COMMENTARY

on

CSA STANDARD S37-01

ANTENNAS, TOWERS, and ANTENNA-SUPPORTING STRUCTURES

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INTRODUCTION

This Commentary sets out the background of CSA Standard S37-01, with the reasons and sources of data for its clauses, so that users may better understand the intent of the requirements. Where no comment has been included the clause is considered to be self-explanatory.

COMMENTARY

C 1. Scope

C 1.1

This Standard applies to the various types of structures which can be generally described as "Communications Structures" as they either act as antennas, which transmit or receive signals, or as supports for such antennas. It covers a broader area than most structural Standards since it prescribes the major loads from wind and atmospheric icing, which are not similarly covered in the National Building Code of Canada (NBCC).

C 1.2

This clause defines the antennas and structures not covered by this Standard. These are small, light antennas whose failure would not cause a failure of the supporting structure and short structures (under 15 m) which support these light antennas.

C 1.4

Generally it is not necessary to consider earthquake effects for towers since the dead loads are small and the horizontal load effects from the wind loads will govern over the seismic loads. In addition communications structures behave very differently than buildings in earthquakes. Appendix M provides the designer with some guidance as to when it is considered necessary to take seismic loading into account.

A full dynamic analysis of a guyed mast is a very complex and lengthy undertaking due to the many modes which must be investigated. Experience has shown that it is not necessary to carry out this analysis except for very tall and heavy masts. Appendix H provides a patch loading method, which can be accommodated on a desktop computer, to simulate a full dynamic analysis within the limits set out.

C 2. Definitions, Symbols, and Reference Publications

C 2.3 Reference Publications

The Standards listed are the applicable editions when the committee approved this version of this Standard, which should be used. The Technical Committee will review any later editions of referenced Standards to ensure that they do not contain clauses not applicable to this Standard before updating the reference. When reference is made to undated publications in specific clauses it is intended that the latest edition and revisions of these publications be used.

C 3. Design Requirements

C 3.5 Existing Structures

Communications Structures which support wireless telecommunications antennas and transmission lines may be subjected to increased forces due to the addition and/or subtraction of antennas and lines. When these changes occur it is important that the structure be re-evaluated for compliance to this Standard. These changes may occur frequently due to expansion and rapidly developing technology in this field.

Generally, structures which have been designed for future antennas and lines do not have to be re-evaluated when these antennas and lines are added. If the size or type of antenna is changed then re-evaluation should be carried out.

For cases where the structure has been designed to an earlier version of this Standard, the current version of this Standard must be used for re-evaluation. It is not acceptable to use the initial design version, as the older Standards are withdrawn when each new version is published, and are no longer valid. When there has been an increase in the environmental loads between versions, the structure may not meet the current requirements. Clause 5.3 and Appendix G provide Reliability Classes and Importance Factors which may be used in evaluating these cases.

C 4 Loads

C 4.1 Dead Load (D)
The dead load to be considered is the weight of all of the structural members, climbing facilities, platforms, and all attachments (antennas, transmission lines, lighting system, etc.) connected to structure. An allowance of about 5% of the structure weight should be made for galvanizing and fasteners. The self-weight of the guy system imposed on the mast of guyed structures should also be applied as appropriate.

C 4.2 Ice Load (I)
The ice load is taken as the weight of glaze ice (900 kg/m^3) which is assumed to form on all the exposed surfaces. The minimum design thickness, t, is obtained from Figure 1. Larger values should be used where experience or a meteorological study indicate a more appropriate value.

The values used for the map are based on ice accretions occurring from freezing rain falling through a layer of freezing air, as indicated in Appendix E. Rime ice has not been considered since there are no instruments or other methods to determine suitable values. The Technical Committee assumes that this is reasonable, as rime ice is less dense than glaze ice and the projected ice area will have a lower drag factor than the cylindrical values employed.

It is generally accepted that ice thickness increases with height, due to in-cloud icing and other effects, however there is currently no data to evaluate this increase. For taller towers, above about 200m, and towers at high elevations, consideration should be given to some appropriate increase. The current ASCE Standard 7 Sect. 10, Atmospheric Icing, provides a formula, but this should be used very carefully since the icing approach differs from the Canadian method.

C 4.3 Design Wind Pressure (P)
This clause is based on the requirements of the National Building Code of Canada (NBCC) modified as required to suit the structures covered by this Standard. Note that both the reference velocity pressure and the height factor are affected by local topography, so this must be taken into consideration in establishing the wind profile, either by a site specific profile as provided by MSC, or by appropriate modification of the flat terrain values.

The requirements of clause 4.3.3 were introduced to match the Standard practice of building designers who are verifying the design of a building to accommodate the addition of antennas of this type.

