previous revisions of the Standard. Weld toe cracks require evaluation. Annex P provides a credible approach to determine the severity of a weld toe crack and an appropriate repair procedure with time frames for completing the repairs.

Refer to Annex J for condition assessments of base plates, Annex M for situations that may lead to cracking, Section 17.0 for design and detailing criteria of base plates to minimize the potential for cracking and Annex Q for tubular pole rigid base plate design criteria to minimize secondary stresses which may result in the formation of toe cracks.

C ANNEX Q: TUBULAR POLE BASE PLATES

The criteria presented in Annex Q is based on the consensus of the committee based on the performance of structures designed in accordance with previous revisions of the Standard and the review of numerous flange plate tests and finite element models.

CQ.3.0 General

Rigid behavior of pole base plates is desired to minimize displacements due to bending in the base plate. Displacements result in secondary bending stresses being introduced into the pole wall, the pole weld to the base plate and the anchor rods. Double nut anchor rod connections (i.e., top nut and leveling nut, refer to Section 4.9.9) cause anchor rods to rotate as a base plate rotates introducing bending stresses into the anchor rods. These secondary stresses may lead to cracking and the eventual propagation under wind loading, which if undetected, may eventually result in the collapse of the pole structure.

The design criteria in Annex Q does not require the use of grout. The use of grout is not desirable due to the potential corrosion of anchor rods when proper drainage is not provided. Drainage holes placed in grout often become plugged over time. Grout is also difficult to place in a reliable manner for transferring loads to the foundation due to limited access for proper placement and the presence of large center openings in the base plate resulting in the absence of a back surface for packing grout.

Limitation for use of the method is necessary because of simplifying assumptions involved with the generation of the design equations presented. Refer to C17.12.4.6, C17.12.5.5 and C17.12.5.6 for commentary addressing the basis of the limitations presented In Annex Q that are unrelated to the generation of the design equations. Section 17.0 specifies more stringent requirements for base plates subjected to fatigue loading including. For example, the minimum number of 8 sides for polygonal cross sections specified in Section 17.12.5.5 is more stringent than the minimum number of 6 sides required for use with Annex Q. Because radial yield lines are included in the determination of the effective width resisting bending, Annex Q does not apply to base plates with radial slots extending to the free edge of the base plate from the anchor rod holes.

CQ.4.0 Design Criteria

Annex Q is based on a yield line approach restricting the geometry of the base plate connection and limiting the effective base plate widths to result in rigid plate behavior. The effective yield strength of the base plate is also limited to 60 ksi to ensure rigid plate behavior.

Anchor rod forces are determined using an equivalent ring of steel with correction factors to account for the higher anchor rod forces that may result when the actual arrangement of the anchor rods is used to determine anchor rod forces. The method using an equivalent ring of steel with correction factors assumes the worst-case orientation of the anchor rods for a given overturning reaction. Anchor rods fully developed into the foundation are assumed to progressively yield in the same manner as a compact pole (e.g., use of the pole plastic vs. elastic section modulus). The anchor rods are considered compact when their strength is not limited by their development into the foundation. The analogy to the pole is that the plastic section modulus only applies when local buckling strength does not govern the strength of the pole cross section. When the anchor rods are not fully developed into the foundation, the correction factors result in anchor rod forces based on the elastic section modulus of the equivalent ring of steel with no redistribution of anchor rod forces due to yielding.

The provision for the base plate shear strength to not be less than the design tensile strength of the pole is required to allow the base plate and anchor rods to develop their full strength without a premature shear failure in the plate. Refer to C4.9.9 for commentary regarding the importance of anchor rod tightening which is also required to allow the base plate and anchor rods to develop their full strength.

CQ.4.1 Anchor Rods

For the transvers bending direction, the moment arm resulting in bending is limited to 3 times the thickness of the base plate for rigid base plate behavior. For the radial bending direction, this equates to a maximum anchor rod spacing of 6 times the thickness of the base plate assuming equal and opposite moments occur in the plate on each side of the radial yield line with the load applied at the anchor rod locations. The 15 inch maximum spacing was considered to be best practice by the committee for properly detailed base plates.

The minimum number of anchor rods equal to 8 was considered to be best practice by the committee for properly detailed rigid base plates. The minimum 0.75 inch anchor rod diameter specified is based on the consensus of the committee based on the performance of pole structure designed in accordance with previous revisions of the Standard.

