CSA Group

CAN/CSA-S37-18
National Standard of Canada

Antennas, towers, and antenna-supporting structures

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Although the intended primary application of this Standard is stated in its Scope, it is important to note that it remains the responsibility of the users to judge its suitability for their particular purpose.

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Antennas, towers, and antenna-supporting structures

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Preface

This is the eighth edition of CAN/CSA-S37, *Antennas, towers, and antenna-supporting structures*. It supersedes the previous editions published in 2013, 2001, 1994, 1986, 1981, 1976, and 1965.

The following is a list of some of the more important changes made in this edition:

a) Clause 4.6 was added for consideration in the structural analysis of known future antenna loading combinations.
b) Clause 5.3.4 was added with recommendations for load cases with discrete attachments installed on a structure for a period of 4 months or less.
c) The wind velocity pressure, q_h, was redefined.
d) The inclusion of drag coefficients, C_d, for attachments of additional shapes.
e) The addition of an effective projected area formula to account for loads normal to the wind direction.
f) The elimination of the C_g of 2.5 in the calculation of the wind load of monopole structures. The dynamic response to wind for monopoles and shrouded tripoles will be taken into account by the new load combination W_F (fatigue loading combination) in accordance with a normative Annex N.
g) Clause 7.3.8 was added to provide factored resistance formulas for link plates.
h) The factor resistance formulas were revised to include bending moments of anchor rods installed without high strength grout.
i) Clause 7.5.8 was added to include U-bolt material, installation, torqueing, and design recommendations.
j) Clause 11.2.2 was added for consideration of guy resultant angle variations in guy anchor shaft analysis and design.
k) Emphasis on additional engineering for construction loads on existing towers and guidance on temporary construction wind speed.
l) Alignment with recommendations from *Canada Labour Code* on load requirements for fall arrestors and vertical rigid rails.
m) Additional requirements for qualified climbers were provided.
n) Additional recommendations on corrosion protection of guy anchorages (Annex F) were provided.
o) Additional recommendations for geotechnical site investigation to include requirements for corrosive soils (Annex L) were provided.
p) Additional recommendations for tower condition assessment to emphasize importance of initial construction inspections and the recording of critical information (Annex D) were provided.

A commentary on this Standard is found in Annexes K and S.

CSA Group acknowledges that the development of this Standard was made possible, in part, by the financial support of Bell, Nav Canada, Hydro-Québec, Rogers Communication Cda Inc., CBC Ontario, NorthWesTel Inc., Telus, and Videotron.

This Standard was prepared by the Technical Committee on Antenna Towers, under the jurisdiction of the Strategic Steering Committee for Construction and Civil Infrastructure, and has been formally approved by the Technical Committee.

This Standard has been developed in compliance with Standards Council of Canada requirements for National Standards of Canada. It has been published as a National Standard of Canada by CSA Group.

Notes:

1) *Use of the singular does not exclude the plural (and vice versa) when the sense allows.*

2) *Although the intended primary application of this Standard is stated in its Scope, it is important to note that it remains the responsibility of the users of the Standard to judge its suitability for their particular purpose.*
3) *This Standard was developed by consensus, which is defined by* CSA Policy governing standardization — Code of good practice for standardization *as "substantial agreement. Consensus implies much more than a simple majority, but not necessarily unanimity". It is consistent with this definition that a member may be included in the Technical Committee list and yet not be in full agreement with all clauses of this Standard.*
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   c) *wording of the proposed change; and*
   d) *rationale for the change.*

CAN/CSA-S37-18
Antennas, towers, and antenna-supporting structures

0 Introduction

Antennas, towers, and antenna-supporting structures comprise a group that can be described as communication structures. They are usually of lattice steel construction, but can be of solid or tubular construction and can use a variety of materials. They can be guyed or self-supporting, and some structures can be mounted on platforms or building rooftops. The principal loads are wind and ice, while earthquake effects require a design check for designated post-critical installations in regions of medium to high seismicity. This Standard is written to address these special characteristics using the applicable sections of other CSA structural design standards. It also refers to the *National Building Code of Canada* for the specification of seismic spectral accelerations.

Most design standards are written to address the requirements of new structures. The Technical Committee for this Standard has also been concerned about the effect of changes on existing structures. While it is not mandatory to upgrade existing towers when new editions of the Standard are published, communication structures are frequently subject to changes in attached equipment. This necessitates conformance with the current edition. Changes are therefore carefully considered so as not to cause significant economic impact as a result of minor changes in equipment.

The latest amendments to Part II of the Canada Occupational Safety and Health Regulations (COSH) of the *Canada Labour Code* require that the design and construction of every tower, antenna, and antenna-supporting structure meet the requirements of this Standard as amended from time to time. Safety of persons who are required to climb a tower is as important a consideration as the safety of the structure; therefore, requirements for ladders, safety devices, platforms, and cages are included as part of this Standard.

It has long been recognized that some structures, particularly tall guyed masts, can be subject to dynamic effects that require a more in-depth study of the loads and responses than that provided by static analysis procedures. This Standard does not have mandatory requirements for dynamic analysis, but it does provide a quasi-dynamic patch-loading procedure, which is recommended for the analysis of wind effects on such towers.

1 Scope

1.1 General

This Standard applies to structural antennas, towers, antenna-supporting structures, and roof- and wall-mounted structures, including their components, such as guys and foundations. It covers the structural design, fabrication, and erection of new structures and the modification of existing structures.

1.2 Exclusions

This Standard is not intended to apply to
a) attachment antennas and arrays or assemblies of such antennas; or

b) towers that support attachment antennas, extending less than 15 m above grade, including the height of any structure on which they are mounted.

1.3 Unusual designs and antennas

This Standard covers the requirements for most structural antennas, antenna towers, and antenna-supporting structures, but it is recognized that structures that are unusual with regard to their height or shape, or with regard to the shape and size of individual members, or that are located on sites having unusual topographical, geological, or climatic conditions might not be adequately covered. In such cases, appropriate engineering principles providing a level of reliability at least equivalent to that provided by this Standard should be applied.

1.4 Dynamic effects of wind

Other than the requirements of Annex N, this Standard does not contain mandatory requirements that take into account the dynamic effects of wind.

1.5 Other design approaches

A rational design based on theory, analysis, and engineering practice, acceptable to the owner and regulatory authority, may be used in lieu of the design procedures or materials described in this Standard. In such cases, the design should be prepared by an engineer qualified in the specific method and knowledgeable about the materials to be used, and should provide a level of safety and performance that is not less than that implicit in this Standard.

1.6 Local, provincial, and federal regulations

Responsibility for observing all applicable local, provincial, and federal regulations is not relieved by compliance with this Standard.

1.7 Terminology

In this Standard, "shall" is used to express a requirement, i.e., a provision that the user is obliged to satisfy in order to comply with the standard; "should" is used to express a recommendation or that which is advised but not required; and "may" is used to express an option or that which is permissible within the limits of the standard.

Notes accompanying clauses do not include requirements or alternative requirements; the purpose of a note accompanying a clause is to separate from the text explanatory or informative material.

Notes to tables and figures are considered part of the table or figure and may be written as requirements.

Annexes are designated normative (mandatory) or informative (non-mandatory) to define their application.

2 Reference publications

This Standard refers to the following publications, and where such reference is made, it shall be to the edition listed below, including all amendments published thereto.

CSA Group

A23.1-14/A23.2-14
Concrete materials and methods of concrete construction/Test methods and standard practices for concrete

CAN/CSA-A23.3-14
Design of concrete structures

CAN/CSA-B72-M87 (R2013)
Installation Code for Lightning Protection Systems

C22.1-18
Canadian Electrical Code, Part I

G4-15
Steel wire rope for general purpose and for mine hoisting and mine haulage

CAN/CSA-G12-14
Zinc-coated steel wire strand

G40.20/G40.21-13
General requirements for rolled or welded structural quality steel/Structural quality steel

G189-1966 (withdrawn)
Sprayed metal coatings for atmospheric corrosion protection

O86-14
Engineering design in wood

S6-14
Canadian Highway Bridge Design Code

S16-14
Design of steel structures

S136-16
North American Specification for the design of cold formed steel structural members

S136.1-16
Commentary on North American Specification for the design of cold formed steel structural members

S157-17/S157.1-17
Strength Design in Aluminum/Commentary on CSA S157-17, Strength Design in Aluminum

CAN/CSA-S832-14
Seismic risk reduction of operational and functional components (OFCs) of buildings

W59-13
Welded steel construction (Metal arc welding)

Z11-12
Portable Ladders

Z259 series of Standards:

Z259.1-05 (R2015)
Body belts and saddles for work positioning and travel restraint

Z259.2.2-17
Self-retracting devices

CAN/CSA-Z259.2.3-16
Descent devices

Z259.2.4-15
Fall arresters and vertical rigid rails

CAN/CSA-Z259.2.5-17
Fall arresters and vertical lifelines

Z259.10-12 (R2016)
Full body harnesses

Z259.11-17
Personal energy absorbers and lanyards

Z259.12-16
Connecting components for personal fall arrest systems (PFAS)

Z259.13-16
Manufactured horizontal lifeline systems

CAN/CSA-Z259.14-12 (R2016)
Fall restrict equipment for wood pole climbing

CAN/CSA-Z259.15-17
Anchorage connectors

CAN/CSA-Z259.16-17
Design of active fall-protection systems

CAN/CSA-Z259.17-16
Selection and use of active fall-protection equipment and system

ACI (American Concrete Institute)

307-08
Code Requirements for Reinforced Concrete Chimneys and Commentary

AISC (American Institute of Steel Construction)
ANSI/AISC 360-10
Specification for Structural Steel Buildings

AS (Standards of Australia Association)
3995-1994
Design of Steel Lattice Towers & Masts

AS/NZS 1170 Part 2:2011 (R2016)
Structural Design Actions — Wind Actions

ASCE (American Society of Civil Engineers)
10-15
Design of Latticed Steel Transmission Structures

Dynamic Response of Lattice Towers and Guyed Masts

48-11
Design of Steel Transmission Pole Structures

ASSE (American Society of Safety Engineers)
ANSI/ASSE A10.48-2016
Criteria for Safety Practices with the Construction, Demolition, Modification and Maintenance of Communication Structures

ASTM International
A27/A27M-17
Standard Specification for Steel Castings, Carbon, for General Application

A123/A123M-17
Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products

A148/A148M-15a
Standard Specification for Steel Castings, High Strength, for Structural Purposes

A153/A153M-16a
Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware

A307-14e1
Standard Specification for Carbon Steel Bolts, Studs and Threaded Rod, 60 000 psi Tensile Strength

A394-08 (2015)
Standard Specification for Steel Transmission Tower Bolts, Zinc-Coated and Bare

A474-03 (2013)
Standard Specification for Aluminum-Coated Steel Wire Strand

A475-03 (2014)
Standard Specification for Zinc-Coated Steel Wire Strand

A586-04a (2014)
Standard Specification for Zinc-Coated Parallel and Helical Steel Wire Structural Strand

A780/A780M-09 (2015)
Standard Practice for Repair of Damaged and Uncoated Areas of Hot-Dip Galvanized Coatings

B695-04 (2016)
Standard Specification for Coatings of Zinc Mechanically Deposited on Iron and Steel

F3125/F3125M-15a
Standard Specification for High Strength Structural Bolts, Steel and Alloy Steel, Heat Treated, 120 ksi (830 MPa) and 150 ksi (1040 MPa) Minimum Tensile Strength, Inch and Metric Dimensions
Note: *The ASTM F3125 specification is a consolidation and replacement of six ASTM standards, including A325, A325M, A490, A490M, F1852, and F2280.*

BSI (British Standards Institution)
BS ISO 5950-1 (1990)
Structural use of steelwork in building. Code of practice for design in simple and continuous construction: hot rolled sections

CARs (Canadian Aviation Regulations)
621.19
Standard Obstruction Markings

Canadian Geotechnical Society
Canadian Foundation Engineering Manual, 4th edition, 2006

CISC (Canadian Institute for Steel Construction)
Handbook of Steel Construction, 11th edition, 2017

European Union
EN 1991-1-4: 2005
Eurocode 1: Actions on structures — Part 1-4: General actions — Wind actions

EN 1993-3-2: 2006
Eurocode 3: Design of steel structures — Part 3-2: Towers, masts and chimneys — Chimneys

EN 1998-3: 2005
Eurocode 8: Design of structures for earthquake resistance — Part 3: Assessment and retrofitting of buildings

Government of Canada
Canada Labour Code — Part II, Regulations respecting occupational safety and health made under Part II of the Canada Labour Code

Industry Canada
Broadcasting Procedures and Rules, Parts I and II

National Research Council Canada
National Building Code of Canada, 2015

User's Guide — NBC 2015 Structural Commentaries (Part 4 of Division B)

Supplement to the National Building Code of Canada User's Guide — NBCC 2015 Structural Commentaries (Part 4 of Division B)

NCHRP (National Cooperative Highway Research Program)
Report 176 (2011)
Cost-Effective Connection Details for Highway Sign, Luminaire, and Traffic Signal Structures

TIA (Telecommunications Industry Association)
ANSI/TIA-222-G-2005
Structural Standards for Antenna Supporting Structures and Antennas

ANSI/TIA-222-H-2017
Structural Standard for Antenna Supporting Structures and Antennas and Small Wind Turbine Support Structures

ANSI/TIA 322-2016
Loading, Analysis, and Design Criteria Related to the Installation, Alteration and Maintenance of Communication Structures

Other publications
Adluri S.M.R. 1990. Ultimate Strength of Schifflerized Angles. Master of Applied Science Thesis. University of Windsor. Windsor, Canada.

Adluri S.M.R. and Madugula M.K.S. 1991. Factored Axial Compression Resistance of Schifflerized Angles, *Canadian Journal of Civil Engineering,* Vol. 18, No. 6.

Durst, C.S. 1960. Wind speeds over short periods of time. *The Meteorological Magazine*, July 1960, Vol. 89, No. 1056.

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Madugula M.K.S. and Adluri S.M.R. 1994. Design of 60° Equal-Leg Steel Angles according to CSA Standard S37-94, *Canadian Journal of Civil Engineering,* Vol. 22, No. 3.

Picard, A. and Beaulieu D. 1987. Design of Diagonal Cross Bracings Part 1: Theoretical Study, *Engineering Journal/American Institute of Steel Construction,* Third Quarter.

Sachs, P. 1978. *Wind Forces in Engineering,* 2nd edition, Pergamon Press.

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Sennah K., Saliba M., Jaalouk N., and Wahba J. 2009. Experimental study on the compressive resistance of stress-relieved solid round steel members, *Journal of Constructional Steel Research,* Vol. 65, No. 5.

3 Definitions and symbols

3.1 Definitions

The following definitions shall apply in this Standard:

Anchor —
a) an individual bar, bolt, dowel, rod, etc, that is embedded in rock, concrete, or soil to provide restraint against uplift and/or shear forces; or
b) concrete blocks, piles, or caissons and similar systems of other materials that provide anchorage for guy systems.

Antenna — a radiating element, used to transmit or receive radio frequency signals, including any reflectors and screens designed to produce a specific coverage.

Antenna array — a system of multiple antennas designed to produce a specific radiation pattern for one signal.

Antenna-supporting structure — a structure that serves to support one or more antennas or antenna arrays.

Antenna tower — a self-supporting tower, or the mast of a guyed tower, that is itself a radiating element.

Approved — approved by the regulatory authority.

Attachment antenna — an antenna that is not an integral component of the structure and is not larger than 1.4 m^2 in projected area, exclusive of ice buildup.

Note: *These antennas include small panel, dipole, screen, grid, and whip types.*

Attachments — antennas, transmission lines, ladders, safety devices, conduits, lighting cables and fixtures, platforms, and any other attachment not considered part of the main structure.

Bonding — a permanent and tight mechanical path of low impedance, joining non-current-carrying parts, to ensure electrical continuity.

Engineer — a person in the engineering profession with specific expertise in structural design and construction who is licensed to practice in a jurisdiction in Canada.

Factored load, αL — the product of a specified load and its load factor.

Factored resistance, ϕR — the product of the resistance of the member or component and the appropriate resistance factor.

Factors —

Cantilever factor — a factor applied to the member forces of cantilevers on guyed masts to take into account the dynamic forces that can occur in certain circumstances.

Efficiency factor — when referring to a guy assembly, the percentage ratio of the breaking strength of the complete guy assembly to the manufacturer's rated breaking strength of the strand or cable in the assembly.

Importance factor, γ — a factor applied to the factored loads to take into account the consequences of a collapse of the structure.

Load factor, α — a factor, appropriate for the limit state under consideration, applied to the specified load to take into account the variability of the load, the load patterns, and the analysis procedures.

Resistance factor, ϕ — a factor applied to a specified material property or to the resistance of a member, connection, or structure that, for the limit state under consideration, takes into account the variability of material properties, dimensions, type of failure, quality of work, and the uncertainty in prediction of member resistance.

Serviceability factor, τ — a factor applied to the wind load, W, to take into account the allowable annual duration of exceeding the serviceability limit state.

Forces — the reactions in the members of the structure due to the application of the defined loads as determined by the appropriate analyses.

Foundation — the base of the structure that transfers the forces resulting from the applied loads, including overturning moments where applicable, to the supporting ground.

Grade — the level of finished ground adjacent to the structure.

Grounding — a permanent electrically conductive path from any point on the structure or its attachments to ground, of sufficient capacity to carry any foreseeable current imposed on it.

Guy assembly — a strand or rope cable used to support a guyed structure, including the components for attaching the ends and adjusting the tension.
Note: *The assembly can include insulators or non-metallic material where applicable.*

Initial tension — the tension in the guy assembly, measured at the anchorage, before wind and ice loads are applied.

Lattice structure — an open framework of interconnected structural components.
Note: *Towers and masts typically have three or four upright members (legs) connected by webs (horizontal and diagonal components).*

Limit states — the conditions of a structure at which it ceases to fulfill the function for which it was designed.

Serviceability limit states — limit states that affect the intended use of the structure, and include deflection, deformation, and vibration.

Ultimate limit states — limit states concerning safety, and include load-carrying capacity, overturning, sliding, fracture, and fatigue.

Loads — actions on the structural system and its attachments of gravity (weight), environmental effects (e.g., wind, ice, and temperature), and other applicable effects (e.g., seismic action, and movement).

Mast — the central shaft of a guyed structure.

Pole structure — a structure consisting of a single slender element, which may be of solid or tubular construction, of uniform or varying cross-sections, and either guyed or self-supporting.

Post-critical telecommunication structures — structures that must remain serviceable during or shortly after a design-level earthquake due to their critical role in the emergency response and rescue effort.

Qualified technician — a person who has a diploma in Civil Engineering Technology with knowledge in structural engineering principles and inspections.

Regulatory authority — a federal, provincial, or municipal ministry, department, board, agency, or commission that has responsibility for regulating, by statute, the use of products, materials, or services.

Secondary structure — a framework, located inside or outside the structure, that is supported, in whole or in part, by the structure.

Snug-tight — the tightness condition of a fastener that exists as a result of a few impacts of an impact wrench or the full effort of a person using an ordinary spud wrench.

Structural antenna — an antenna that is an integral component of the structure, or any antenna larger than 1.4 m^2 in projected area, exclusive of ice buildup.

Note: *These include microwave, satellite earth station, and broadcast antennas.*

Structure (where used without qualification) — any structural antenna, antenna tower, or antenna-supporting structure, including its foundations and anchorages.

Tilt — the rotation of the structure in the vertical plane, measured as the angle between the vertical and the tangent to the final deflected position of the structure at the level in question.

Tower (where used without qualification) — a vertical, relatively slender structure, either guyed or self-supporting, used as an antenna or antenna-supporting structure.

Transmission line — a cable or waveguide that transmits the signal between an antenna and the transmitting or receiving equipment.

Twist — the horizontal rotation of the structure in its final deflected position at the level in question.

Ultimate geotechnical resistance — resistance of the soil or rock consistent with a failure state generally corresponding to the maximum shear resistance of the soil or rock at appropriate strain in the soil or rock.

Note: *In specific cases in the text, the term "geotechnical" may be replaced by one of the following: "bearing", "pull-out", or "horizontal".*

3.2 Symbols

The following symbols shall apply in this Standard:

A = area

A_b = cross-sectional area of a bolt, based on its nominal diameter

A_f = face area of flat members, bare

A_g = gross area

A_I = face area of radial ice

A_n = critical net area

A'_{ne} = effective net area reduced for shear lag

A_{ne} = effective net area

A_r = face area of round members, bare

A_s = net projected area

B_r = factored bearing resistance of a member

b = width of leg of angle

C_a = wind speed-up factor for wind on antennas or structures located on or near the roof surface of buildings

C_d = drag factor

C_{df} = drag factor for flat members

C_{dr} = drag factor for round members

C_e = height factor

C_f = axial compressive force under factored loads

C_g = gust effect factor

C_r = factored axial compressive resistance

D = dead load

D_o = outside diameter of tubular section

D_p = average exterior diameter or average least width of a tubular pole structure

d = diameter of bolt or round member

E = earthquake effects

e = end or edge distance from centre of hole in line with the direction of the force

F_a = acceleration-based site coefficient, as defined in Sentence 4.1.8.4(4) of the *National Building Code of Canada*

F_u = tensile strength of the plate, angle member, or bolt

F_v = velocity-based site coefficient, as defined in Sentence 4.1.8.4(4) of the *National Building Code of Canada*

F_y' = effective yield stress

F_y = yield stress

g = transverse spacing between fastener gauge lines; distance from the heel of the angle to the bolt centroid

H_a = building height, or width of the building wall facing the wind

H_x = height above grade of the portion of the structure being considered

I = ice load

K = effective length factor

KL = effective length

k_m = local equivalent length factor

L = length

L_n = net length

L_{xx}, L_{yy} = member lengths about the x and y axes of a built-up member

ℓ = distance between intermittent fillers

M_f = bending moment under factored loads

M_r = factored moment resistance

m = number of shear planes in a bolted joint

n = number of bolts

P = design wind pressure

q = reference velocity pressure

q_h = site-specific wind pressure profile

R = seismic force response modification coefficient, defined in Clause 5.12.5.3

R_a = the ratio of the projected area of the attachments (perpendicular to the wind direction) to the projected area of the tubular structure for the section being considered

R_s = solidity ratio

r = radius of gyration

r_{min} = minimum radius of gyration of an individual component of a built-up member

r_{xx}, r_{yy} = radius of gyration of the built-up member about the principal axes, x and y, of a built-up member

S = elastic section modulus of a section

$S(T_n)$ = design spectral response acceleration, expressed as a ratio-to-gravitational acceleration, for a period of T_n seconds, as defined in Sentence 4.1.8.4. of Part 4 Division B Volume 2 of the *National Building Code of Canada*

$S_a(0.2)$ = uniform hazard spectral acceleration, expression as a ratio to gravitational acceleration, for a period of 0.2 s, as defined in Appendix C Division B Volume 2 of the *National Building Code of Canada*

s = centre-to-centre longitudinal spacing (pitch) of any two successive fastener holes

T = temperature; temperature effects

T_f = tensile force in a member under factored load

T_n = natural period of the structure

T_r = factored tensile resistance

t = nominal leg thickness; wall thickness; thickness of member material; ice thickness

V = wind velocity

V = horizontal seismic force, defined in Clause 5.12.5

V_f = shear force in a member under factored load

V_r = factored shear resistance of a member

W = wind load

W = weight of the structure for seismic force calculation, defined in Clause 5.12.5

W_F = wind load for fatigue loading combination

w = effective leg width, as shown in Figure 13; actual flat side dimension, but not less than the dimension calculated using a bend radius equal to $4t$

w_n = net width, determined from gross width less design allowance for holes within the width

α = load factor

α_D = load factor for dead load

α_E = load factor for earthquake load

α_I = load factor for ice load

α_T = load factor for temperature effects

α_W = load factor for wind load

β = an interaction factor derived from test results

γ = importance factor

λ = non-dimensional slenderness ratio

τ = serviceability factor

ϕ = resistance factor

ϕ_b = bolt resistance factor

ψ = load combination factor

4 Design requirements

4.1 General

Antennas, towers, and antenna-supporting structures shall be designed to be serviceable and safe from structural failure during the useful life of the structure.

Note: *The procedure to assess such structures, as set out in this Standard, uses limit states design, the object of which is to keep the probability of reaching any limit state below the acceptable limit. This is achieved by applying load factors to the specified loads and resistance factors to the specified resistances.*

4.2 Ultimate limit states

A structure designed to this Standard shall have sufficient strength and stability to ensure that the factored resistance equals or exceeds the forces due to the factored loads and that fatigue requirements are met.

4.3 Serviceability limit states

A structure designed to this Standard shall have sufficient rigidity to ensure that the specified limits of tilt and twist, or combinations thereof, equal or exceed the effects of the factored loads.

4.4 Engineering

4.4.1 General

Design or evaluation of a structure in accordance with this Standard shall be carried out by an engineer and shall take into account the conditions prevailing at the location of the structure and their effect on the design loads.

4.4.2 Communication structure on a building

Where a structure designed to this Standard is to be supported by a building or other type of structure, a suitably qualified engineer shall confirm that such a building or other structure conforms to the appropriate standard under the added loads.

4.4.3 Drawings

Engineering drawings and diagrams should include the information outlined in Annex A.

4.5 Existing structures

4.5.1 Inspections and maintenance

Existing structures shall be inspected on a regular basis, and any recommended maintenance shall be carried out in a timely manner to ensure that the structure continues to perform as intended.

Note: *See Annex D for recommendations.*

4.5.2 Modifications in structure or antenna loading

To be in conformance with this edition of this Standard, when an existing structure or its attachments are modified,

a) the physical condition and information required for analysis of the structure shall be verified by inspection;
b) the adequacy of the structure shall be evaluated in accordance with this Standard; and
c) any structural deficiencies relative to this Standard shall be corrected.

Note: *Both addition and removal of attachments can result in increases in the forces in some members of the structure.*

4.6 Analysis scenarios

The analysis of the tower shall consider scenarios that include, as a minimum, the existing and proposed antenna loading scenario and any other scenarios with known future antenna loading combinations.

Note: *It is important to recognize that the highest member forces do not necessarily occur with the highest loading.*

5 Loads

5.1 Dead load, *D*

Dead load, *D*, shall be determined as the weight of the structure itself plus the weight of all attachments.

5.2 Ice load, *I*

5.2.1 General

Ice load, *I*, shall be determined as the weight of glaze ice formed radially on all exposed surfaces of the structure, including guys and attachments. Where the gap between two adjacent parallel members or attachments is equal to, or less than, twice the radial ice thickness, the ice between them shall be considered to form a flat surface, tangential to the radial ice around each. The density of the ice shall be taken as 900 kg/m^3.

5.2.2 Local conditions

The minimum glaze ice thickness for any location shall be as given by Figure 1. Local topography and other factors specific to each site shall be considered in determining the class of icing. The probability of ice thickness increasing at sites adjacent to large bodies of water or exposed to in-cloud icing should be considered.

Note: *For a further discussion of icing, see Annex E.*

5.2.3 Escalation with height

The reference ice thickness, t_i, as given by Figure 1, shall be escalated with height in accordance with the following formula:

$$t_{iz} = t_i \times K_{iz}$$

where

$$K_{iz} = \left(\frac{H_x}{10}\right)^{0.1}$$

$0.9 \le K_{iz} \le 1.25$

5.2.4 Ice on guy cables

For guyed structures, the value of t_{iz} at the mid-height of the guy may be used for its full length.

5.3 Design wind pressure, *P*

5.3.1 General

For structures other than those covered by Clause 5.3.3, the design wind pressure, *P*, shall be determined by the following formula:

$P = q_h C_g C_a$

where

P = the pressure of the undisturbed flow, independent of drag factor

q_h = the 50-year return wind velocity pressure at elevation 'h' above the base of the tower as developed in a site-specific assessment, see Clause 5.3.2.2
In the absence of a site-specific assessment of the wind, $q_h = qC_e$ may be used

where

q = the reference velocity pressure, see Clause 5.4

h = the reference height, m

C_e = the height factor, see Clause 5.5

C_g = the gust effect factor, see Clause 5.6

C_a = the speed-up factor for wind on antennas or structures on roofs, see Clause 5.7

Note: *The formula $q_h = qC_e$ applies over flat terrain away from large bodies of water for a distance of at least 1.0 km or 10 times the height of the structure, whichever is greater. For further discussion of wind, see Annex E.*

5.3.2 Local conditions

5.3.2.1 General

The wind pressure profile providing values of wind pressure as a function of height above grade, over flat ground away from large bodies of water, shall be obtained from the formula $q_h = qC_e \ge 290$ Pa. C_e shall be based on the appropriate terrain as specified in Clause 5.5.1 or as determined from Clause 5.3.2.2. Topography and other local conditions shall be considered since these conditions can result in higher values of q_h.

Note: *Clause 5.4.1 limits the minimum value of q to 320 Pa.*

5.3.2.2 Site specific wind values

Site-specific wind values and profiles shall be obtained in a two-step process:

a) a reference wind speed representative of the region is determined; and
b) the reference wind speed is adjusted for site roughness and topographical features using generally accepted methods.

Site specific wind velocity pressure profiles shall be obtained from Environment and Climate Change Canada (ECCC) or other qualified atmospheric scientists or wind engineers using appropriate wind historical data.

Note: *Refer to Annex E for determination of site-specific wind values and profiles.*

5.3.3 Small antennas on buildings

Where an antenna of less than 5 m^2 in a projected area is mounted on a building or other structure that has been designed in accordance with the *National Building Code of Canada* or other applicable codes, the design wind pressure shall be calculated as in Clause 5.3.1, except that q may be taken as the once-in-10-year return period hourly wind pressure for the site, with C_g = 2.5.

5.3.4 Short-term loading on a structure

For load cases with attachments being installed on a structure for a period of 4 months or less, the reference wind velocity pressure, q, may be taken as the 10-year period mean hourly wind pressure at 10 m above ground level, as appropriate for the site. In addition, the ice load may be omitted if the short-term loading period falls outside the typical freezing temperature range for the applicable region.

Note: *This Clause is not intended to include construction loads.*

5.4 Reference velocity pressure, *q*

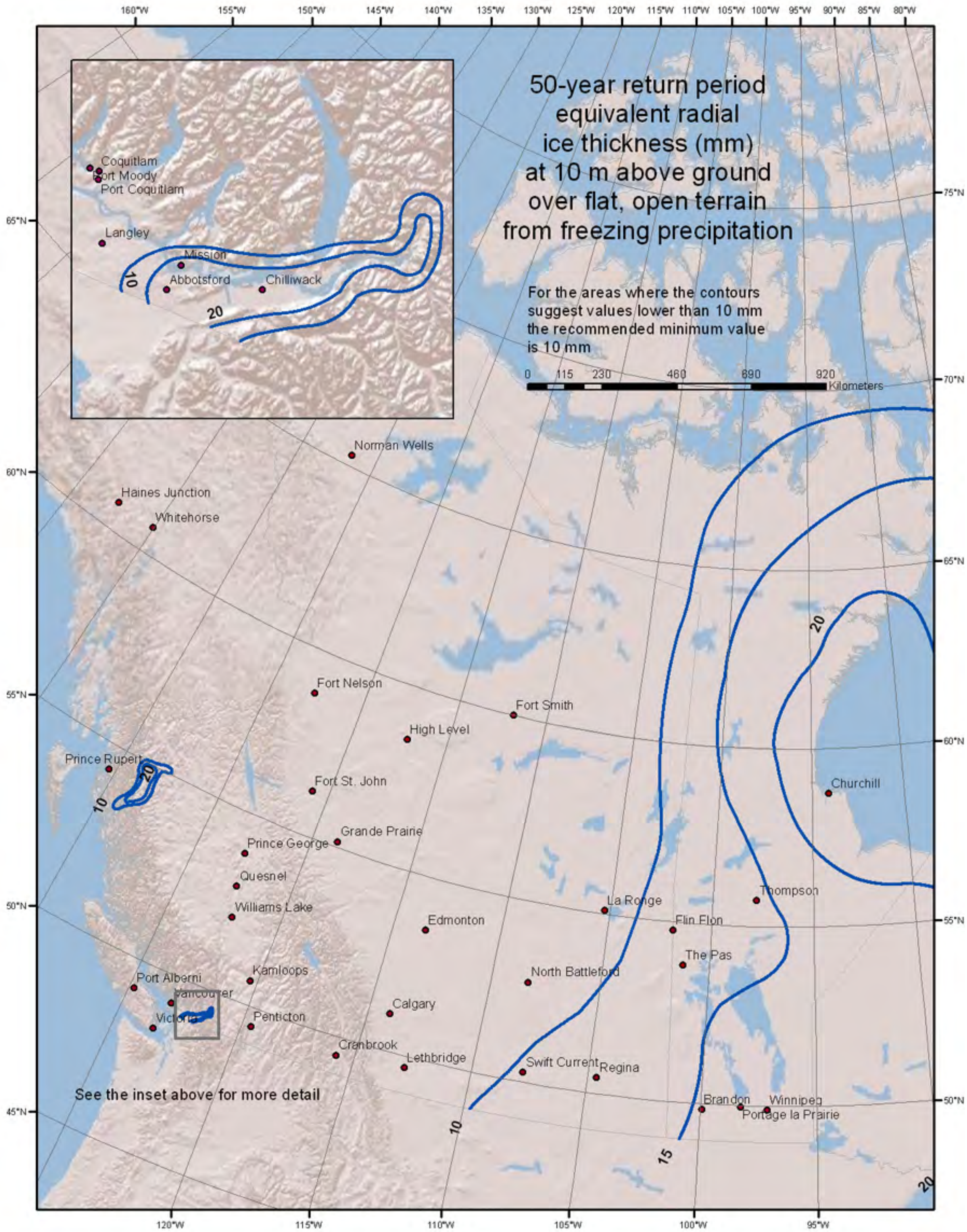
5.4.1 General

The reference velocity pressure, q, shall be the 50-year return period mean hourly wind pressure at 10 m above ground level, as appropriate for the site, but not less than 320 Pa.

5.4.2 Sources of data

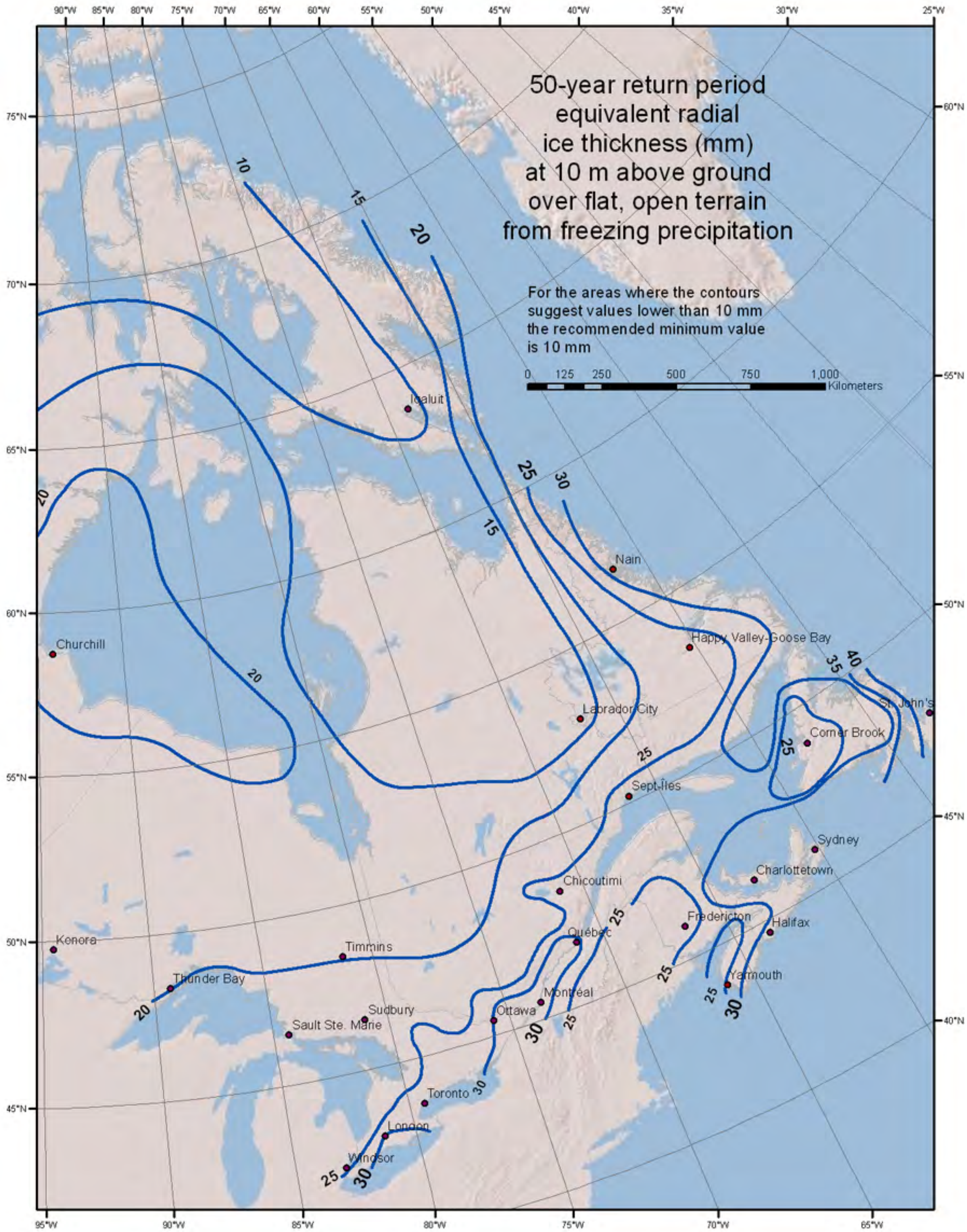
Subject to Clause 5.3.2.1, values of q representative of flat terrain, 10 m above ground and away from large bodies of water, may be obtained from the *National Building Code of Canada* or, in the absence of other available information, from Annex E.