C 4.4 Reference Velocity Pressure (q)
The 30-year return period is the value supplied by Environment Canada and used in the NBCC. Most other countries use the 50-year return period for towers; the difference between the two values is relatively small. A minimum value of 300 Pa was introduced when a number of site-specific studies produced very low q values, especially in North-western Ontario. It was felt that these low values might result in small member sizes which would not be able to provide the necessary stability.

The recommended procedure is to have a site-specific study preformed by the Meteorological Service of Canada (MSC). The values given in the NBCC are for the cities listed based for the most part on the meteorological reading at the nearest airport. The tower site might be a considerable distance from these locations. The wind zone map provided in Appendix E gives the maximum average value for that zone but will not cover any special conditions. It may be of value when designing a number of towers in the same geographical area.

C 4.5 Height Factor (C_e)

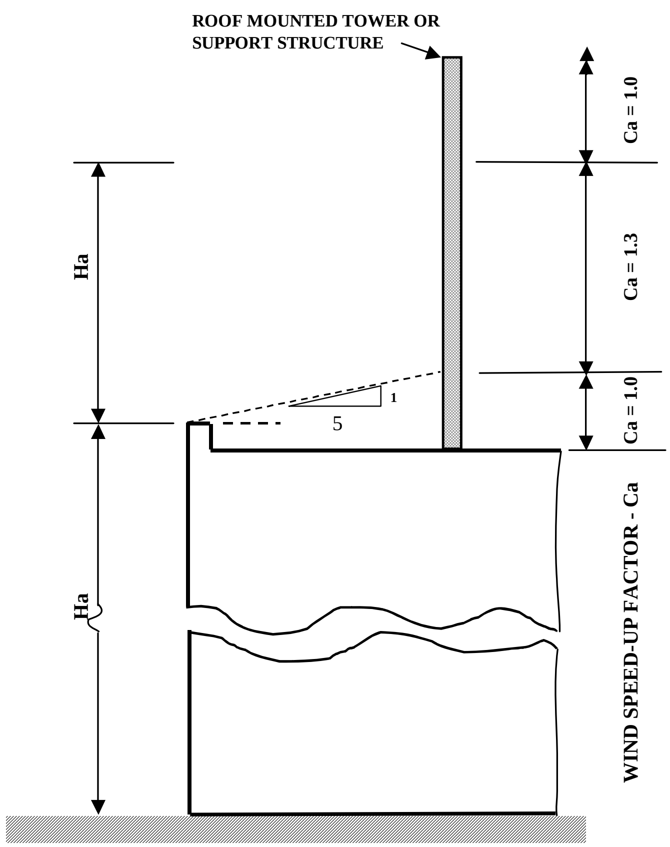
The formula for C_e is the exponential expression for wind over flat terrain. Rough ground will change the curve and is usually given by a log expression. A site-specific study by MSC will provide the more exact values designated by q_h.

C 4.6 Gust Effect Factor (C_g)

Towers and masts are should be designed for a 3 to 5 second gust velocity. Since the reference velocity factor, q, is a one hour average value it is necessary to factor this value. Some dynamic effects are also covered by this value. Conditions for reducing the gust effect factor for pole structures have been introduced to this version of the Standard, if the design includes consideration of vortex shedding. This method may be useful in the re-evaluation of existing poles.

C 4.7 Roof Wind Speed-up Factor (C_a)

This clause is provided to give some guidance for this situation. Note that it is essentially based on an isolated building exposed directly to the wind.



Ha = Height or Width of the roof of the windward face of the building, whichever is less.

C 4.8 Wind Load (W)

C 4.8.1
As ice is considered to form radially on the members the drag factor for ice area is the same as for round members, but note that where two adjacent parallel members are close enough that the ice spans the gap between them the surface between is considered flat. (See Clause 4.2.1)

C 4.9 Drag Factor (C_d)
Three-dimensional models were used to determine the drag factors given. The effects of internal horizontal bracing and members in the face not being considered (i.e. bracing in the faces parallel to the wind in square towers) are incorporated in these drag factors.

C 4.9.1 Lattice towers and Masts
The drag factors presented in this clause are based on the research carried out at the British Naval Research Station at Whitbread UK. Models of various bracing configurations on square and triangular mast sections were used. These values have been used by numerous international Standards, such as BS 8100 : Part 1, IASS WG4, ASCE 7, and TIA/EIA-222.

C 4.9.1.4
In Clause 4.9.1.4 the drag factors given for this situation are less than would normally be used for these shapes. They are values that have been used in earlier versions of the Standard and provide for some shielding by the main structure.

C 4.9.2 Pole Structures
The values given in Table 2 are based on AASHTO Standards which were derived from wind tunnel tests on round, 8, 12, 16 sided poles. The 18 sided pole values were interpolated between the round and 16 sided values. Eighteen sided poles are a new configuration resulting from requirements for larger stiffer poles and manufacturing limitations.