Refer to Section 9.6 for the minimum spacing between anchor rods to prevent splitting failures in a concrete foundation based on the diameter of the anchor rods. Anchor rods with leveling nuts are not considered torqued anchor rods justifying a smaller spacing between anchor rods to avoid splitting compared to torqued anchor rods.

CQ.4.2 Base Plate Thickness

The minimum base plate thickness based on anchor rod diameter was considered best practice by the committee and on recommendations of references reviewed by the committee. The 0.25 reduction for lower strength anchor rods was included to allow the use of 2 inch base plates (when all strength requirements are meet) with ASTM A615 18J anchor rods (2.25 inch diameter) which are commonly used for tubular pole structures.

Q4.3 Yield Strength

The limitation of the design yield strength equal to 60 ksi is intended to result in rigid plate behavior when higher yield strength base plates are utilized. The use of higher yield strength base plates may require special welding procedures and appropriate ductile mechanical properties for the base plate material to allow yielding and redistribution of bending stresses under limit state loading conditions.

CQ.4.4 Center Openings for Butt Welded Base Plates

The minimum center opening diameter equal to 30% of the pole diameter is based on the experience of the committee for not hindering the hot-dip galvanizing process. Additional holes or slots are not considered to impact the design of external base plates as the assumed yield lines are external to the pole. For internal base plates, the diameter of additional holes and the length of slots are required to be deducted from the length of the radial yield line (B_{er}) in the equation for the effective base plate width in Section Q6.2.

CQ.4.4.1 External Base Plates

The base plate material extending inside the diameter of the pole is ignored for strength calculations but contributes to the rigid base behavior of the base plate (e.g., beam action spanning across the pole diameter). The center opening limitations were based on the consensus of the committee based on correlations with existing pole structures which have performed successfully based on previous revisions of the Standard.

CQ.4.4.2 Internal Base Plates

Refer to CQ.4.4.1. The base plate material inside the anchor rod bolt circle for an internal base plate adds stiffness in a similar manner as the base plate material inside the pole diameter for an external base plate.

CQ.5.0 Anchor Rod Force

The maximum anchor rod force is used to determine the strength requirements of a base plate. Refer to CQ.4.0 for the method of calculating anchor rod forces. Correction factors are used to result in anchor rod forces using an elastic section modulus when the anchor rods are not fully developed into a foundation. In this case, there is only one anchor rod with the calculated anchor rod force (assuming the worst-case direction of the applied overturning moment). When a plastic section modulus is used, the assumption is that the anchor rod forces are distributed to adjacent anchor rods as yielding occurs in an anchor bolt. For the condition where the full capacity of the anchor rod group is not utilized (i.e., yielding may not occur), it is still acceptable to use the plastic section modulus to determine anchor rod forces. For this condition, the base plate would be capable of yielding and redistributing the anchor rod forces in the same manner as the anchor rods would upon their yielding.

The minimum strength requirement equal to 50% of the pole bending strength was adopted from the ASCE 48 Standard as best practice for pole structures governed by serviceability requirements.

CQ.6.0 Base Plate Bending

The equation for base plate thickness is based on the equality of bending strength $\phi[B_{eff}(t^2)]/4)(Fy)$ to the applied moment $[P_u(x)]$ and solving for the required plate thickness. The moment arm is equal to the distance to the centerline of the anchor rod as opposed to the face of the anchor rod nut. Using this approach, the equations derived using yield line theory can also be derived from a conservation of energy approach and also found to best match test data and finite element models.

CQ.6.1 External Base Plates

Refer to Figure Q-1. The extension beyond the bolt circle considered to be effective contributing to the effective radial yield line is equal to 3 times the thickness of the base plate. This limitation is analogous to the limitation based on stiffness for the distance between the effective outside pole diameter and the bolt circle and the spacing between anchor rods (refer to CQ.4.1).

The effective transverse yield line width is limited to the following: the length of a line connecting the midpoint between anchor rods on the bolt circle, 12 times the thickness of the base plate and a location of the transverse yield line no closer than the midpoint between anchor rod on the bolt circle and the outside effective diameter of the pole. When these limitations are exceeded, non-rigid behavior would be expected to result in premature failure of the anchor rods, the pole wall or the weld as a result of secondary stresses due to base plate bending. It should be noted that the maximum spacing between anchor rods from Q.4.1 governs over the limitation for the effective transverse yield line width of 12 times the thickness of the base plate.

Only the component of the effective radial yield line that is parallel to the transverse yield line contributes to strength.