Figure 1
Ice map
(See Clauses 5.2.2, 5.2.3, E.1, and S.5.2.)



(*Continued*)

Figure 1 (Concluded)



5.5 Height factor, C_e

5.5.1 General

Unless modified by site considerations (see Clause 5.3.2.1), the height factor, C_e, shall be determined from the following expressions, depending upon the roughness of the surrounding terrain:

a) Open terrain: For level terrain with only scattered buildings, trees or other obstructions, open water, or shorelines:

$C_e = C_{eo}$

where

C_{eo} = the height factor for open terrain = $\left(\dfrac{H_x}{10}\right)^{0.2}$ $0.9 \le C_{eo} \le 2.0$

where

H_x = the height above grade, m

b) Rough terrain: Suburban, urban, or wooded terrain extending upwind from the structure in question uninterrupted for at least 1 km or 10 times the height of the structure, whichever is greater:

$C_e = C_{er}$

where

C_{er} = the height factor for rough terrain = $0.7\left(\dfrac{H_x}{12}\right)^{0.3}$ $0.7 \le C_{er} \le 2.0$

c) Intermediate terrain: Suburban, urban, or wooded terrain extending upwind from the structure in question uninterrupted for at least 0.05 km, but no more than 1.00 km or 10 times the height of the structure, whichever is greater:

when $0.05\text{ km} < X_r < 1.00\text{ km}$: $C_e = C_{er}\left[0.816 + 0.184\log_{10}\left(\dfrac{10}{X_r - 0.05}\right)\right] \le C_{eo}$

when $X_r >= 1.00$ km: $C_e = C_{er}$

where

X_r = the upstream extent of the rough terrain

5.5.2 Maximum extent for one value of q_h

Lattice towers with small panels and pole structures may be considered in sections not exceeding 6 m in height, using the value of q_h at the mid-height of the section. Large panels shall be considered individually, using the value of q_h at the mid-height of the panel.

5.5.3 Wind on guy cables

For guyed structures, wind loads on the guys shall be considered. For this purpose, the value of q_h at mid-height may be used for its full length.

5.5.4 Wind on attachments

For structural antennas and discrete attachments, the value of q_h shall be taken at the centroid of the projected area.

5.6 Gust effect factor, C_g, and dynamic response

5.6.1 General

For structures designed to this Standard, the dynamic response to wind shall be taken into account by the gust effect factor, C_g, applied to the wind pressure for static analysis where C_g = 2.0 for the structure, its attachments and structural antennas.

Note: *See also Annex N for tubular structures (poles and shrouded tripoles).*

5.6.2 Consideration of dynamic effects

A more detailed consideration of the dynamic effects of wind may be warranted for towers over 250 m in height or structures with unusual design or loading considerations. The loads corresponding to the peak dynamic response should be used for design. When dynamic effects are adequately considered, a static analysis in accordance with this Standard shall not be required.

Note: *A patch loading procedure for evaluating the peak dynamic response is given in Annex H.*

5.7 Roof wind speed-up factor

5.7.1 General

Except as given in Clause 5.7.2, the wind speed-up factor, C_a, shall be taken as 1.0.

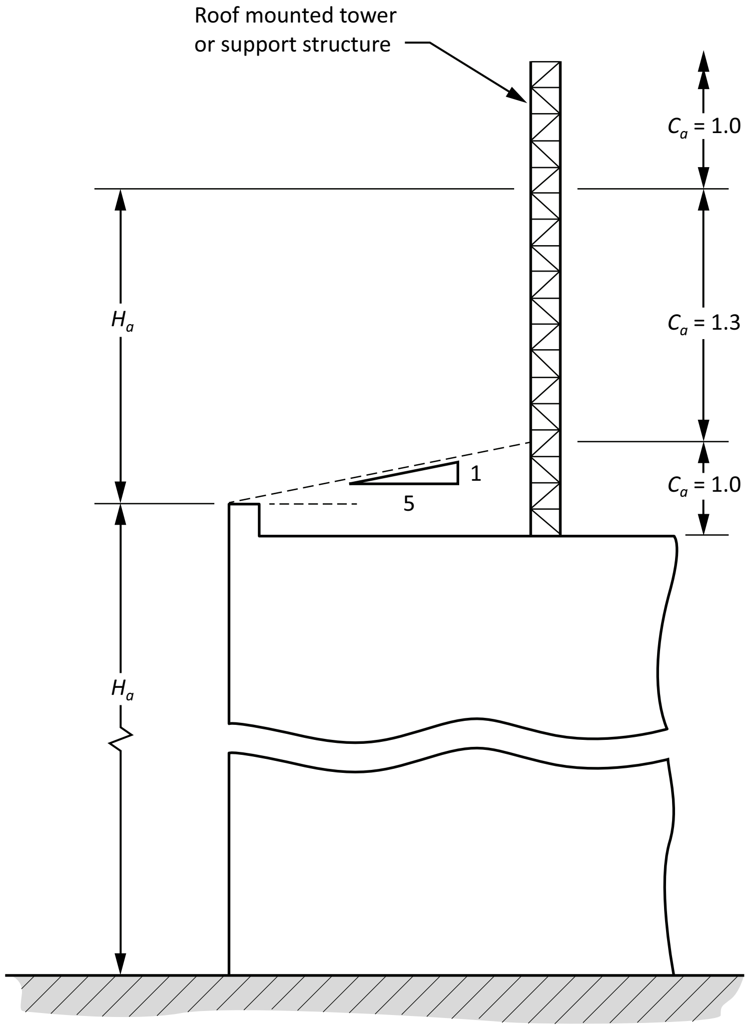
5.7.2 Speed-up on roofs

On roofs, accelerated wind flows causing increased wind loads can be experienced. Unless a different value can be shown by wind tunnel tests or other information, C_a shall be taken as 1.3 for that portion of the projected area of a roof-mounted structure between a line extending upwards at a slope of 1 (vertical) to 5 (horizontal) from the top edge of the roof or parapet and a height, H_a, above the top edge of the roof or parapet, where H_a is equal to the height or the width, whichever is less, of the windward face of the building. See Figure 2.

Notes:

1) *Near roof edges and close to corners, values of C_a higher than 1.3 may be appropriate. Caution is advised in these areas.*
2) *For roof installations affected by the wind flows around adjacent taller structures, or where the building on which the antenna is mounted is of unusual shape, appropriate boundary layer wind tunnel studies are recommended.*

Figure 2
Roof wind speed-up factor (C_a)
(See Clause 5.7.2.)



H_a = height or width of the roof of the windward face of the building, whichever is less.

C_a = wind speed-up factor

5.8 Wind load, *W*

5.8.1 Lattice towers

For lattice structures with round, flat, or a combination of round and flat members, or for iced structures, the wind load, *W*, shall be the wind pressure, *P*, times the sum of each partial face area times its appropriate drag factor, as given by Clause 5.9.1:

$$W = P\left(C_{df} \times A_f + C_{dr} \times A_r + C_{dr} \times A_i\right)$$

where

W = wind load

P = wind pressure

C_{df} = drag factor for flat members

A_f = face area of flat members, bare

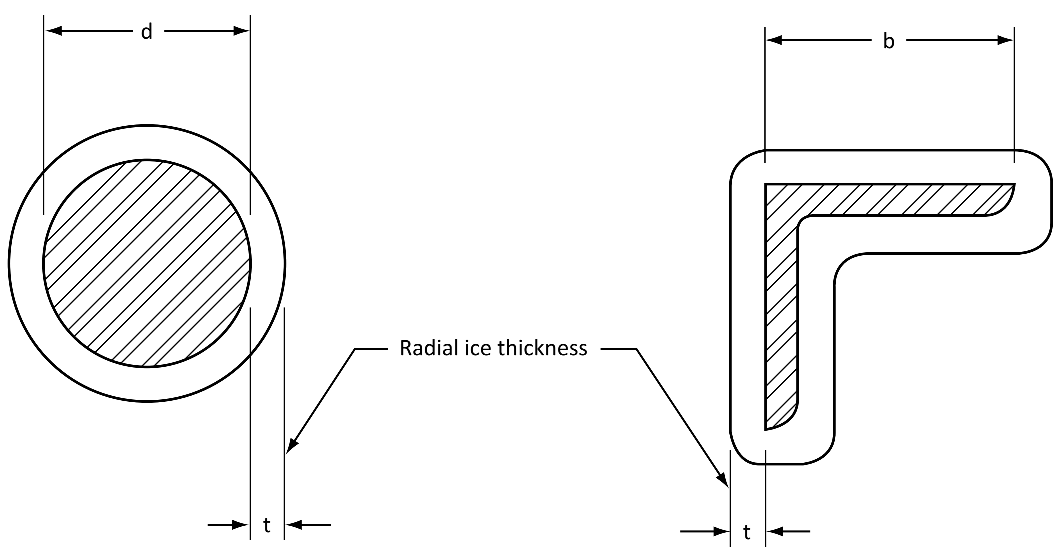
C_{dr} = drag factor for round members

A_r = face area of round members, bare

A_i = face area of radial ice, in accordance with Figure 3

Face area is the net area of the members of one face of the structure, projected normal to that face. Members not in the face being considered, including internal horizontal bracing, may be ignored in determining the projected area of the structure.

Figure 3
Face area of radial ice
(See Clause 5.8.1.)



$A_r = d \times L$
$A_f = b \times L$
$A_i = 2t \times L$
where
L = length of member
t = ice thickness

5.8.2 Pole structures

For pole structures, the wind load shall be the wind pressure, P, times the net projected area, A_s, taken as the actual projected area, based on the diameter or overall width, times the appropriate drag factor as given by Clause 5.9.2. For iced pole structures, the value of A_s shall include the area of the radial ice.

5.8.3 Attachments

For attachments, the wind load shall be the wind pressure, P, times the net projected area, A_s, times the appropriate drag factor. Radial ice shall be considered as in Clause 5.8.1. Any secondary structure shall be considered as in Clause 5.8.1. Shielding of the secondary structure by the main structure may be considered where appropriate.

5.9 Drag factor, C_d

5.9.1 Latticed towers and masts

For latticed towers and masts, the drag factor, C_d, is dependent upon the solidity ratio, R_s, and shall be obtained as follows:

a) for flat members on
   i) square towers, $C_{df} = 4.0\ (R_s)^2 - 5.9\ (R_s) + 4.0$; and
   ii) triangular towers, $C_{df} = 3.4\ (R_s)^2 - 4.7\ R_s + 3.4$;

b) for round members,
   $C_{dr} = C_{df}\ (0.57 - 0.14\ R_s + 0.86\ R_s^2 - 0.24\ R_s^3)$ when $N < 3.25$ and for all iced conditions (subcritical flow)
   $C_{dr} = C_{df}\ (0.36 + 0.26\ R_s + 0.97\ R_s^2 - 0.63\ R_s^3)$ when $N > 6.5$ for all no-ice conditions (supercritical flow); linear interpolation may be used when $3.25 \le N \le 6.5$ using the appropriate value of C_{df} from Item a), except that C_{dr} shall not exceed C_{df},
   where
   C_{dr} = drag factor for round members
   C_{df} = drag factor for flat members
   $R_s = A_s/A_g$
      where
      A_s = net projected area of one face of the structure, i.e., for one panel, the projected area of all members within that panel, including ice thickness but excluding any attachments
      A_g = gross area of one face of the structure, i.e., for one panel, the panel height times the out-to-out face width, including ice thickness where appropriate
      $N = D\ (q\ Cg\ Ce\ Ca)^{0.5}$
      D = diameter of round member, m

c) for square towers, C_d shall be multiplied by the factor K_d, which shall be calculated using the following formula:
   $K_d = 1 + K_1 K_2 \mathrm{Sin}^2 2\theta$
   where
   K_1 = 0.55 for flat members; 0.80 for round members
   $K_2 = 0.2$ for $0.0 < R_s \le 0.2$
      $= R_s$ for $0.2 < R_s \le 0.5$
      $= (1 - R_s)$ for $0.5 < R_s \le 0.8$
      $= 0.2$ for $0.8 < R_s \le 1.0$
   θ = the angle of incidence of the wind measured from a direction normal to face in plan $-45° \le \theta \le 45°$

d) for triangular towers, C_d shall be multiplied by the factor K_d, which shall be calculated using the following formula:
   $K_d = 1 - 0.1 \mathrm{Sin}^2(1.5\theta)$
   K_d = 1.00 for round members
   θ = the angle of incidence of the wind measured from a direction normal to face in plan $-60° \le \theta \le 60°$

5.9.2 Pole structures

5.9.2.1 General

The drag factors, C_d, for smooth pole structures, whether guyed or cantilevered, shall be as given in Table 1. These values are for smooth shapes and include transcritical and supercritical flow conditions.

5.9.2.2 Attachments

Where there are attachments on the outside of a smooth pole structure, the values given in Table 1 shall be modified to obtain the effective drag factor, C'_d. Values of C'_d shall be calculated as follows:

a) When $R_a \leq 0.1$, $C'_d = (1 + 3\ R_a)C_d$ and $C'_d \leq 1.4$. In this case, the projected areas of the attachments may be ignored.

b) When $R_a > 0.1$, $C'_d = 1.3C_d$ and $C'_d \leq 1.4$. In this case, the projected areas of the attachments shall be considered separately (see Clause 5.9.4) and added to the wind load on the pole

where

R_a = A_a/A_p

where

A_a = the total exposed area of attachments, perpendicular to the wind direction, for the section being considered

A_p = the area of the smooth pole structure for the section being considered

Table 1
Drag factor, C_d, for smooth pole structures
(See Clause 5.9.2 and S.5.9.2.)

	Round	18-sided	16-sided	12-sided	8-sided
$N < 3.25$	1.2	1.2	1.2	1.2	1.2
$3.25 \leq N \leq 6.5$	$\frac{5.64}{N^{1.3}}$	$\frac{3.44}{N^{0.89}}$	$\frac{2.58}{N^{0.65}}$	$\frac{2.45}{N^{0.6}}$	1.2
$N > 6.5$	0.5	0.65	0.76	0.8	1.2

where

$N = D_p(q\ C_g\ C_e\ C_a)^{0.5}$

D_p = average exterior diameter or average least width of a tubular pole structure of the section being considered, m

Note: *Reynolds number = N multiplied by 83 000.*

5.9.3 Guys

For guys, whether bare or iced, the value of C_d shall be taken as 1.2.

5.9.4 Antennas and other attachments

5.9.4.1 Discrete attachments

Drag coefficients, C_d, for antennas and other attachments shall be based on published data. In the absence of published data, Table 2 shall be used.

Where attachments are located inside the inclusion zone, as defined in Figure 4, a drag factor as determined by Table 2 shall be used, multiplied by K_a,

where

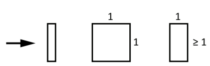
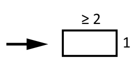
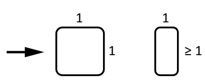
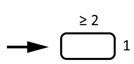
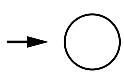
K_a = $(1 - R_s) \leq 0.6$

= 1.0 for round attachments when $N \geq 3.25$

Where attachments are located outside the inclusion zone in clusters of 3 or more, at approximately the same elevation and distributed around the mast, a value of $K_a = 0.8$ may be used with the C_d from Table 2 for flat attachments and for round attachments with $N \leq 3.25$.

When a value of K_a of 0.8 is applied, no other shielding factors shall be used for the same attachments, antennas mounted on them, or the mast.

Table 2
Drag coefficients, C_d, for attachments
(See Clause 5.9.4 and S.5.9.4.1.)

Member type		Aspect ratio ≤ 2.5	Aspect ratio = 7	Aspect ratio ≥ 40
		C_d	C_d	C_d
	Flat, square, or rectangle	1.20	1.40	2.00
	Rectangle	0.90	1.05	1.50
	Square or rectangle with rounded corners	0.90	1.05	1.50
	Rectangle with rounded corners	0.70	0.80	1.15
	$N < 3.25$ (Subcritical)	0.70	0.80	1.2
	$3.25 \leq N \leq 6.5$	$1.47/(N)^{0.63}$	$1.72/(N)^{0.65}$	$5.65/(N)^{1.3}$
	$N > 6.5$ (Supercritical)	0.45	0.50	0.50

where

$N = D\,(q\,C_g\,C_e\,C_a)^{0.5}$

D = the outside diameter of the attachment, m

Aspect ratio is the overall length/width ratio in the plane normal to the wind direction.

Notes:
1) *For iced conditions, C_d is based on subcritical flow for all values of N.*
2) *For intermediate values of aspect ratio and member shape, use linear interpolation between columns and rows values respectively.*

5.9.4.2 Linear attachments

Where linear attachments are located inside the inclusion zone, as defined in Figure 4, a drag factor as determined by Table 2 shall be used, multiplied by K_a,

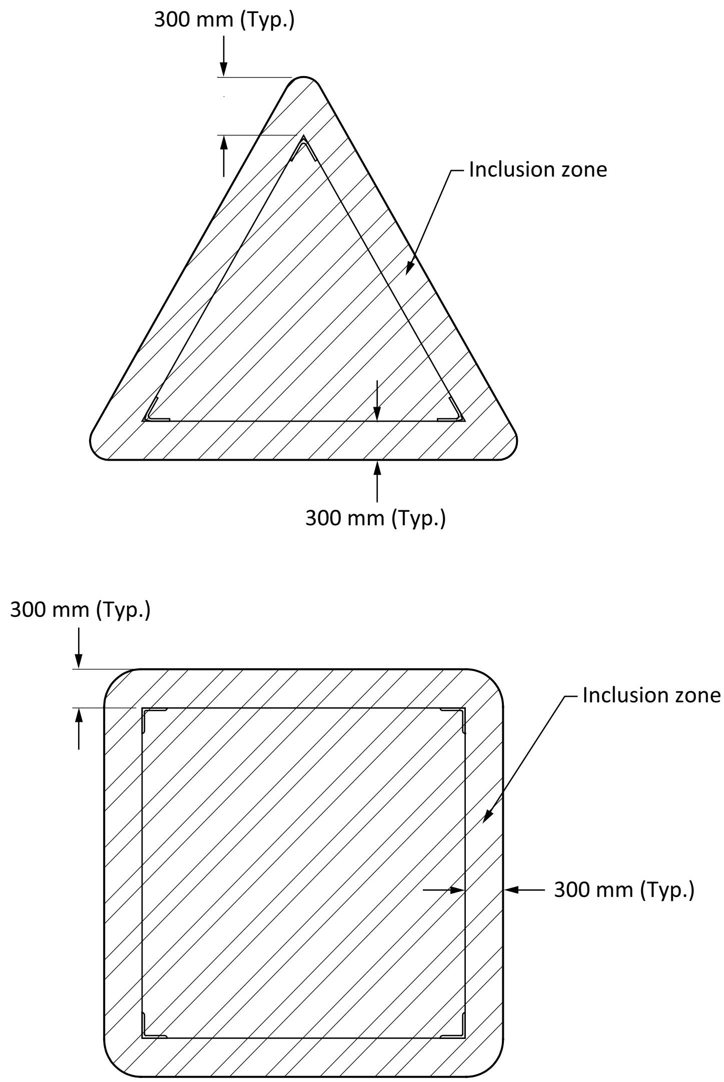
where

K_a = $(1- R_s) \le 0.6$

= 1.0 for round attachments under super critical flow conditions

Where linear attachments do not meet the requirements of being within the inclusion zone, as defined in Figure 4, K_a of 1.0 shall be used.

Figure 4
Inclusion zone for attachments
(See Clause 5.9.4.2.)



5.9.4.3 Special antenna assemblies

Certain types of antenna assemblies (e.g., those that incorporate curved reflective surfaces) can give rise to loads exceeding the "full-face" load when subjected to an oblique wind. Drag factors for these antennas shall be based on published data.

5.9.4.4 Total antenna loads

Where two or more antennas or antenna panels are at the same elevation, the total load at that level may be considered as the sum of the loads from each antenna due to the wind from the direction being considered (see Clause 6.5).

5.9.4.5 Effective projected area

The effective projected area of an attachment is equal to the drag factor of the shape of the attachment multiplied by its projected area. The effective projected area, $(EPA)_\theta$, of an attachment shall be determined from the following equation (see Figure 5):

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

$$(EPA)_{\theta+90} = K_a[(EPA)_T \sin(\theta)\cos(\theta) - (EPA)_N \cos(\theta)\sin(\theta)]$$

where

$(EPA)_\theta$ = effective projected area for wind direction at an angle θ measured from the normal

$(EPA)_{\theta+90}$ = effective projected area for wind direction at an angle θ+90 measured from the normal

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

$(EPA)_T = C_dT(P_A)_T$ = effective projected area associated with the windward face parallel to the azimuth of the attachment

P_A = projected area of the attachment as applicable

C_{dN} = drag factor associated with the windward face normal to the azimuth of the attachment

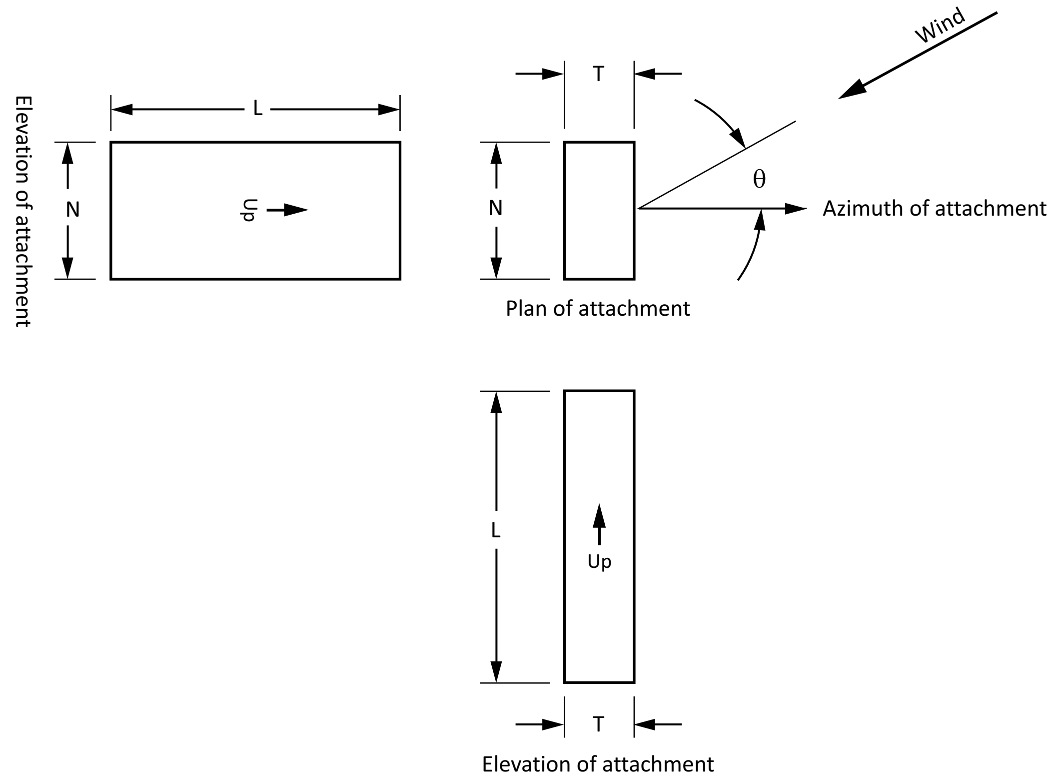
C_{dT} = drag factor associated with the windward face parallel to the azimuth of the attachment

$(P_A)_N = L \times N$

$(P_A)_T = L \times T$

Note: *$(EPA)_\theta$ and $(EPA)_{\theta+90}$ effective projected areas are to be considered concurrently in the load calculations.*

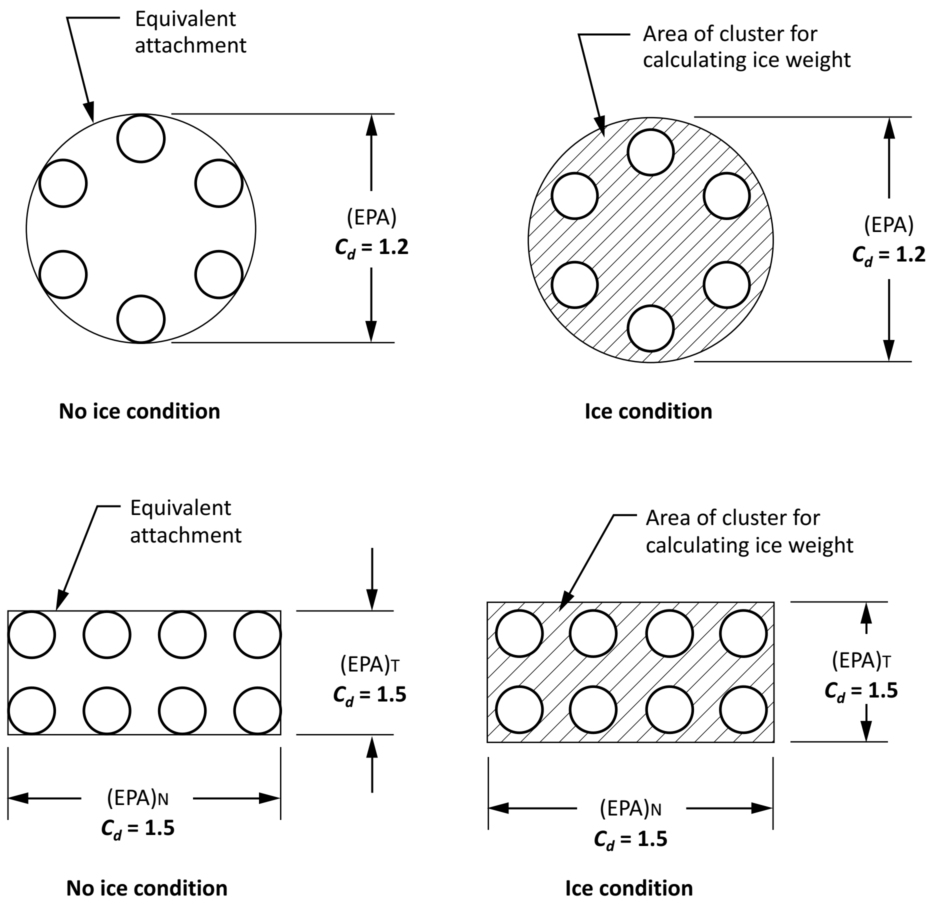
Figure 5
Effective projected area of the attachment
(See Clause 5.9.4.5.)



5.9.4.6 Clusters of linear attachments

Where linear attachments are mounted in a cluster, the projected area of each line shall be considered except that the group of lines need not be considered larger than an equivalent attachment with a width equal to the maximum out-to-out dimension of the group for both the normal and transverse sides with a drag factor, C_d, equal to 1.5 for square or rectangular clusters and 1.2 for round clusters. See Figure 6.

Figure 6
Linear attachments in a group or cluster
(See Clause 5.9.4.6.)



5.9.4.7 Limiting wind load on tower section

The total wind load on the structure plus attachments shall not exceed the load on the structure determined in accordance with a solidity ratio of 1.0 (solid-faced), plus the wind load on externally-mounted attachments not shielded by the structure when it is considered to be solid-faced. Shielding by the solid-faced mast may be considered for attachments that are at a distance behind the windward face of the structure of not more than 1.5 times the face width for triangular towers and 2.0 times the face width for square towers.

5.9.4.8 Unusual shapes

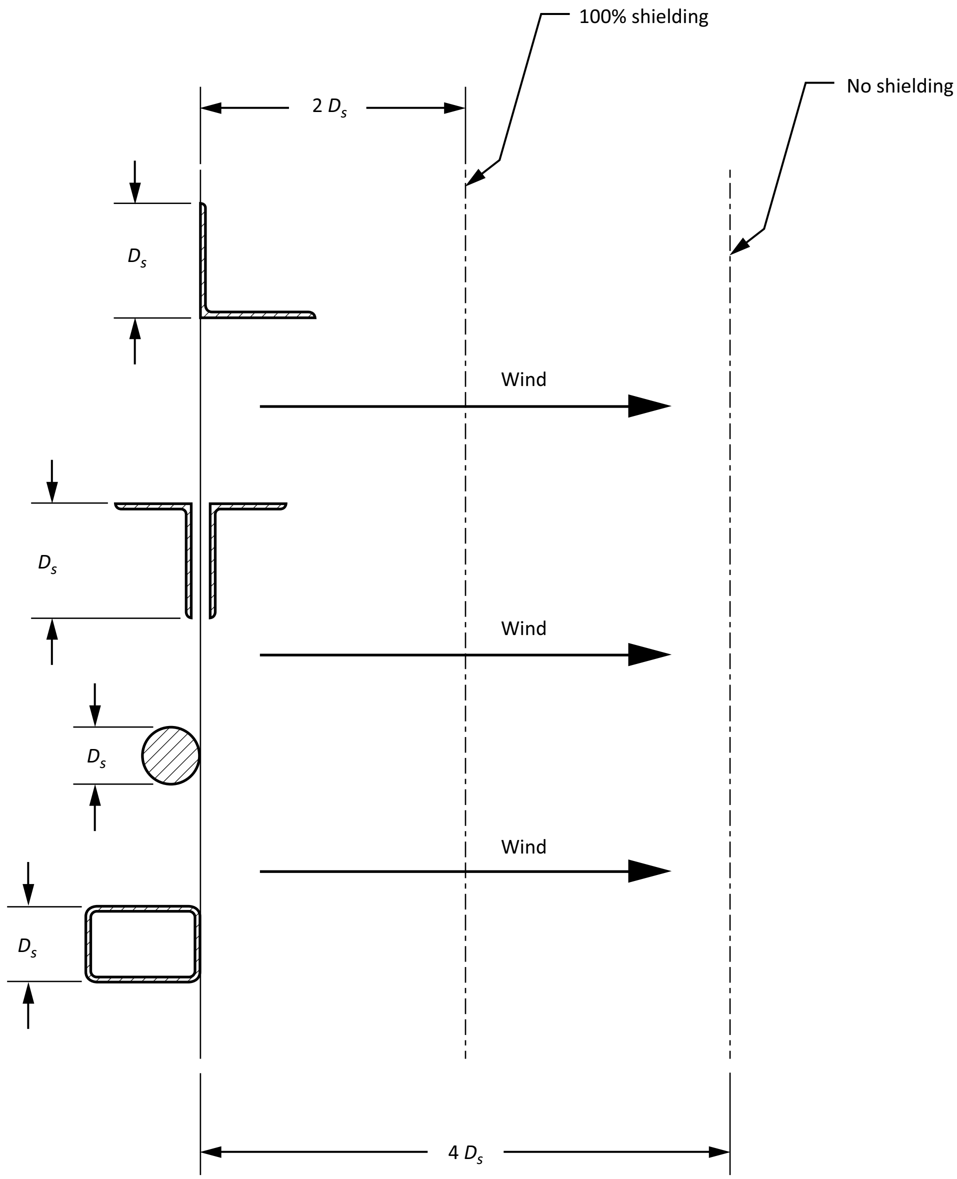
For members with unusual shapes and large cross-sections, the drag factor shall be based on experimental data or on published information such as that contained in the *National Building Code of Canada* or other accepted standards and publications.

5.10 Shielding

5.10.1 Shielding of members and attachments

Full shielding of one member or attachment by another shall be considered to occur if the distance between the members is not greater than twice the projected width of the upwind member, measured from the closest point at which that width occurs. No shielding shall be considered if the distance is greater than four times the projected width of the upwind member. Linear interpolation shall be allowed if the distance is between twice and four times the projected width (see Figure 7).

Figure 7
Limits for winds shielding
(See Clause 5.10.1.)



5.10.2 Shielding of antennas

Shielding of a leeward antenna by a windward antenna may be considered if, for the wind direction being considered,

a) the back-to-back distance between the antennas is less than twice the minimum dimension of the windward antenna, measured perpendicular to the wind direction;

b) the leeward antenna has at least 50% of its projected area, as measured in a plane perpendicular to the wind direction, in the shadow of the projected area of the windward antenna viewed from the direction from which the wind is blowing; and
c) the loading on the leeward antenna is not less than 50% of its fully exposed loading.

Note: *It is important to recognize that the highest combined load does not necessarily occur when the wind direction is perpendicular to the largest antenna.*

5.11 Temperature effects, *T*

The effects of loads and displacements caused by change in temperature shall be considered for the load combinations that include ice. The temperature to be considered for iced conditions shall be –10 °C. A lower temperature may be used based on local data and experience.

5.12 Earthquake load and effects, *E*

5.12.1 General

The effects of earthquake loading shall be considered for post-critical telecommunications structures in accordance with the requirements of this Clause. Telecommunication structures that are not designated as post-critical shall satisfy the minimum performance level of life safety if they are located in an area of human occupancy. See Annex M.

5.12.2 Exclusion

All structures located in zones of low seismicity where spectral accelerations values $S_a(0.2)$, as defined in Sentence 4.1.8.4 of the *National Building Code of Canada (NBC)*, are less than or equal to 0.12 are exempt from Clause 5.12.

5.12.3 Equipment mounted on building rooftops

Communication equipment and its connections to the building structure shall be designed in accordance with the seismic load requirements of Sentence 4.1.8.18 of the *NBC*. Guidance for proper anchoring of equipment on rooftops of new and existing buildings is provided in CAN/CSA-S832. Serviceability requirements of post-critical rooftop equipment shall be those provided by this Standard.

5.12.4 Site properties

The peak ground acceleration (PGA) and the design spectral acceleration response values, $S(T_n)$, shall be determined on accordance with Subsection 4.1.8.4 of the *NBC*. They are based on a 2% probability of exceedance in 50 years. The spectral response acceleration values were obtained for a constant modal viscous damping of 5% of critical.

A geotechnical investigation shall be performed to determine the site classification for seismic site response and the site coefficients (F_a and F_v) defined in Tables 4.1.8.4.A to C of the*NBC*. See Clause M.6.

5.12.5 Seismic analysis procedures

5.12.5.1 General

Rational seismic analysis procedures shall be performed. See Table M.1.

5.12.5.2 Monopoles and short towers

A simplified static force procedure may be used for monopoles and relatively short towers with a fundamental period less than or equal to 0.2 s:

a) The minimum lateral earthquake force, V, shall be calculated using the following formula:
$V = F_a S_a(0.2)\, W$

b) The total weight of the structure, W, shall include attachments and, for guyed masts, one-half of the weight of the guy cables.

c) The force V determined in Item a) shall be considered acting at the centre of mass of the tower for the calculation of base reactions.

Note: *This procedure neglects the effects of vertical accelerations. Structures eligible for this calculation procedure are assumed to move rigidly with the ground.*

5.12.5.3 Equivalent static force procedure

An equivalent static force procedure may be used where $F_a S_a(0.2)$ is less than 0.35 and for structures with height not exceeding 50 m:

a) This procedure neglects the effects of vertical accelerations on the structure.

b) The effects of vertical accelerations shall be accounted for in the design of the connections between the antennas and their mounts. To this end, vertical accelerations shall be taken as 2/3 $F_a S_a(0.2)$ and the d'Alembert force generated by the antenna's vertical shaking is taken as the antenna mass times the vertical acceleration. This force is then assumed to be acting at the centre of mass of the antenna.

c) The minimum lateral earthquake force, V, shall be calculated using the following formula:

$$V = \frac{F_a S_a(T_n) W}{R}$$

where R is a response modification coefficient. For ultimate limit states life safety checks, R is equal to 1.5 for tubular pole structures, 2.5 for latticed guyed masts, and 3.0 for latticed self-supporting towers. R shall be taken as 1 when calculating displacements and rotations for checking serviceability requirements.

d) The total weight of the structure, W, shall include attachments and, for guyed masts, one-half of the weight of the guy cables.

e) The force V determined in Item c) shall be considered as distributed vertically along the height of the structure according to an assumed mode shape corresponding to the deflected shape the structure would take if its weight were applied in the horizontal direction.

f) The static analysis of the structure shall then be carried out by considering the distributed horizontal force profile determined in Item e).

5.12.5.4 Equivalent modal analysis procedure

An equivalent modal analysis procedure as described in Annex M may be used for free-standing towers and masts with height not exceeding 150 m.

5.12.5.5 Dynamic analysis procedures

Dynamic analysis procedures shall be performed for all towers and masts with height exceeding 150 m. The dynamic analysis procedure shall be in accordance with one of the following methods (see Annex M):

a) For free-standing structures: Linear dynamic analysis by either the Modal response spectrum method, the Truncated modal superposition method, or the Direct time integration method using a structural model that is representative of the magnitude and spatial distribution of the mass and the stiffness of the structure and its attachments, and structural damping.

b) For guyed masts: Simplified linearized dynamic analysis procedure for masts with height ranging from 150 to 350 m, which accounts for horizontal accelerations only, or nonlinear dynamic analysis procedure, in which case a special study shall be performed. For tall guyed masts with height exceeding 350 m, vertical accelerations and the effects of asynchronous ground displacements imposed at the supports shall be considered.

6 Analysis

6.1 Initial condition

The initial condition of a structure for analysis shall be taken as that under the unfactored dead load, with the guys, if any, at their design initial tensions. In the absence of more specific information, a temperature of 10 °C for sites at or south of latitude 55° N and 0 °C for sites north of latitude 55° N, may be used.