C 4.9.4 Antennas

C 4.9.4.1
The major manufacturers of parabolic antennas publish drag factors and force coefficients for all wind directions.

C 4.9.4.3
This clause sets out the method to be used to determine the amount of shielding which may be considered for the leeward antennas of a group at the same level. Computer programs have been written to determine these values.

C 4.10 Temperature Effects (T)
The temperature for the initial condition of the analysis is set out in clause 5.1. For the ice load conditions, a lower temperature is usually assumed by the designer. Ice loads form on the structure in the temperature range from -5°C to 2°C. This lower value is usually considered in determining the temperature effect in the structure, especially guyed masts. For design purposes the maximum design ice load is not considered to occur at any lower temperatures which might occur at the site.

C 4.11 Earthquake Effects (E)

The effects of earthquakes on self-supporting towers, poles, and guyed masts and the method of analysis differ significantly from those for buildings. Methods for evaluating these structures are not sufficiently developed, at this time, to be included in the main body of this Standard. Appendix M does provide some guidance based on studies carried out to date.

C 5. Analysis

C 5.3 Factored Loads for Ultimate Limit States

Dead Load Factor, α_D

A load factor of 1.25 is used for dead loads (weight) since these values can be determined much more accurately than the environmental loads. The load factor for the self-weight of guy cable systems is taken a 1.0 since this will be known precisely. Also the weight of the guy is integral to the guy stiffness, so a factored guy weight would not provide accurate analysis results.

Importance Factor, γ

The importance factors given in Table 3 allows the designer and the owner, subject to the approval of the regulatory authority, to adjust the applied environment loads to suit the reliability class for the site. Most new towers should be reliability Class I. A structure intended for temporary use, two years or less, may have a lower reliability if it meets the criteria. Existing structures may also be evaluated to a lower reliability class if they meet the criteria.

Load Combination Factor, ψ

A combination factor of 0.5 is applied to the wind loads for the ice condition, since the probability of the maximum wind load occurring simultaneously with the specified ice load is extremely low.

Wind Load Factor, α_W

The wind load factor of 1.5 was determined after an extensive evaluation of many satisfactory existing tower designs.

Ice Load Factor, α_I

The load factor is applied to the weight of the ice only. If the load factor were applied to the ice thickness the project area would increase linearly, but the weight would increase as the square of the thickness. Studies indicate that this increases the load on guy cables to an unacceptable level. These studies showed that the method adapted produced structures comparable to existing satisfactory ones. As we gain more information about atmospheric icing of structures this approach may be reviewed.

No load factor is applied to the guy initial tension as this is a known value, which should be maintained regularly, and factoring this value would distort the analysis results.

C6 Structural Steel

Note that Appendix K of the Standard also provides a commentary on Clause 6 addressing the changes from the 1995 version to this version. This commentary deals with the background and reasons for the requirements of this Clause. For convenience some parts of Appendix K have been incorporated into this Commentary on page 17 and are identified in this section by **[REF]**.

C 6.1 General

CSA Standard CAN/CSA-S16.1 is the basic Standard for the design of steel structures in Canada. Structures conforming to CSA Standard S37-01 are typically lattice self-supporting towers and guyed masts made up of angles, solid rounds and tubular members, or tubular pole type structures. S16.1, which is applicable primarily to buildings, does not, in some areas, adequately address the use of these types of members. Clause 6 sets out acceptable procedures for their design. Many of the requirements are based on those for the design of electric power transmission towers. While these may appear more liberal than standard structural practise they are based on full scale testing to failure, and have been codified in such Standards as ASCE 10 and Euro Code ENV 1993-3-2;1997.

C 6.1.4 Minimum Thickness

These values are based on past practices which have been successful to prevent local buckling, vibrations and damage, due to usages not normally considered in the design

C 6.1.5 Member Shapes

Shifflerized 60^o angles are weaker in torsional-flexural buckling and stronger in flexural buckling than regular 90^o steel angles, hence, it is rational to assume that the design of 60^o angles should account for torsional-flexural buckling. However, the results of experiments show that the design strength according to CSA Standard S16.1-M94 taking into account the effects of torsional-flexural buckling is very conservative. This conservatism stems partially from the inherent underestimation of the axial strength by the basic compression curve when applied to the design of 60^o steel angles. **[REF]**

C 6.1.6 Minimum Charpy V-Notch Value

Communications structures are skeleton frames exposed to the ambient temperatures at the site. Some structural steels are intended only for use in building which are heated and not subjected to the same wide range of temperatures as towers and masts. The designer must be aware of these circumstances and specify appropriate properties for the steel to be used.