CQ.6.2 Internal Base Plates

Refer to Figure Q-2. The extension towards the center of the pole cross section from the bolt circle considered to be effective contributing to the effective radial yield line is equal to 3 times the thickness of the base plate. This limitation is analogous to the limitation based on stiffness for the distance between the effective inside pole diameter and the bolt circle and the spacing between anchor rods (refer to CQ.4.1).

The effective transverse yield line width is limited to the following: the length of a line connecting the midpoint between anchor rods on the bolt circle and 12 times the thickness of the base plate. When these limitations are exceeded, non-rigid behavior would be expected to result in premature failure of the anchor rods, the pole wall or the weld as a result of secondary stresses due to base plate bending. It should be noted that the maximum spacing between anchor rods from Q.4.1 governs over the limitation for the effective transverse yield line width of 12 times the thickness of the base plate.

Only the component of the effective radial yield line that is parallel to the transverse yield line contributes to strength.

CQ.7.0 Base Plate Shear

The shear strength of the base plate is required to equal or exceed the tensile strength of the pole wall. This requirement assures that the yield lines in the base plate will be capable of forming prior to a premature shear failure in the base plate. The length for determining shear strength is conservatively considered equal to the perimeter of the centerline of the pole wall (i.e., equal to the length considered for the tensile strength of the pole wall) for external flanges. The length for determining shear strength would increase for locations towards the bolt circle.

For internal flanges, the length for determining shear strength decreases towards the bolt circle which is the location used for determining the shear strength of the base plate. The shear strength per unit length must therefore be larger than the tensile strength of the pole wall per unit length by the ratio of the effective inside pole diameter to the bolt circle diameter.

CQ.8.0 Socketed Connections

The proper fit-up of large diameter multi-sided pole cross sections into socketed flange plate connections is difficult due to the manufacturing tolerances associated with producing multi-sided pole cross sections. Significant gaps may occur at bend lines as well as along the flat sides of the cross section. Large diameter round cross section poles may have out-of-roundness tolerances that result in similar fit-up issues. It was the consensus of the committee to limit the use of socketed connections to pole diameters 24 inches or less. Refer to C4.9.10.1 and C17.12.4.6 and C17.12.5.6.1 for additional commentary. Socketed base plates connections are not allowed for internal base plate due to the difficulties producing quality fillet-welds on the inside of the pole.

The minimum insertion of a pole section into a base plate is 0.75 inches which was considered best practice by the committee. The maximum insertion would be dependent on the inner fillet-weld size.

The minimum inner and outer fillet-weld sizes are based on providing a strength per unit length equal to the tensile strength of the pole wall per unit length to allow the base plate and anchor rods to develop their full strength. Fillet-welds less than 3/16 inch are not considered structural welds. AWS D1.1 allows an increase in weld strength for fillet-welds loaded normal to the longitudinal weld axis which is allowed by the Standard for an outer fillet-weld when the strength of the inner fillet-weld is ignored. The increase is not allowed when both outer and inner fillet-welds are considered for strength due to the limited distortion capacity of transversely loaded fillet-welds preventing both welds from obtaining their ultimate strength values.

Requiring the outer fillet-weld strength to not be less than the inner fillet-weld strength was considered best practice by the committee for transferring the load from the pole to the base plate. The requirement of an unequal leg outer fillet-weld with a 30 degree termination angle was adopted from the AASHTO Standard to minimize the stress concentration at the fillet-weld leg termination on the pole wall.

The provision addressing gaps between the pole wall and the base plate was adopted from AWS D1.1.

CQ.9.0 Butt Welded Connections

Refer to CQ.8.0 for the restricted use of socketed connections.

Full-penetration groove-welds with unequal leg reinforcing fillet-welds are required to minimize the stress concentration at the fillet-weld leg termination on the pole wall. Partial penetration welds result in a lack of penetration normal to the direction of stress from the pole wall and result in significant stress concentrations at the root of the partial penetration welds.

The maximum through-thickness stress was adopted from the ASCE 48 Standard. The width for calculating the area per unit length for determining the through-thickness stress is equal to the pole wall thickness plus the inner and outer reinforcing fillet-weld leg sizes. This requirement is intended to prevent lamellar tearing due to weld shrinkage and/or normal stresses from the pole wall.

CQ.10.0 Base Plate Anchor Rod Holes

The edge distance for anchor rods must be adequate to develop the required shear loading based on the shear reaction resisted by an anchor rod. The minimum edge distances specified in Section 4.9.4 for structural connections are not applicable to base plates. The minimum edge distance considered as best practice by the committee for base plates is based on preventing a nut or washer from extending over the edge of the base plate.