6.2 Load combinations

The following load combinations shall be considered:

a) $D + W$;
b) $D + W + I + T$;
c) $D + E$;
d) $D + I + E$, whenever applicable in accordance with Clause 5.12; and
e) W_F.

where

D = dead load

W = wind load

I = ice load

T = temperature; temperature effects

E = earthquake load

W_F = wind load for fatigue loading combination whenever applicable in accordance with Annex N

6.3 Factored loads for ultimate limit states

Member forces shall be those due to the loads determined in accordance with Clause 5 multiplied by the appropriate load factors, α, the load combination factor, ψ, and the importance factor, γ, when

$$\alpha_D D + \gamma\ (\psi \alpha_W W + \alpha_I I + \alpha_T T)$$

$$\alpha_D D + \gamma \alpha_E E$$

$$\alpha_D D + \gamma(\psi\ \alpha_I I + \alpha_E E)$$

where

α_D = 1.25 for the dead load of the structure and attachments, excluding guy assemblies

= 0.85 when the dead load resists overturning, sliding, uplift, or reversal of load effect

= 1.00 acting together with E

= 1.00 for the self-weight of guy assemblies and for earthquake load

α_E = 1.00

γ = the importance factor as given in Table 3

ψ = 1.0 with only D and W acting

ψ = 1.0 with only D and E acting

ψ = 0.5 with D, W, I, and T acting together

ψ = 0.5 with I and E acting

α_W = 1.40

α_I = 1.45, except

= 0.82 when the ice load resists overturning, sliding, uplift, or reversal of load effects

= 1.0 when the ice load is considered in combination with earthquake effects

α_T = 1.0

No load factor shall be applied to the guy initial tension.

Table 3
Reliability classification
(See Clauses 6.3, G.2, and S.6.3.)

Reliability class	Importance factor, γ	Classification criteria
I	1.0	Failure would result in risk of injury or unacceptable disruption of service.
II	0.9	Failure would result in negligible risk of injury. Loss of service is not critical.
III	0.8	All consequences of failure are tolerable. No foreseeable risk of injury.

Note: *See Annex G.*

6.4 Factored loads for serviceability limit states

6.4.1 Reference velocity pressure for serviceability

The reference velocity pressure, q, for serviceability limit states shall be the 10-year return period mean hourly wind pressure at 10 m above ground level, as appropriate for the site.

6.4.2 Factored loads

Member forces shall be those due to the unfactored loads determined in accordance with Clause 4 multiplied by the serviceability factor, τ , given in Clause 6.4.3, and the load combination factor, ψ, as given by

$$D + (\psi \tau W + I + T)$$

$$D + E$$

$$D + 0.5I + E$$

where

ψ = 1.0 for the load combination in Clause 6.2, Item a)

= 0.5 for the load combination in Clause 6.2, Item b)

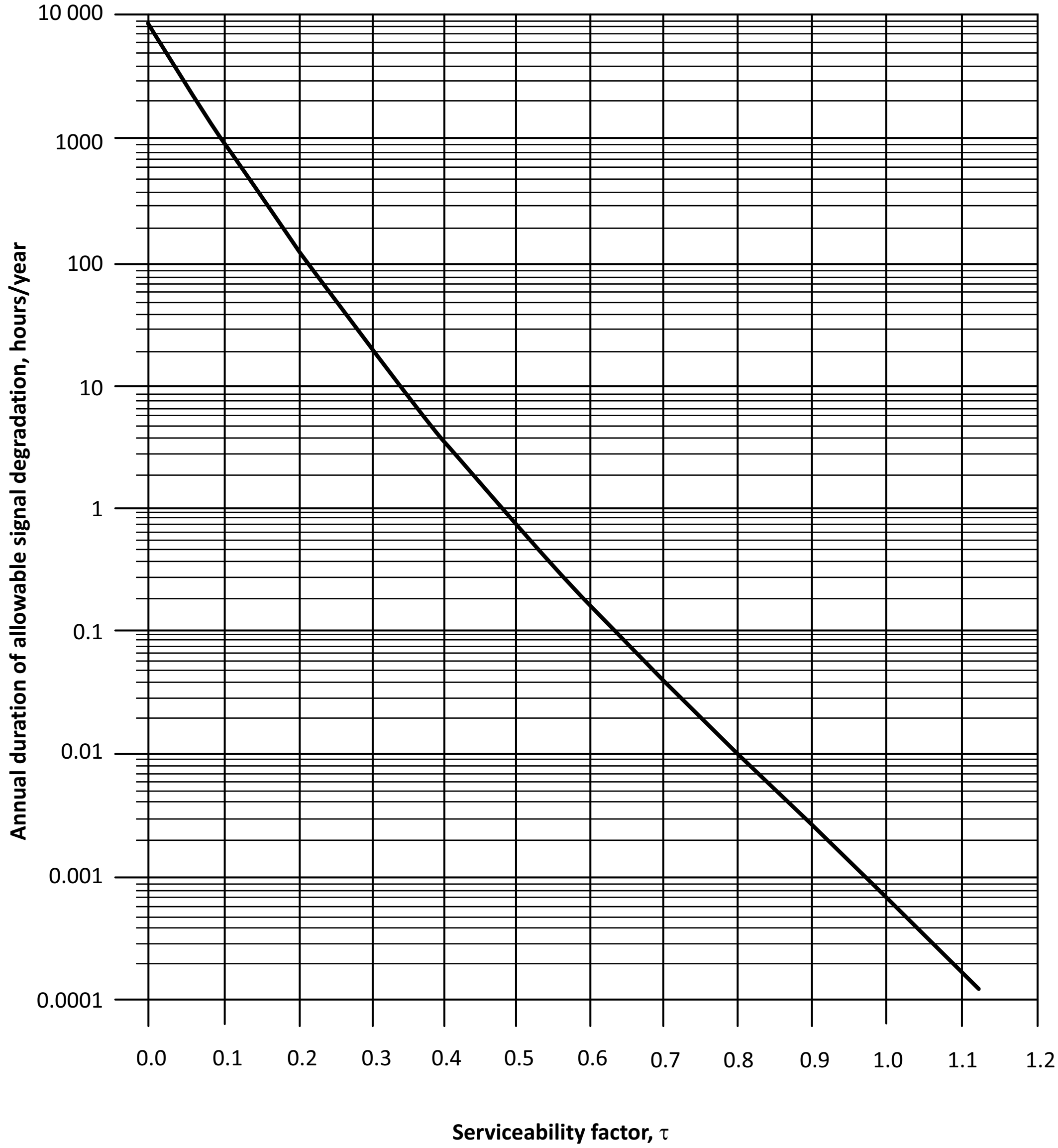
τ = the serviceability factor as given in Clause 6.4.3

6.4.3 Serviceability factor

The serviceability factor, τ, shall be determined from Figure 8, based on the annual duration of allowable signal degradation due to wind effects on un-iced structures as specified by the owner. If neither the permissible annual duration nor the serviceability factor is specified, the value for τ shall be taken as 1.0. For wind in combination with ice, τ shall be taken as 1.0.

Note: *See Annex J for further information on serviceability.*

Figure 8
Serviceability factor, τ (un-iced case)
(See Clauses 6.4.3, E.4, and J.3.)



6.4.4 Earthquake load combination

The earthquake load combination shall be $D + E$ for serviceability limit states, where E is based on the *NBC* uniform hazard spectra with a probability of occurrence of 2% in 50 years in accordance with Clause 5.12. All those installations required to perform at the continuous serviceability level PL3 defined in Annex M shall be checked for serviceability limit states under this load combination.

6.5 Wind direction

6.5.1 Ultimate limit states

For ultimate limit states, the factored wind loads shall be considered to be acting in the directions that produce the maximum forces in each member of the structure. For triangular towers, at least four wind directions (parallel to and at 30°, 60°, and 90° to any face) shall be considered, and for square towers, at least three wind directions (parallel to and at 45° and 90° to any face) shall be considered. More wind directions can be required to obtain maximum forces (e.g., if there is asymmetrical guying or complex antenna loadings).

Note: *It is important to recognize that the highest combined load does not necessarily occur when the wind direction is perpendicular to the largest antenna.*

6.5.2 Serviceability limit states

For serviceability limit states, the analysis shall include, for each of the designated antennas, the wind direction that produces the maximum twist or tilt. The maximum twist or tilt of an antenna is dependent on the torque or load applied to the tower as a whole, and might not be caused by the wind direction producing the maximum torque or load at that level.

6.6 Earthquake direction

6.6.1 General

For structures that are regular in stiffness and mass distribution, horizontal seismic input may be applied separately and independently along their principal directions. The worst effect of two orthogonal horizontal components acting simultaneously shall be evaluated for those structures with large eccentric masses or irregular geometry, with 100% of the prescribed design motion for one direction and 30% of the same motion in the perpendicular direction.

6.6.2 Vertical ground motions

In the absence of better information on appropriate design earthquake signatures for the site, vertical ground motions of amplitude of 2/3 of the horizontal motions may be considered acting simultaneously with the horizontal motions when required in dynamic analysis procedures in accordance with Clause 5.12. (See Table M.1.)

6.6.3 Phase lag effects to be considered

Phase lag effects due to large distances between ground anchor points of guyed masts with height exceeding 350 m shall be accounted for in nonlinear dynamic analysis. (See Table M.1.)

6.6.4 Post-disaster communication structures

Checks on serviceability limit states shall be made for post-disaster telecommunication structures. The analysis shall include, for each of the designated post-critical antennas, the earthquake direction that produces the maximum twist or tilt.

6.7 Displacement effects

The analysis of all structures shall take into account the effects of displacements on member forces. For guyed structures, the effect of the displacement of the guy points and the stability of the mast shall be considered. As a minimum, the mast shall be analyzed as a beam column on elastic supports, taking into account the effects of all significant displacements and rotations in three-dimensional space.

6.8 Cantilever factor

The forces determined from the analysis, including those in the interfacing connections, shall be increased by a factor of 1.25 for the design of

a) pole structures, cantilevered or guyed, mounted on top of lattice structures; and
b) cantilevered lattice sections of guyed masts with KL/r equal to or greater than 120.

where

K = 2.0

L = the length of cantilever

r = the radius of gyration of the cantilevered section

7 Structural steel

7.1 General

7.1.1 Scope

The requirements of this Clause relate to structural steel for the types of structures covered by this Standard. The relevant clauses of CSA S16 are referenced in Clause 7.1.2. When the requirements in referenced clauses differ, this Standard shall govern.

Note: *See Annex K for a commentary on Clause 7.*

7.1.2 Relevant clauses of CSA S16

The following relevant clauses of CSA S16 shall be considered a part of this Standard, unless otherwise indicated:

a) Clause 5, Material — Standards and identification;
b) Clause 12, Gross and net areas;
c) Clause 13, Member and connection resistance;
d) Clause 14, Beams and girders;
e) Clause 18, Composite columns;
f) Clause 19, Built-up members;
g) Clause 21, Connections;
h) Clause 22, Design and detailing of bolted connections;
i) Clause 23, Installation and inspection of bolted joints;
j) Clause 24, Welding;
k) Clause 28, Shop and field fabrication and coating; and
l) Clause 30, Inspection.

7.1.3 Other steel members

Cold-formed steel members not covered by this Standard shall conform to the requirements of CSA S136.

7.1.4 Minimum thickness

Unless the member is designed in accordance with Clause 7.1.3, the minimum thickness shall be

a) 5 mm for connection plates;
b) 4 mm for steel tubular structures; and

c) 3 mm for structural steel shapes.

7.1.5 Member shapes

The following Clauses are applicable to angle members, solid round sections, and tubular sections as indicated. The factored resistances of Schifflerized angle members may be determined using the procedures for 90° angles. The design properties from ASTM A500/A500M products shall be determined from the nominal wall thickness. For other member shapes, a general philosophy of design similar to that in this Standard shall be applied.

Note: *See Annex Q for properties of Schifflerized (60°) angles.*

7.1.6 Minimum Charpy V-notch value

Structural steels used in structures designed to this Standard shall have properties to resist brittle fracture at the lowest temperature likely to occur at the site. Steels that conform to Clause 5 of CSA S16 may be used. When other standards are used, a minimum Charpy V-notch value (CVN) of 20 J shall be specified. Higher values shall be used for high strength steels and thicker members subject to tension forces. The minimum specified value need not exceed 70 J.

Note: *See Annex K for a proposed formula to determine the minimum CVN value.*

7.1.7 Normal framing eccentricity

7.1.7.1 General

Joints framing eccentricities defined in this Clause shall be considered normal. When joint eccentricities exceed normal framing eccentricity, the member resistance shall be determined by a more detailed method, which takes into account the interaction of flexure and axial force.

7.1.7.2 Leg members

Normal framing eccentricities shall be defined by one of the following conditions:

a) For solid round and tubular leg members, the lines of action of the bracing members meet at a point within the diameter of the leg.

b) For angle leg members, the lines of action of the bracing members meet at a point within a distance equal to 75% of the angle width b, on either side of the centroid of the angle.

c) For built-up leg members, the lines of action of the bracing members meet at a point within the cross-section of the built-up leg.

7.1.7.3 Bracing members

Normal framing eccentricity shall be defined as the condition where the centroid of the bolt or weld group is located between the heel of the angle and the centerline of the connected leg of the member. When a joint eccentricity exceeds this condition, the bracing member resistance shall be multiplied by the factor $b/2g$, where b is the width of the connected leg and g is the distance from the heel of the angle to the centroid of the connection. If the width of the connected leg is 76 mm or less or the slenderness ratio, L/r, is greater than 120, the reduction factor need not be applied.

7.1.8 Secondary bracing members

7.1.8.1 General

Secondary members are used to reduce the unbraced length of a leg or bracing member. They are not considered to directly resist the applied loads.

7.1.8.2 Resistances required of secondary members

A secondary member that is considered to provide lateral support to a compression member in the plane of buckling under consideration, shall be capable of resisting a force, F_S, in tension or compression, as determined by

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

where

$0.015C_f \leq F_S \leq 0.025C_f$

F_S = the force applied at the node point where the secondary member providing the support connects to the primary member, and acting in a direction perpendicular to the corresponding radius of gyration of the primary member

C_f = the axial design compressive force in the supported member

KL/r = effective slenderness ratio of the supported member in the plane of support

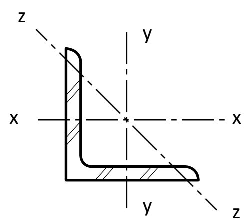
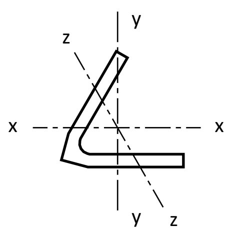
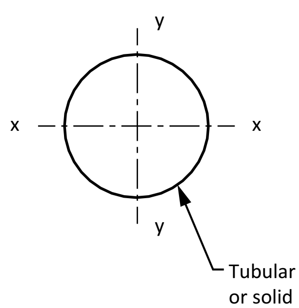
A different value may be used when determined by a suitable analysis.

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

The minimum required design strengths of multiple members connecting at a panel point within a face shall be determined from Table 5.

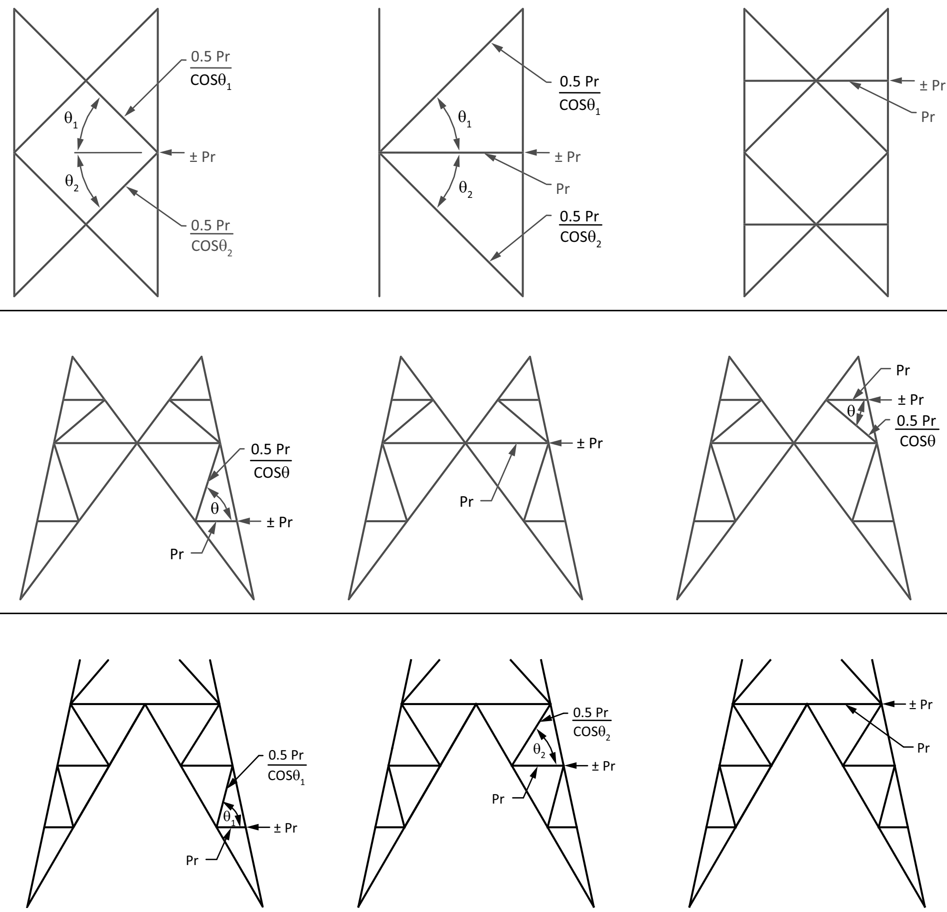
A secondary diagonal member that is connected to either end of a horizontal secondary member shall have a minimum design strength equal to one-half of the required design strength of the horizontal divided by the cosine of the angle between the members (see Table 5) unless a more rigorous analysis is performed.

Table 4
Bracing resistance required for leg members
(See Clause 7.1.8.2 and Figure 10.)

Leg shape	Tower cross section	Bracing resistance, P_r, required in a face at a panel point
	Square	When weak-axis buckling (KL/r_z) governs: $P_r = F_s$ / (2 x 0.707) = 0.707 F_s When in-plane buckling (KL/r_x or KL/r_y) governs: $P_r = F_s$
	Triangular	When weak-axis buckling (KL/r_z) governs: $P_r = F_s$ / (2 x 0.866) = 0.577 F_s When in-plane buckling (KL/r_y) governs: $P_r = F_s$ When out-of-plane buckling (KL/r_x) governs: $P_r = F_s$ / (0.866) = 1.15 F_s
	Square	In-plane buckling (KL/r_x or KL/r_y) governs: $P_r = F_s$
	Triangular	Out-of-plane buckling (KL/r_x) governs: $P_r = F_s$ / (0.866) = 1.15 F_s

Notes:
1) *Alternatively, P_r may be determined using the worst case effective slenderness ratio (highest ratio) to determine F_s and multiplying the result by 1.15 for triangular tower cross sections or by 1.00 for square tower cross sections.*
2) *One value of P_r applies for both faces when investigating a segment of a leg. The larger value shall be used considering the leg segment above and below a panel point.*
3) *Weak-axis buckling governs angle legs for symmetrical bracing patterns. In-plane, out-of-plane, or weak-axis buckling may govern staggered bracing patterns.*

Table 5
Minimum required resistance at panel points
(See Clause 7.1.8.2 and Figure 10.)



Note: *For leg slopes greater than 15° from vertical, P_r shall be divided by the cosine of the leg slope.*

7.1.8.3 Exception

When the angle between the leg and the diagonal of a K-braced panel is less than 25°, the value of F_S determined by the preceding equation might not be adequate. In such cases, a more detailed analysis shall be performed, taking into account eccentric and secondary loads and member deformations.

Each value of F_S shall be applied, independently of all other loads, at the intersection point of the primary and secondary members. In conventional lattice towers and masts, the effects in the primary members resulting from this application need not be added to those effects calculated in the analysis of the entire structure (the primary design effects). When the primary design effects are smaller than the forces due to those calculated during the application of F_S forces, the larger value shall be used.

7.1.9 Resistance factor

The resistance factor, ϕ, shall be 0.90 unless otherwise specified.

7.2 Compression members

7.2.1 General

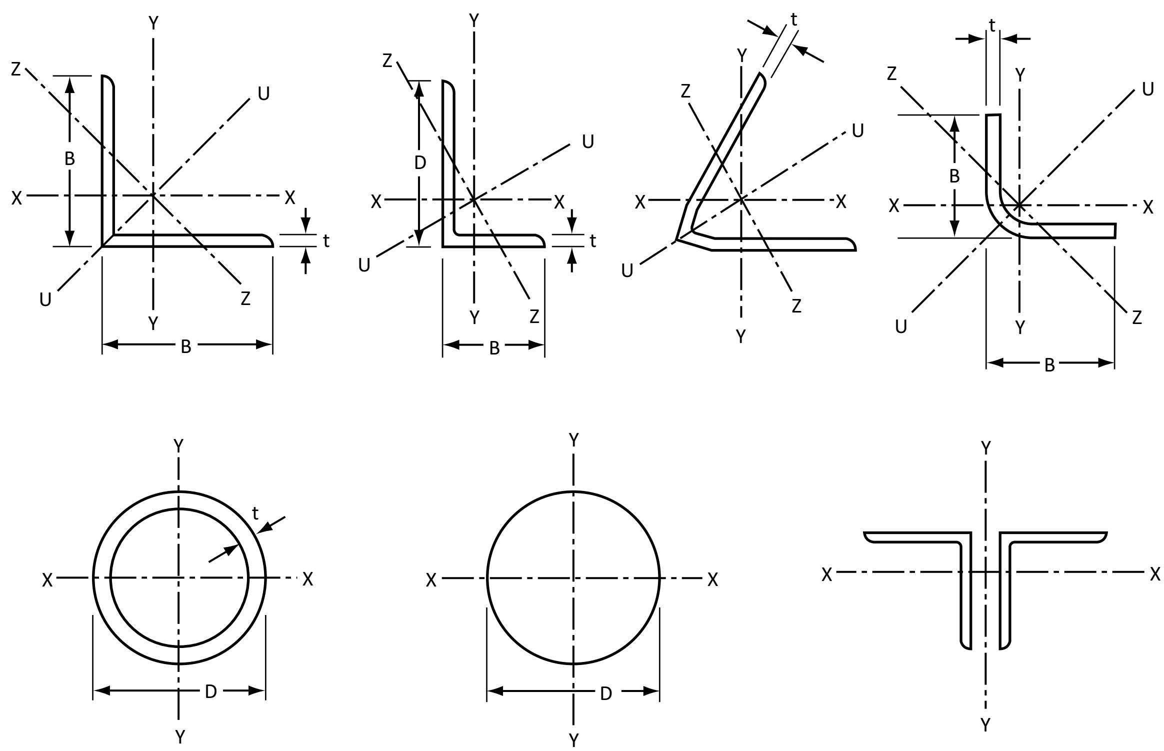
7.2.1.1 Unbraced length, *L*

The unbraced length, *L,* for any cross-sectional axis of a member shall be taken as the distance along the longitudinal axis of the member between the points at which it is intersected by the longitudinal axes of the members providing support for that cross-sectional axis. The unbraced length may vary for different cross-sectional axes of the member. Secondary members that do not directly resist the applied loads may be used to reduce the unbraced length of a member.

7.2.1.2 Slenderness ratio, *L/r*

The slenderness ratio of a member shall be *L/r*, where *r* is the radius of gyration of the member around the cross-sectional axis perpendicular to the unbraced length under consideration. See Figure 9 for typical cross-sectional axes.

Figure 9
Typical member section axes
(See Clause 7.2.1.2.)



7.2.1.3 Effective length factor, *K*

The effective length factor, *K*, shall be used to adjust the unbraced length, *L*, to take into account the structural configuration, including the rotational restraints and end connections.

7.2.1.4 Rotational restraint

A single-bolt connection, either at the end of a member or at an intermediate support, shall not be considered to provide rotational restraint. A multiple-bolt connection, or equivalent welding, shall be considered to provide rotational restraint where the connection is to a member capable of resisting rotation in the required plane. Where a bracing member is connected to a leg member through a gusset plate by a multiple bolt connection, the bracing member may be considered restrained in the plane of the plate at the centroid of the connection. A gusset plate shall not be considered to provide restraint transverse to the plane of the plate. Where analysis or tests show that a specific detail provides restraint differing from the requirements of this Clause, the effective slenderness ratio may be adjusted accordingly.

7.2.1.5 Effective slenderness ratio, *KL/r*

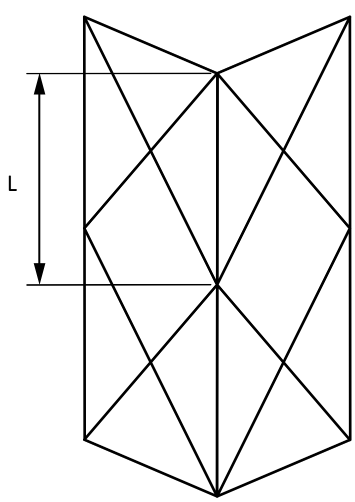
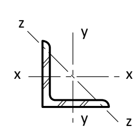
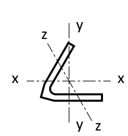
The effective slenderness ratio is the ratio of the effective length, *KL*, to the corresponding radius of gyration, *r*.

The maximum effective slenderness ratio for compression members shall be
a) 120 for leg members;
b) 200 for main members other than leg members carrying design compression forces; and
c) 240 for secondary members.

7.2.2 Leg members

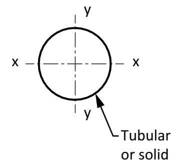
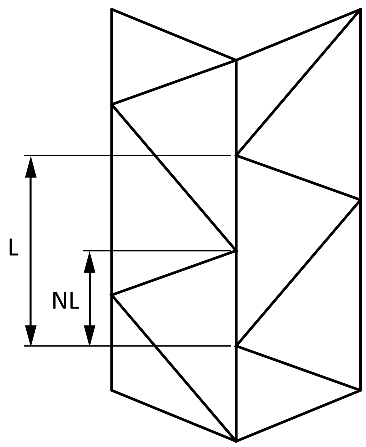
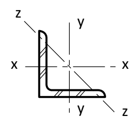
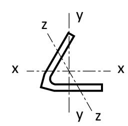
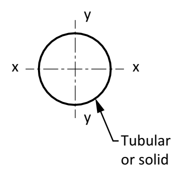
The effective slenderness ratios, *KL/r*, for leg members shall be as provided in Table 6. The minimum effective length factor, *K*, shall be equal to 1.0 for latticed structures.

Table 6
Effective slenderness ratios for leg members
(See Clause 7.2.2.)

Symmetrical bracing patterns		
	Leg shapes	**Effective slenderness ratios** *K* = 1.0
		*KL/rz*
		*KL/rz*

(Continued)

Table 6 (Concluded)

<table>
<tr><td></td><td></td><td>*KL/rx*</td></tr>
<tr><td colspan="3">**Staggered bracing patterns**</td></tr>
<tr><td rowspan="4">
N ≥ 0.5</td><td>**Leg shapes**</td><td>**Effective slenderness ratios**
K = 1.0</td></tr>
<tr><td></td><td>1.1*KL/rx*, 1.1*KL/ry*, *K(NL)/rz*</td></tr>
<tr><td></td><td>*KL/rx*, *K(NL)/rz*</td></tr>
<tr><td></td><td>*KL/rx*</td></tr>
</table>

Notes:

1) *L shall equal the panel spacing measured along the axis of the leg.*
2) *The maximum effective slenderness ratio shall be used to determine the design compression strength and the bracing resistance required to provide lateral support.*

7.2.3 Bracing members

7.2.3.1 General

The effective slenderness ratios, *KL/r*, for bracing members shall be determined taking into account the loading condition, bracing pattern, member end restraints, and framing eccentricities. Effective slenderness ratios shall be determined from Table 7, except for round members welded directly to leg members where the effective length factors, *K*, shall be taken from Table 8. Effective lengths and slenderness ratios for commonly used bracing patterns shall be as shown in Tables 9 and 10. Effective length, *L*, shall be the distance between the centroids of the end connections.

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

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

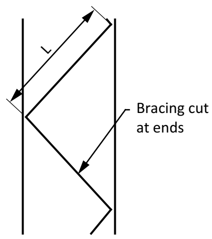
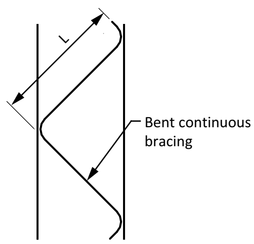
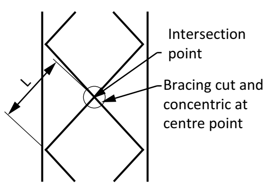
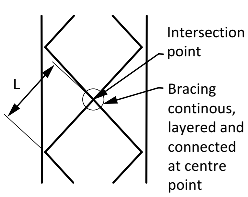
Table 7
Effective slenderness ratios for bracing members

(See Clause 7.2.3.1.)

Slenderness ratios < 120, Eccentricity governs		
Formula number	**Equation**	**Conditions at ends of the buckling length under consideration**
1	$KL/r = L/r$	Concentric at both ends
2	$KL/r = 30 + 0.75\ L/r$	Concentric at one end and normal framing eccentricity at the other end
3	$KL/r = 60 + 0.50\ L/r$	Normal framing eccentricity at both ends
Concentric conditions	Double angles or channels Round members with concentric end plate	
Normal framing eccentricity conditions	Single angles or channels Round members with eccentric end plate Round members with flattened ends	
Slenderness ratios ≥ 120, Restraint governs		
Formula number	**Equation**	**Conditions at ends of the buckling length under consideration**
4	$KL/r = L/r$	Unrestrained against rotation at both ends
5	$KL/r = 28.6 + 0.762\ L/r$	Partially restrained at one end and unrestrained at other end
6	$KL/r = 46.2 + 0.615\ L/r$	Partially restrained against rotation at both ends
Unrestrained conditions	Single bolt connections	
Partially restrained conditions	Multiple bolt or welded connections to a stiffer member/component or group of members/components	

Notes:
1) *Formula 2 applies to single angles in cross bracing patterns when L/r < 120 and the angles are connected back-to-back at the crossover point.*
2) *Different equations may apply for each direction of buckling under consideration. The maximum effective slenderness ratio shall be used to determine the compressive strength.*

Table 8
Effective slenderness ratios for solid round bracing members directly welded to legs
(See Clause 7.2.3.1.)

Bracing pattern	Slenderness ratio of bracing member		
	$L/r < 80$	**$80 \le L/r \le 120$**	**$L/r > 120$**
	$K = 1.0$	$K = 0.70 + 0.30\ (120 - L/r) / 40$	$K = 0.70$
	$K = 1.1$	$K = 0.70 + 0.40\ (120 - L/r) / 40$	$K = 0.70$
 [See Note 2)]	$K = 1.0$	$K = 0.75 + 0.25\ (120 - L/r) / 40$	$K = 0.75$
 [See Note 2)]	$K = 1.1$	$K = 0.90 + 0.20\ (120 - L/r) / 40$	$K = 0.90$

Notes:
1) *L shall be determined using the panel spacing and the clear distance between the legs.*
2) *In loading cases when both diagonal members in any face of a double braced pattern simultaneously develop compression forces, the value of KL for determining the diagonal's resistance to these compressive forces shall be based on the single braced condition.*

7.2.3.2 Cross bracing (tension/compression)

7.2.3.2.1 Resistance to out-of-plane buckling

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

a) One of the diagonal members is continuous and one of the diagonal members is subjected to tension [see Clause 7.5.5, Item d)].
b) Triangulated horizontal plan bracing (Figure 10) is provided at the intersection point with sufficient resistance as defined in Clause 7.1.8.2.
c) A continuous horizontal member meeting the following criteria is connected at the crossover point:
   i) The continuous horizontal has sufficient strength to provide resistance to the leg as defined in Clause 7.1.8.2.
   ii) The strength of the continuous horizontal is determined ignoring the out-of-plane buckling resistance of the diagonals.

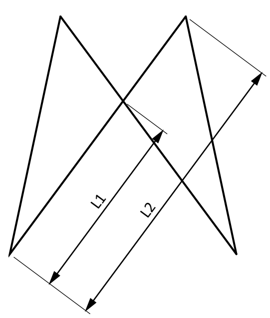
Otherwise, the crossover point shall not be considered as providing support resisting out-of-plane buckling. See Table 9.

7.2.3.2.2 Discontinuous diagonals

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

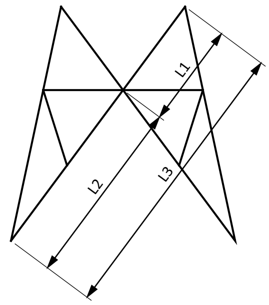
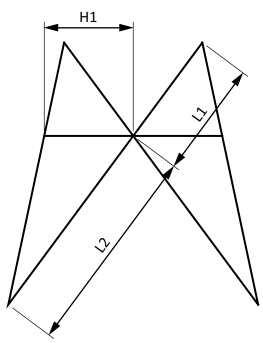
a) Triangulated horizontal plan bracing with sufficient resistance as defined in Clause 7.1.8.2 is provided at the crossover point.
b) A continuous horizontal with sufficient strength as defined in Clause 7.2.3.2.1, Item c) is provided through the crossover point.

Table 9
Buckling length considerations for cross bracing
(See Clauses 7.2.3.1 and 7.2.3.2.1.)

	Crossover point providing support	Crossover point connected but no support provided
	$L1/r_{min}$	$L1/r_{min}$ $L2/r_{out}$

(Continued)

Table 9 (Concluded)

	$L1/r_{min}$ $L2/r_{out}$	$L1/r_{min}$ $L3/r_{out}$
	**Internal bracing at crossover point** $L1/r_{min}$ $L2/r_{min}$ $H1/r_{min}$	**No internal bracing at crossover point** $L1/r_{min}$ $L2/r_{min}$ $H1/r_{min}$ $2H1/r_{out}$

Notes:

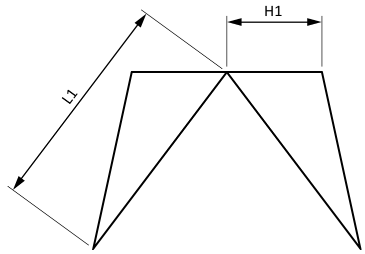
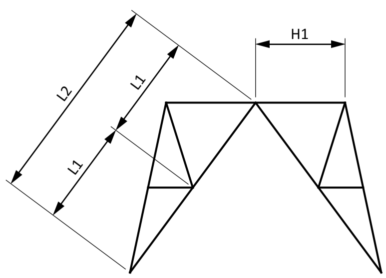
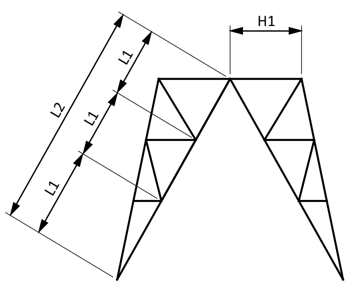
1) *L shall be determined using the length between the centers of connecting bolt or weld patterns.*
2) *"r_{min}" refers to the minimum radius of gyration for a member (i.e., the z-z axis for a single angle member, r_x or r_y for a round member).*
3) *"r_{out}" refers to the radius of gyration associated with out-of-plane buckling.*
4) *See Clause 7.2.3.2 for criteria to determine support at the crossover point.*
5) *Horizontals shall meet the requirement of Clause 7.2.3.2 when diagonals are not continuous through the crossover point.*

7.2.3.3 K bracing

Triangulated plan bracing shall be provided at the bracing apex point with sufficient resistance as defined in Clause 7.1.8.2 when the horizontal member is not a continuous member.

When triangulated plan bracing is not provided with a continuous horizontal, the out-of-plane unbraced length of the horizontal shall be considered to be 0.75 times the total length of the horizontal. The horizontal member shall have sufficient strength to provide support to the legs as defined in Clause 7.1.8.2 determined using the full length of the horizontal. (See Table 10.)