Resistance to brittle fracture in members subject to tension due to axial forces or moments is an important consideration in specifying the minimum physical properties of the steel to be used at a particular site. This is usually done by specifying a Charpy V- Notch value, however the method for determining the minimum value has not been readily available. The British Standard BS 5950: Part 1 gives this formula utilizing the thickness and yield of the steel, the two critical values. **[REF]**

The minimum Charpy V-Notch (CVN) value may be determined from the following.

$$CVN = F_Y t/710c$$

where

F_Y = yield strength, MPa,

t = thickness of an angle, plate, the thickness of a tube, flanges of rolled sections and diameter of a solid round

c = 1 when the design tensile stress is greater than 100 MPa at a weld or unreamed punched hole.

= 2 when the design tensile stress is equal to or less to 100 MPa or if the location is at drilled or reamed holes or a non welded location.

For the extreme cold temperatures which occur in Canada, it may not be possible to obtain a steel with the required values. In these cases, a modified yield may be employed based on a slower rate of loading if applicable. Reference to CSA S6-94 Canadian Highway Bridge Design Code will provide guidance in selecting this value.

C 6.1.7 Framing Eccentricity

It is nearly impossible to connect angle members without introducing some eccentricity at the connection. If the eccentricity is minimized, as set out in this Standard, the members perform satisfactorily. This has been verified by laboratory and full scale testing, and many years of experience in the field on thousands of lattice towers. When the "normal framing " requirements are exceed, an adjustment factor has been provided based on recent laboratory testing at the University of Windsor. It should be noted that the gauges for angles in the CISC Manual may exceed the "normal framing" requirement, as these values are based on the use of ¾ " bolts or larger. Tower designs frequently utilize smaller size bolts which allow the "normal framing" requirement to be meet. The designer must check the connection detailing carefully to ensure that the design assumptions are achieved.

C 6.1.8 Secondary Bracing Members

The design of secondary members, often referred to as "redundant members", is an important consideration in the analysis and design of lattice towers and masts. Over the years the requirements, and the interpretation of these requirements, have varied widely. In recent years there have been attempts in Europe to more precisely define these requirements (BS 8100, Euro Code) but there are still differences. There has also been some reluctance to adopt the new methods, as the old method seems to have worked adequately in the field. This version of the Standard provides a Standard procedure, but has left the designer some discretion on the design of the more eccentric connections.

In the past the specified resistance has been a single value. The newer Standards, including this one, now provide for a variable value which is a function of the L/r ratio of the member being supported. This seems to be accepted as logical. As a historical note the failure of the first Quebec Bridge was due to the inadequate strength of the secondary bracing of the main chords and bracing. Most values were based on the tests and recommendations following this event. Modifications to these original values were based on testing for the type of structure being considered.

In previous versions of this Standard, the axial force in secondary members has been taken as 2% of the force in the member being supported. This version has adapted a variable load, depending on the slenderness ratio (L/r) of the member being supported. **[REF]**

C 6.1.8.3

This cautionary note was adopted from the Euro Code ENV 1993-3-2:1997. There is a possibility that the leg and diagonals in this situation will act together, and the actual distribution of forces will differ from those obtained from the assumed analysis.

In guyed masts the leg loads, especially near the bottom, can become very large, while the diagonal loads may be small. In this case the load in the diagonal resulting from the distribution of the nodal force, F_s, may become the governing value.

C 6.2.1.1 Unbraced Length, L

When modeling a lattice tower or mast it is customary to define the structure as a series of nodes at the intersection of the axes of the members and to use the distance between nodes for determining the L/r value. This is satisfactory for angle legs and bracing members. For other shapes where gusset plates are utilized, the value may be overly conservative. Conservatively the gusset will provide restraint for the x-x, and z-z- axes so that the value of L for these axes can be taken from the centroid of the connection to the gusset plate at the leg and the L for the y-y axis taken between the node points. **[REF]**

C 6.2.1.4 Rotational Restraint

Clause 6.2.5.1 utilizes the effects of different end restraints in determining the effective slenderness ratio to be used in calculating the compressive capacity of the member. This clause sets out the criteria for deciding the type of rotational restraint to be used.

C 6.2.2 Leg Members

An effective length factor of K=1 is utilized for leg members although they have continuity. This conservative value provides for any moments introduced from any eccentricity at the joints. This is the traditional value and has provided satisfactory results over the years. **[REF]**

C 6.2.3 Bracing Members

The clauses dealing with bracing members have been significantly modified for this version of the Standard. The section utilizes the same concepts as used in the 94 version, however the Technical Committee has brought some of the material from the 94 Appendix K into the main body and extended the type of bracing patterns utilized which answers some of the questions raised in the past. In addition descriptions for solid rounds and tubes are provided since these were not adequately covered in the previous versions. **[REF]**

C 6.2.3.3 Cross Bracing (Tension/Compression)