CQ.11.0 Grouted Base Plates

Grout is not recommended due to difficulties with proper placement and drainage. Improper placement often leads to cracking and the eventual loss of bearing strength. Improper drainage often leads to accelerated anchor rod corrosion. Refer to CQ.3.0 for additional commentary. Use of grout may be required for correcting excessive projections of anchor rods; however, the bearing strength of the grout is required to be ignored for determining anchor rod tension and compression forces (refer to Section 4.9.9).

Leveling nuts are required per Section 4.9.9 as grout is not allowed to be considered to transfer anchor rod compression forces to the foundation. When smooth anchor rods with nuts or embedded plates at their base are utilized in pile caps, mats or other situations where the punching shear of the concrete below the anchor rods is minimal, an additional nut may be required to transfer anchor rod compression forces to the foundation due to the gap between the leveling nut and the top of the foundation.

C ANNEX R: ASSUMED MATERIAL STANDARDS

The material standards associated with members and components of an existing structure are not always available for the analysis of a changed condition. It was the consensus of the committee, based on knowledge of structural materials in common use over specific time periods, to provide material standard that may conservatively be used with the Standard for a structural analysis of an existing structure. It should be noted that use of the material standards presented may be overly conservative for some structures.

C ANNEX S: ANALYSIS OF EXISTING ANTENNA SUPPORTING STRUCTURES BASED ON TARGET RELIABILITIES

Annex S provides load modification factors reducing the loads calculated in accordance with Section 2.0 for the analysis of Risk Category II existing structures. The load modification factors result in a reduction in loading for the extreme wind and extreme ice loading conditions based on equating the target reliabilities of an existing structure to that of a new structure. The use of Annex S may be useful when a significant change results in a demand-capacity increase greater than the 5% threshold specified in Section 15.0 which would require an existing structure to conform to the current Standard. When applying Annex S, the structure must conform to the current Standard with no components exceeding a demand-capacity ratio of 1.05.

The approach adopted by the committee for existing structures is to require the analysis of all structures to be based on the current Standard as opposed to attempting to use the revision of the Standard used for the initial installation. Many issues arise attempting to use previous revisions of the Standard for different risk categories for the extreme ice and earthquake loading conditions. Topographic and terrain exposure consideration are also an issue for applying older revisions of the Standard. Issues also can arise using previous revisions of the Standard that were not recognized by national building codes at the time of installation. Using the current Standard for the evaluation of existing structures is intended to result in consistent conclusions regarding the capacity of existing communication structures and eliminate the confusion among building officials and engineers on how to address the issues involved attempting to use previous revisions of the Standard.

Load modification factors are applied to factored loads as opposed to specifying a higher acceptable demand-capacity ratio greater than 1.05 for all loading conditions. This approach has the advantage of providing specific load modification factors based on the variables associated with each extreme loading condition in keeping with LRFD philosophy. The use of load modification factors for an analysis also captures the impact of P-delta effects for flexible structures.

The use of Annex S requires periodic inspection evaluations (i.e., condition assessments or other structural health monitoring evaluations) in accordance with a site-specific management plan per Note 2 for Table S-1.

A load modification factor equal to 1.0 is specified for Risk Categories I, III and IV. The extreme wind loading specified in the Standard for Risk Category 1 is considered as the lower bound for communication structures and a reduction for existing structures was not considered justified by the committee. Load modification factors equal to 1.0 are used for Risk Category III and IV structures as a minimum level of reliability is generally required and load modification factors less than 1.0 were not considered justified by the committee.

Earthquake loading rarely governs the design of communication structures and involves more complex reliability considerations compared to extreme wind and ice loading conditions. For this reason, a 1.0 load modification factor was specified for seismic load effects.

The existing structure modification factors specified in Table S-1 are considered conservative and with further research and experience with existing structures, lower modification factors may be presented in future revisions of the Standard.

The following were considerations of the committee for the use of load modification factors for the analysis of existing structures:

1. Communication structures are not inhabited structures and do not support significant dead or live loads. The governing design criteria for communication structures is based on reliability requirements for the structure to remain stable under extreme loading events due to wind, ice and earthquakes loading that may occur over the lifetime of the structure. Over the life of a communication structure, there are often several iterations of changes related to the appurtenances supported by the structure, with different durations associated with each change. The remaining life or the duration of a changed condition is an important consideration for the determination of the return periods appropriate for the extreme loading events to meet the desired target reliabilities for a structure.