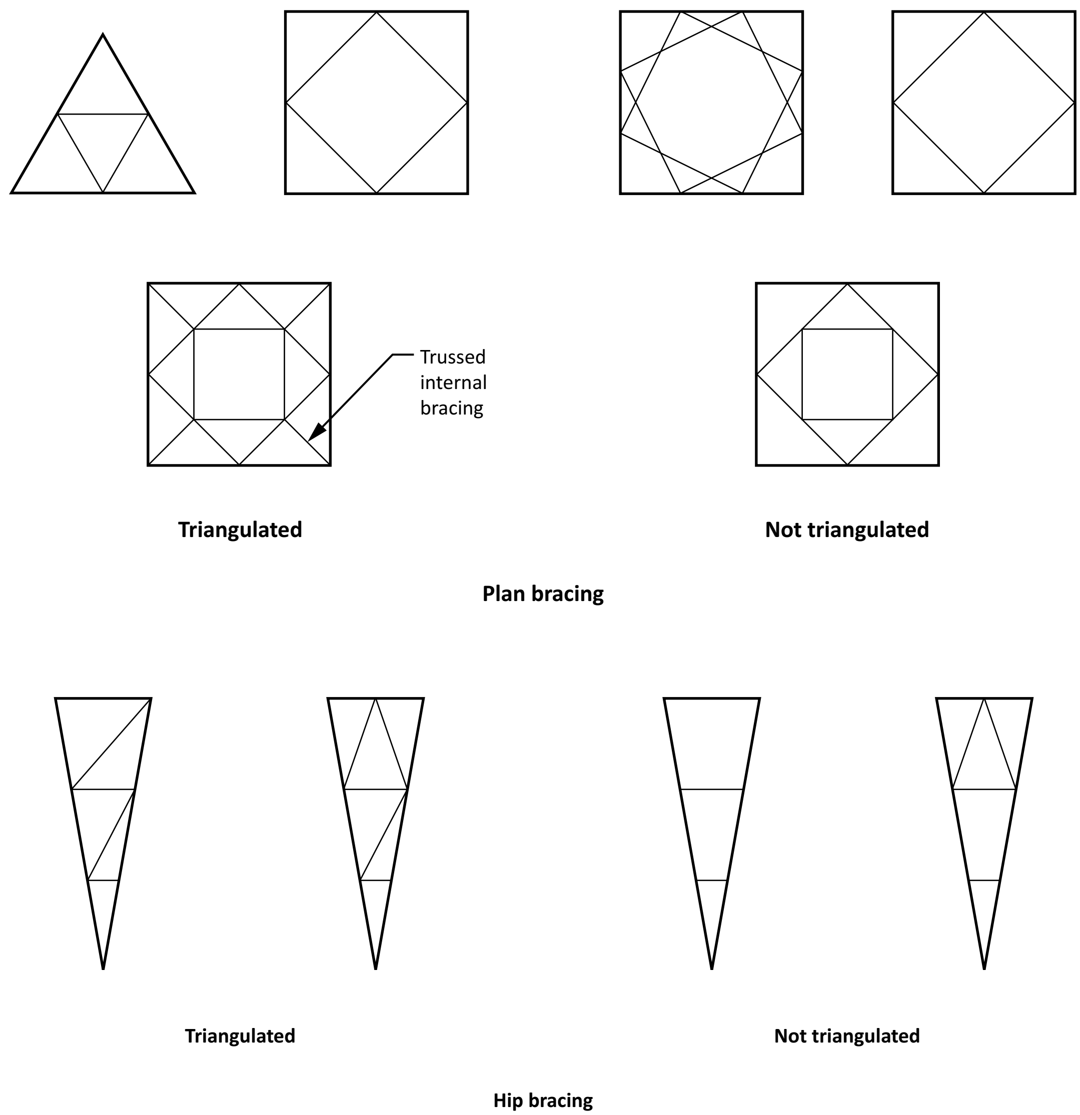
Table 10
Buckling length considerations for K-type or portal bracing
(See Clauses 7.2.3.1 and 7.2.3.3.)

<table>
<tr><td rowspan="3">
</td><th colspan="2">Horizontal member continuous</th></tr>
<tr><td>Plan bracing support at apex [See Note 5)]</td><td>No plan bracing</td></tr>
<tr><td>$L1/r_{\mathrm{min}}$
$H1/r_{\mathrm{min}}$</td><td>$L1/r_{\mathrm{min}}$
$H1/r_{\mathrm{min}}$
$1.5\ H1/r_{\mathrm{out}}$
[see Note 4)]</td></tr>
<tr><td>
</td><td>$L1/r_{\mathrm{min}}$
$L2/r_{\mathrm{out}}$
$H1/r_{\mathrm{min}}$</td><td>$L1/r_{\mathrm{min}}$
$L2/r_{\mathrm{out}}$
$H1/r_{\mathrm{min}}$
$1.5\ H1/r_{out}$
[see Note 4)]</td></tr>
<tr><td rowspan="3">
</td><th colspan="2">Horizontal member not continuous
Plan bracing required at apex [see Note 5)]</th></tr>
<tr><td>Hip bracing provided [see Note 5)]</td><td>Hip bracing not provided</td></tr>
<tr><td>$L1/r_{\mathrm{min}}$
$H1/r_{\mathrm{min}}$</td><td>$L1/r_{\mathrm{min}}$
$L2/r_{\mathrm{out}}$
$H1/r_{\mathrm{min}}$</td></tr>
</table>

Notes:

1) *L shall be determined using the length between the centers of connecting bolt or weld patterns.*
2) *"r_{min}" refers to the minimum radius of gyration for a member (i.e., the z-z axis for a single angle member, r_x or r_y for a round member).*
3) *"r_{out}" refers to the radius of gyration associated with out-of-plane buckling.*
4) *2.0 $H1/r_{out}$ shall be considered to determine strength for providing support to a leg as defined in Clause 7.1.8.2.*
5) *Plan and Hip bracing shall be triangulated (see Figure 10) and meet the requirement of Clause 7.1.8.2.*

Figure 10
Plan and hip bracing
(See Clause 7.2.3.2.1 and Tables 10.)



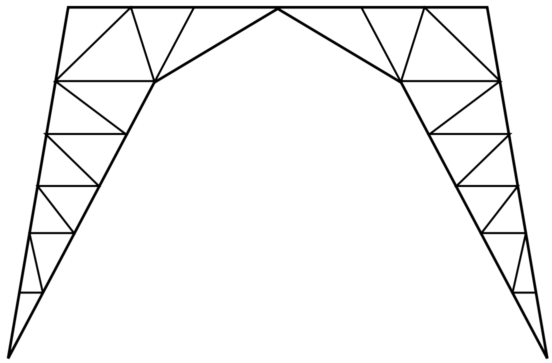
Notes:
1) *Plan or hip bracing shall be triangulated and meet the requirements of Clause 7.1.8.2 to provide lateral support to a member.*
2) *Effective slenderness ratios for main diagonals with staggered hip/face bracing shall be determined in accordance with Table 5 using the equations provided for leg members with staggered bracing patterns.*
3) *The minimum required resistance of hip bracing shall be determined in accordance with Table 4 with $P_r = 1.15\ F_s$ (F_s based on the governing effective slenderness ratio of the main diagonal).*

7.2.3.4 Cranked K and portal frame bracing

7.2.3.4.1 Cranked K

For large face widths, a crank or knee joint may be introduced into the main diagonals (see Figure 11), to reduce the length and size of the secondary members. This produces high forces in the members at the bend, and transverse support shall be provided at the knee joint. Diagonal and horizontal members shall be designed as for K-type bracing, with the design length of the diagonals taken as the length to the knee joint.

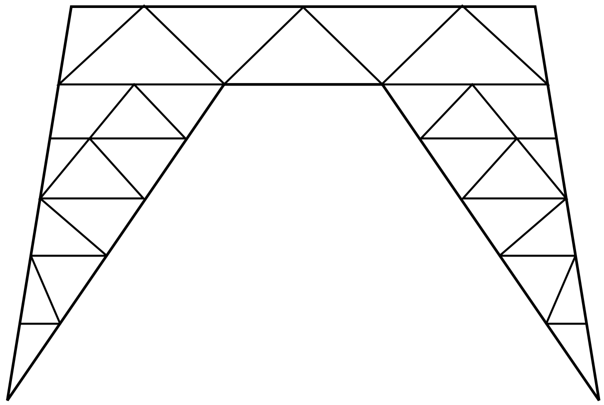
Figure 11
Cranked K-type bracing
(See Clause 7.2.3.4.1.)



7.2.3.4.2 Portal frame

If a secondary horizontal member is introduced on the face at the knee joint to reduce the stresses at the bend (see Figure 12), the panel becomes a portal frame, with corresponding stiffnesses and moments in the panel. Special consideration should be given to the analysis and design for these conditions.

Figure 12
Portal Frame-Type Bracing
(See Clause 7.2.3.4.2.)



7.2.3.4.3 Special considerations

Special consideration should be given to local conditions, such as wind load at an angle to the bracing system, that might introduce bending forces into the horizontals and to the potential effect of foundation settlement or movement.

7.2.4 Built-up members

7.2.4.1 General

Clause 7.2.4 is applicable to compression members composed of two back-to-back angles separated from one another by intermittent fillers.

7.2.4.2 Maximum slenderness of a single element

The slenderness ratio of a single element between points of interconnection shall not exceed the slenderness ratio of the built-up member. The minimum radius of gyration of a single element shall be used in computing the slenderness ratio of that element between points of interconnection.

7.2.4.3 Slenderness ratio of built-up member

The slenderness ratio of the built-up member shall be taken as the larger value determined by the following formulas:

a)

$$\frac{L_{xx}}{r_{xx}}$$

b)

$$\sqrt{\left(\frac{L_{yy}}{r_{yy}}\right)^2 + \left(\frac{k_m \ell}{r_{\min}}\right)^2}$$

where

L_{xx}, L_{yy} = unbraced member lengths about the xx-axis and yy-axis of the built-up member, respectively

r_{xx}, r_{yy} = radius of gyration of the built-up member about the xx-axis and yy-axis

k_m = local equivalent length factor equal to 1.0, except if the intermittent fillers are fastened with welds or pretensioned high-strength bolts, in which case k_m = 0.65

ℓ = distance between intermittent fillers

$r_{\min}$ = minimum radius of gyration of a single element

7.2.4.4 Requirement for double stitch bolts

If the connected leg is 125 mm or greater, at least two bolts or the equivalent in welding should be provided in line across the width of the leg at each intermittent filler.

7.2.5 Effective yield stress

7.2.5.1 Angle members

For angle members, the effective yield stress, F_y', shall be determined in accordance with the following formulas:

a) if $\frac{w}{t} \le \frac{210}{\sqrt{F_y}}$ then $F_y' = F_y$

b) if $\frac{210}{\sqrt{F_y}} < \frac{w}{t} \le \frac{380}{\sqrt{F_y}}$ then $F_y' = F_y\left(1.677 - 0.677\frac{w\sqrt{F_y}}{210t}\right)$

c) if $\frac{380}{\sqrt{F_y}} < \frac{w}{t} \leq 25$ then $F_y' = \frac{65500}{\left(\frac{w}{t}\right)^2}$

where

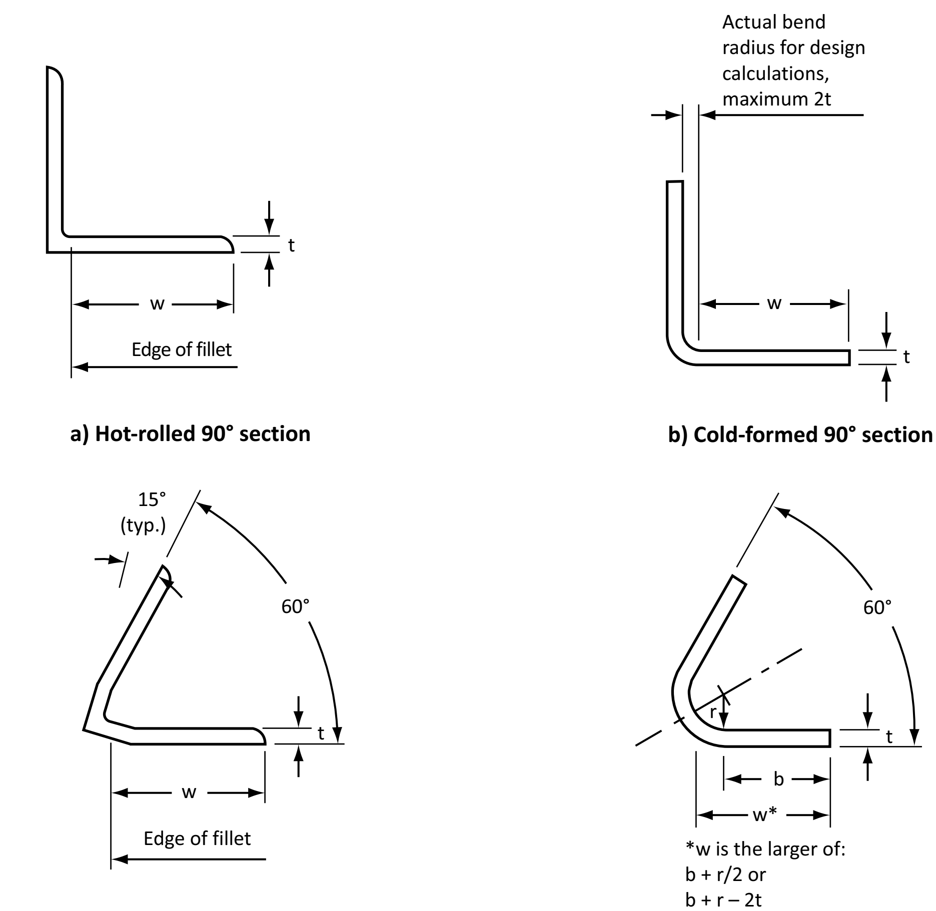
F_y = yield stress, MPa

w = effective leg width (see Figure 13)

t = nominal leg thickness (see Figure 13)

Note: *See Annex K for a discussion of effective yield stress and fillet radii of hot-rolled 90° sections.*

Figure 13
Angles
(See Clauses 3.2 and 7.2.5.1.)



a) Hot-rolled 90° section **b) Cold-formed 90° section**

c) Schifflerized (60°) section **d) Cold-formed 60° section**

Note: *See Annex K for a discussion of effective yield stress and fillet radii of hot-rolled 90° sections.*

7.2.5.2 Limit on width/thickness ratio
For unequal leg angles, the larger *w/t* ratio shall be used. For back-to-back angles, the larger *w/t* ratio for the single angle shall be used. For tubular round members, the diameter to thickness ratio (*D/t*) shall not exceed 400.

7.2.5.3 Width/thickness ratio
The *w/t* ratio of an angle member in compression shall not exceed 25.

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

7.2.5.5 Tubular round members
For tubular round members, the effective yield stress, F'_y, shall be determined as follows:

$D/t \le 0.114E/F_y$	$F_y' = F_y$
$0.114E/F_y < D/t \le 0.448E/F_y$	$F_y' = \left(\dfrac{0.0379E}{(D/t)F_y} + \dfrac{2}{3}\right)F_y$
$0.448E/F_y < D/t \le 400$	$F_y' = \dfrac{0.337E}{(D/t)}$

where

D = outer diameter of tubular member

t = wall thickness of tubular member

E = elastic modulus of the tubular member

7.2.5.6 Polygonal tubular members
For polygonal tubular steel members, the maximum width to thickness ratio (*w/t*) and effective yield stress, F'_y, shall be determined from Table 11.

7.2.6 Compressive resistance

7.2.6.1 General
The factored axial compressive resistance, C_r, of a member is determined by the following formula:

$$C_r = \phi \frac{AF_y'}{(1 + \lambda^{2n})^{1/n}}$$

where

$$\lambda = \frac{KL}{r}\sqrt{\frac{F_y'}{\pi^2 E}}$$

A = cross-sectional area

F_y'= effective yield stress for members as determined by Clause 7.2.5.1

ϕ = resistance factor

n = 1.34 for members consisting of structural shapes including angles; hot rolled solid round bars and hollow structural sections manufactured according to CSA G40.20, Class C (cold-formed non-stress-relieved); 2.24 for hollow structural sections manufactured according to CSA G40.20/G40.21, Class H (hot-formed or cold-formed stress-relieved)

7.2.6.2 Factored resistance of 60° angle

The factored resistance of 60° angle leg members may be determined using the procedures for 90° angles. Torsional-flexural buckling need not be explicitly accounted for in the design of 60° steel angles if compressive resistances are determined in accordance with Clause 7.2.6.1.

7.2.6.3 Hot-rolled bars greater than 51 mm diameter

All hot-rolled bars greater than 51 mm in diameter shall be produced to the mill special straightness tolerance of L/500.

Note: *The mill standard straightness tolerance for hot-rolled solid round bars is L/250.*

7.3 Tension members

7.3.1 Leg members

The factored tension resistance of a leg member and its splices shall not be less than one-third of the calculated factored compressive resistance, unless erection and all other forces have been considered.

7.3.2 Other tension members

The maximum slenderness ratio for tension-only members, and for components of built-up members not in tension under the initial loading condition, shall be 300, unless other means are provided to control flexibility, sag, vibration, slack, and other similar effects.

7.3.3 Net area

7.3.3.1 General

The net area, A_n, shall be determined by summing the critical net areas, A_n, of each segment along a potential path of minimum resistance as follows:

a) for a segment normal to the force (i.e., in direct tension), $A_n = w_n t$; and

b) for a segment inclined to the force, $A_n = w_n t + \frac{s^2 t}{4g}$

where

w_n = net width (gross width less design allowance for holes within the width)

t = thickness of member material

s = centre-to-centre longitudinal spacing (pitch) of any two successive fastener holes

g = transverse spacing between fastener gauge lines

7.3.3.2 Bolt holes

In calculating w_n, the width of bolt holes shall be taken as 2 mm larger than the specified hole diameter. Where it is known that drilled holes will be used, this allowance may be waived.

7.3.4 Effective net area — Shear lag

7.3.4.1 General

When fasteners transmit load to each of the cross-sectional elements of a member in tension in proportion to their respective areas, the effective net area shall be equal to the net area:

$A_{ne} = A_n$

7.3.4.2 Angles connected by one leg

When angle members are connected by one leg and the critical net area includes the area of the unconnected leg, the effective net area, A_{ne} to allow for shear lag shall be calculated as follows:

a) when connected with four or more transverse lines of fasteners, $A_{ne} = 0.90A_n$; and
b) when connected with fewer than four transverse lines of fasteners, $A_{ne} = 0.75A_n$

A_n shall be calculated in accordance with Clause 7.3.3.1, Items a) and b). When the outstanding leg has been clipped or coped, the reduction due to clipping or coping only needs to be considered when it exceeds the reduction in the area determined by the formula given in Item a).

7.3.4.3 Unequal leg angles

If the angle legs are unequal and only the short leg is connected, the unconnected leg shall be considered to be the same size as the connected leg.

7.3.5 Tensile resistance

The factored tensile resistance, T_r, developed by a member subjected to an axial tensile force shall be taken as the lesser of

a) $T_r = \phi A_g F_y$; or
b) $T_r = 0.85\,\phi A_{ne} F_u$.

For tensile resistance of link plates, refer to Clause 7.3.8.

7.3.6 Tensile resistance (block shear)

The factored resistance for a potential failure involving the simultaneous development of tensile and shear component areas shall be taken as follows:

$$T_r = 0.85\phi\left[U_t A_n F_u + 0.6A_{gv}\frac{\left(F_y + F_u\right)}{2}\right]$$

where

a) U_t is an efficiency factor, and $U_t = 1.0$ is used for symmetrical blocks or failure patterns and concentric loading or is taken from the following for specific applications:

Connection type	U_t
Angles connected by both legs, Flange-connected T_s	1.0
Angles connected by one leg, channels connected by web only and stem-connected T_s	0.75

b) A_n is the net area in tension, as specified in Clause 7.3.3; and
c) A_{gv} is the gross area in shear.

For steel grades with $F_y > 485$ MPa, $(F_y + F_u)/2$ shall be replaced with F_y in the determination of T_r.

7.3.7 Tensile resistance (tear-out)

The factored resistance for a potential plate tear-out failure of one or more bolts along parallel planes tangent to the bolt hole(s) and directed towards the edge of the plate shall be taken as follows:

$T_r = 0.5\phi A_{gv} F_u$

7.3.8 Link plates

The factored resistance of a pin connected link plate, T_r, shall be taken as the lowest value of
a) tension on the effective area:
$\phi = 0.90 \quad T_r = 0.85\ \phi 2t\ b_{eff} F_u$
b) shear on the effective area:
$\phi = 0.90 \quad T_r = 0.85\ \phi 0.6 A_{gv}(f_y + \mathrm{F_u})/2$
c) bearing on the projected area at the pin:
$\phi = 0.90 \quad T_r = \phi 1.8 A_{pb} F_y$
d) yielding on the gross area:
$\phi = 0.90 \quad T_r = \phi A_g F_y$

where

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

A_{pb} = projected bearing area of pin = dt

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

where

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

d = the pin diameter

t = the thickness of link plate

The pin hole shall be located midway between the edges of the member in the direction normal to the applied force. The width of the plate beyond the pin hole shall not be less than 2 b_{eff} + d and the minimum extension, a, beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than 1.33 b_{eff}.

The corners beyond the pin hole may be cut at 45° to the axis of the member, provided the net area beyond the pin hole, on a plane perpendicular to the cut, is not less than that required beyond the pin hole parallel to the axis of the member.

7.4 Flexural members

7.4.1 Round tubular members

The factored moment resistance, M_r, shall be determined as follows:

$$\frac{D}{t} \leq 0.0714\frac{E}{F_y} \qquad M_r = \phi F_y Z$$

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

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

where

D = outer diameter of tubular member

t = wall thickness of tubular member

E = modulus of elasticity, 200 000 MPa

S = elastic section modulus

Z = plastic section modulus

7.4.2 Polygonal tubular structures

For polygonal tubular members, the factored moment resistance M_r shall be determined as follows:

$M_r = \phi F'_y S$

where

F'_y = effective yield stress as determined from Table 11

S = minimum elastic section modulus

Table 11
Effective yield stress for polygonal tubular members
(See Clauses 7.2.5.6 and 7.4.2.)

Shape	(w/t) Ratios	Effective yield stress
18-sided	$(F_Y/E)^{1/2}(w/t) < 0.759$	$F'_Y = 1.27\ F_Y$
	$0.759 \leq (F_Y/E)^{1/2}(w/t) \leq 2.14$	$F'_Y = 1.560\ F_Y\,[1.0 - 0.245\ (F_Y/E)^{1/2}(w/t)]$
16-sided	$(F_Y/E)^{1/2}(w/t) < 0.836$	$F'_Y = 1.27\ F_Y$
	$0.836 \leq (F_Y/E)^{1/2}(w/t) \leq 2.14$	$F'_Y = 1.578\ F_Y\,[1.0 - 0.233\ (F_Y/E)^{1/2}(w/t)]$
12-sided	$(F_Y/E)^{1/2}(w/t) < 0.992$	$F'_Y = 1.27\ F_Y$

(Continued)

Table 11 (Concluded)

Shape	(w/t) Ratios	Effective yield stress
	$0.992 \leq (F_Y/E)^{1/2}(w/t) \leq 2.14$	$F'_Y = 1.611\, F_Y\, [1.0 - 0.220\, (F_Y/E)^{1/2}(w/t)]$
8-sided	$(F_Y/E)^{1/2}(w/t) < 1.10$	$F'_Y = 1.27\, F_Y$
	$1.10 \leq (F_Y/E)^{1/2}(w/t) \leq 2.14$	$F'_Y = 1.578\, F_Y\, [1.0 - 0.194\, (F_Y/E)^{1/2}(w/t)]$

where

F_Y = specified minimum steel yield strength, MPa

t = wall thickness, mm

w = flat side dimension calculated using an inside bend radius equal to $4t$, mm

E = modulus of elasticity, MPa

For polygonal members, w/t shall not exceed $2.14\,(E/F_Y)^{1/2}$.

Note: *Polygonal members with more than 18 sides shall be considered as round members for strength investigation purposes using a diameter equal to distance across flats.*

7.4.3 Solid round members

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

$$M_r = \phi F'_y Z$$

where

F'_y = effective yield stress as determined from Clause 7.2.5.4

Z = plastic section modulus

7.4.4 Combined flexural and axial compression

When subject to both axial compressive force and bending moment, members shall be proportioned such that

$$\frac{C_f}{C_r} + \frac{M_f}{M_r} + \left[\left|\frac{V_f}{V_r}\right| + \left|\frac{Mt_f}{Mt_r}\right|\right]^2 \leq 1.0$$

where

C_f = axial compressive force due to factored loads

C_r = factored axial compressive resistance = $\phi F'_y A$ for self-supporting tubular pole structures

M_f = bending moment due to factored loads

M_r = factored moment resistance

V_f = factored shear forces

V_r = factored shear resistance

Mt_f = factored torsional moment

Mt_r = factored torsional resistance

Notes:

1) *In accordance with Clause 6.7, the analysis of self-supporting and guyed tubular pole structures must take into account the effects of displacements (e.g., P-delta and other effects) on element forces and moments.*
2) *See Clause K.7.4 for leg angle members subjected to combined axial and flexural forces.*

7.5 Bolted connections

7.5.1 Bolts

7.5.1.1 ASTM bolts

ASTM A307, ASTM A394, ASTM F3125 Grades A325 and A325M, and ASTM F3125 Grades A490 and A490M may be used for bolt assemblies. ASTM F3125 Grade A490 high-strength bolts shall not be hot dipped or mechanically galvanized. Galvanized bolts that have been pretensioned beyond the snug-tight level shall not be reused.

7.5.1.2 Other bolts

Other suitable bolts that conform to a recognized standard may be used when appropriate minimum tensile strengths, F_u, are specified.

7.5.1.3 Compatibility

Bolts shall be compatible with the structural steel being connected. Galvanized bolts, nuts, and washers shall be used with galvanized structural steel.

7.5.2 Connection resistance

7.5.2.1 Factored bearing and shear resistance

The factored resistance developed at the bolts in a bolted joint subjected to bearing and shear shall be taken as the least of

a) the factored bearing resistance at bolt holes (except for long slotted holes loaded perpendicular to the slot), B_r, as follows:

$$B_r = 3\phi_b ntdF_u$$

b) the factored bearing resistance perpendicular to long slotted holes, B_r, as follows:

$$B_r = 2.4\phi_b ntdF_u$$

or

c) the factored shear resistance of the bolts, V_r, as follows:

$$V_r = 0.6\phi_b mnA_bF_{ub}$$

where

ϕ_b = 0.8

n = number of bolts

t = thickness of member material

d = diameter of the bolt

m = number of shear planes in the joint

A_b = cross-sectional area of a bolt, based on its nominal diameter

F_u = tensile strength of the member

F_{ub} = tensile strength of the bolt

Where the threaded section of a bolt intercepts any shear plane, the factored shear resistance of the bolt shall be taken as 70% of V_r.

7.5.2.2 Tensile strength

The tensile strength, F_{ub}, used in computing bolt resistance shall be the minimum value stipulated in the applicable standard for the bolt.

7.5.2.3 Factored tensile resistance

The factored tensile resistance, T_r, developed by bolts subjected to tension only, shall be determined by the following formula:

$$T_r = \phi_b n (0.75 A_b) F_{ub}$$

where

ϕ_b = 0.8

n = number of bolts

A_b = cross-sectional area of a bolt, based on its nominal diameter

F_{ub} = specified minimum tensile strength of the bolt

Alternatively, 0.75 A_b may be substituted with A_n as follows:

$$A_n = \frac{\pi}{4}(d - 0.9382p)^2$$

where

d = nominal diameter of bolt

p = pitch of threads, mm

7.5.2.4 Combined tension and shear

A bolt in a joint that is required to develop resistance to both tension and shear shall be proportioned so that the following relationship is satisfied:

$$\left(\frac{V_f}{V_r}\right)^2 + \left(\frac{T_f}{T_r}\right)^2 \le 1$$

where

V_f = bolt shear force under factored load

V_r = the factored shear resistance determined in accordance with Clause 7.5.2.1

T_f = bolt tensile force under factored load

T_r = the factored tensile resistance determined in accordance with Clause 7.5.2.3

7.5.3 Anchor rods

For anchor rods, the following interaction equation shall be satisfied:

$$\left(\frac{T_f}{T_r}\right)^2 + \left(\frac{V_f}{V_r}\right)^2 \le 1$$

and

$$\left|\frac{C_f}{C_r}\right| + \left(\frac{V_f}{V_r}\right)^2 \leq 1$$

where

T_f = anchor rod tension force under factored loads

C_f = anchor rod compression force under factored loads

V_f = anchor rod shear force (direct shear and torsion components) corresponding to T_f or C_f

T_r = the factored tensile resistance determined in accordance with Clause 7.5.2.3

C_r = the factored compressive resistance of anchor rod = $\phi A_n F_y$ and ϕ = 0.90

V_r = the factored shear resistance determined in accordance with Clause 7.5.2.1

When the clear distance from the top of concrete to the bottom levelling nut exceeds the diameter of the anchor rod, and without properly installed high strength grout, the following interaction equation shall also be satisfied:

$$\left(\frac{V_f}{V_r}\right)^2 + \left(\left|\frac{T_f}{T_r}\right| + \left|\frac{M_f}{M_r}\right|\right)^2 \leq 1$$

and

$$\left(\frac{V_f}{V_r}\right)^2 + \left|\frac{C_f}{C_r}\right| + \left|\frac{M_f}{M_r}\right| \leq 1$$

where

M_f = factored bending moment corresponding to V_f

= 0.65 $l_{ar} V_f$

l_{ar} = length from top of concrete to bottom of anchor rod levelling nut

M_r = anchor rod flexural resistance determined based on the tensile root diameter in accordance with Clause 7.4.3

7.5.4 Fillers

When bolts transmitting shear force pass through fillers with a total thickness greater than 6 mm, the fillers shall be extended beyond the connection. The filler extension shall be secured by sufficient fasteners or welded to distribute the total force in the member at the ultimate limit state uniformly over the combined section of the member and the filler. Alternatively, an equivalent number of fasteners may be included in the connection or the filler may be welded within the connection.

7.5.5 Connections

Clauses 21, 22, and 23 of CSA S16 shall apply, except that

a) bearing-type connections may be used for connections subject to stress reversal;
b) nuts shall be prevented from working loose by pretensioning of the bolts, the use of jam or lock nuts, lock washers, or other methods acceptable to the engineer;
c) for tension connections, where it can be shown that the stiffness of the connected parts is sufficient to reduce prying forces to insignificance, tension connections may be made with high-strength bolts tightened to a snug-tight condition; and
d) connecting bolts at the crossover point shall be pretensioned.

Note: *See also Clause 12.2 of this Standard.*

7.5.6 Installation and field review

All connections at a minimum shall be installed to the snug-tight condition and the installation inspected to the satisfaction of the engineer. For pretensioned connections, the engineer shall be satisfied that proper pretensioning has been achieved in the field based on the method used to achieve proper bolt tensions.

7.5.7 Splices

7.5.7.1 General

The minimum resistance of a splice shall not be less than the maximum design force in the member and, where practicable, should equal the design resistance of the member. The splice shall provide sufficient stiffness to ensure the continuity of the member.

7.5.7.2 Minimum resistance in tension

For splices that transfer compression by bearing, the mating surfaces shall be finished to bear. The splice shall have a minimum tensile resistance equal to the maximum tension force, but not less than 33% of the compressive resistance of the member, except where the design force in the splice is compression only and a lower tension value can be justified by an engineering study taking into account the dynamic effects of wind and erection forces in the member.

7.5.7.3 Gap splices

For gap splices, the eccentricity between the centroid of the member and the centroid of the splice plates shall be taken into account in the design of the splice plates and bolt groups.

7.5.7.4 Prying action

For flange type splices in tension, the effects of prying action shall be considered in the design of the bolts and connecting parts.

7.5.8 U-bolts

7.5.8.1 General

Nut locking devices shall be used for all U-bolt connections. Nuts and U-bolts shall be provided from the same source to ensure compatibility of the nuts with the threads of the U-bolts after galvanizing.

Pretensioned U-bolts shall not be re-used once they have been placed in service.

Square U-bolts or V-bolts shall be stress-relieved after forming.

U-bolts shall conform to one of the following material specifications: CSA G40.21, AISI 1018, ASTM A36, A529, A572 Gr 42, 50 or 55, or F1554 Gr 36 or 55.

The inside nominal width of a square U-bolt shall be detailed to not exceed by more than 3 mm the width of the attached member. The inside corner radius of a square U-bolt shall not be less than the outside corner radius of an HSS member or the diameter of the U-bolt. In lieu of more accurate information, the outside corner radius of an HSS member shall be assumed to equal 2.25 times the nominal wall thickness of the HSS.

7.5.8.2 Factored resistance

U-bolt connections shall not be utilized to transfer moments about the longitudinal axis of a round supporting member (torsion) from a structural member required to maintain strength and stability of a structure. The limitation under this loading condition shall not apply to connections used for attachments.

Shear and axial forces on each leg of a U-bolt shall meet the resistance requirements for bolted bearing connections as per Clause 7.5.2. In addition, the tensile resistance strength, T_r, of each leg of the U-bolt shall not exceed $\phi A_b F_y$ where F_y is the U-bolt minimum specified yield strength and A_b is the gross area of one leg of the U-bolt based on nominal dimensions.

The strength of a U-bolt connection in transferring forces parallel to the longitudinal axis of a supporting member (sliding) and moments about the longitudinal axis of a supporting member (torsion) shall satisfy the following interaction equation:

$$\left(\frac{V_f}{V_r}\right)^2 + \left(\frac{T_{uf}}{T_{ur}}\right)^2 \le 1.0$$

where

V_f = factored shear force applied parallel to the supporting member (sliding)

T_{uf} = factored torsional moment applied about the longitudinal axis of the supporting member (torsion)

V_r = sliding resistance in accordance with the following:
$\phi\ 0.3(2T_p - T_f) > 0$

T_{ur} = torsional strength = $0.5\ DV_r$

T_p = installed pretension in each leg of U-bolt

T_f = factored tensile force applied to the U-bolt assembly

D = diameter of supporting member

ϕ = 0.9

The installed pretension in each leg of a U-bolt shall be considered to equal 140 A_b but not to exceed the maximum force that a U-bolt can be tightened based on the U-bolt bracket strength.

7.6 Welding

7.6.1 General

Welding design and practice shall conform to CSA W59.

7.6.2 Welding of steel tubular pole structures

7.6.2.1 Circumferential welds

Complete penetration welds shall be used for tubular sections joined by circumferential welds.

7.6.2.2 Welds of flanges

The weld connection of tubular sections to flange and base plates shall be given special attention considering the type of joint used and the forces to be resisted.

7.6.2.3 Longitudinal seam welds

Longitudinal seam welds for tubular sections shall have 60% minimum penetration, except in the following areas:

a) longitudinal seam welds within 150 mm of circumferential welds shall be complete-penetration welds; and
b) longitudinal seam welds on both sections of telescopic (slip-type) field splices shall be complete-penetration welds for a distance of the nominal splice length plus 150 mm.

7.7 Telescoping field splices for tubular pole structures

7.7.1 General

The minimum length of any telescoping (slip-type) field splice shall be 1.5 times the inside dimension of the larger section.

7.7.2 Fabrication tolerances

Good quality of fabrication is an important factor in minimizing field construction and performance problems. Fabrication tolerances for tubular pole sections are not specified in standards and should be agreed upon between the owner and designer, and specified in fabrication documents.

Note: *See Annex P for suggested tolerances.*

7.8 Openings in tubular members

The cross-section properties, such as section modulus and minimum moment of inertia of the tubular member at reinforced openings, shall be equal to or greater than the cross-sectional properties of the member without openings at the level in question, unless the design has considered the reduced section properties.

8 Corrosion protection

8.1 General

The exposed, open steel framework of most communications structures requires corrosion protection. Hot dip galvanizing has proven to be the most satisfactory and economical method for normal atmospheric corrosion protection. In areas with a highly corrosive industrial or environmental atmosphere, consideration should be given to providing other protection in addition to the galvanizing. Where other coatings, such as painting, are required, they should be applied before erection due to the height and operational restrictions of most towers.

8.2 Structural steel

8.2.1 Zinc coatings

All structural steel members shall have a zinc coating. Hot dip galvanizing, conforming to the requirements of ASTM A123/A123M, shall be used. Alternatively, zinc metal spray, conforming to the requirements of CSA G189, may be used, subject to the approval of the engineer. The minimum thickness of the zinc spray shall equal or exceed the requirements of ASTM A123/A123M.

8.2.2 Entrapped moisture

Corrosion likely to occur due to entrapped moisture or other factors shall be eliminated or minimized by appropriate design and detailing. Positive means of drainage shall be provided, unless the member is completely sealed.

8.2.3 Permanently sealed surfaces

Corrosion protection is not required for the inner surfaces of enclosed spaces that are permanently sealed from any external source of oxygen.

8.3 Guy assemblies

8.3.1 General

Guy cables and preformed guy grips of ferrous materials shall be constructed of hot-zinc-coated wire in accordance with the applicable standard for the type and construction being utilized. Aluminum-coated steel wire shall be considered acceptable.

8.3.2 Zinc coatings

Guy hardware of ferrous materials shall be zinc-coated in accordance with applicable standards. Acceptable processes are hot dip galvanizing, zinc metal spray, and mechanical galvanizing. The minimum thickness shall meet the requirements of ASTM A123/A123M.

8.3.3 Non-ferrous components

Nonferrous components shall be of corrosion-resistant materials, electrochemically compatible with adjacent materials. Aluminum or aluminum-coated fittings in contact with zinc coatings shall be considered electrochemically compatible.

8.4 Fasteners

All steel fasteners shall be zinc-coated, either hot dip galvanized in accordance with ASTM A153/A153M or mechanically galvanized in accordance with ASTM B695. Precautions shall be taken to ensure that harmful effects of hydrogen embrittlement are controlled.

8.5 Anchorages

8.5.1 Zinc coatings

All exposed steel items for anchorages, including anchor bolts, shall be zinc-coated in accordance with Clause 8.2, or otherwise suitably protected. Where anchorage steel is partially embedded in concrete, the zinc coating shall extend a minimum of 50 mm into the concrete.

8.5.2 Steel below grade

Anchorage steel below grade that is not encased in concrete shall be galvanized and further corrosion protection shall be provided. Special corrosion protection methods shall be considered for anchorage steel in corrosive soil conditions. For these cases, galvanizing may be sacrificed at similar rates to bare steel under similar conditions thereby reducing design life.

Where galvanizing of below grade anchorage steel is not practical (such as pile foundations), the designer shall consider other suitable means to provide corrosion protection over the design life of the structure. (See Annex F.)