The technical committee has reviewed the requirements for cross bracing in the previous version of this Standard and found it to be overly conservative in many cases. A review of other Standards for lattice structures and research papers confirms these findings. The committee has adopted this new approach as being the most suitable for our applications. The formula adjusts the effective length for the out of plane buckling in accordance with the ratio of the tension force to the compression force. In the case of equal tension and compression in the bracing, the adjusted length ADL is DL1. When the tension load goes to zero the adjusted length is DL1 + 0.45 DL2 for the out of plane buckling. This compares to the $L_1 + 0.5L_2$ given in the 94 edition of this Standard. This formula is based on the work of Picard and Beaulieu (1987). **[REF]**

C 6.2.3.4 Discontinuous Cross Bracing with Continuous Mid-Horizontal

This type of bracing has not been used extensively in lattice towers, but it appears frequently enough that the committee felt that it should be included. The 65% value adopted as the limit for support is conservative, since there were no experiment values available to justify a lower value. Recent anecdotal information from electric transmission tower engineers indicates that a value as low as 20% is acceptable based on full scale testing. However this is proprietary data and has not yet been verified. Until approved by the S37 Committee the 65% value should be used.

The designer is also cautioned that if the diagonals are assumed to have rotational restraint at the ends the centre joint must be capable of providing the assumed restraint. The orientation of the diagonals and the stiffness of the horizontal affect these considerations.

C 6.2.3.5 K Bracing
In the past, K bracing has created many discussions, especially with regard to the design of the horizontals and the need to provide internal plan bracing. This clause should resolve these questions.

C 6.2.4 Built-up Members
Previously the requirements for the spacing between the stitch bolts of built up members have often varied between different Standards and between versions of the same Standard. The requirements given in CSA S37-01 are now the same as those in CSA S16.1 and other North American structural Standards.

C 6.2.5 Effective Slenderness Ratio, KL/r
Determining the effective slenderness ratio, KL/r, is an efficient method of adjusting the basic compressive Resistance, Cr , of bracing members to take into account the eccentricity and end restraint of the connection. This method is based on the values specified in ASCE 10 –97 Design of Lattice Steel Transmission Structures and earlier design guides. Previously only the formula for angle members was provided. The formulas have been expanded to cover solid round welded bracing members and single tubular bracing members. The values for the welded solid round members are based on testing of full-scale welded mast sections at the University of Windsor under the direction of Dr. Madugula.

K 6.2.6 Width-Thickness Ratio
As a large number of steel angles used in towers fall above the maximum width-thickness ratio imposed by CAN/CSA-S16.1-94, the Technical Committee decided to take into account these larger width-thickness ratios by determining a reduced effective yield stress based on the susceptibility of the individual plate elements to local buckling. This approach is similar to ASCE [1992]. **[REF]**

Table K1 contains fillet radii of metric size angles.

6.2.7 Compressive Resistance
This edition of the Standard has adopted the single compression formula given in the 1994 version of S16.1. This replaces the numerous equations of the previous edition which were adapted from the Structural Stability Research Council. The maximum difference between the two sets of values is approximately 3%. [See reference 4]. **[REF]**

C 6. 3.4 Effective Net Area Reduction – Shear Lag
This clause follows CSA S16, with one exception. In clause 6.3.4.2 the reduction of the effective net area to allow for shear lag with fewer than four fasteners is taken as 0.70 rather than 0.60 given in S16. This higher value was adopted by the technical committee after a review of many tension tests of angle members at the University of Windsor. The small number of tests and larger bolts and member sizes used to establish the values in S16 may account for the difference.

C 6.4 Tubular Pole Structures
The formulas and requirements presented in this clause are based on ASCE Draft Standard for Design of Steel Transmission Pole Structures, ASCE Manual No. 72. Since these Standards are in US customary units for Allowable Stress Design (AWS) the conversion has been made to metric units and Limit States Design (LSD).

C 6.4.2.1 Polygonal Tubular Pole Structures
Eighteen sided poles are included in this version of the Standard. This configuration has been introduced to facilitate the fabrication of the larger diameter poles required to achieve the deflection requirements needed for communication structures. There are no known tests on this type of pole, so the values have been determined by interpolating between the 16 sided and round shapes

C 6.5.2.1 Connection Resistance Factors
The resistance factor, Φ_b, for bolts has been increased from 0.67 to 0.80 following the change being made by CSA S16.1. A review of the research on bolts concluded that the earlier value was too conservative. The new value still maintains a generous reliability index and is still conservative. **[REF]**

C 6.5.2.2 Factored Bearing and Shear Resistance
In earlier editions of CSA S16.1 the bearing value, B_r, in bearing type connections was given by $B_r = \Phi_b\ t\ n\ e\ F_u$ with the limiting value of $\Phi_b\ t\ 3\ n\ d\ F_u$ and $\Phi_b = 0.67$. CSA S37-94 utilized the same expressions. **[REF]**

A review of these values indicated that the limiting value e = 3d was unconservative, but gave acceptable values when used with $\Phi_b = 0.67$. With the change in the values for Φ_b the Technical Committee re-examined the expressions and chose to utilize the ones expressed in the AISC, which were based on the same research results [Reference (4)]. These expressions are more suitable for one or two bolt type connections usually used in communications structures. A value of $\Phi = 0.9$, the normal value used for steel, was selected for checking the bearing value of the member. These considerations resulted in changing the limiting of value for B_r to $2\ \Phi\ t\ d\ F_u$.