2. The following illustrates remaining life estimates that result in the probabilities of occurrence for the ASCE 7 extreme wind and ice loading events, conservatively assuming for illustration purposes, that the modification factors are solely based on the expected remaining life of an existing structure (refer to Note 3 below).

Extreme Wind Loading

Target reliability = 7% over a 50-year period for Risk Category II, 700-year MRI wind speed
Refer to ASCE 7-16 Equation C26.5-3:
Probability of occurrence = $1 - (1 - 1/700)^{50} = 0.07$ or 7% for a 50-year period
Load modification factor = 0.95
Effective load factor = 0.95(1.6) = 1.52
Refer to ASCE 7-10 Equation C26.5-5:
Return period = $0.00228\exp[10(1.52)^{0.5}] = 516$

For a 7% target reliability for a 516 MRI, the period for occurrence would be 37 years per the following:
Probability of occurrence = 1 - $(1 - 1/516)^{37}$ = 0.07 or 7% for a 37-year period (remaining life)

Note that the effective load factor and the equation for the associated return period is from ASCE -10 used to determine the return periods associated with the wind speed maps based on risk category.

Extreme Ice Thickness

Target reliability = 10% over a 50-year period for Risk Category II, 500-year MRI ice thickness
Refer to ASCE 7-16 Equation C26.5-3:
Probability of occurrence = 1 - $(1 - 1/500)^{50}$ = 0.10 or 10% for a 50-year period
Load modification factor = 0.85
Effective load factor = 0.85(2.0) = 1.70
Refer to ASCE 7-16 Table C10.4-1
Return period = 300 years
For a 10% target reliability for a 300 MRI, the period for occurrence would be 32 years per the following:
Probability of occurrence = 1 - $(1 - 1/300)^{32}$ = 0.10 or 10% for a 32-year period (remaining life)

Note that the effective load factor is based on the load factor used for ice loading in ASCE -10 used prior to the extreme ice thickness maps published in ASCE 7-16.

3. The Annex S load modification factors were not solely justified based on estimating the remaining life of an existing structure. The determinations of the periods of occurrence above are for illustration purposes only and are not intended to convey a limiting life expectancy of an existing structure under a changed condition when using the Annex S load modification factors. The required enhanced periodic inspection evaluations per a site-specific management plan required for use of Annex S also justifies lowering the effective load factor. Communication structures generally have exposed load carrying structural members, allowing comprehensive inspections over the life of the structure. Regular inspections increase the reliability of a structure by identifying and mitigating structural issues before they escalate to a partial failure or collapse. In addition, condition assessments allow verification of the performance of a structure due to a changed condition and can identify local conditions that warrant an analysis under more stringent wind or ice loading conditions.

4. The limiting demand-capacity ratio of 1.05 for an existing structure from Section 15.0 is also justified for use with Annex S. A demand-capacity ratio equal to 1.05 is analogous to multiplying resistance factors for strength design (also referred to as strength reduction factors) by 1.05.

Higher resistance factors are justified for an existing structure due to condition assessments. The unknowns related to fabrication and erection for an existing structure are less compared to a proposed new structure. This, along with other factors, effects the prediction of design strength and is reflected in the magnitudes of strength reduction factors used for strength design. Strength design in accordance with the Standard is independent of the loading conditions used for analysis (i.e., independent of the use of load modification factors).

5. There is a risk/benefit consideration involved with the decision to modify a structure considering the risks related to construction (accidents, etc.) compared to the benefits of modifying the structure. Because of the risks involved, it has been common Industry practice to use, as a minimum, a 5% increase in strength when considering a changed condition without requiring modifications to a communication structure. There generally has been acceptable performance of existing structures using this approach. In addition, despite the trend in specifying higher ice loading, many existing structures designed in accordance with previous revisions have performed satisfactorily for their intended purpose. Based on the history of performance of existing structures, the load modification factors specified in Annex S are believed to result in a conservative approach for the analysis of existing structures when periodic inspection evaluations are performed in accordance with a site-specific management plan for the structure.

C ANNEX T: SI CONVERSION FACTORS

Annex T provides conversion factors for use with the Standard for conversions to the International System of Units (SI) commonly used in other international standards. Equivalent SI units are also provided in square brackets [ ] throughout the Standard.

C ANNEX U: REFERENCES

Annex U provides references used for the development of the Standard. Additional references are included in the commentary for the sections and annexes of the Standard.