Note: *It has been shown that the useful design life of driven bare steel piles or screw piles installed in undisturbed soil (regardless of the soil properties) is not significantly affected. This is attributed to an oxygen deficiency in the soil a few feet below the ground line or below the water table zone of undisturbed soils. However, where in-situ soils are disturbed during construction, corrosion protection must be considered.*

8.6 Repair

Reworked or damaged zinc coatings shall be repaired in accordance with the processes and requirements of ASTM A780/A780M and its annexes.

9 Other structural materials

9.1 General

Structures covered by this Standard may be constructed of materials other than structural steel. Design and manufacture for conventional materials shall conform to the standards listed in Clauses 9.3, 9.4, and 9.5. For other materials, that do not have an established CSA standard, factored resistances shall be provided such that the level of reliability is equivalent to that provided by this Standard.

9.2 Loads

Factored loads and member forces shall be calculated in accordance with this Standard.

9.3 Concrete

9.3.1 Factored resistances

The factored resistances of components, including foundations and guy anchorages made of concrete, shall conform to the requirements of CAN/CSA-A23.3.

9.3.2 Construction and testing

The construction and testing of concrete structures and components shall conform to the requirements of CSA A23.1/A23.2.

9.3.3 Cylindrical concrete towers

The resistance of the shafts of cylindrical, self-supporting, reinforced-concrete towers may be in accordance with the requirements of ACI 307 and the resistance factor used with such shafts shall provide a level of reliability equivalent to that provided by this Standard.

9.4 Structural aluminum

The factored resistances and construction of components made of aluminum shall conform to the requirements of CAN/CSA-S157. For bolted connections, the threads of the fasteners shall be excluded from the bearing area.

9.5 Timber

The factored resistances and construction of components made of timber shall conform to the requirements of CSA O86.

10 Guy assemblies

10.1 General

10.1.1 Assemblies

Guy assemblies shall be constructed of appropriate materials. Requirements for commonly used materials are given in Clause 10.2. Other materials, including non-metallic materials, may be used subject to the approval of the engineer.

10.1.2 Tests

The manufacturer of guy assembly components, such as preformed guy grips, mechanical sleeves, clips, and other fittings, shall provide results of static and dynamic tests, establishing the performance, including strength and efficiency, of the component. The results shall be evaluated on a statistical basis, unless otherwise accepted by the engineer.

10.1.3 Quality control

The manufacturer shall provide, on request, information on the methods used for quality control of the component.

10.1.4 Component properties

In addition to strength requirements, due consideration shall be given, in the choice of materials, to fatigue, ductility, toughness, and brittle fracture. In particular, any heat treatment shall be such that the properties of strength, ductility, and toughness necessary for the intended use are obtained.

10.2 Wire rope and wire strand

10.2.1 Material standards

Material shall conform to the following standards:

a) wire rope: CSA G4, using hot-zinc-coated wire;
b) guy strand: CAN/CSA-G12 or ASTM A475;
c) bridge strand: ASTM A586; and
d) aluminum-coated steel wire strand: ASTM A474.

10.2.2 Pre-stretching

Guys may be pre-stretched prior to installation, subject to the approval of the engineer. Pre-stretching shall be done in accordance with the manufacturer's recommendations.

10.2.3 Splicing

Strand or wire ropes shall not be spliced by means of a running or woven type of splice.

10.2.4 Bending

Wire rope or strand greater than 29 mm in diameter shall not be field-bent to form an end attachment. If bent in a shop or plant, the end attachment shall meet the requirements of Clause 10.1.2.

10.3 Other components

10.3.1 Clips

Twin-base or U-bolt wire rope clips may be used to secure looped ends. The clips shall be the same size as the guy within a tolerance of ± 1.2 mm.

10.3.2 Preformed guy grips

Preformed guy grips shall be designed specifically for the type of guy being used. Design considerations shall include the electrochemical compatibility of the material and the number, size, and lay of wires in the strand. The preformed guy grip manufacturer shall perform static and dynamic tests to demonstrate the capacity and efficiency of the product for different guy sizes.

Note: *It should be noted in choosing components associated with strand that the lay of the wire in the strand can vary from one manufacturer to another according to their production techniques.*

10.3.3 Mechanical or pressed sleeves

Loops may be secured by means of a mechanical type of splice consisting of a metal ferrule applied cold under heavy pressure. The ferrule shall be of a corrosion-resistant material, electrochemically compatible with the strand or rope being spliced. This type of splice shall not be used for wire rope having a fibre core. Care shall be taken in the application of the sleeves to ensure that the individual wires of the strand or rope being spliced are not broken or crushed.

10.3.4 Sockets

Sockets for strand and wire rope shall be cast in accordance with ASTM A148/A148M or ASTM A27/A27M, heat-treated. They shall be designed to transmit the breaking strength of the guy without permanent deformation. Sockets manufactured for use with wire rope shall not be used for strand, nor shall sockets manufactured for galvanized strand be used for aluminum-coated strand without the approval of the engineer. Sockets for use with other types of cables shall conform to recognized standards and shall provide a level of reliability equivalent to that provided by this Standard.

10.3.5 Thimbles

A thimble or other means shall be used to maintain an adequate radius at the inside of end attachments consisting of loops. Thimbles shall be of the heavy duty or solid type, and the size shall be in accordance with the manufacturer's recommendations.

10.3.6 Shackles

Shackles shall be forged from AISI grade 1035 steel, or equivalent, suitably heat-treated.

10.3.7 Turnbuckles

Turnbuckles shall be forged from AISI grade 1035 steel, or equivalent, suitably heat-treated.

10.3.8 Guy link plates

Where guy link plates are used, any resultant binding or bending in the components shall be accounted for in the design.

10.3.9 Initial tension tags

Permanent tags shall be affixed at the anchor end of each guy assembly above guy hardware, where they are clearly visible to the inspection and maintenance personnel.

10.4 Design of guys

10.4.1 General

The factored resistance of the guy assembly shall be equal to or greater than the maximum factored guy forces determined in accordance with Clause 6.

10.4.2 Efficiency factors for guy assemblies

10.4.2.1 General

The effective breaking strength of strand or cable shall be taken as the manufacturer's rated breaking strength times an efficiency factor for the type of end attachment, expressed as a percentage, based on static and dynamic tests in accordance with Clause 10.1.2.

10.4.2.2 Efficiency factors

Clips on strand shall be considered to have a maximum efficiency factor of 75%, except that twin-base and U-bolt clips on seven-wire guy strand may be considered to have a maximum efficiency factor of 90%. For other types of end connections and for cable, the manufacturer's published efficiency factor for the attachment, based on tests in accordance with Clause 10.1.2, shall be used.

10.4.3 Effective resistance

The effective resistance of a guy assembly shall be the lesser of

a) the effective breaking strength of the strand or cable; or
b) the ultimate strength of any other component of the assembly.

The ultimate strength of any other component shall be not less than 90% of the manufacturer's rated breaking strength of the strand or cable.

10.4.4 Factored resistance

The factored resistance of the guy assembly shall be the effective resistance times the resistance factor, $\phi = 0.6$, except that the resistance factor for guy insulators shall be in accordance with Clause 15.1.3.

10.4.5 Initial tensions

Guy tensions shall be measured at the anchorage. For the initial condition, as defined in Clause 6.1, guy tensions are normally set at 10% of the rated breaking strength of the strand or cable, and generally within limits of 8 to 15%. In setting the initial guy tension, consideration shall be given to the response of the structure, including its dynamic behaviour and to variations in the guy initial tension.

10.4.6 Articulation

Full articulation shall be provided at both ends of any guy assembly.

10.4.7 Take-up devices

Turnbuckles or other means of length adjustment that do not necessitate disconnecting end attachments shall be provided for all guy assemblies. The minimum available adjustment length shall be

a) 300 mm for guys of nominal diameter 13 mm or less; and
b) 450 mm for guys of nominal diameter greater than 13 mm.

For long guys, additional take-up should be provided.

The take-up device shall be secured against self-loosening.

10.4.8 Non-metallic material

10.4.8.1 General
Where non-metallic material is used, the effect of both temporary and permanent elongation of such material on the stability of the structure shall be taken into consideration.

10.4.8.2 High RF
When used on towers with high RF power, the connection between the guy assembly and the tower shall include, as a minimum, a corona socket or shield. For higher power applications, insulators as described in Clause 15 may be used to reduce the risk of damage to the synthetic guy materials from high RF fields surrounding the end connections.

11 Foundations and anchorages

11.1 General

11.1.1 References

11.1.1.1 *National Building Code of Canada*
The requirements of Section 4.2 (foundations) of the *National Building Code of Canada (NBC)* shall apply, except as modified in Clause 11 of this Standard.

11.1.1.2 *Canadian Foundation Engineering Manual*
The *Canadian Foundation Engineering Manual* provides information on geotechnical aspects of foundation engineering, as practiced in Canada.

11.1.2 Geotechnical site investigation

11.1.2.1 General
A geotechnical investigation, carried out by or under the direction of an engineer having sufficient knowledge and experience in planning and executing such investigations, shall be provided for the site. Geotechnical investigation for sites located in permafrost areas shall be carried out under the direction of an engineer with sufficient knowledge and experience in permafrost geotechnical investigation.

11.1.2.2 Information required
The geotechnical report shall include all necessary information for the design and/or design review of the foundations and anchorages in soil, rock, or permafrost. The information provided in the geotechnical report shall be based on or compatible with limit states design methodology.

Note: *See Annex L.*

11.1.2.3 Failure mechanisms
The geotechnical report shall specify all failure mechanisms that could involve any geotechnical component. For each failure mechanism, the geotechnical report shall specify either the ultimate geotechnical resistance or the method of calculating the ultimate geotechnical resistance when the

resistance varies according to the foundations' and/or the anchorages' geometry and depth. The geotechnical report shall specify the information required to associate applied pressure with corresponding displacement over a range up to the limit values specified in the geotechnical report.

Note: *See Annex L.*

11.1.2.4 Modifications to resistance factor, ϕ

When required, the geotechnical report shall address modifications to the recommended resistance factors, ϕ, described in Clause 11.2.6.

11.1.2.5 Sensitivity of soil

The geotechnical report shall address the sensitivity of the soil or rock to earthquakes and other design considerations as they apply to the site and, if required, identify the Site Class for seismic site response as defined in Section 4.1.8.4 of the*NBC*.

Note: *See Clause M.6.*

11.1.2.6 Pre-existing report

If a pre-existing geotechnical report is available and values provided in the report are "allowable values" only, the safety factors used in arriving at these allowable values shall be used to determine the ultimate resistance. In cases where the safety factors are not specified, a safety factor of 2.0 shall be used.

Note: *See Annex L.*

11.2 Design

11.2.1 General

The design of tower foundations and guy anchors shall be based on the geotechnical investigation and report referred to in Clauses 11.1.2.1 to 11.1.2.6.

11.2.2 Anchor shaft design

The guy resultant angle on anchor shafts connected to foundation elements can vary over the design life of guyed tower structures. For new towers, the anchor shaft design shall consider the effects of guy initial tension only loading, initial design loading, and initial plus future design loading. For existing towers, guy resultant angle variations due to changes in the tower geometry, or loading shall be considered. The use of bolted connections shall not be used along the shaft below the guy anchor plate unless specifically designed to account for guy resultant angle variations. As an alternative, a pin connection in the lower part of the anchor shaft may be used to minimize bending moments from variations in the guy resultant angle.

11.2.3 Soil density

For design purposes, soil shall be considered as having a density of not more than 1600 kg/m^3, unless otherwise indicated in the geotechnical report, and a concrete density of 2300 kg/m^3.

11.2.4 Submerged soil density

For design purposes, the submerged density of any portion of a foundation or an anchorage placed below the water table level shall not exceed 1000 kg/m^3 for soil, unless otherwise indicated in the geotechnical report, and 1300 kg/m^3 for concrete.

11.2.5 Ultimate resistance

The foundation and/or anchorage shall be designed to resist the effects of all factored loads as determined in accordance with Clause 6. The ultimate foundation and/or anchorage capacity shall be calculated using the ultimate geotechnical resistances as recommended in the geotechnical report.

11.2.6 Resistance factors

The factored resistance of the foundation or anchor shall be calculated as the ultimate calculated capacity, multiplied by the resistance factor, ϕ, where

a) ϕ = 0.75 for bearing resistance of foundations (rock or soil), for self-supporting structures;

b) ϕ = 0.60 for bearing resistance of foundations that provide positive engagement with rock and/or soil, for guyed masts;

c) ϕ = 0.50 for bearing resistance of foundations that provide engagement with rock and/or soil by friction or adhesion, for guyed masts;

d) ϕ = 0.75 for pull-out and uplift resistance of foundations and anchors in soil that provide positive engagement with the soil;

e) ϕ = 0.50 for pull-out and uplift resistance of foundations and anchors in soil for driven piles or drilled caissons with the base equal to the top;

f) ϕ = 0.40 for pull-out and uplift resistance of foundations and anchors in soil for driven piles or drilled caissons with the base narrower than the top (tapered cross-section);

g) ϕ = 0.50 for pull-out and uplift resistance of foundations and anchors in rock that utilize only one rock bolt, dowel, or anchoring device as described in Clause 11.4;

h) ϕ = 0.75 for pull-out and uplift resistance of foundations and anchors in rock that utilize more than one rock bolt, dowel, or anchoring device as described in Clause 11.4; and

i) ϕ = 0.75 for lateral resistance of foundations and anchors in soil or rock for all types of foundations and anchors.

Note: *See Table 12.*

Table 12
Resistance factors, ϕ, for geotechnical resistances
(See Clause 11.2.6.)

		Resistance by bearing on rock or soil (positive engagement)		Resistance by friction with the soil	
Self-supporting structures	**Uplift**	g) ϕ = 0.50 when resisted by only one rock anchor bolt as described in Clause 11.4	d), h) ϕ = 0.75	f) ϕ = 0.40 for driven piles or drilled caissons with the base narrower than the top	e) ϕ = 0.50 for driven piles or drilled caissons with the base equal to the top
	Bearing	a) ϕ = 0.75		a) ϕ = 0.75	
Guyed masts	**Lateral**	i) ϕ = 0.75		i) ϕ = 0.75	

(Continued)

Table 12 (Concluded)

		Resistance by bearing on rock or soil (positive engagement)		Resistance by friction with the soil	
	Uplift	g) ϕ = 0.50 when resisted by only one rock anchor bolt as described in Clause 11.4	d), h) ϕ = 0.75	f) ϕ = 0.40 for driven piles or drilled caissons with the base narrower than the top	e) ϕ = 0.50 for driven piles or drilled caissons with the base equal to the top
	Bear-ing	b) ϕ = 0.60		c) ϕ = 0.50	

11.3 Foundations and anchorages in soil

11.3.1 Bearing against undisturbed soil

Where possible, the sides of tower foundations and the front face of guy anchors shall be placed against undisturbed soil. Where this is not possible, the engineer shall specify the type of material required for the backfill and compaction effort used to provide the necessary resistance against sliding, uplift, and overturning.

11.3.2 Base of foundation below frost line or into permafrost

Tower foundations and guy anchors in soil shall extend below the frost line, except in permafrost areas, unless frost action is considered in the design of the foundations and guy anchors.

11.4 Rock anchors

11.4.1 Pull-out strength

The pull-out strength of rock anchors may be achieved by one of the following:

a) mechanical anchorage by a device such as an expanding shield;

b) bonding of grout to deformed bars and to the rock. Plain bars shall not be considered to develop bonding with the grout; or

c) bearing on grout by an end plate connected to a bar, a nut and washer, or a similar device, and bonding of the grout to the rock.

11.4.2 Drilled holes

All drilled holes shall be completely filled with grout. Grout shall be a non-shrinking type with a minimum compressive strength of 25 MPa at 28 days.

11.4.3 Weathered rock

Unless special deep anchors are employed, anchors in weathered and fractured rock shall be designed as anchorages in soil.

11.4.4 Combined effects of shear and tension

If rock anchors are required to provide resistance to horizontal forces, they shall be designed for the combined effects of shear and tension from the horizontal, overturning, and uplift loads.

11.5 Roof installations

Anchorage of roof-mounted structures and antennas shall be designed to resist the effects of all factored loads as determined in accordance with Clause 6.

Note: *See Clause 4.4.2. See Clause 5.12.3 and Annex M for seismic considerations.*

11.6 Non-penetrating mounts

11.6.1 General

Anchorages may be provided by non-penetrating mounts designed to resist the effects of all factored loads as determined in accordance with Clause 6. When resistance to sliding and overturning is provided by use of ballast, the resistance to sliding shall be determined by tests on the materials to be used for the faying surfaces, subject to all conditions that can reasonably be expected during the installation, including effects of temperature, moisture, dirt, and other contaminants. Accepted statistical procedures shall be used to establish the design value of the coefficient of friction from the test results such that the probability of occurrence of a lower coefficient of friction is not greater than 5%. Lateral resistance to earthquake effects shall not be provided with systems relying on friction only.

Note: *Typical roof installations are for antennas up to 5 m^2 surface area. Various types of rubber matting are often used under the mount to protect the roof surface and to provide improved resistance to sliding. These mats have acceptable performance on a wide variety of roof surfaces, while typically providing a minimum coefficient of friction of 0.6.*

11.6.2 Ultimate load effects for overturning and sliding

Factored loads for ultimate limit states, including overturning and sliding, shall be determined in accordance with Clause 6.3. Where resistance to movement is provided by dead load alone, the factored loads shall be determined using a dead load factor, α_D, of 0.85. The mount shall be proportioned so that sliding will occur before overturning, and movement shall be limited to the extent that it does not endanger people, the building, or any other attachment.

11.6.3 Serviceability load effects for sliding

Where loss of service is considered acceptable, and subject to Clause 11.6.2, sliding of the antenna may be considered as serviceability limit state. Serviceability loads shall be determined in accordance with Clause 6, with a resistance factor of 0.8 applied to the coefficient of friction.

11.7 Foundation and anchorage installation — Field review

In order to ensure that subsurface conditions are consistent with those used for design, foundations, and anchorages shall be reviewed at all critical stages of the work by the engineer responsible for the design or by qualified persons reporting to the engineer.

12 Tower and pole structure installation

12.1 General

12.1.1 Construction loads

Suitable provision shall be made to ensure that the construction loads shall be safely sustained during the erection procedure without permanent deformation or other damage to any member of the structure. Special consideration shall be given to existing structures undergoing construction or modifications. Any increase in stresses to the structure from construction loads (winches, hoists, gin poles and other construction equipment) or changes to tower stability (member replacement, guy changeout) shall be considered.

Note: *For guidance on temporary construction wind speeds, refer to ANSI/TIA-322.*

12.1.2 Members and surfaces not to be damaged

Iron sledges shall not be used for driving or hammering any members. Hammers of plastic, lead, wood, or other soft material shall be used to minimize damage to the galvanizing. Care shall be taken during the placing of members to prevent impact on other members.

12.2 Connections

After the erection has been completed, all bolted connections, including connections on attachments, shall be checked and any loose nuts retightened. Joints using ASTM F3125 Grade A325 bolts shall be assembled in accordance with Clause 23 of CSA S16, except that for bolts in shear other means may be provided to prevent loosening of the nuts, in accordance with Clause 7.5.5. Galvanized high-strength bolts (ASTM F3125 Grade A325 or equivalent) shall not be reused. Retightening of previously tightened bolts that have been loosened by the tightening of adjacent bolts shall not be considered as a reuse.

12.3 Tolerances

12.3.1 Guy tensions

Guy initial tensions determine the stiffness of the guying system and affect the performance of the structure. They shall therefore be set and maintained as follows:

a) For new structures, the guy tensions shall be set within +15% and –5% of the specified initial tension at anchorage, corrected for the ambient temperature. For existing structures where the constructional stretch of the cables has occurred, guy tensions shall be set within +10% and –10% of the specified initial tension.

b) When guy tensions cannot be set to the tolerances given in Item a), guy tensions shall be considered satisfactory if the average tension of the guys at the same level meets the tolerances of Item a).

Note: *The initial tensions in the guys at one level on the structure may differ due to differences in the elevations of the guy anchors and twist in the tower shaft. It is important that the tower be plumbed before the specified initial tensions are applied within the tolerances. Extreme cases may require differences beyond the recommended tolerances.*

12.3.2 Verticality

For guyed and self-supporting structures, the out-of-plumb between any two elevations on the structure shall not exceed 1 in 500.

12.3.3 Twist

The twist between any two elevations shall not exceed 0.5° in 3 m. The maximum twist over the structure height shall not exceed 5°.

12.3.4 Straightness

The straightness of individual members shall be within a tolerance of 1 in 500.

12.3.5 Measurements

Measurements shall be made at a time when the wind velocity is less than 25 km/h at ground level and there is no ice on the guys of guyed structures.

Note: *See Annex C for measuring methods.*

12.4 Articulation

Care shall be taken to ensure that articulation is maintained at both ends of the guy assemblies, with particular attention paid to the installation of turnbuckles at anchorages.

12.5 Take-up devices

Adjustment of turnbuckles shall not be made by means of leverage through the body of the turnbuckle. Where a cable or other device is used to prevent rotation of a turnbuckle, the arrangement shall avoid damage due to guy vibrations.

For initial installations, the minimum take-up adjustment available after the structure is plumb and the guy tensions are set, unless otherwise accepted by the engineer, shall be

a) 150 mm for guys with nominal diameter of 13 mm and smaller; and

b) 250 mm for guys with nominal diameter greater than 13 mm.

12.6 Clips

Clips on wire rope shall be installed in accordance with the manufacturer's instructions with regard to number, spacing, and torque. Clips on bridge and guy strand should be installed in accordance with Annex B. The bolts on U-bolt and twin-base clips shall be tensioned using the specified torque within a tolerance of ± 5%. After installation, an inspection shall be made to determine whether any damage has been done by the tensioning of the nuts. The final torque shall be checked after the initial tension has been applied. Guy clips may be reapplied, provided an inspection indicates that the bolts have not been damaged or distorted by overtensioning.

12.7 Preformed guy grips

Guy grips shall be new and installed in accordance with the manufacturer’s recommendations. During installation, guy grips may be reapplied not more than three times. Guy grips shall not be reused after having been in service.

12.8 Erection equipment

Erection equipment and tests performed during erection shall not cause damage or distortion to any part of the guy assembly or to the tower.

12.9 Grounding

All structures shall be grounded during erection.

12.10 Welding

Field welding shall be in accordance with the requirements of CSA W59.

12.11 Telescoping field splices

All telescoping (slip-type) field splices of tubular pole structures shall be jacked together using a jacking force at least 1.3 times the factored axial compressive force in the pole at the splice location and shall provide the specified minimum splice length after jacking. Installation drawings shall indicate the jacking procedure and the jacking force to be used for each splice. Shims or spacers shall not be used in telescoping splices.

12.12 Structure installation — Field review

New or modified structures shall be field reviewed during critical stages of the work. The structure shall be reviewed by the engineer responsible for the design or by qualified persons under direct supervision of the engineer responsible for the design in accordance with the requirements specified on the design documents. Recommendations for the field review stages are contained in Annex D.

13 Obstruction marking

When required to identify the presence of structures posing a hazard to aircraft navigation, obstruction marking shall be in accordance with CARs 621.19.

14 Bonding and grounding

14.1 General

14.1.1 Grounding against damage

Bonding and grounding provisions for structures shall conform to CSA C22.1 and CAN/CSA-B72. The grounding configuration shall be acceptable to the owner as it is dependent on the requirements of the particular site.

Note: *Section 10 of CSA C22.1 covers general bonding and grounding requirements and defines the size of grounding conductors and electrodes, their placement, and methods of connection. CAN/CSA-B72 covers lightning protection systems for towers (Class IV installations) that are erected on buildings.*

14.1.2 Grounding for performance

Grounding considerations related to the transmission characteristics of antenna towers for AM broadcasting do not form part of this Standard. These are outlined in Industry Canada's *Broadcasting Procedures and Rules, Parts I and II.*

14.2 Bonding

14.2.1 All components bonded to tower and building

For the safety of personnel, and to prevent arcing and possible damage, conductive components within 2 m of each other shall be bonded together and to a mutual ground. Antennas and other attachments shall be bonded to the supporting structure, and transmission lines shall, as a minimum, be bonded to the tower at their top and bottom ends and to the building on the outside of the waveguide entry. The tower and equipment shelter grounding systems shall be interconnected, and all metal fences, transmission line support structures, and other metallic structures within 2 m of the tower structure or shelter shall be bonded to the mutual underground grounding system.

14.2.2 Metal-to-metal contact

To ensure a low-resistance, continuous path to ground for lightning strikes and stray electrical currents, all paint, scale, and rust shall be removed from the entire area of the mating surfaces of the leg member splices, including butt joints and splice plates, filler plates, and gusset plates forming the splice.

In preparing the mating surfaces, care shall be taken that the zinc coating of galvanized steel is not removed.

14.2.3 Grounding cable

Where the structure itself does not provide a continuous electrical path to ground, lightning protection and grounding of the structure and its attachments shall be provided by a lightning rod or other protective device at the top of the tower, bonded to a suitably sized grounding cable. The grounding cable shall run the full height of the structure and connect to the underground grounding system at the base of the structure without running under waveguide bridges. All conductive attachments and components shall be bonded to the grounding cable.

14.3 Grounding

14.3.1 Anchorages and foundations

A direct continuous electrical path to ground shall be provided at each tower foundation, centre pier, and guy anchorage. Ground rods and ground collector wires shall be installed at least 600 mm away from any foundation or anchor constructed of concrete. All conductors shall be free of sharp bends or kinks.

14.3.2 Guy assemblies

For guy assemblies, the grounding connection shall be made directly to the guy cable above the anchor assembly and take-up devices. Where a guy is sectionalized by means of insulators, the lowest section of the guy shall be grounded, unless otherwise instructed by the engineer.

Only approved mechanical-type connectors of non-corrosive, conductive material shall be used for connecting grounding conductors to guy cables.

14.3.3 Spark gap

The bottom section of a spark gap, where used, shall be connected to the grounding system by means of a grounding conductor.

14.3.4 Grounding lugs and plates

The locations where holes may be drilled, or the provision of lugs or plates, for the connection of grounding conductors shall be specified by the engineer.

14.3.5 Thermal connection

Where grounding specifications call for the connection of grounding conductors by thermal connection, the location and method shall be approved by the engineer.

14.3.6 On buildings

Towers on buildings shall be grounded in accordance with CAN/CSA-B72.

15 Insulators and insulation

15.1 Design

15.1.1 Ceramic materials
Guy insulators utilizing ceramic materials shall be designed so that these materials will be subject to compressive loads only, and their failure shall not result in failure or severance of the guy assembly.

15.1.2 Base insulators
In the design of base insulators, tension, compression, horizontal shear, and other applicable forces shall be taken into consideration.

15.1.3 Factored resistance
The factored resistance of a load-carrying component of an insulator shall be the ultimate resistance of the component times the resistance factor, ϕ, which shall be taken as 0.60 for metallic components made of steel or aluminum, 0.50 for non-metallic components of fail-safe insulators, and 0.40 for other non-metallic insulators.

15.1.4 Deterioration
Manufacturers shall provide details on the effects on their product of impact loading, vibration, high electrical voltages (both radio frequency and atmospheric), temperature extremes, weathering, radiation, and pollution. Any product that is subject to deterioration over a period of time may be installed only if

a) it is clearly specified that it shall be replaced after a specified period;
b) an appropriately reduced value of ultimate strength is used; or
c) the deterioration can be prevented by proper maintenance, in which case a maintenance schedule shall be specified.

15.1.5 End fittings
Where end fittings are used they shall be forged from AISI grade 1030, 1035, or 1045 steel (normalized) or its equivalent, or cast from steel in accordance with the requirements of ASTM A27/A27M or A148/A148M, and shall be suitably heat-treated (quenched and tempered, normalized, or annealed) and hot dip galvanized.

15.1.6 Cast metal pins
Cast metal pins shall not be used in insulators.

15.2 Spark gap
If base insulation is used, at least one insulation assembly shall be provided with an adjustable air gap. The bottom section of the spark gap assembly shall be grounded.

15.3 Inspection
Insulators shall be inspected on a regular basis, for deterioration, cracking, contamination, and oil leaks, where applicable.

16 Ladders, safety devices, platforms, and cages

16.1 General

16.1.1 Purpose
Clause 16 prescribes the minimum requirements for the design, construction, and use of ladders, safety devices, platforms, and cages used to provide safe conditions for persons climbing or working on structures covered by this Standard.

16.1.2 Load factor
Unless otherwise noted, a load factor, α = 1.5, shall be applied to the design loads specified in Clause 16 to determine the ultimate factored load.

16.1.3 Compliance
To comply with this Standard, towers and antenna support structures exceeding 3 m in height shall be equipped with the climbing and safety devices described in Clauses 16.1.4 and 16.1.5.

16.1.4 Ladders and climbing facilities
Ladders or climbing facilities shall be provided in one of the following forms:
a) a fixed ladder equipped with fall-arresting devices, cages, or hoops;
b) a fixed climbing facility equipped with fall-arresting devices, cages, or hoops;
c) a portable ladder that complies with the conditions set forth in Clause 16; or
d) a combination of the above with appropriate warning signs where changes take place.

16.1.5 Platforms
Platforms for use by workers shall comply with the following:
a) a rest platform shall be provided at least every 18 m of vertical height, unless the structure is equipped with fixed rail type fall-arresting devices;
b) a work platform shall be provided at locations defined in Clause 16.3.4.2; and
c) an interchange platform shall be provided at locations indicated in Clause 16.3.4.3.

16.1.6 Non-compliant structures
Any new or existing structure covered by this Standard that does not meet the requirements of Clause 16.1.3 shall be
a) climbed only by qualified persons as defined in Clause 16.10; and
b) equipped with appropriate warning signs in accordance with Clause 16.9.

16.2 Ladders and climbing facilities

16.2.1 Definitions

16.2.1.1 Ladders
A ladder consists of two side rails joined at regular intervals by cross pieces called steps, or rungs, on which a person may step while ascending or descending. Ladders may be portable or permanently fixed to the structure, but are not an integral part of the structure.

16.2.1.2 Climbing facility

A climbing facility is a series of properly spaced attachments permanently installed on a tower, support structure, or antenna, on which a person may step while ascending or descending, and which may incorporate or employ

a) steps, rungs, cleats, and/or structural members that form an integral part of the structure;
b) rungs, cleats, or step bolts that are permanently attached to the structure; or
c) corner steps consisting of individual cleats or step bolts that are permanently attached to, and project from, the corner of the support structure.

16.2.2 Load requirements

16.2.2.1 Fixed ladders and climbing facilities

The design of rails, rungs, steps, cleats, bolts, and connections of fixed ladders and climbing facilities shall consider the weight of the ladder and attachments, and live loads due to expected usage. The live loads imposed by persons occupying the ladder shall be considered to be concentrated at such point or points as will cause the maximum stress in the structural member being considered.

The minimum design unfactored live load on a rung, step, or cleat shall be the greater of

a) a single concentrated load of 1.1 kN; or
b) the anticipated impact loads resulting from the use of ladder safety devices.

The minimum design unfactored live load on a section of a fixed ladder between adjacent supports shall be a minimum of two concentrated live load units of 1.1 kN. Where more than two climbers use the same section of the ladder or climbing facility at the same time, additional live load units shall be applied, as appropriate.

16.2.2.2 Portable ladders

Portable ladders shall meet the requirements of the latest edition of CSA Z11. They may be constructed of materials other than steel. Portable ladders shall be capable of sustaining all loads to which they might reasonably be subjected.

16.2.3 Design requirements

16.2.3.1 All ladders and climbing facilities

Except as permitted in Clause 16.2.3.2 or 16.2.3.3, for all new ladders and climbing facilities

a) centre-to-centre spacing between rungs shall be 300 mm ± 5 mm;
b) clear spacing between side rails shall be not less than 300 mm;
c) side rails or equivalent support to the climber shall extend at least 1000 mm above the level of any platform or landing, unless the extension is precluded by other considerations;
d) when applicable, ladder connections to the structure shall prevent movement or rotation about the connections;
e) all rungs, cleats, steps, step bolts, rails, and fasteners shall be fastened to prevent rotation;
f) all rungs, cleats, steps, step bolts, and rails shall be free from splinters, sharp edges, burrs, or projections that can pose a hazard;
g) round rungs shall be between 19 and 50 mm in diameter, and step bolts and integral climbing facilities between 16 and 50 mm in diameter. Flat or angle ladder rungs or steps shall be not less than 25 mm in the horizontal plane and not more than 75 mm in all planes (see Figure 14);

h) there shall be a minimum clearance of 180 mm from the centre line of the ladder rung to any obstruction behind the ladder rung to provide adequate clearance for the climber's feet (see Figures 15 to 19);
i) there shall be a space clear of all obstructions at any rung, step, cleat, step bolt, or applicable tower member where the foot of a climber would be placed. This space shall measure at least 100 mm vertically, 125 mm horizontally, and 180 mm deep [see Item h)], above, and Figure 15);
j) there shall be at least 660 mm clearance from the centre line of a rung, cleat, step, step bolt, or applicable tower member to any obstruction on the climbing side to provide for clearance of the climber's head and body (see Figures 15 to 17); and
k) the slope of any ladder or climbing facility used shall be between 90° and 60° to the horizontal, but preferably between 90° and 75°. In no case shall the ladder or climbing facility slope toward the climber (see Figure 20).

16.2.3.2 Fixed ladders

In addition to the requirements of Clause 16.2.3.1, the specific features of a fixed ladder shall include the following:

a) Where a centre-mounted safety rail or cable is used, the clear distance between side rails shall be not less than 400 mm (see Figure 14).
b) Splices between ladder sections shall meet the strength design requirement or ladder sections shall be supported to carry all the design load between splices.
c) Steel ladders shall be hot dip galvanized.
d) Ladder rungs shall not be painted.

16.2.3.3 Climbing facilities

In addition to the requirements of Clause 16.2.3.1, the specific features of a climbing facility shall include the following:

a) Where a centre mounted safety rail or cable is used, the clear distance between side rails shall be not less than 400 mm (see Figures 14 and 15).
b) Centre-to-centre spacing between rungs, cleats, steps, or structural members used for climbing may be increased to not more than 410 mm, provided that the spacing remains uniform between interchange platforms, where a change in spacing may take place (see Figure 15).
c) The tower legs and other vertical support members may be considered as side rails (see Figure 15).
d) Provision shall be made to ensure that the climber’s foot cannot slide off the end of any rung, cleat, step, step bolt, or applicable tower member during a climb.

16.2.4 Use and location of ladders and climbing facilities

16.2.4.1 Fixed ladders and climbing facilities

Fixed ladders or climbing facilities may be installed to allow climbing on either the inside or the outside of the tower. However, the system selected shall be installed on the outside of a tower when the clearance inside the tower does not satisfy the requirements of Clause 16.2.3.1, Item j), since there is insufficient room inside smaller towers for a person with equipment to climb safely (see Figures 16 to 18).

16.2.4.2 Portable ladders

16.2.4.2.1 Maximum length

The maximum length of a portable ladder, measured along its side rail, shall not be more than
a) 6.0 m for a step ladder; and
b) 9.0 m for a single ladder.

16.2.4.2.2 Use

While in use, a portable ladder shall
a) be placed on a firm level footing and secured in such a manner that it cannot be dislodged accidentally from its position;
b) be sloped at between 4 and 5 vertical to 1 horizontal; and
c) if a step ladder, have its legs fully spread and its spreader locked.

A portable ladder shall not be lashed to another portable ladder to increase its length.

16.3 Platforms

16.3.1 Definition

A platform is a support on which a climber may stand while ascending or descending a tower in order to rest or perform work (see Clause 16.3.3).

16.3.2 Load requirements

The design of platforms and related connections shall consider the weight of the platform and attachments, and live loads due to expected usage. The live loads imposed by climbers occupying the platform shall be considered to be concentrated at such points as will cause the maximum stress on the platform and the structure.

The minimum unfactored live load on a platform shall be a single concentrated load of 1.1 kN. Where anticipated usage indicates that more than one climber may use the platform at the same time, additional live load units shall be applied, as appropriate.

16.3.3 Design requirements

Specific features of platforms shall be as follows:
a) platforms shall be within 330 and 762 mm of the centre line of the ladder or climbing facility, but located so that they will not interfere with climbing (see Figure 21);
b) steel platforms shall be hot dip galvanized;
c) there shall be a minimum head clearance of 2150 mm above the platform; and
d) unless equipped with an approved fall-arresting device or engineered anchorage for personal fall arrest equipment, all work and interchange platforms shall be equipped with hand rails and kick plates as shown in Figure 21.

16.3.4 Location of platforms

16.3.4.1 Rest platforms

If rest platforms are required, they shall be provided at a maximum vertical spacing of 18 m. Handrails shall not be required on rest platforms inside towers that have a face width of 2000 mm or less.

16.3.4.2 Work platforms

Work platforms shall be located to provide access to

a) antennas
b) obstruction lights; and
c) other attachments critical to the intended use and safe operation of the communication structure.

16.3.4.3 Interchange platforms

Interchange platforms shall be provided at

a) any point where the ladder or climbing device changes from one location on the tower to another; or
b) where the ladder rung spacing changes.

16.4 Fall arrestors and vertical rigid rails

16.4.1 Definition

Fall arrestors and vertical rigid rails are appliances incorporating special body harnesses with slide attachments and rails, cables, or special safety lines that, by means of a friction brake, locking device, or similar facility in the slide attachment, prevent climbers from falling more than a short distance should they slip while climbing on the structure. Fall arrestors and vertical rigid rails and their associated equipment shall conform to the applicable requirements of the CSA Z259 series of Standards.