Two different limit states, strength and deformation at the hole, are being checked when determining the bearing resistance. This can give conflicting but acceptable values.

C 6.5.5Splices
This section has been completely rewritten to include the current thinking on this subject. The minimum tensile strength of the splice has been reduced from 50% to 33%, bringing this version in line with other tower Standards and allows the lower value permitted of bearing splices to be used for all types of splices. The consideration of eccentric moments has been expanded to include the splice plates as well as the bolt group. There has been confusion about this in the past. The need to check for prying in tension flange splices has been added to remind the designer that this is a requirement. **[REF]**

C 6.6.2.2 Welding of Steel Tubular Pole Structures
The weld detail must consider vibrations and fatigue in addition to the direct forces. For sections which butt to the plate a full penetration groove weld with a reinforcing fillet should be used. This type of detail is usually used with polygonal sections. When a slip through connection is employed two rows of fillet weld may be used. This type of detail is often used with circular sections. **[REF]**

C 7. Corrosion Protection

C 7.5.2 Anchors

This clause specifies additional corrosion protection to the normal galvanizing. When this protection takes the form of cathodic protection, the owner or maintenance engineer must be aware that this method has a limited life and the anchorage must be inspected and the anodes replaced, if necessary, at required intervals. The system supplier should advise on the maintenance schedule, procedures and precautions to be taken.

C 8. Other Structural Materials

This clause covers requirements for structures made of other than structural steel, such as aluminum or concrete, and references appropriate CSA standards.

C9. Guy Assemblies

C 9.1 General

The guy assemblies used in guyed masts require special attention. The strand and hardware used for lighter and small masts are unique as a structural component and therefore are given extensive coverage. The hardware is usually that used for running rigging, as opposed to standing rigging hardware as used in structures. This necessitates different factors which are spelled out in this clause. For taller, heavier, masts the guy assemblies are similar to those used in bridge work. The dividing line between light and heavy systems is usually taken at the 32 mm strand size.

C 9.2.2 Prestretching

Constructional stretch occurs due the bedding down of the wires in the strand when load is applied to it. The amount of this stretch will depend on the method of manufacture and the number of layers of wire in the strand. Once it has occurred it does not re-occur. For strand less than 32 mm in diameter and/or of short length, it is usually not necessary to prestretch the strand if sufficient take up and adjustment is provided. Longer and heavier guy assemblies will require more take-up and/or prestretching. The decision on this operation is best left to the design engineer in consultation with the strand manufacturer.

C 9.3.1 Clips

Clipping is an acceptable method of securing loops in wire rope or strand for the smaller size cables. The effective breaking strength of the cable will be reduced due to the crushing effect of the clips. At one time it was the only method used for making loops both in the shop and in the field, but new technologies, such as mechanical splices and preformed guy grip, have proven to be more economical and convenient.

C 9.3.4 Sockets

For strand sizes larger then 32 mm, socketing is the only method for attaching the end fittings. Most socketing is achieved using poured hot zinc. Resin socketing may be used, but the designer must ensure that deterioration due to ultra-violet light does not occur. It is important that the strand be straight where it enters the socket otherwise the load distribution will be unequal and premature wire failure will occur.

C 9.3.5 Thimbles

It is important that the strand be properly supported as well as at the correct radius when a loop is used as the end attachment. A thimble serves this purpose. Some “brooming” will occur at the top of the thimble and this is acceptable. Near the rated breaking strength of the cable a heavy-duty thimble will start to show some deformation. This will serve to indicate that the structure has been severely overloaded to near the breaking strength.

C 9.3.7 Turnbuckles

In the late 1960’s there were four turnbuckle failures on major guyed masts. While fortunately none of these caused actual collapse of the structure, they did raise serious concerns about potential failures. The resulting investigation and studies identified two significant problem areas. The first was lack of articulation at the anchor end of the turnbuckle, which introduced bending for which the turnbuckle was not designed. The second was the coarse grain structure of the metal due to the forging operation, which caused low ductility, particularly at low temperatures. Based on this, the requirement for full articulation of the guy assembly, and specification of the minimum grade of steel and suitable heat-treatment, were introduced for shackles and turnbuckles.

C 9.3.9 Initial Tension Tags

This is a new requirement in the ’01 version of the Standard. In the past, the designer specified the initial tension, at the referenced design temperature, that had been used in the design. Usually this was about 10% of the rated breaking strength of the cable, but could be adjusted to between 8% and 15% to control the deflections at the design load. These values were incorporated on the design drawings and erection diagrams. There may also have been adjustments required to the tensions in guys at the same level when their anchors were at different elevations. Usually these design values and a table of tensions for the various ambient temperatures were supplied with the erection diagrams. Copies were also provided to the owner. Unfortunately these were often lost and/or not provided to the inspectors or crews doing subsequent maintenance work on the mast. Some times, when they did not get the information, they erroneously set the tensions at the 10% value, thereby reducing the rated capacity of the structure. To over come these problems a requirement was introduced into the Standard for the attachment of a permanent tag, providing the necessary data, at the anchor end of the guy assembly. As most existing installations do not have such a tag attached, it is recommended that, in any future inspection or maintenance contract, the owner/consultant specify the supply and installation of the necessary tags. It will be the responsibility of the owner/ consultant to obtain and supply the required information.

C 9.4.4 Factored Resistance

The resistance factor for guy cables and hardware is taken as 0.6 so as to have the same ratio of load to resistance as was used in the AWS versions of the Standard.
The factor of 2.5 times the maximum working load to determine the effective breaking strength of the guy assembly was specified for LSD. The 2.5 value has three components;

- 1.67 safety factor, to the yield point, for tension members,
- 1.30 the recommended Ultimate Tensile Strength to Yield Point ratio,
- 1.15 a redundancy factor to ensure that any failure does not occur in the guy assembly before the mast.

The inverses of the UTS/YP ratio (0.77) and of the redundancy factor (0.87) are used to adjust the regular load factor of 0.9 for tension members to obtain the adjusted load factor for the guy assemblies.

$$0.9 \times 0.87 \times 0.77 = 0.60$$

C 9.4.5 Initial Tensions

While the initial tension in the guy cable is usually specified as 10% of the rated breaking strength of the cable, at the anchor end, for the initial site temperature, it is sometime necessary to select another value in the 8% to 15% range to straighten out .the deflected mast shape or modify the load distribution. In special circumstances these values can be exceeded, but in such cases additional studies need to be carried out to ensure that other effects do not occur. Low initial tensions may result in “galloping” of the cable and high initial tensions in high frequency, low amplitude Aeolian vibrations.

C 9.4.6 Articulation

The commentary on clause 9.3.7 for turnbuckles gives the background for this requirement.

C 9.4.7 Take-up Devices

Take-up devices are provided to allow the field crew to readily adjust the length to achieve the specified tension at the ambient temperature and to permit future adjustments that may be required due to the constructional stretch in the cable. See also clause 11.5, which specifies the amount of take-up the crew must leave, at the time of the initial installation, for future adjustment.

C 9.4.8 Non-Metallic Material

Non-metallic guy cables are occasionally used as an insulating component on radiating masts or to prevent re-radiation from adjacent antennas. These materials do not have the same characteristics as steel, therefore special consideration is required.

C10. Foundations and Anchorages

There are many types of foundations and anchorages required for communications structures.

- **a.** Spread footings for guyed towers - primarily in compression.
- **b.** Spread footings for self-supporting towers - subjected to compression, uplift and high shear loads.
- **c.** Concrete dead-man anchors for guyed towers - primarily in uplift and sliding.
- **d.** Pile footings for guyed towers - primarily in compression.
- **e.** Pile footings for self-supporting towers - subjected to compression, uplift and high shear loads.
- **f.** Pile anchors for guyed towers - primarily in uplift and sliding
- **g.** Drilled caisson or drilled pile footings for guyed towers - primarily in compression.
- **h.** Drilled caisson or drilled pile footings for self-supporting towers - subjected to compression, uplift and high shear.
- **i.** Drilled caissons or drilled pile anchors for guyed towers – primarily in uplift and sliding.

C 10.1.1 Reference Standards

This clause references Section 4.2 of the NBCC, which in turn references the Canadian Foundation Engineering Manual. Since the Canadian Foundation Engineering Manual, is geared more to buildings and other structures, some clauses that may be applicable to towers are not always obvious. Also other clauses that apply to foundations and anchorages for other structures may not necessarily apply to towers.

C 10.1.2 Site Investigations
This section makes it mandatory to have a site geotechnical report, and provides guidance for providing such a report. The requirements for foundations for communication structures differ significantly from those for buildings, and direction is required for the engineer carrying out the investigation.
The section has been expanded considerably to emphasize the importance of correct site-specific soils information to the integrity of the overall structure.

C 10.2 Design

C 10.2.2
The previous version of the Standard provided only two values for the resistance factor, Φ =0.75 for soil or rock, and 0.5 for single rock anchors. These were considered to provide the same "safety factors" as were used for "WSD" designs. The technical committee undertook a review of these values, as there was some consideration that they could be too low for piles in tension when using the Foundation Engineering Manual procedures. The revised load factors listed in clause 10.2.2 were ultimately adopted.