16.4.2 Load requirements

The anchorage point for all fall arrestors and vertical rigid rails shall be a minimum of 17.8 kN factored load unless designed in accordance with the CSA Z259 series of Standards.

16.4.3 Design requirements

Specific features of fall arrestors and vertical rigid rails shall be as follows:

a) The rails or cables shall be continuous. For rails, sections shall be connected to permit smooth operation of the slide over the splice.
b) The rails or cables shall have a "stop" at the top so that the slide cannot inadvertently run off the top end of the rail (see Figure 21).
c) The rails and cables shall be installed as close as possible to the centre line of a fixed ladder or climbing facility.

However, these rails or cables may also be installed to the side or behind the climber provided the rails, cables, related attachments, and the system as a whole are designed to accommodate this mounting configuration and that the mounting configuration does not interfere with safe use of the climbing ladder or facility and the person using the rail or cable is correctly equipped and trained for its use.

16.4.4 Use and location of fall-arresting devices

Fall arrestor and vertical rigid rails shall be provided even when ladder cages or hoops are provided, except

a) when the fixed ladder or climbing facility is less than 3000 mm high;
b) when the climbing device is a portable ladder; or
c) when accessed by a qualified climber utilizing proper fall arrest safety equipment.

Fall-arresting devices may be used in lieu of platforms when platforms are neither available nor feasible.

At work levels, horizontal lifelines certified as meeting the requirements of the CSA Z259 series of Standards shall be considered acceptable.

Note: *It is not the intent of this Clause to suggest that fall arrestors and vertical rigid rail devices replace platforms. The intent is to recognize that some towers and antenna-supporting structures either do not have the space for platforms or are not conducive to having platforms installed on them. Under these conditions, the proper use of fall arrestors and vertical rigid rail devices or engineered anchorage for personal fall arrest equipment in conjunction with other safety procedures, permits properly qualified persons to work on the structure without platforms.*

16.5 Ladder cages and hoops

16.5.1 Definition

A ladder cage or hoop is an enclosure fastened to the side rails of the fixed ladder or directly to the climbing facility or structure to encircle the climbing space for the safety of the climber.

16.5.2 Design requirements

All ladder cages or hoops shall be designed to support a single concentrated load of 1.1 kN plus their own weight.

16.5.3 Specific features

Specific features of ladder cages and hoops (see Figure 19) shall be as follows:

a) The cage shall have an inside diameter of not less than 660 mm and not more than 762 mm (the centre line of the ladder being on the perimeter of the circle so formed).
b) The cage shall have vertical support bars equally spaced around the outside of the safety hoop at the lesser of a maximum of 40° or centre-to-centre spacing of 240 mm.
c) Hoops shall be equally spaced and separated vertically by not more than 1220 mm.
d) All hoops shall be securely fastened to the ladder and to each vertical support bar.
e) Vertical bars shall be continuous for the full height of the tower, unless an opening is required to gain access to a platform. Continuity may be achieved with the use of splice plates.
f) The bottom of the cage shall extend to not lower than 2140 mm and not higher than 2440 mm from the base of the structure, fixed ladder, or climbing device.
g) All bars, hoops, and fasteners shall be free from splinters, sharp edges, burrs, or projections, and shall be fastened in such a manner as to prevent rotation.
h) The top of the cage shall extend a minimum of 1070 mm above the top of the tower or platform.

16.5.4 Use and location of cages and hoops

Ladder cages and hoops shall be used when fall-arresting devices are not used, except when the fixed ladder or climbing facility is less than 3000 mm high or when the climbing device is a portable ladder.

Where fixed ladders or climbing facilities are installed to permit climbing inside small cross-section towers, the tower bracing shall not be considered as a cage, regardless of the tower face width.

Note: *Ladder cages and hoops should not be used for communication structures, except in special circumstances. They are only suitable for protection when moving from one point to another. On communication structures, it is frequently necessary to service the transmission lines or inspect the tower at various locations, which requires leaving the ladder to go to these points. Ladder cages make this operation more hazardous than would be the case with open ladders equipped with fall-arresting devices.*

16.6 Mud grating

At sites that are apt to be muddy, facilities such as steel grating or scraper bars shall be provided near the base of the structure to enable climbers to remove all mud from the soles of their footwear prior to climbing.

16.7 Access through obstructions

Major structural features on towers, such as platforms and antenna mounts, that might form an obstruction to the climber shall be designed to provide safe access through or around the obstruction. Where possible, clearances shall conform to the requirements of Clause 15.

16.8 Retrofitting

16.8.1 Procedure

As part of their maintenance inspection program, existing structures covered by this Standard shall be observed and inspected with regard to fixed ladders, climbing facilities, platforms, hoops and cages, and fall-arresting devices. An engineer shall review those observations and make appropriate recommendations to the owner in order to meet the requirements of this Standard. Except as noted in Clause 16.8.2, those recommendations shall be implemented in accordance with the following schedule:

a) towers undergoing modification shall incorporate the recommendations of the engineer as part of the modification;
b) frequently climbed structures (i.e., those that are accessed at least monthly) shall meet these requirements within two years of the date of the engineer's report; and
c) other structures shall have these requirements implemented within five years of the date of the engineer's report.

Following this schedule shall not relieve the owner of the responsibility to ensure that all personnel climbing the tower follow the requirements of Clause 16.8.2 in the period before the modifications are completed.

Note: *TV and FM antennas pose the greatest danger to personnel having to climb communication structures. Every effort should be made by the owner to fit these antennas with approved climbing or fall-arresting devices.*

16.8.2 Exceptions

16.8.2.1 General

Existing structures and antennas that cannot be equipped with specified fall-arresting devices because it would make the antenna inoperable or create other hazards because of the RF or other electrical power in the vicinity, or because of other justifiable reasons, shall not be required to be retrofitted with climbing and safety devices covered by this Standard.

16.8.2.2 Without fall-arresting devices

Any structure that is not fitted with a fall-arresting device shall only be climbed by qualified climbers, as defined by Clause 16.10, using other certified non-permanent safety equipment and devices, and established procedures.

16.8.2.3 Importance factors

When existing structures designed to earlier editions of this Standard are evaluated, the appropriate importance factor, γ, should be applied.

Note: *Existing structures designed to earlier editions of this Standard might be overloaded when evaluated against this current edition. The addition of fall-arresting devices could add to this overload. However, the application of the appropriate importance factor,* γ*, might alleviate this.*

16.8.2.4 Irregular spacing

Existing structures with climbing facilities that are an integral part of the structure and that have an increased rung spacing or an irregular rung spacing shall be considered acceptable, provided the condition is identified by appropriate warning signs.

16.9 Warning signs

For structures that are not in conformance with the requirements of this Standard, suitable warning signs shall be posted at the base of the tower. In addition, the location of restricted clearances, irregular alignment or rung spacing, and other hazards shall be identified to the climber by warning markers or other suitable means both above and below the location.

16.10 Qualified climbers

For the purposes of this Clause, a qualified person shall be one who is properly trained, experienced, and has demonstrated capability to:

a) perform safely
   i) on similar structures;
   ii) at the heights required;
   iii) with the equipment installed on the structure;
   iv) with his/her personal fall arrest equipment; and
   v) with the equipment or tools to be used while on the structure; and

b) identify and select proper fall arrest anchorage points. Refer to Annex R.

Figure 14
Typical ladder arrangement
(See Clause 16.2.3.)

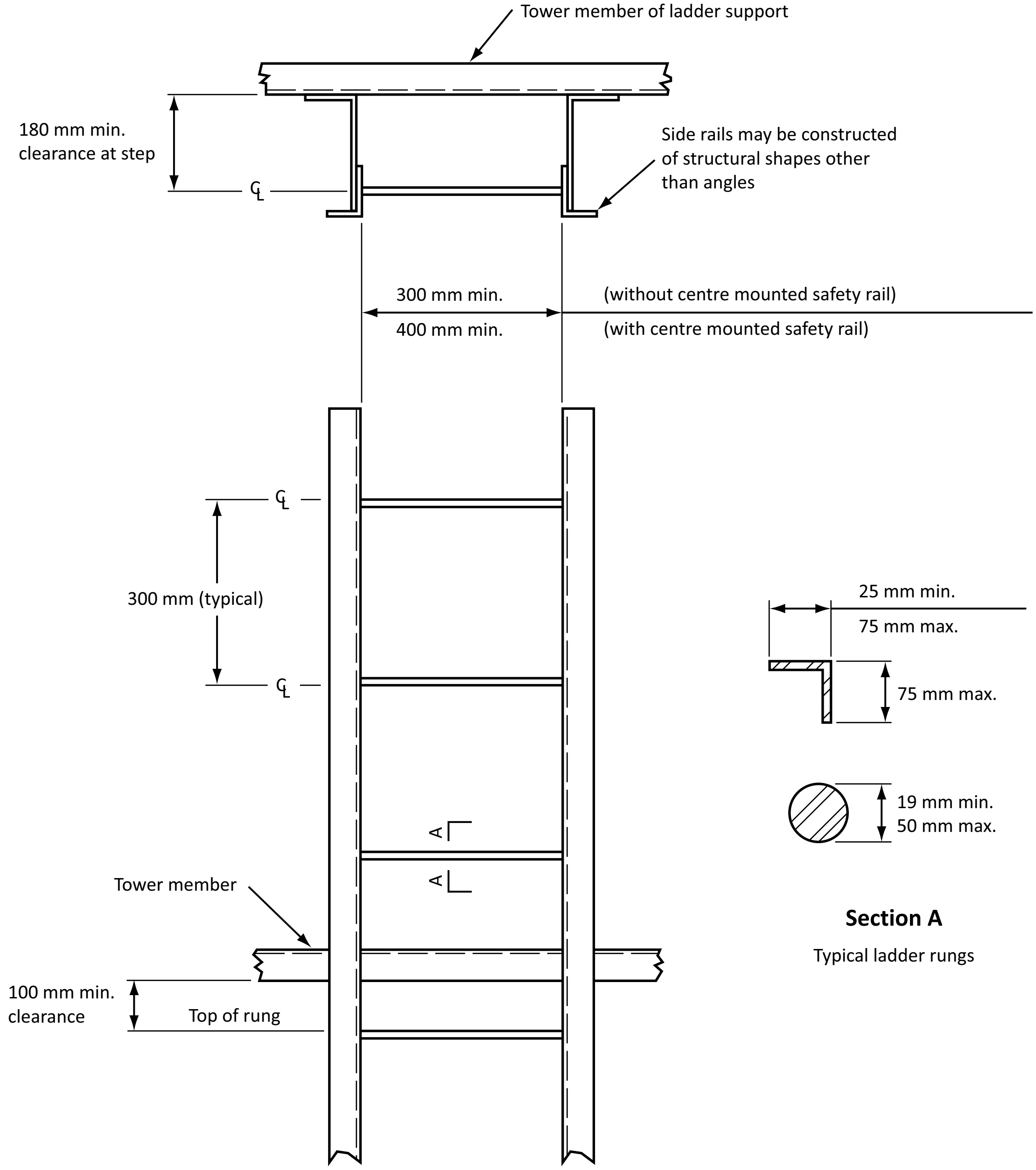
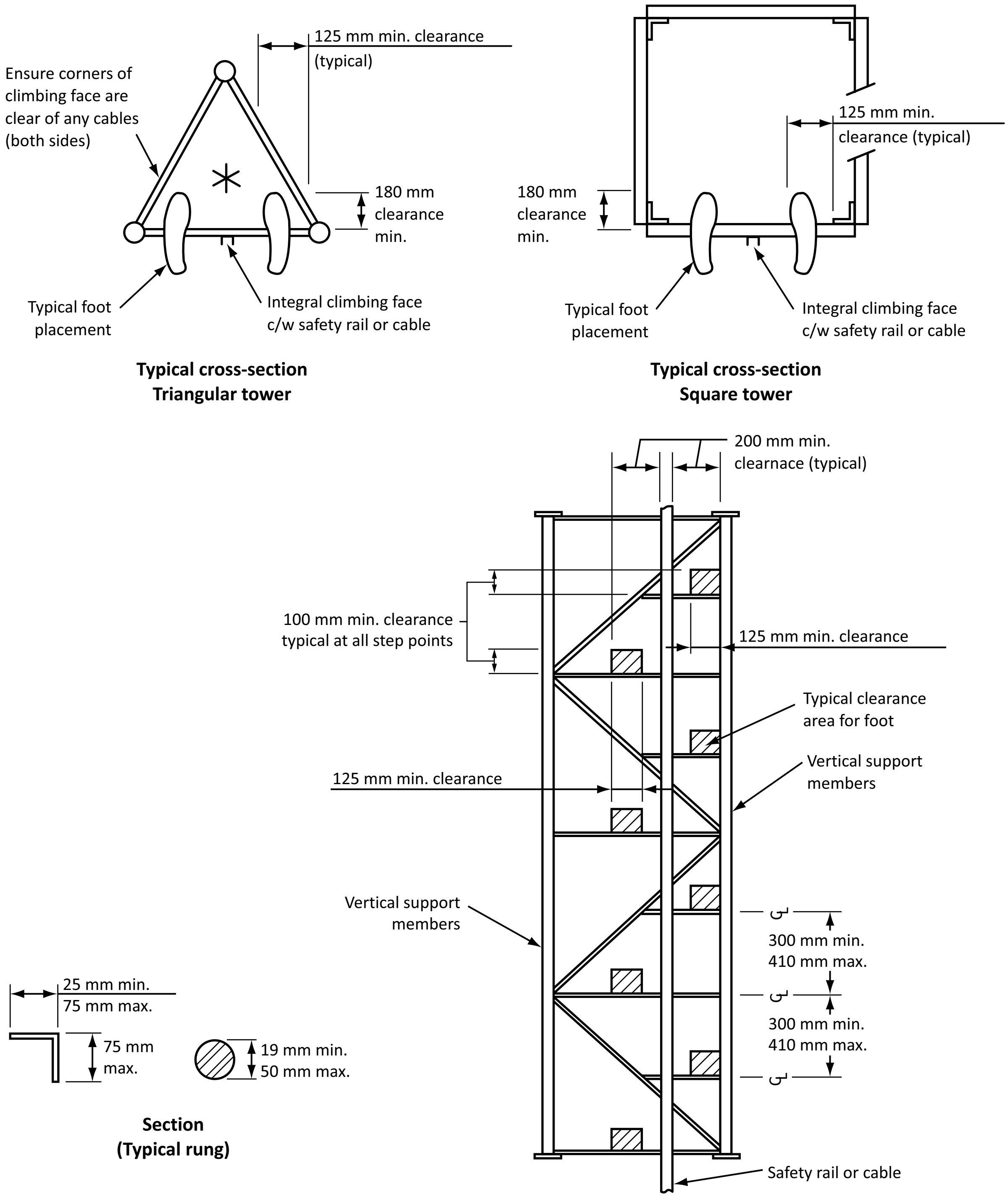
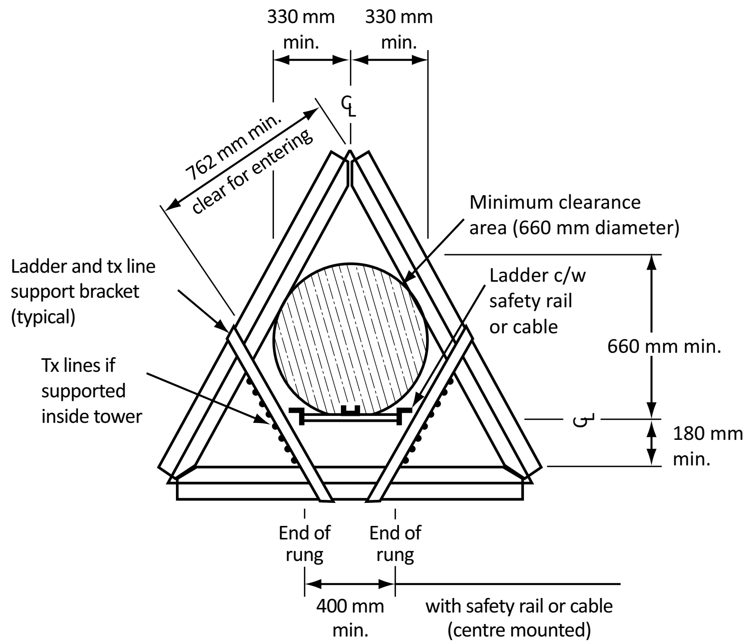


Figure 15
Typical climbing facilities arrangements
(See Clauses 16.2.3.1 and 16.2.3.3.)



Notes:
1) *Rung spacing shall be uniform (± 5 mm) for height of structure unless a warning sign is provided and platforms are installed where changes take place.*
2) *Step bolts and integral climbing facilities may be 16 mm in diameter.*

Figure 16
Minimum face width for internal ladders
(See Clauses 16.2.3.1 and 16.2.4.1.)



Inside face mount ladder

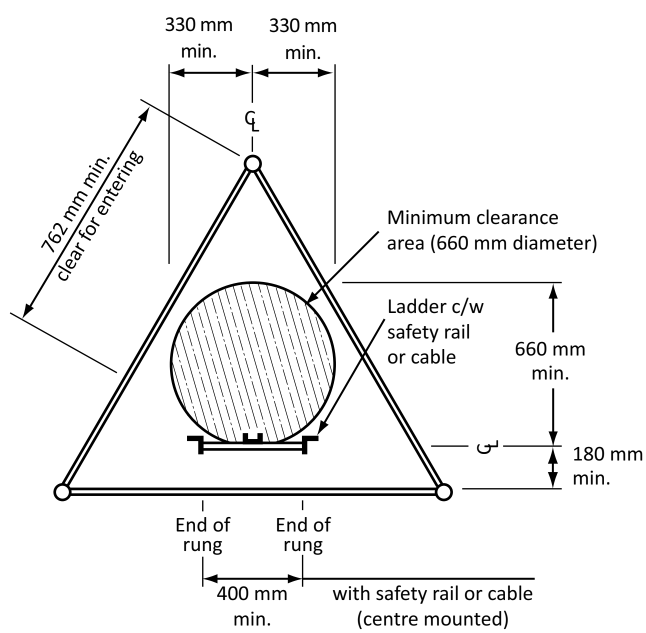
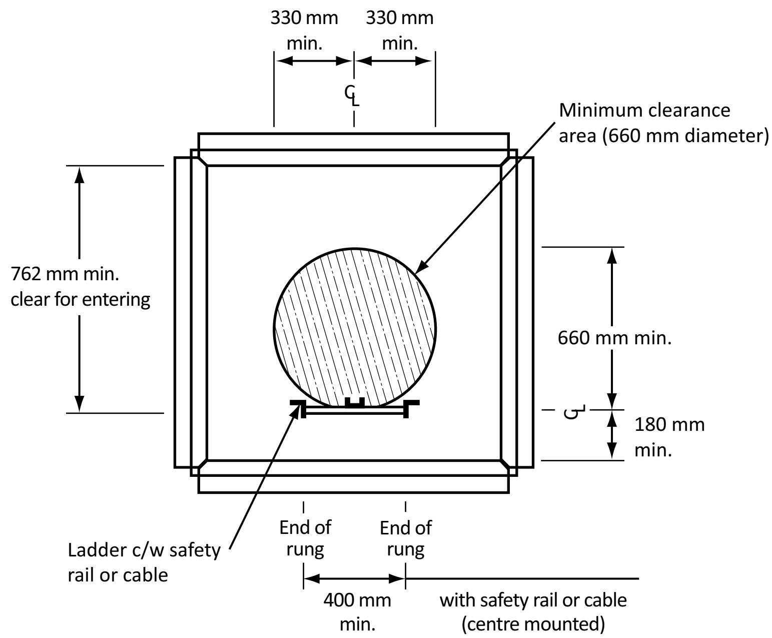


Figure 17
Minimum face width for internal ladders
(See Clauses 16.2.3.1 and 16.2.4.1.)



Inside face mount ladder

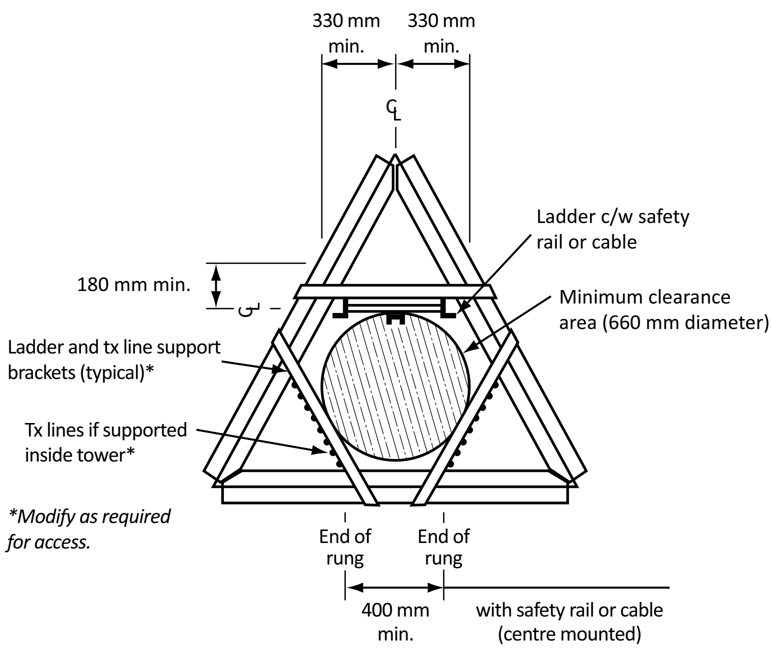


Figure 18
Minimum face width for internal ladders
(See Clauses 16.2.3.1 and 16.2.4.1.)

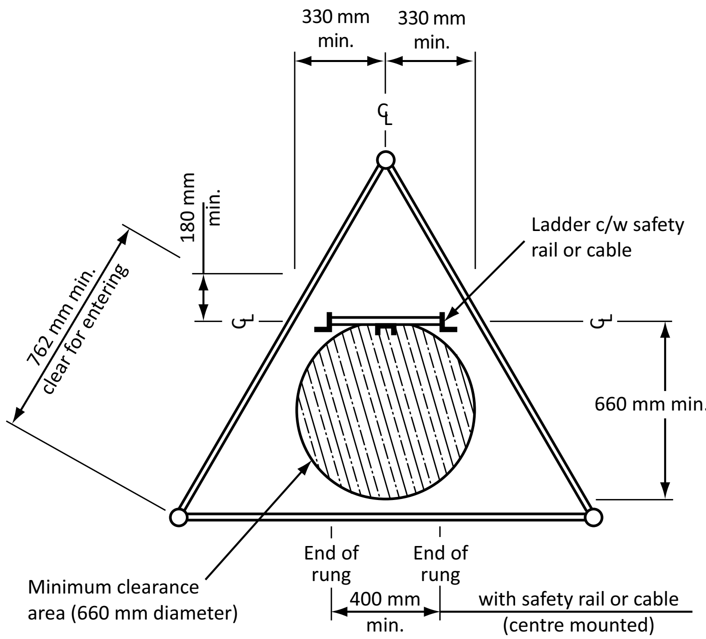
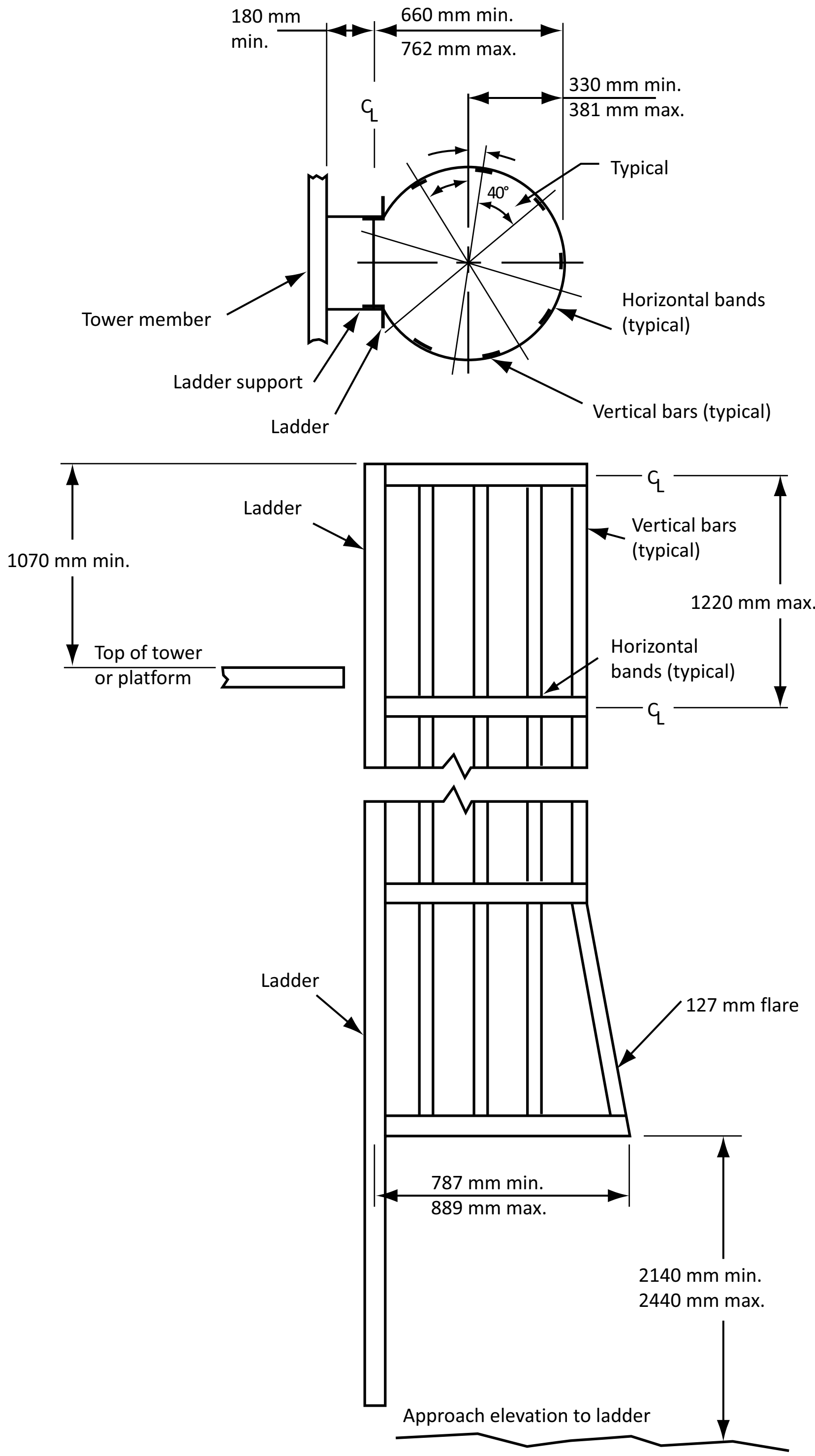


Figure 19
Ladder cages and hoops
(See Clauses 16.2.3.1 and 16.5.3.)



Note: *Modify ladder and cages at platforms as required for access.*

Figure 20
Slope of fixed ladders
(See Clause 16.2.3.1.)

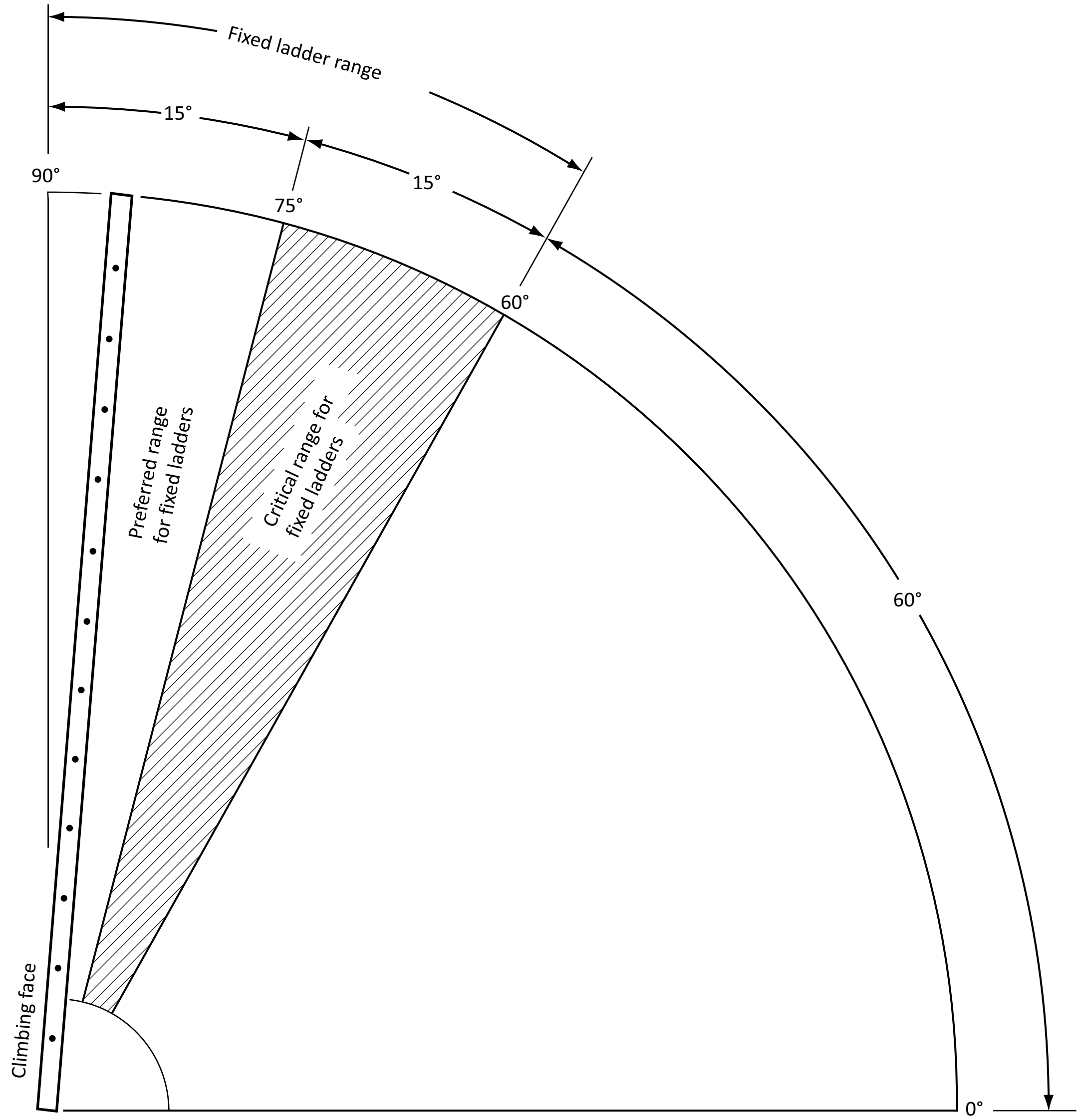
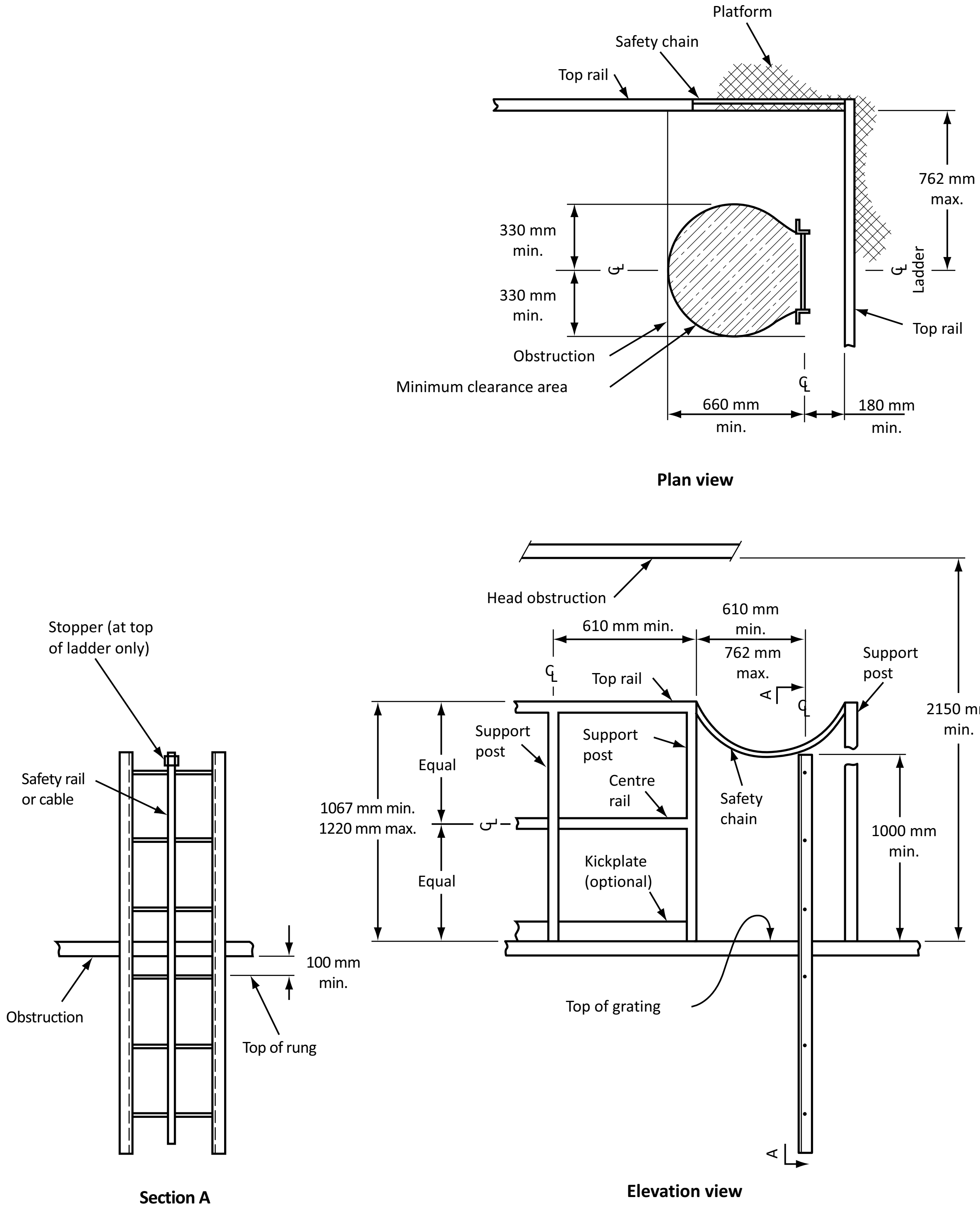


Figure 21
Specific features — Platforms, safety rails, and cables
(See Clauses 16.3.3 and 16.4.3.)



Annex A (informative) ***Recommended information to be shown on drawings***

Note: *This Annex is not a mandatory part of this Standard.*

A.1 Structure profile

For tower profiles, the following information should be shown:

a) overall dimensions (tower height, section height, panel height, tower face width gauge to gauge, etc.);
b) tower member sections and shapes (required for legs, diagonals, horizontals, and secondary members, etc.);
c) guy elevations, anchor radii for flat terrain, guy sizes, and terminations; and
d) visual representation of the tower loading (existing and future antennas and accessories considered in the tower design such as pinwheels, ice-guards, platforms, etc.).

A.2 Design data

For design data, the following information should be shown:

a) governing standard (CAN/CSA-S37) with year or edition;
b) wind pressure (specify return period, source of the wind data, wind pressure profile if different from Clause 5.3);
c) radial ice (specify source of ice data, rime icing values if present, etc.);
d) seismic design data if relevant [peak ground acceleration (PGA) and design spectral acceleration response values, $S(T_n)$, site classification for seismic site response, and site coefficients (F_a and F_v)];
e) steel grades (required for at least the main tower structural elements, bolts, and/or welds);
f) distance to bend line for Schifflerized angles;
g) list of tower design loading (antennas, lines, and other accessories considered in the design including their elevations, azimuths, and assumptions that affect the structural behaviour of the tower, etc.); and
h) anchorage and guy table with anchorage radii, elevations for each azimuth, and guy design initial tensions).

A.3 Cross-section(s)

For cross-section(s), information on the position and size of transmission lines, conduits, ladders, etc., in relation to tower members should be shown.

A.4 Tower details

For tower details, the following information should be shown:

a) tower base (base plate, star base or tapered base dimensions and information, etc.);
b) member connections and splice details (size and number of bolts and/or welds, plates, etc.);
c) antenna mount details;
d) transmission line supports and restrainer details;
e) outriggers and guy attachment plate details;
f) ladders, platforms, and safety device details;
g) anchorage and guy table including anchor radii, elevations for each azimuth, and design guy tensions; and
h) grounding systems and obstruction marking system details.

A.5 Guy assemblies

For guy assemblies, the following information should be shown:

a) description of guys (i.e., diameter, type of strand or rope, breaking strength, initial tension at 10 °C at anchorage, etc.);
b) size and description of all guy assembly hardware;
c) clips: number and spacing;
d) preformed guy grips: diameter and grip lay which must be the same as the guy lay and also the percentage of efficiency if different from 100%;
e) mechanical or pressed sleeves;
f) turnbuckles: diameter and take-up;
g) bridge sockets: size and take-up; and
h) table of guy tensions for temperature intervals between –30 °C and 30 °C in intervals of 10 C°.

A.6 Foundations and guy anchors

For foundations and guy anchorages, the following information should be shown:

a) factored and service design reactions;
b) dimensions of the foundations and guy anchors (specify height, width, depth including the depth below ground level, etc.);
c) concrete strength and reinforcing steel characteristics (specify yield strength, diameter, and spacing, etc.);
d) description of anchorage bolts and embedded steel in the foundations and in the rock (specify grade, strength, diameter, length, and bond characteristics, etc.); and
e) description of soil conditions (rock or soil), including design bearing pressure and design water table depth (specify the references for the soil report, if it exists), and characteristics of backfill material.