Many communications structures have relatively low dead weight in comparison to the applied live loads of wind and ice. This ratio is significantly different to that for most other types of structures. These transient live loads are based on the one in 30 year return period, and are the governing conditions. For the bearing load condition this difference is recognized by introducing two values for Φ, depending on the ratio of dead to live load.

For foundations which resist pullout, the Foundation Engineering Manual indicates that a "factor of safety" of 6, or a Φ value of 0.25, is required. These values were considered to be too conservative, for communication structures, due to the nature of the loading as outline above. A Φ value of 0.375, or F of S of 4, was adopted. The other values remained the same as previously. These increased requirements may cause some difficulties for pile foundations of existing structures when loadings are increased. In these cases a risk evaluation analysis, similar to that outlined in Appendix G, should be carried out.

C 10.7 Installation – General
This clause is new to this edition of the Standard. (See also **C 11.2**)
This clause is applicable to the design engineer, and to the owner/purchaser and the foundation contractor, who each need to be aware of the requirement for the necessary inspections.

It is recognized for both clauses 10.7 and 11.12 that the design of most communication structures is carried out at centralized locations, not necessary local to the site, and that many towers are located in remote regions so that it is not always feasible or economically justifiable for the design engineer of record to visit the site. However a report by a qualified person is required at each critical stage.

C 11.11 Telescoping Field Splices
This is a new clause. It was added because there was a need to set out the minimum requirements and values for this type of splice

C 11.12 Installation – General
This clause is new to this edition of the Standard. (See also **C 10.7**)
This Clause is applicable to the design engineer, and to the owner/purchaser and the erecting contractor, who each need to be aware of the requirement for the necessary inspections.

C 12 Obstruction Marking
This clause is provided to alert the owner and/or engineer that obstruction marking, either lighting or painting and lighting, may be required for the structure. It also directs the responsible persons to the governing document(s). The design engineer must include any lighting equipment to be installed on the structure in the loading calculations.

C 13 Bonding and Grounding
This clause does not have any direct impact on the design of the structure unless any special components affect the loading. It is included to advise that the supply and installation of these components should be included for all towers. Many tower owners have their own requirements for these items. If nothing is specified then the tower should meet the minimum requirements set out in the referenced documents. A good bonding and grounding system is recommended to protect the equipment installed on the structure and in the equipment shelter at the base from lightning and other stray currents.

C 14 Insulators and Insulation
These components are utilized in towers and guyed masts used for AM radio broadcasting, and other structures which may re-radiate other signals. Apart from the strength requirements determined by the structural engineer, the radio engineer for the project must provided the electrical requirements.

C 14.1.1
The insulators utilized in guy assemblies, particularly the primary insulator at the radiating mast, present a challenge to the designer to conform to the requirements of this clause. There are basically two types available. The one, generally referred to as the “cage” type, places a ceramic cone in compression using a metal cage and a metal rod through the cone to transfer the loads. These have a low flash over rating and several units may have to be placed in series to achieve the required rating. This appears to be a “fail safe” design, but has not always proved to be so. The second type, referred to as an “oil-filled” insulator, utilizes a fibreglass loop in tension to place the ceramic cylinder in compression and contain the oil surrounding the loop. This protects it from any arcing which may occur. If the loop fails then the insulator will separate. A high factor of safety in the components has ensured long service provided the oil has not leaked out. A good inspection and maintenance program is required with both types.

C 14.1.3
The lower value for the resistance factor applied to materials other than steel or aluminium is to account for the uncertainty in predicting the failure mode of the insulating material. The ratio of the factors is the same as the ratio used in AWS,

$$2.5/3.0 \times 0.6 = 0.5.$$

C15. Ladders, Safety Devices, Platforms and Cages

C 15.1 General
This clause has been prepared in concert with the requirements of the Federal Department of Labour as set out in the Canada Gazette Part II, Volume 134, Number 21, dated October 11, 2000.

With the publication of this Gazette, the Section on "Towers, Antennas and Antenna Support Structures" set out in Division II, CSA Standard S37 automatically became part of the Canada Labour Code.

The wording of Section 15 of S37 was approved as written by Labour Canada before it was published in the Gazette Part I. It was subsequently reviewed after the comments were received regarding the wording used in Gazette Part I. This review included a review by the Labour Canada (HRDC) legal department.

It must be noted that by adding this section to the regulations, the Department of Labour is recognizing the special requirements for towers and antenna support structures; thus the new Division II for Towers, Antennas and Antenna-Support Structures. Buildings are covered under Division I.

Division I requirements should not be applied to Division II applications; this has been a problem for tower designers and contractors for years. With this revision to the Canada Labour Code Regulations, S37 requirements govern even if they conflict with Division I requirements.

S37 was also revised to meet the requirements for a Fall Protection System built in accordance with section 12.10 of the Canada Labour Code.

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