Annex B (informative)
Guidelines for clips on bridge and guy strands

Note: *This Annex is not a mandatory part of this Standard.*

Table B.1
Installation specifications for twin-base and U-bolt clips on bridge and guy strand
(See Clause 12.6.)

Guys			Clips		Torque, Nm	
Nominal strand, mm	Diameter, in	Designated strand size	Minimum no. of clips	Spacing of clips centre-to-centre, mm	Twin-base clips	U-bolt clips
6.3	1/4	6	3	40	40	20
8.4	5/16	8	3	50	40	40
9.0	3/8	9	3	60	60	60
10.8	7/16	10	3	75	90	90
12.6	1/2	12	4	80	90	90
14.4	9/16	14	4	85	170	130
16.2	5/8	16	4	90	170	130
18.2	11/16	18	5	115	300	170
20.2	3/4	20	5	115	300	170
22.2	7/8	22	5	130	300	300
24.2	15/16	24	6	140	300	300
26.8	1	26	6	160	300	300
29.0	1-1/8	28	6	170	490	300
32.0	1-1/4	32	6	190	490	490

Annex C (informative)
Measuring guy tensions

Note: *This Annex is not a mandatory part of this Standard.*

C.1 Measuring initial tension

C.1.1 General
There are two basic methods for measuring initial guy tensions in the field: the direct method and the indirect method. These methods assume that the measurements are taken on a calm day.

C.1.2 Direct method
A dynamometer (load cell) with a length adjustment device such as a come-along is attached to the guy system by clamping onto the guy just above the turnbuckle and onto the anchor shaft below the turnbuckle, thus making the turnbuckle redundant (see Figure C.1).

The come-along is then tightened until all the load is taken off the turnbuckle. At this point, the dynamometer carries all of the guy load to the anchor and the guy tension may be read directly off the dynamometer dial.

To set the correct tension, adjust the come-along until the proper tension is read on the dynamometer. Two control points are marked, one above the clamping point on the guy and one on the anchor shaft, and the control length is measured. The dynamometer and come-along are then removed, and the original turnbuckle is adjusted to maintain the control length previously measured.

Other clamping devices incorporating load cells and allowing the bypass of the linking pin may be used for direct measurements provided they are properly calibrated.

C.1.3 The indirect methods

C.1.3.1 General
There are three common techniques for the indirect measurement of initial guy tensions: the pulse, the swing, and the tangent intercept or sag technique. An alternative sag technique is possible, but requires considerably more work in acquiring the data and in calculating the corresponding guy tension. This is a form fitting procedure, which requires accurate surveying and precise calculations using hyperbolic functions.

C.1.3.2 Pulse technique
This technique relies on starting a pulse (wave) that travels up and down the length of the guy cable a convenient number of times, *n*. The time it takes for the pulse to go up and down the cable *n* times is measured with a stopwatch to establish its speed. The speed of the pulse is a function of the average mass and tension of the cable.

The governing equation is

$$T = \frac{4ML_c{}^2n^2}{p^2}$$

where
T = average guy tension, N
M = mass of guy cable, kg/m
L_c = guy chord length, m
n = number of times the pulse has traveled up and down the cable
P = time measured while counting n returns of the pulse, s

A convenient value should be chosen for n, keeping in mind that one's response time in starting and stopping the watch and the error in measuring time are constant for any number of vibrations. The greater the number of vibrations timed, the more accurate the result is likely to be.

In practice, time, P, is known for n vibrations for the average guy tension, T. The tower inspector measures the time in order to determine the guy tension at a given temperature.

C.1.3.3 Swing technique

This technique relies on causing the guy cable to swing like a pendulum about the line joining its two end connections a convenient number of times, n. The time it takes for the cable to swing back and forth n times is measured with a stopwatch to establish its period of swing. The period of swing is a function of the length of the pendulum, which is a function of the tension of the cable.

The governing equation is

$$T = \frac{3.24ML_c^{\,2}n^2}{P^2}$$

where
T = average guy tension, N
M = mass of guy cable, kg/m
L_c = guy chord length, m
n = number of complete swings of the cable
P = time measured while counting n swings, s

A convenient value should be chosen for n, keeping in mind that one's response time in starting and stopping the watch and the error in measuring time are constant for any number of pulses or swings. The greater the number of swings timed, the more accurate is the result likely to be.

In practice, time, P, is known for n swings for the average guy tension, T. The tower inspector measures the time in order to determine the guy tension at a given temperature.

C.1.3.4 Formula to convert average guy tension to guy tension at anchorage

The pulse and swing techniques give the average guy tension, T, while the direct and tangent intercept techniques give the guy tension, T_a, at the anchorage. The average guy tension is converted to the tension at the anchorage through the formula

$$T_a = \sqrt{\left(T - 4.9V_gM\right)^2 + \left(4.9H_gM\right)^2}$$

where

T_a = guy tension at the anchorage, N

V_g = vertical projection of guy length, m

M = mass of guy cable, kg/m

H_g = horizontal projection of guy length, m

C.1.3.5 Tangent intercept technique

This technique assumes that the guy is suspended as a parabola and that the weight of the guy is uniformly distributed along the chord (see Figure C.2).

A line of sight is established that is tangential to the guy cable near the anchor end and that intersects the tower leg a distance D (tangent intercept) below the guy attachment point on the mast. This tangent intercept distance is either measured or estimated and the tension, T_a, is calculated from the following equation:

$$T_a = \frac{4.9ML_cH_g}{D\cos\alpha}$$

$$\cos\alpha = \frac{H_g}{\sqrt{\left(H_g^2 + \left(V_g - D\right)^2\right)}}$$

where

D = tangent intercept, m

C.1.3.6 Form fitting technique

This technique assumes that the guy is suspended as a catenary and that the inspector is able to get precise locations of at least three well separated points on the guy cable.

The inspector establishes accurate positions in space of at least three well-separated points on the guy cable as well as the two anchorage points. Using the basic characteristics of the guy cable and the locations of the two anchorage points, the closest fitting geometry of the catenary is calculated. In the process, the corresponding tension that would yield the measured geometry is also calculated.

This is too complicated a process for most applications, but is listed here because it does have some advocates and may be considered where the required accuracy justifies all the effort required.

C.2 Calculating guy tension for a change in temperature

When guy tension tables are not readily available, it might be necessary to calculate the change in guy tensions due to the difference between the ambient temperature at the time of measurement and that assumed for setting the design initial tension. The inspector may use the following formula for such calculations:

$$T_a' = T_a - \frac{\Delta t\left(\alpha_g L_c - \frac{\alpha_m V_m V_g}{L_c}\right)}{\left(\frac{L_c}{A_g E_g} + \frac{V_g^2 V_m}{L_c^2 E_m A_m}\right)}$$

where

T_a' = tension at ambient temperature, N

Δt = change in temperature, °C

α_m = thermal expansion coefficient of tower

L_c = guy chord length, m

α_g = thermal expansion coefficient of guy

V_m = height of tower at guy attachment, m

V_g = vertical projection of guy length, m

A_g = cross-sectional area of guy, mm^2

E_g = elastic modulus of guy

E_m = elastic modulus of tower

A_m = cross-sectional area of tower leg, mm^2

This formula is based on the following assumptions:
a) all guys at each guy level have the same lengths and slopes;
b) there is only one guy per leg;
c) what happens between the tower and guys other than those at the elevation being considered does not matter;
d) there is no interaction between guy levels;
e) the effects of temperature are linear both in the guys and in the tower;
f) there is restraint against displacement in the horizontal direction at the tops of the guys; and
g) the guy sag may be ignored for this calculation.

Although it is possible to get more accurate results by using an appropriate computer program, for the majority of small towers and small changes in temperature, the formula given above should suffice.

C.3 Guys consisting of segments of different size and/or material

When guys consist of several segments of different size or type of cable, an equivalent cable may be calculated such that its properties of cross-sectional area, elastic modulus, and coefficient of thermal expansion will yield the same response to applied load and temperature change as would the composite cable itself.

A guy cable composed of two lengths, L_1 and L_2, having different characteristics of cross-sectional area, A_1 and A_2, elastic modulus, E_1 and E_2, weight, W_1 and W_2, diameter, D_1 and D_2, and coefficient of thermal expansion, ε_1 and ε_2, may be replaced by a single length of cable of length $L = L_1 + L_2$ where the diameter is $(D_1L_1 + D_2L_2)/L$, the weight is $(W_1L_1 + W_2L_2)/L$, the coefficient of thermal expansion is $(\varepsilon_1L_1 + \varepsilon_1L_2)/L$ and the material stiffness $AE = (LA_1 E_1 A_2 E_2)/(L_1 A_2 E_2 + L_2 A_1 E_1)$.

Figure C.1
Measuring guy tension by dynamometer and vibration methods
(See Clause C.1.2.)

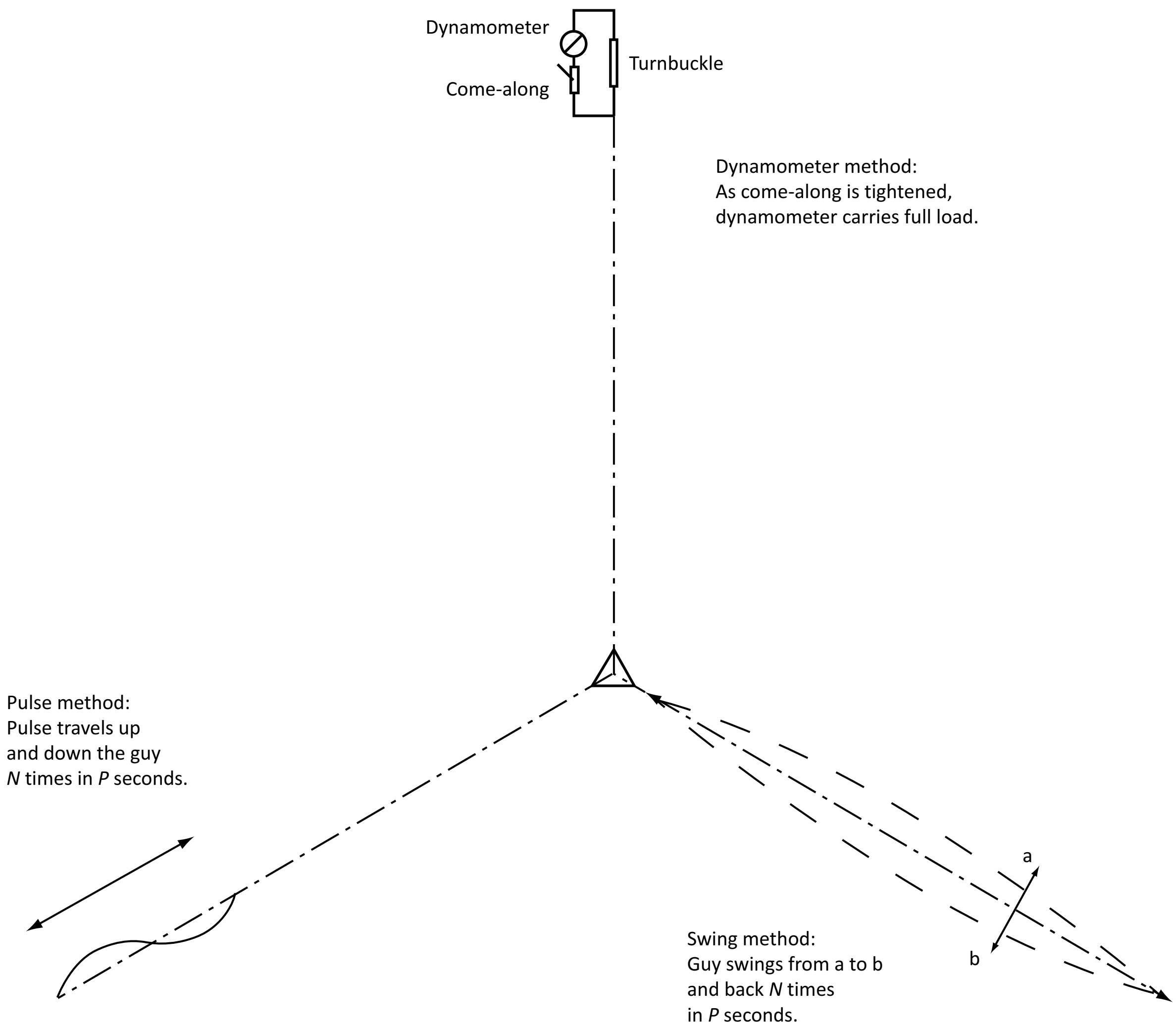
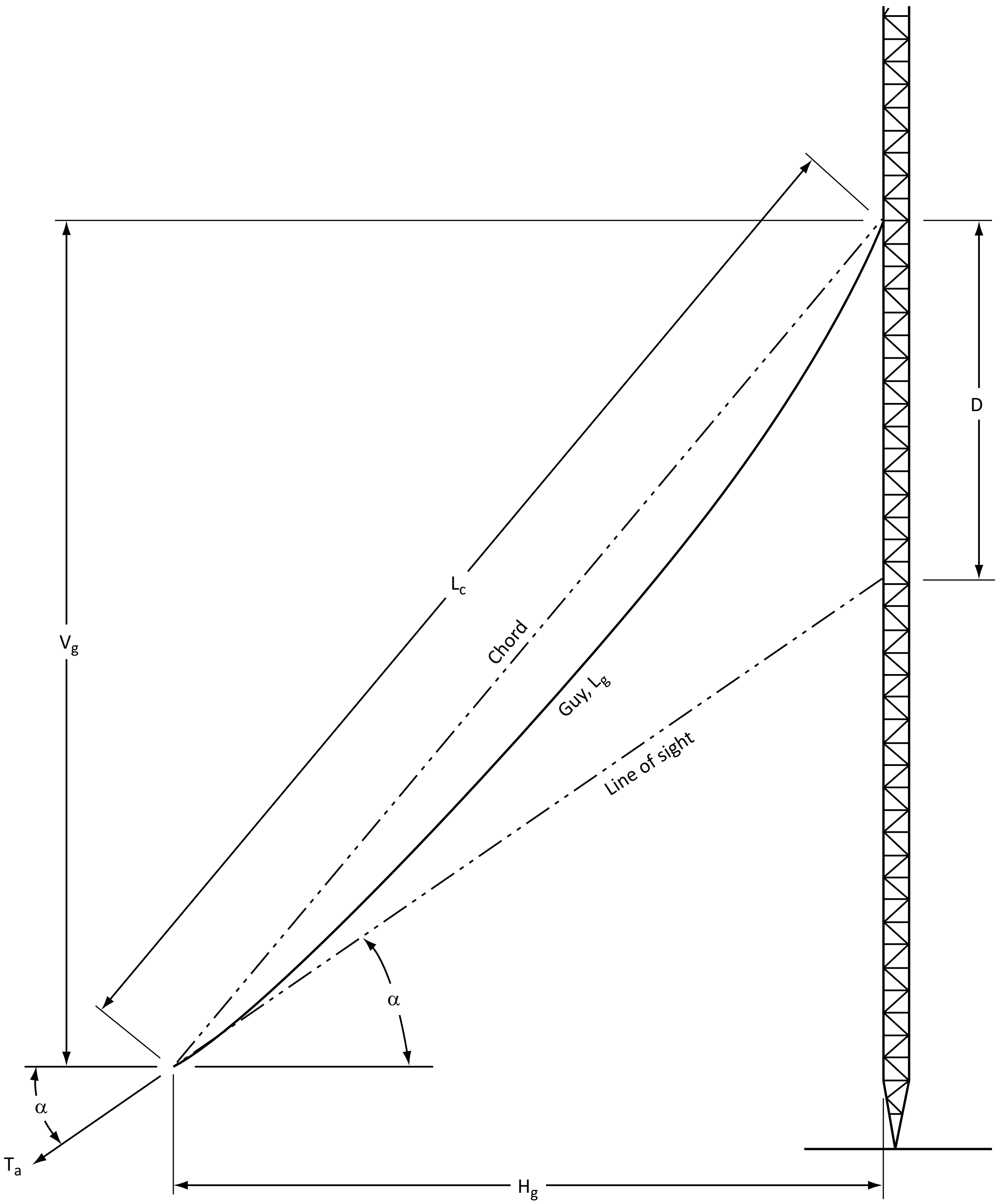


Figure C.2
Tangent intercept method
(See Clause C.1.3.5.)



Annex D (informative) ***Recommendations for condition assessment***

Note: *This Annex is not a mandatory part of this Standard.*

D.1 General

All antenna towers and antenna-supporting structures should be given a detailed inspection by an engineer or a qualified technician during critical stages during initial construction, regularly scheduled intervals not exceeding four years for guyed structures, monopoles and shrouded tripoles; five years for rooftops; six years for self-supporting structures; and after any modifications to the existing structure. The inspector should submit a written report to the owner, who should have the inspector's recommendations implemented.

D.2 Initial construction inspection

D.2.1 Sub-grade inspection

An inspection should be completed by the engineer of record and client or client representative to ensure all sub-grade installations are in accordance with design drawings and/or client specifications. It is critical to inspect such features as the foundations, grounding, cathodic protection, or other buried components for compliance. The following are some of the important things to note:

a) parameters that require verification in accordance with the recommendations of the geotechnical report for the site or as noted in the design documents;
b) foundation dimensions, placement, and orientations (i.e., horizontal azimuths);
c) rock anchor installation (performance/pull test results);
d) reinforcing steel grade, size, condition, support, placement, and cover;
e) concrete mix design documentation matches strength, durability requirements, etc.;
f) concrete tests required to be performed prior to placement of concrete (i.e., slump, temperature, air content, test cylinder);
g) anchorage dimensions and placement (i.e., size, embedment depth, projection above concrete, orientation, pattern, alignment);
h) condition of subgrade immediately prior to concrete placement;
i) proper concrete placement (i.e., avoid segregation of aggregates) and curing;
j) grounding material, placement, and required resistance tests;
k) structural backfill material and placement (i.e., maximum lift thickness, moisture content and density); and
l) installation of the cathodic protection and initial electrical measurements to ensure proper installation and design.

D.2.2 Post-installation inspection

A detailed inspection (refer to Clause D.3) should be completed once the new tower is constructed. It is critical to assess whether there are deviations from the original design drawings or client specifications.

D.3 Detailed inspection

All antenna towers and antenna-supporting structures should be given a detailed inspection by an engineer or a qualified technician at intervals not exceeding four years for guyed structures; five years for rooftop, monopoles, and shrouded tripoles; and six years for self-supporting structures. The inspector should submit a written report to the owner, who should have the inspector's recommendations implemented. The inspection should include the following:

a) Verticality, straightness (alignment), and twist — Measure by transit, as shown in Figures D.1 and D.2, apparent displacements of legs at guy levels and other elevations of interest. Three lines of sight are required for triangular towers and four for square towers.
   The verticality, straightness, and twist can be calculated using the formulas in Figures D.1 and D.2.
b) Guy tension — For guy tensions, if the drawings are not available or if the tensions are not noted on the drawings, take the specified initial tensions as 10% of the breaking strength of the guy at an ambient temperature of 10 °C south of latitude 55° N, and 0 °C north of latitude 55 °N. All guy tension measurements should be adjusted for the temperature at the time of making the measurements.
c) Twist — For tower twist, take measurements from three directions for triangular towers and from four directions for square towers.
d) Visual examination of tower members and connections — Check for bent, fractured, or missing members, cracks in welds, and bolts that are loose, missing, or short.
e) Visual examination of ladders, gratings, and handrails — Check for fractured members or welds and loose or missing supports.
f) Visual examination of guy assemblies — Check for
   i) broken strand wires;
   ii) slippage of guy grips, clips, etc;
   iii) loose, worn, cracked, bent, or missing hardware;
   iv) articulation at guy ends and, in particular, at turnbuckles;
   v) turnbuckle adjustment available;
   vi) corrosion in guy cables and hardware; and
   vii) mechanical and compression connectors.
g) Foundations — Check concrete and grout for movement, cracking, spalling, deterioration, etc. Check visible steel of anchor shafts, anchor shaft vertical azimuth, guy anchor plates, base plates, and anchor bolts for bending, fractures, etc. Check for adequate drainage and backfill, and check base plates for full bearing.
   Note: *See Annex F for comments on below-grade inspections and the need for corrosion protection.*
h) Antennas and antenna mountings — Check all visible components of the antennas and mountings for damage, e.g., loose or missing bolts, cracked welds, bent, fractured, or missing members, etc. Particular attention should be given to U-bolt assemblies supporting cantilevered pinwheels at top of tower.
i) Transmission lines — Check for fractured, loose, or missing supports and connections; missing protective sleeves under wraplock connections, dents, or damage in the lines; and lines rubbing against structural members, etc.
j) Grounding — Check lightning rod and ground wires on the tower base, guys, transmission lines, and waveguide bridge for electrical continuity and loose or missing connections.
k) Insulators — Check all insulators in guy assemblies and tower bases for cracks, flaws, chipping, leaking, etc., where applicable.
l) Electrical installation — Check for kinks, loose or missing supports, loose or missing junction box covers and screws, blocked drainage holes, deterioration or cracks, etc. Check lamp fixtures for missing, loose, or broken hardware and fittings, broken globes, burned-out bulbs, etc. Check that obstruction lights are in accordance with Transport Canada requirements. (See Clause 13).
m) Painting — Check that painting is in accordance with Transport Canada specifications with regard to length and positioning of colour bands, and that colours are correct. Check paint for peeling, blistering, flaking, fading, oxidation, and ensure that the pattern is still visually effective.

n) Galvanizing and other anti-corrosion treatment — Check all galvanized components of the tower (i.e., tower members, connections, ladders, guys and guy hardware, base plates and anchor bolts, waveguide bridge, conduit, etc.) for scratching or scoring, flaking, rusting, peeling, blistering, etc., of the galvanized surfaces. Check all non-galvanized metal surfaces (e.g., antenna and transmission line connections) for corrosion or other surface deterioration. Refer to Annex F.

D.4 Modification inspections

D.4.1 General

All antenna towers and antenna-supporting structures should be inspected by an engineer or a qualified technician when there are modifications to existing structures. The inspection should contain all information that would be critical to the existing and future modifications.

D.4.2 Pre-construction

The inspection should include the following:

a) welding data — confirmation of competency of the welder and welding procedures are in accordance with CWB recommendations; and
b) material data — mill certifications for all structural steel. These certifications should detail all critical information that confirms the grade, strength, and composition of the material.

D.4.3 Post construction

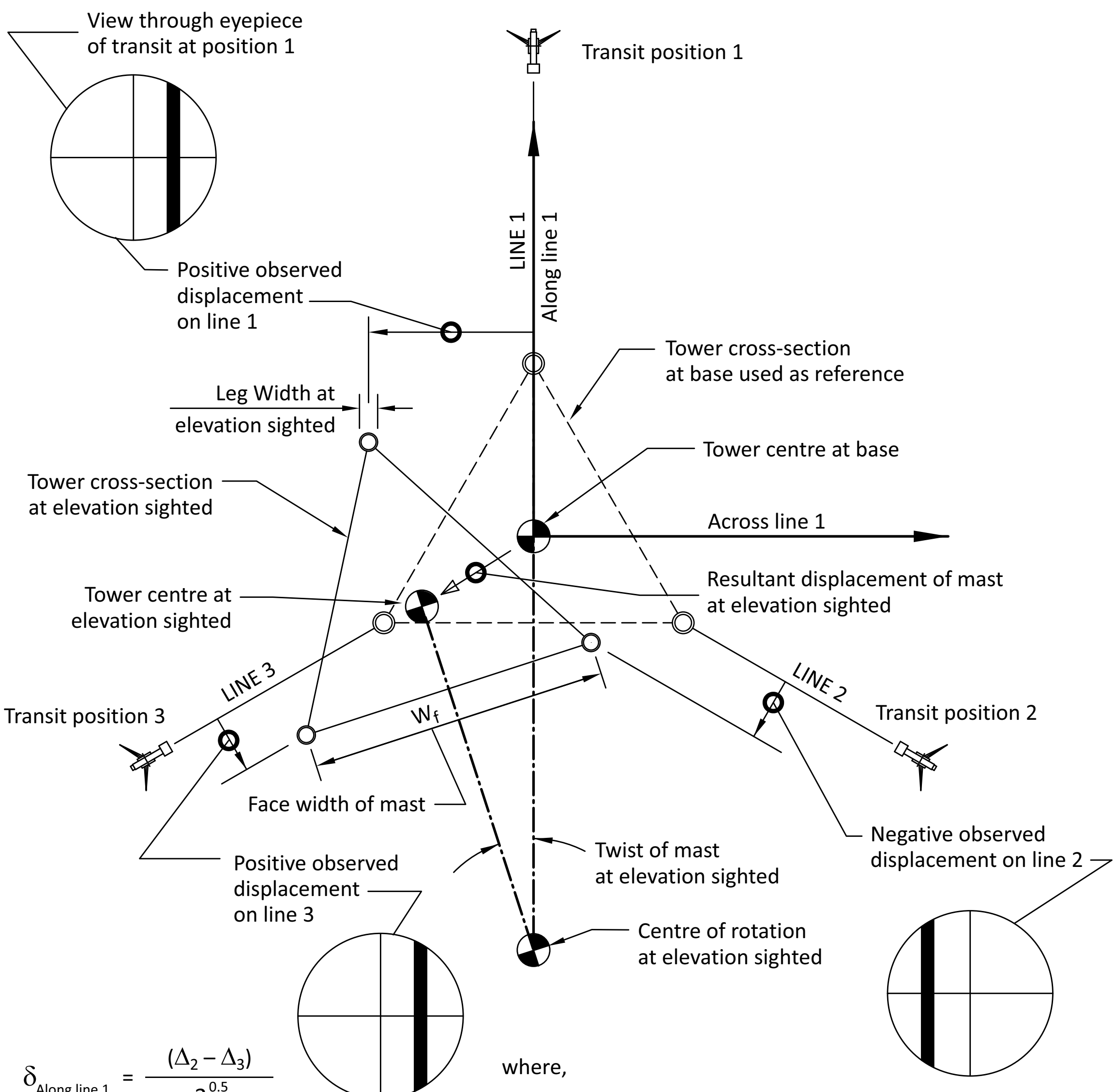
The inspection should be completed to verify the installation is in accordance with the design drawings and/or client specifications.

Note: *Refer to Clause D.2.*

D.5 Inspection report

A report should be submitted based on the above observations and measurements. The report should comment on materials and quality of work that do not comply with the requirements of this Standard. The report should include a brief description of the tower, the date of inspection, weather conditions, and the name of the inspector.

Figure D.1
Measuring tower alignment and verticality — Triangular tower
(See Clause D.3.)



$$\delta_{\text{Along line 1}} = \frac{(\Delta_2 - \Delta_3)}{3^{0.5}}$$

$$\delta_{\text{Across line 1}} = \frac{(\Delta_2 + \Delta_3 - 2\Delta_1)}{3}$$

$$\delta_{\text{Total}} = (\delta^2_{\text{Along line 1}} + \delta^2_{\text{Across line 1}})^{0.5}$$

$$\text{Twist, } \Theta = \arcsin\left[\frac{(\Delta_1 + \Delta_2 + \Delta_3)}{3^{0.5}\, W_f}\right]$$

where,

Δ_n = observed displacement of leg across line "n";

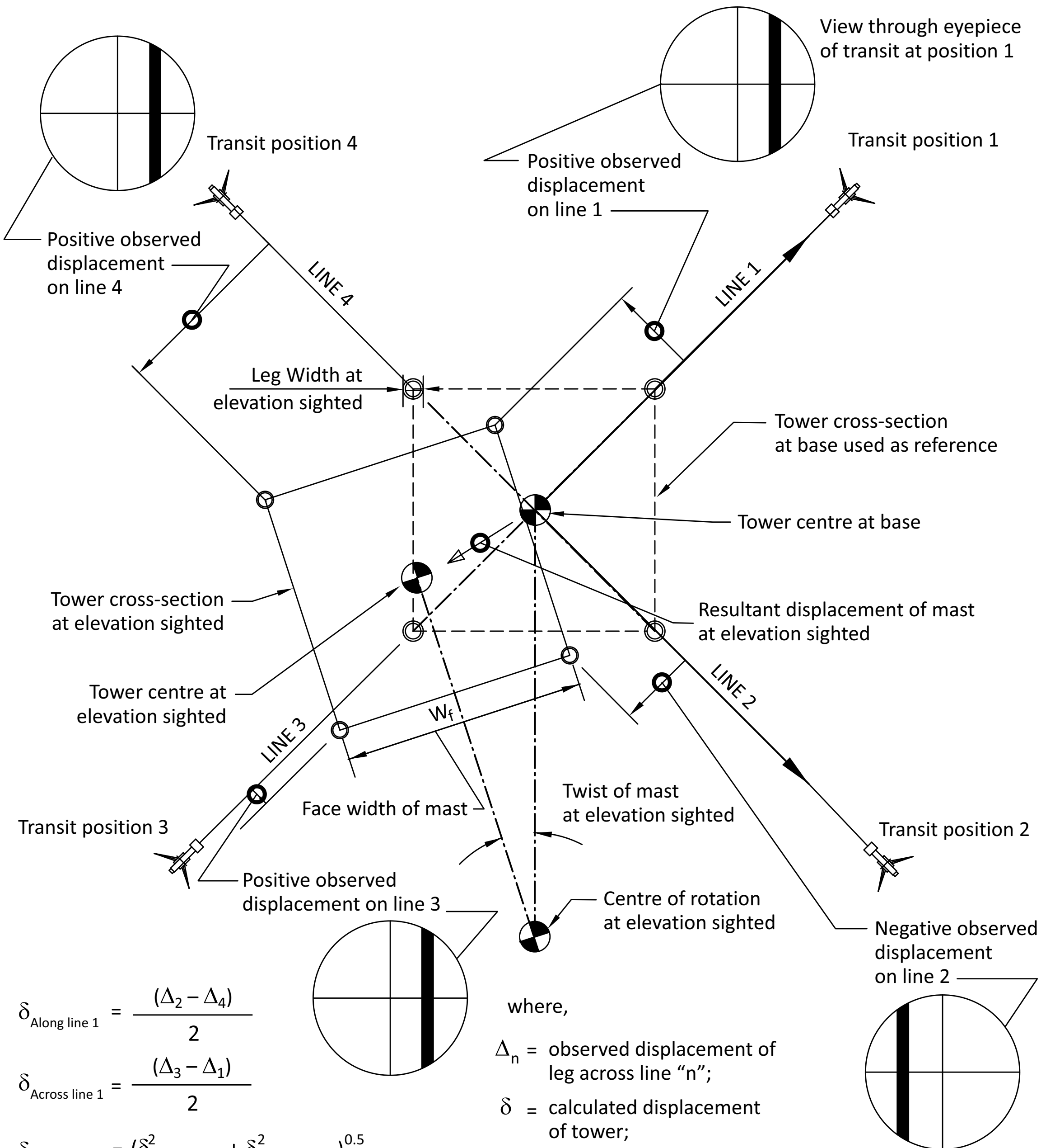
δ = calculated displacement of tower;

Θ = calculated twist;

W_f = face width of tower at elevation sighted

Figure D.2
Measuring tower alignment and verticality — Square tower

(See Clause D.3.)



$$\delta_{\text{Along line 1}} = \frac{(\Delta_2 - \Delta_4)}{2}$$

$$\delta_{\text{Across line 1}} = \frac{(\Delta_3 - \Delta_1)}{2}$$

$$\delta_{\text{Total}} = (\delta^2_{\text{Along line 1}} + \delta^2_{\text{Across line 1}})^{0.5}$$

$$\text{Twist, } \Theta = \arcsin\left[\frac{(\Delta_1 + \Delta_2 + \Delta_3 + \Delta_4)}{2 \times 2^{0.5} W_f}\right]$$

where,

Δ_n = observed displacement of leg across line “n”;

δ = calculated displacement of tower;

Θ = calculated twist;

W_f = face width of tower at elevation sighted

Annex E (informative) ***Supplementary meteorological information***

Note: *This Annex is not a mandatory part of this Standard.*

E.1 Ice

There are three major processes leading to significant ice accretion on structures: glaze icing, rime icing, and freezing wet snow. Ice and snow have their greatest impact on guyed towers, since they can considerably increase guy weight and consequently guy tensions, even without the presence of wind.

Glaze icing occurs when liquid precipitation (rain or drizzle) falls through a layer of air based at the ground whose temperature is below 0 °C. The precipitation freezes on impact with vegetation, structures, objects, or the ground. Glaze ice is usually clear with a density of 900 kg/m^3. It occurs everywhere in Canada. Mainland coastal inlets and valleys in British Columbia will experience local glaze icing that is thicker than that indicated on Figure 1. Values greater than 25 mm should be used for certain areas in British Columbia such as the areas near Terrace and Stewart, and from Abbotsford to Hope.

Experience indicates that glaze ice due to freezing drizzle can be expected at thicknesses greater than 50 mm on exposed windward coastal slopes in eastern Newfoundland and Labrador.

Rime icing occurs when vegetation, structures, and objects are exposed to cloud or fog droplets whose temperature is below 0 °C. It usually is only significant in exposed hill, mountain, or ridge-top locations or near the top of their windward slopes, especially in cold maritime areas such as coastal British Columbia, eastern Québec, or Atlantic Canada. In these areas, extreme rime ice exceeding 1 m in thickness and lasting for several days or weeks during the cold season can occur, although it is normally restricted to the highest and most exposed locations such as mountain tops exceeding 1000 m in elevation and near large sources of open water. Rime ice amounts ranging from a few millimetres to a few centimetres can occur anywhere in Canada, even on level ground, when fog occurs during subfreezing conditions. Rime ice generally grows into the direction of the wind.

Soft rime ice usually has a feathery, opaque appearance, and can have a density below 100 kg/m^3. Hard rime ice is milky and opaque with density typically in the range of 400 to 700 kg/m^3, depending on the microphysical conditions during growth (temperature, wind speed, liquid water content, and size of the cloud droplets).

It is essential to note that extremes of rime ice values greater than those indicated in Figure 1 occur on hill, mountain, and ridge tops, especially in Western and Atlantic Canada. Local experience and knowledge may be used in determining design icing amounts.

Rime icing, resulting in a greater ice thickness than indicated in Figure 1, can also be experienced when the tops of very tall towers (especially those higher than 200 m) are in cloud due to a relatively low cloud base during the cold season.

Freezing wet snow occurs when snow falls through air having a temperature above 0 °C and adheres to a surface. After the air temperature falls below 0 °C, the snow freezes to the surface, structure, or object where it has adhered. Amounts can be significant (up to a few centimetres or more) and it can occur anywhere in Canada. The density of frozen wet snow ranges from 500 to 900 kg/m^3.

E.2 Wind

E.2.1 Reference wind speed

The reference wind speed is obtained by thorough assessment of the wind climatology of the region. It is provided by reference to Table E.1 of this Standard or Table C-2 in Appendix C, Division B of the *National Building Code of Canada*. Alternatively, historical wind data for extended periods in nearby locations, and onsite wind measurements whenever available, may be processed for the determination of reference wind speed using an appropriate extreme statistical method. The extended wind data must be quality controlled for validity and completeness and adjusted for averaging time, anemometer height, and terrain exposure of the anemometer.

E.2.2 Dynamic wind pressure

Dynamic wind pressure due to the speed of the wind is calculated from the following general equation:

$$p = \frac{1}{2}\rho v^2$$

Equation E-1

where

p = pressure, Pa

ρ = air density, kg/m^3

v = wind speed, m/s

Figure E.1 provides 50-year return period values of hourly mean wind pressure (reference velocity pressure) representative of winds 10 m above ground, over flat, open terrain. Wind pressure values were calculated from 50-year return period values of hourly mean wind speed on flat, open terrain, 10 m above ground, using ρ = 1.29 kg/m^3 (0 °C, standard pressure).

Clause 5.3.2.1 requires that consideration be given to local topography or other conditions, in determining the value of the reference velocity pressure. Clause E.2.3 is a general description of cases in which terrain features interact with the wind flow to produce significant changes in wind speed.

E.2.3 Local variations of wind speed

Wind accelerates over small-scale features (smaller than about 10 km in horizontal scale) such as hills, ridges, and escarpments. Speed-up due to such terrain can increase wind speed by as much as a factor of 2. The largest increases are usually very localized, being restricted to the tops of such terrain features and can disappear within a range of about 50 m to a few hundred metres from the top of the terrain feature. Procedures to account for increases in wind speed due to terrain are available in the *Supplement to the National Building Code of Canada User's Guide — NBCC 2015 Structural Commentaries (Part 4 of Division B)*.

Dynamic interactions between the wind flow and larger terrain features (such as constrictions in broad valleys, the outflow exit of mountain passes, exposed coastal headlands, and smooth lee slopes of ranges of hills, mountains, and plateaus) can result in increases in the basic wind speed over areas extending several kilometres. Advice from meteorological experts, tower maintenance or operations personnel, or others who have relevant local knowledge can aid in identifying and accounting for such effects.

E.3 Combined ice and wind loads

Clause 6.3 specifies factored load calculations for a variety of design cases. The load combination factor, ψ, is specified as 0.5 when ice and wind act together. This is equivalent to using half of the reference velocity pressure when calculating wind loads on an iced tower and reflects the low probability of experiencing extreme wind pressure and ice thickness at the same time.

In locations where extreme rime ice is experienced at higher elevations on mountain, hill, and ridge tops, as noted in Clause E.1, ice can accumulate throughout the cold season, resulting in a much greater probability of experiencing extreme wind and ice conditions at the same time. An increased value of ψ is recommended in these cases. A value as high as 1.0 will be appropriate for locations most prone to significant rime icing conditions, since the extreme wind pressure can occur at the same time as the fully iced condition.

E.4 Serviceability factor

Clause 6.4 provides for the application of a factored wind pressure in the design of a tower for the serviceability limit state for signal transmission in a telecommunications system. This Clause describes the development of the serviceability factor, τ.

Information on the annual wind speed frequency distribution is required to ensure that the loss of signal transmission due to tower and antenna twist and deflection under wind load does not exceed specified criteria. For instance, if the loss of signal is allowable for an average of 2 h per year, then the annual frequency of occurrence of the wind pressure used for design of the tower twist and deflection should not exceed 2 h per year. Knowledge of the annual wind speed frequency distribution is required at any location where there is a need to design to a serviceability limit. A simple and convenient method is used to relate the 10-year return period wind pressure (the reference velocity pressure) to the annual wind speed frequency distribution, as a result of a study conducted by Environment Canada. A description of this method follows.

The Rayleigh cumulative frequency distribution, which is commonly used to describe the wind speed frequency distribution, is determined by the following formula:

Equation E-2

$$F\left(v\right) = e^{-\left(\frac{v}{c}\right)^2}$$

where

v = the wind speed in suitable units

$F(v)$ = the probability of occurrence (fraction of time) during which the wind speed is v or higher

c = the scale parameter in units of wind speed

Then, n, the number of hours during which the wind speed is equal to or greater than v, is determined by the following formula:

Equation E-3

$$n = 8766e^{-\left(\frac{v}{c}\right)^2}$$

where 8766 represents the number of hours in a year.

A value for c, the scale parameter, is required in order to make use of the formula. However, the 10-year return period wind speed, v_{10}, is known. Although not strictly accurate, the probability of occurrence of v_{10} can be approximated by setting n = 1/10, yielding the following formula:

Equation E-4

$$c^2 = \frac{v_{10}^2}{11.4}$$

Substitution of c^2 into Equation E-2 yields the following formula for n in terms of v_{10}:

Equation E-5

$$n = 8766e^{-11.4\left(\frac{v^2}{v_{10}^2}\right)}$$

or, after inverting

Equation E-6

$$\frac{v^2}{v_{10}^2} = \frac{1}{11.4} ln\left(\frac{8766}{n}\right)$$

Equation E-6 describes the frequency distribution of hourly mean wind speeds. However, serviceability limit applications are often concerned with wind speeds that only occur on average a few minutes each year. Thus a relationship is required that accounts for the fact that each hourly mean wind speed is the composite of continuously varying wind speed that typically has significant excursions both above and below the mean value for several minutes each hour.

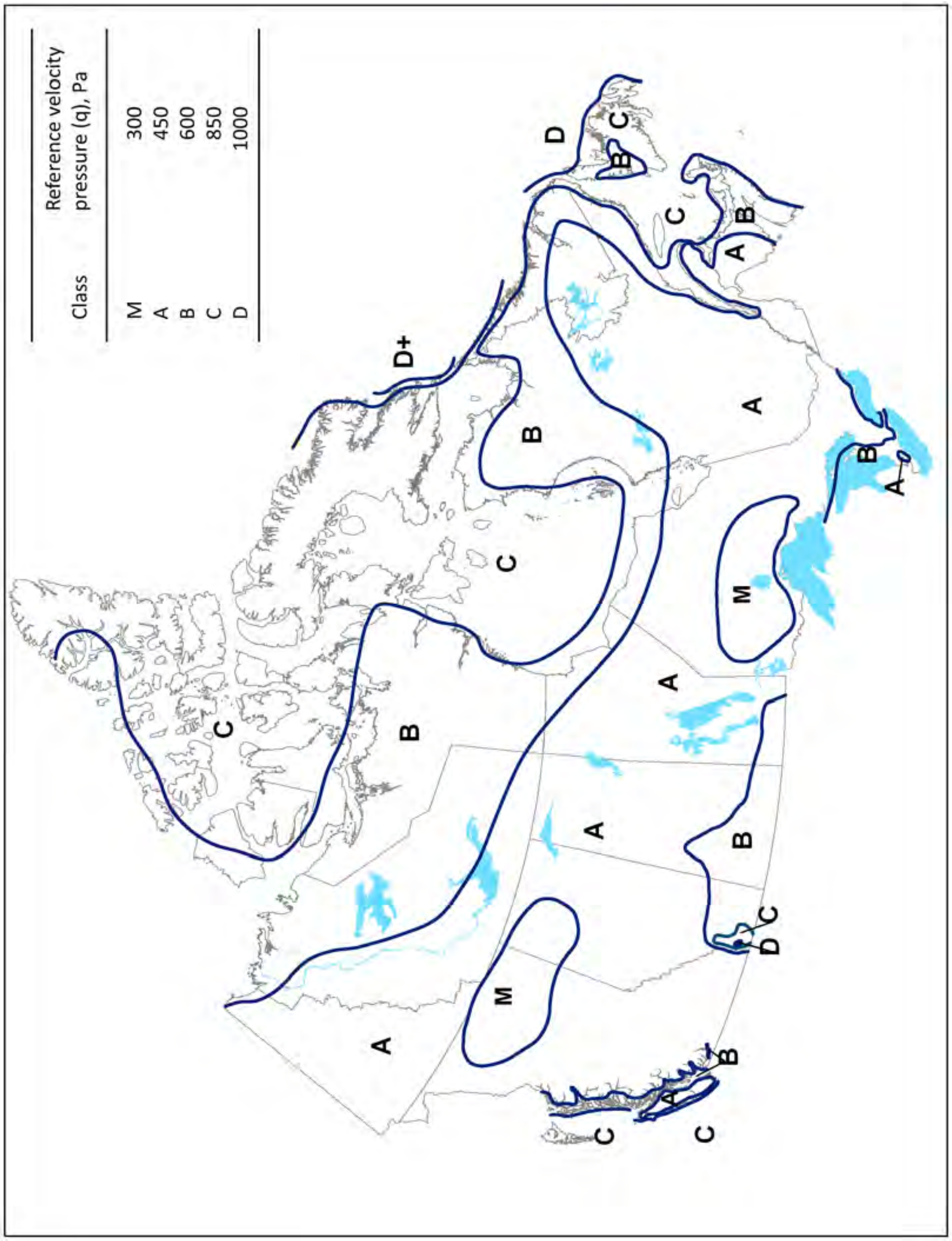
Assuming that each hour is composed of a series of shorter-duration (10 s, for example) mean speeds that are distributed normally about the mean hourly speed with a particular standard deviation, an estimate can be made of the annual frequency distribution of shorter duration periods. Durst (1960) reported from short duration observations of wind speed over open, flat terrain that the coefficient of variation of 10 s duration periods contributing to the hourly mean speed is 0.15. The following steps were undertaken to estimate the frequency duration of 10 s duration wind speeds, and hence to the development of Figure 8:

a) Equation E-5 was used to estimate the annual number of hours of hourly mean wind speed in bins 1 km/h wide from 0 to 220 km/h; a value of v_{10} of 100 km/h was assumed.
b) For the hourly speed corresponding to each of the wind speed bins, the number of 10 s duration wind speed occurrences ranging from 3.9 standard deviations below the mean speed in steps of 1 km/h to 3.9 standard deviations above the bin wind speed were calculated, assuming a normal distribution and a value of coefficient of variation equal to 0.15 (i.e., the standard deviation of 10 s wind speed averages divided by the hourly mean).
c) The total time contribution in each wind speed bin from each 10 s duration wind speed was calculated by multiplying the relative frequencies of each 10 s speed corresponding to an hourly mean speed with the frequency of occurrence of each hourly mean speed.
d) The annual frequency of 10 s duration speeds was then compiled by summing the total time contributions from each hourly mean speed over the entire range of wind speeds (0 to 220 km/h).
e) Noting that the ratio of wind pressures is equivalent to the ratio of the squares of wind speeds and that the design gust speed (corresponding to the 10-year return period 10 s duration gust) is obtained by use of the gust effect factor, C_g (usually 2.0), a table was composed relating the ratios (serviceability factor, τ) of wind pressures corresponding to the 10 s duration wind speeds obtained in Item d) to the 10-year return period gust with the annual number of hours of occurrence; this information is presented in Figure 8.

For example, if the design wind pressure, *P*, is 840 Pa and the annual allowable budget for wind induced twist and deflection beyond the serviceability limit is 6 min (0.1 h), then the serviceability factor, τ, from Figure 8 is 0.7 and the wind pressure will be equal to or exceed 0.7 × 840 Pa = 588 Pa for an average of 6 min every year.

This method is based on a number of approximations, but because most of the assumptions and approximations are believed to be conservative, the resulting serviceability factor should be safe.

Figure E.1
Wind map
(See Clause E.2.2.)



Notes:
1) *Source Environment and Climate Change Canada.*
2) *For details by town and city, use Table E.1.*

Table E.1
Climatic data
(See Clause E.2.1 and Figure E.1.)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
* *50-year return period.* † *Ice thickness from freezing precipitation.*						
British Columbia						
100 Mile House	1040	51.65	121.28	350	270	10
Abbotsford	70	49.05	122.33	440	340	23
Agassiz	15	49.23	121.77	470	360	21
Alberni	12	49.27	124.80	320	250	10
Ashcroft	305	50.72	121.28	380	290	10
Bamfield	20	48.83	125.13	500	390	10
Beatton River	840	57.38	121.40	300	230	10
Bella Bella	25	52.16	128.14	500	390	10
Bella Coola	40	52.37	126.75	390	300	10
Burns Lake	755	54.23	125.75	390	300	10
Cache Creek	455	50.8	121.32	390	300	10
Campbell River	20	50.02	125.24	520	400	10
Carmi	845	49.5	119.12	380	290	10
Castlegar	430	49.32	117.67	340	270	10
Chetwynd	605	55.7	121.63	400	310	10
Chilliwack	10	49.17	121.95	470	360	21
Comox	15	49.68	124.93	520	400	10

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Courtenay	10	49.68	124.98	520	400	10
Cranbrook	910	49.5	115.77	330	250	10
Crescent Valley	585	49.45	117.55	330	250	10
Crofton	5	48.87	123.65	400	310	10
Dawson Creek	665	55.77	120.23	400	310	10
Dease Lake	800	58.44	130.00	300	230	10
Dog Creek	450	51.58	122.30	350	270	10
Duncan	10	48.78	123.70	390	300	10
Elko	1065	49.3	115.12	400	310	10
Fernie	1010	49.5	115.07	400	310	10
Fort Nelson	465	58.83	122.70	300	230	10
Fort St. John	685	56.25	120.85	390	300	10
Glacier	1145	51.27	117.52	320	250	10
Gold River	120	49.78	126.05	320	250	10
Golden	790	51.3	116.97	350	270	10
Grand Forks	565	49.03	118.45	400	310	10
Greenwood	745	49.1	118.68	400	310	10
Hope	40	49.38	121.44	630	480	20
Jordan River	20	48.42	124.05	550	430	10
Kamloops	355	50.67	120.32	400	310	10
Kaslo	545	49.92	116.92	310	240	10
Kelowna	350	49.88	119.48	400	310	10

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Kimberley	1090	49.68	115.98	330	250	10
Kitimat Plant	15	54.05	128.63	480	370	20
Kitimat Townsite	130	54.07	128.65	480	370	20
Ladysmith	80	48.99	123.82	400	310	10
Langford	80	48.45	123.50	400	310	10
Lillooet	245	50.68	121.93	440	340	10
Lytton	325	50.23	121.57	430	330	10
Mackenzie	765	55.3	123.08	320	250	10
Masset	10	54.02	132.10	610	480	10
McBride	730	53.3	120.17	350	270	10
McLeod Lake	695	54.98	123.03	320	250	10
Merritt	570	50.1	120.78	440	340	10
Mission City	45	49.13	122.30	430	330	20
Montrose	615	49.08	117.58	350	270	10
Nakusp	445	50.23	117.80	330	250	10
Nanaimo	15	49.17	123.93	500	390	10
Nelson	600	49.48	117.28	330	250	10
Ocean Falls	10	52.35	127.70	590	460	10
Osoyoos	285	49.03	119.50	400	310	10
Parksville	40	49.32	124.33	500	390	10
Penticton	350	49.5	119.58	450	350	10
Port Alberni	15	49.23	124.80	320	250	10

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Port Alice	25	50.38	127.45	320	250	10
Port Hardy	5	50.7	127.42	520	400	10
Port McNeill	5	50.58	127.10	520	400	10
Port Renfrew	20	48.56	124.40	520	400	10
Powell River	10	49.83	124.52	510	390	10
Prince George	580	53.92	122.75	370	290	10
Prince Rupert	20	54.32	130.32	540	420	10
Princeton	655	49.5	120.51	360	280	10
Qualicum Beach	10	49.35	124.45	530	410	10
Queen Charlotte City	35	53.26	132.08	610	480	10
Quesnel	475	52.98	122.48	310	240	10
Revelstoke	440	50.98	118.20	320	250	10
Salmon Arm	425	50.7	119.28	390	300	10
Sandspit	5	53.25	131.82	780	600	10
Sechelt	25	49.47	123.75	480	370	10
Sidney	10	48.65	123.40	420	330	10
Smith River	660	59.88	126.43	300	230	10
Smithers	500	54.78	127.17	400	310	10
Sooke	20	48.38	123.72	480	370	10
Squamish	5	49.7	123.15	500	390	10
Stewart	10	55.93	129.98	360	280	10
Tahsis	25	49.93	126.65	340	260	10

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Taylor	515	56.2	120.69	400	310	10
Terrace	60	54.52	128.60	360	280	24
Tofino	10	49.12	125.88	680	530	10
Trail	440	49.1	117.70	350	270	10
Ucluelet	5	48.93	125.55	680	530	10
Greater Vancouver						
Burnaby (Simon Fraser Univ.)	330	49.28	122.92	470	360	10
Cloverdale	10	49.1	122.73	440	340	10
Haney	10	49.22	122.60	440	340	10
Ladner	3	49.08	123.08	460	360	10
Langley	15	49.1	122.65	440	340	10
New Westminster	10	49.22	122.92	440	340	10
North Vancouver	135	49.32	123.07	450	350	10
Richmond	5	49.17	123.10	450	350	10
Surrey (88 Ave. & 156 St.)	90	49.16	122.79	440	340	10
Vancouver (city hall)	40	49.25	123.12	450	350	10
Vancouver (Granville & 41 Ave.)	120	49.23	123.14	450	350	10
West Vancouver	45	49.33	123.17	480	370	10
Vernon	405	50.3	119.27	400	310	10
Victoria	10	48.43	123.37	570	440	10
Victoria Gonzales Height	65	48.42	123.32	570	440	10
Victoria Mt Tolmie	125	48.47	123.33	630	480	10

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Whistler	665	50.12	122.96	320	250	10
White Rock	30	49.02	122.80	440	340	10
Williams Lake	615	52.13	122.15	350	270	10
Youbou	200	48.88	124.20	320	250	10
Alberta						
Athabasca	515	54.72	113.28	360	280	10
Banff	1400	51.17	115.57	320	250	10
Barrhead	645	54.13	114.40	440	340	10
Beaverlodge	730	55.22	119.43	360	280	10
Brooks	760	50.58	111.88	520	400.	10
Calgary	1045	51.05	114.08	480	370	10
Campsie	660	54.13	114.65	440	340	10
Camrose	740	53.02	112.83	390	300	10
Canmore	1320	51.09	115.35	370	290	10
Cardston	1130	49.2	113.30	720	560	10
Claresholm	1030	50.03	113.58	580	450	10
Cold Lake	540	54.45	110.17	380	290	10
Coleman	1320	49.63	114.50	630	480	10
Coronation	790	52.08	111.45	370	290	10
Cowley	1175	49.57	114.08	1010	780	10
Drumheller	685	51.47	112.70	440	340	10
Edmonton	645	53.55	113.47	450	350	10

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Edson	920	53.58	116.43	460	360	10
Embarras Portage	220	58.45	111.47	370	290	10
Fairview	670	56.07	118.38	350	270	10
Fort MacLeod	945	49.72	113.42	680	530	10
Fort McMurray	255	56.73	111.38	350	270	10
Fort Saskatchewan	610	53.72	113.22	430	330	10
Fort Vermilion	270	58.4	116.00	300	230	10
Grande Prairie	650	55.17	118.80	430	330	10
Habay	335	58.83	118.73	300	230	10
Hardisty	615	52.67	111.30	360	280	10
High River	1040	50.58	113.87	650	500	10
Hinton	990	53.4	117.58	460	360	10
Jasper	1060	52.88	118.08	320	250	10
Keg River	420	57.75	117.62	300	230	10
Lac la Biche	560	54.77	111.97	360	280	10
Lacombe	855	52.47	113.73	400	310	10
Lethbridge	910	49.7	112.82	660	510	10
Manning	465	56.92	117.62	300	230	10
Medicine Hat	705	50.05	110.67	480	370	10
Peace River	330	56.23	117.28	320	250	10
Pincher Creek	1130	49.48	113.95	960	750	10
Ranfurly	670	53.42	111.68	360	280	10

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Red Deer	855	52.27	113.80	400	310	10
Rocky Mountain House	985	52.37	114.92	360	280	10
Slave Lake	590	55.28	114.77	370	290	10
Stettler	820	52.32	112.72	360	280	10
Stony Plain	710	53.53	114.00	450	350	10
Suffield	755	50.2	111.17	490	380	10
Taber	815	49.78	112.15	630	480	10
Turner Valley	1215	50.67	114.28	650	500	10
Valleyview	700	55.07	117.28	420	330	10
Vegreville	635	53.5	112.05	360	280	10
Vermilion	580	53.37	110.85	360	280	10
Wagner	585	55.35	114.98	370	290	10
Wainwright	675	52.82	110.87	360	280	10
Wetaskiwin	760	52.97	113.37	390	300	10
Whitecourt	690	54.15	115.68	370	290	10
Wimborne	975	51.87	113.58	400	310	10
Saskatchewan						
Assiniboia	740	49.63	105.98	490	380	10
Battrum	700	50.55	108.33	540	420	10
Biggar	645	52.07	108.00	450	350	10
Broadview	600	50.37	102.58	460	360	10
Dafoe	530	51.75	104.53	370	290	10

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Dundurn	525	51.82	106.50	460	360	10
Estevan	565	49.13	102.98	520	400	10
Hudson Bay	370	52.85	102.38	370	290	10
Humboldt	565	52.2	105.12	390	300	10
Island Falls	305	55.53	102.35	350	270	10
Kamsack	455	51.57	101.90	400	310	10
Kindersley	685	51.47	109.17	460	360	10
Lloydminster	645	53.28	110.00	400	310	10
Maple Creek	765	49.92	109.48	450	350	10
Meadow Lake	480	54.13	108.43	400	310	10
Melfort	455	52.87	104.62	360	280	10
Melville	550	50.92	102.80	400	310	10
Moose Jaw	545	50.4	105.53	520	400	10
Nipawin	365	53.37	104.00	380	290	10
North Battleford	545	52.78	108.28	460	360	10
Prince Albert	435	53.2	105.77	380	290	10
Qu'Appelle	645	50.55	103.88	420	330	10
Regina	575	50.45	104.62	490	380	10
Rosetown	595	51.55	108.00	490	380	10
Saskatoon	500	52.12	106.63	430	330	10
Scott	645	52.37	108.83	450	350	10
Strasbourg	545	51.07	104.95	420	330	10

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Swift Current	750	50.28	107.80	540	420	10
Uranium City	265	59.57	108.62	360	280	10
Weyburn	575	49.67	103.85	480	370	10
Yorkton	510	51.22	102.47	400	310	10
Manitoba						
Beausejour	245	50.07	96.52	410	320	16
Boissevain	510	49.23	100.05	520	400	15
Brandon	395	49.83	99.95	490	380	15
Churchill	10	58.75	94.12	550	430	21
Dauphin	295	51.15	100.05	400	310	15
Flin Flon	300	54.77	101.88	350	270	12
Gimli	220	50.64	96.99	400	310	16
Island Lake	240	53.87	94.67	370	290	15
Lac du Bonnet	260	50.27	96.06	370	290	16
Lynn Lake	350	56.85	101.05	370	290	12
Morden	300	49.18	98.10	520	400	16
Neepawa	365	50.23	99.47	440	340	15
Pine Falls	220	50.57	96.22	390	300	16
Portage la Prairie	260	50	98.29	460	360	15
Rivers	465	50.03	100.23	460	360	15
Sandilands	365	49.36	96.30	400	310	16
Selkirk	225	50.15	96.87	410	320	16

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Split Lake	175	56.25	96.10	390	300	18
Steinbach	270	49.53	96.68	400	310	16
Swan River	335	52.14	101.27	350	270	14
The Pas	270	53.87	101.25	370	290	12
Thompson	205	55.79	97.86	360	280	12
Virden	435	49.86	100.93	460	360	14
Winnipeg	235	49.89	97.15	450	350	16
Ontario				0		
Ailsa Craig	230	43.13	81.55	500	390	25
Ajax	95	43.85	79.03	480	370	25
Alexandria	80	45.32	74.63	400	310	30
Alliston	220	44.15	79.87	360	280	23
Almonte	120	45.23	76.20	410	320	26
Armstrong	340	50.3	89.03	300	230	15
Arnprior	85	45.43	76.35	370	290	27
Atikokan	400	48.75	91.62	300	230	15
Attawapiskat	10	52.93	82.43	410	320	17
Aurora	270	44	79.47	440	340	25
Bancroft	365	45.05	77.85	320	250	25
Barrie	245	44.4	79.67	360	280	23
Barriefield	100	44.23	76.47	470	360	30
Beaverton	240	44.43	79.15	360	280	25

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Belleville	90	44.17	77.38	430	330	25
Belmont	260	42.88	81.08	470	360	30
Bowmanville	95	43.91	78.68	480	370	25
Bracebridge	310	45.03	79.30	350	270	21
Bradford	240	44.12	79.57	360	280	25
Brampton	215	43.68	79.77	440	340	25
Brantford	205	43.13	80.27	420	330	25
Brighton	95	44.03	77.73	480	370	25
Brockville	85	44.59	75.68	440	340	30
Burk's Falls	305	45.62	79.40	350	270	22
Burlington	80	43.32	79.80	460	360	25
Cambridge	295	43.38	80.32	360	280	25
Campbellford	150	44.3	77.80	410	320	25
Cannington	255	44.35	79.03	360	280	25
Carleton Place	135	45.13	76.15	410	320	27
Cavan	200	44.2	78.47	440	340	25
Centralia	260	43.28	81.47	490	380	21
CFB Borden	225	44.27	79.88	360	280	22
Chapleau	425	47.83	83.40	300	230	21
Chatham	180	42.4	82.18	430	330	28
Chesley	275	44.28	81.08	480	370	21
Clinton	280	43.62	81.53	490	380	21

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Coboconk	270	44.65	78.80	350	270	25
Cobourg	90	43.97	78.17	490	380	25
Cochrane	245	49.07	81.02	350	270	17
Colborne	105	44	77.88	490	380	25
Collingwood	190	44.48	80.22	390	300	20
Cornwall	35	45.03	74.73	410	320	30
Corunna	185	42.88	82.43	470	360	22
Deep River	145	46.1	77.50	350	270	22
Deseronto	85	44.2	77.05	430	330	25
Dorchester	260	42.98	81.07	470	360	30
Dorion	200	48.78	88.53	390	300	16
Dresden	185	42.58	82.18	430	330	25
Dryden	370	49.78	92.75	300	230	15
Dundalk	525	44.17	80.39	420	330	24
Dunnville	175	42.9	79.62	460	360	25
Durham	340	44.17	80.82	440	340	24
Dutton	225	42.67	81.50	470	360	30
Earlton	245	47.72	79.82	450	350	21
Edison	365	49.8	93.55	310	240	15
Elliot Lake	380	46.38	82.66	380	290	24
Elmvale	220	44.58	79.87	360	280	21
Embro	310	43.15	80.90	480	370	28

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Englehart	205	47.82	79.87	410	320	21
Espanola	220	46.25	81.77	420	330	24
Exeter	265	43.35	81.48	490	380	22
Fenelon Falls	260	44.53	78.75	360	280	25
Fergus	400	43.7	80.37	360	280	25
Forest	215	43.1	82.00	480	370	22
Fort Erie	180	42.9	78.93	460	360	27
Fort Erie (Ridgeway)	190	42.88	79.05	460	360	27
Fort Frances	340	48.61	93.39	310	240	15
Gananoque	80	44.33	76.17	470	360	30
Geraldton	345	49.73	86.95	300	230	15
Glencoe	215	42.75	81.72	430	330	26
Goderich	185	43.75	81.72	550	430	20
Gore Bay	205	45.92	82.47	440	340	21
Graham	495	49.25	90.57	300	230	15
Gravenhurst (Muskoka Airport)	255	44.92	79.37	360	280	21
Grimsby	85	43.2	79.57	460	360	25
Guelph	340	43.55	80.25	360	280	25
Guthrie	280	44.47	79.55	360	280	23
Haileybury	210	47.45	79.63	440	340	22
Haldimand (Caledonia)	190	43.07	79.93	440	340	28

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Haldimand (Hagersville)	215	42.97	80.05	460	360	30
Haliburton	335	45.05	78.52	350	270	25
Halton Hills (Georgetown)	255	43.65	79.92	370	290	25
Hamilton	90	43.25	79.86	460	360	25
Hanover	270	44.15	81.03	480	370	22
Hastings	200	44.3	77.95	410	320	25
Hawkesbury	50	45.6	74.62	410	320	30
Hearst	245	49.68	83.67	300	230	15
Honey Harbour	180	44.87	79.82	390	300	20
Hornepayne	360	49.22	84.78	300	230	15
Huntsville	335	45.33	79.22	350	270	22
Ingersoll	280	43.03	80.88	480	370	30
Iroquois Falls	275	48.77	80.68	370	290	18
Jellicoe	330	49.68	87.52	300	230	15
Kapuskasing	245	49.42	82.43	310	240	15
Kemptville	90	45.02	75.64	410	320	30
Kenora	370	49.82	94.43	310	240	15
Killaloe	185	45.55	77.42	350	270	20
Kincardine	190	44.18	81.63	550	430	20
Kingston	80	44.23	76.48	470	360	30
Kinmount	295	44.78	78.65	350	270	25

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Kirkland Lake	325	48.15	80.03	390	300	21
Kitchener	335	43.45	80.48	370	290	26
Kitchenuhmaykoosib	215	53.83	90.00	420	330	18
Lakefield	240	44.43	78.27	380	290	25
Lansdowne House	240	52.23	87.88	320	250	15
Leamington	190	42.08	82.57	470	360	28
Lindsay	265	44.35	78.73	380	290	25
Lion's Head	185	44.98	81.25	480	370	20
Listowel	380	43.73	80.95	470	360	23
London	245	42.98	81.23	470	360	30
Lucan	300	43.18	81.40	500	390	25
Maitland	85	44.63	75.62	440	340	30
Markdale	425	44.32	80.65	410	320	22
Markham	175	43.87	79.27	440	340	25
Martin	485	49.25	91.13	300	230	15
Matheson	265	48.53	80.47	390	300	19
Mattawa	165	46.32	78.70	320	250	23
Midland	190	44.75	79.88	390	300	20
Milton	200	43.52	79.88	430	330	25
Milverton	370	43.57	80.92	430	330	25
Minden	270	44.92	78.73	350	270	25
Mississauga	160	43.58	79.65	440	340	25

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Mississauga Port Credit	75	43.55	79.58	480	340	25
Mitchell	335	43.47	81.20	480	370	24
Moosonee	10	51.32	80.72	350	270	15
Morrisburg	75	44.9	75.18	410	320	30
Mount Forest	420	43.98	80.73	410	320	24
Nakina	325	50.17	86.70	300	230	15
Nanticoke (Jarvis)	205	42.88	80.10	480	370	30
Nanticoke (Port Dover)	180	42.78	80.20	480	370	30
Napanee	90	44.25	76.95	430	330	26
New Liskeard	180	47.5	79.67	430	330	22
Newcastle	115	43.92	78.58	480	370	25
Newmarket	185	44.05	79.47	380	290	25
Niagara Falls	210	43.1	79.07	430	330	25
North Bay	210	46.32	79.47	340	340	23
Norwood	225	44.38	77.98	410	320	25
Oakville	90	43.45	79.68	470	360	25
Orangeville	430	43.92	80.10	360	280	25
Orillia	230	44.62	79.42	360	280	22
Oshawa	110	43.9	78.85	480	370	25
Ottawa (Barrhaven)	98	45.28	75.76	410	320	30
Ottawa (City Hall)	70	45.42	75.69	410	320	30

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Ottawa (Kanata)	98	45.31	75.91	410	320	28
Ottawa (M-C Int'l Airport)	125	45.32	75.67	410	320	30
Ottawa (Orleans)	70	45.48	75.52	410	320	30
Owen Sound	215	44.57	80.93	480	370	21
Pagwa River	185	50.02	85.22	300	230	15
Paris	245	43.2	80.38	420	330	27
Parkhill	205	43.15	81.68	500	390	24
Parry Sound	215	45.35	80.03	390	300	20
Pelham (Fonthill)	230	43.03	79.28	420	330	25
Pembroke	125	45.82	77.12	350	270	24
Penetanguishene	220	44.78	79.92	390	300	20
Perth	130	44.9	76.25	410	320	26
Petawawa	135	45.9	77.33	350	270	24
Peterborough	200	44.3	78.32	410	320	25
Petrolia	195	42.87	82.15	470	360	22
Pickering (Dunbarton)	85	43.82	79.10	480	370	25
Picton	95	44	77.13	490	380	25
Plattsville	300	43.3	80.62	420	330	27
Point Alexander	150	46.13	77.57	350	270	23
Port Burwell	195	42.65	80.82	470	360	30
Port Colborne	180	42.9	79.23	460	360	25

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Port Elgin	205	44.43	81.40	550	430	20
Port Hope	100	43.95	78.30	480	370	25
Port Perry	270	44.1	78.95	440	340	25
Port Stanley	180	42.67	81.22	470	360	30
Prescott	90	44.72	75.52	440	340	30
Princeton	280	43.17	80.53	420	330	27
Raith	475	48.83	89.93	300	230	15
Rayside-Balfour (Chelmsford)	270	46.58	81.20	450	350	23
Red Lake	360	51.05	93.82	300	230	15
Renfrew	115	45.47	76.68	350	270	25
Richmond Hill	230	43.87	79.45	440	340	25
Rockland	50	45.55	75.29	400	310	30
Sarnia	190	42.97	82.38	470	360	22
Sault Ste. Marie	190	46.52	84.33	440	340	23
Schreiber	310	48.8	87.25	390	300	20
Seaforth	310	43.55	81.40	480	370	22
Shelburne	495	44.08	80.20	400	310	24
Simcoe	210	42.83	80.30	450	350	33
Sioux Lookout	375	50.07	91.98	300	230	15
Smiths Falls	130	44.9	76.02	410	320	26
Smithville	185	43.1	79.55	420	330	25
Smooth Rock Falls	235	49.28	81.63	320	250	16

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
South River	355	45.83	79.38	350	270	22
Southampton	180	44.48	81.38	530	410	20
St. Catharines	105	43.17	79.25	460	360	25
St. Mary's	310	43.25	81.13	470	360	25
St. Thomas	225	42.78	81.20	470	360	30
Stirling	120	44.3	77.55	400	310	25
Stratford	360	43.37	80.95	450	350	27
Strathroy	225	42.95	81.63	470	360	27
Sturgeon Falls	205	46.37	79.92	350	270	23
Sudbury	275	46.5	81.00	460	360	22
Sundridge	340	45.77	79.40	350	270	22
Tavistock	340	43.32	80.83	450	350	27
Temagami	300	47.07	79.78	370	290	23
Thamesford	280	43.07	81.00	480	370	30
Thedford	205	43.15	81.85	500	390	22
Thunder Bay	210	48.4	89.32	390	300	18
Tillsonburg	215	42.85	80.73	440	340	30
Timmins	300	48.47	81.33	350	270	18
Timmins (Porcupine)	295	48.5	81.17	370	290	18
Toronto Area						
Etobicoke	160	43.7	79.57	440	340	25

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
North York	175	43.77	79.42	440	340	25
Scarborough	180	43.78	79.25	470	360	25
Toronto (City Hall)	90	43.65	79.38	440	340	25
Toronto (LBP Int'l Airport)	170	43.68	79.61	440	340	25
Trenton	80	44.1	77.58	470	360	25
Trout Creek	330	45.98	79.37	350	270	22
Uxbridge	275	44.1	79.12	420	330	25
Vaughan (Woodbridge)	165	43.78	79.60	440	340	25
Vittoria	215	42.77	80.32	470	360	33
Walkerton	275	44.12	81.15	500	390	23
Wallaceburg	180	42.6	82.38	450	350	25
Waterloo	330	43.47	80.52	370	290	26
Watford	240	42.95	81.88	470	360	24
Wawa	290	47.98	84.78	390	300	20
Welland	180	42.98	79.25	430	330	28
West Lorne	215	42.6	81.60	470	360	30
Whitby	85	43.87	78.93	480	370	25
Whitby (Brooklin)	160	43.95	78.95	450	350	25
White River	375	48.6	85.28	300	230	15
Wiarton	185	44.75	81.15	480	370	20
Windsor	185	42.3	83.02	470	360	25
Wingham	310	43.88	81.32	500	390	23

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Woodstock	300	43.13	80.75	440	340	28
Wyoming	215	42.95	82.12	470	360	24
Québec						
Acton-Vale	95	45.65	72.57	350	270	26
Alma	110	48.55	71.65	350	270	25
Amos	295	48.57	78.12	320	250	16
Asbestos	245	45.77	71.93	350	270	25
Aylmer	90	45.4	75.83	410	320	27
Baie-Comeau	60	49.22	68.15	500	390	25
Baie-Saint-Paul	20	47.44	70.51	480	370	27
Beauport	45	46.87	71.18	420	330	30
Bedford	55	45.12	72.98	410	320	26
Beloeil	25	45.57	73.20	370	290	32
Brome	210	45.2	72.57	370	290	27
Brossard	15	45.45	73.47	420	330	32
Buckingham	130	45.58	75.42	400	310	30
Campbell's Bay	115	45.73	76.60	320	250	25
Chambly	20	45.45	73.28	400	310	30
Coaticook	295	45.13	71.80	350	270	20
Contrecoeur	10	45.85	73.23	430	330	30
Cowansville	120	45.2	72.75	410	320	28
Deux-Montagnes	25	45.53	73.88	370	290	32

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Dolbeau	120	48.88	72.23	350	270	25
Drummondville	85	45.88	72.48	350	270	28
Farnham	60	45.28	72.98	370	290	28
Fort-Coulonge	110	45.85	76.73	320	250	24
Gagnon	545	51.93	68.17	390	300	18
Gaspé	55	48.83	64.48	480	370	25
Gatineau	95	45.5	75.65	410	320	30
Gracefield	175	46.1	76.05	320	250	25
Granby	120	45.4	72.73	350	270	28
Harrington-Harbour	30	50.5	59.48	720	560	30
Havre-St-Pierre	5	50.23	63.60	630	480	28
Hemmingford	75	45.05	73.58	400	310	30
Hull	65	45.43	75.73	410	320	30
Iberville	35	45.35	73.23	410	320	30
Inukjuak	5	58.48	78.10	600	470	12
Joliette	45	46.02	73.45	360	280	30
Kuujjuaq	25	58.1	68.40	600	470	12
Kuujjuarapik	20	55.28	77.75	550	430	12
La Pocatière	55	47.37	70.04	500	390	27
Lachute	65	45.65	74.33	400	310	32
Lac-Mégantic	420	45.58	70.88	350	270	23

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
La-Malbaie	25	47.65	70.15	480	370	26
La-Tuque	165	47.43	72.78	350	270	24
Lennoxville	155	45.37	71.85	320	250	20
Léry	30	45.35	73.80	420	330	32
Loretteville	100	46.85	71.35	410	320	30
Louiseville	15	46.25	72.95	430	330	30
Magog	215	45.27	72.13	350	270	20
Malartic	325	48.13	78.13	320	250	18
Maniwaki	180	46.38	75.97	310	240	25
Masson	50	45.53	75.42	400	310	30
Matane	5	48.85	67.53	600	470	27
Mont-Joli	90	48.58	68.18	520	400	27
Mont-Laurier	225	46.55	75.50	300	230	22
Montmagny	10	46.98	70.55	470	360	29
Montréal Area						
Beaconsfield	25	45.43	73.87	420	330	32
Dorval	25	45.45	73.75	420	330	32
Laval	35	45.58	73.75	420	330	32
Montréal (City Hall)	20	45.51	73.55	420	330	32
Montréal-Est	25	45.63	73.52	420	330	32
Montréal-Nord	20	45.6	73.63	420	330	32
Outremont	105	45.52	73.62	420	330	32

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Pierrefonds	25	45.48	73.87	420	330	32
St-Lambert	15	45.5	73.50	420	330	32
St-Laurent	45	45.5	73.67	420	330	32
Ste-Anne-de-Bellevue	35	45.42	73.93	420	330	32
Verdun	20	45.45	73.57	420	330	32
Nicolet (Gentilly)	15	46.4	72.28	420	330	30
Nitchequon	545	53.21	70.91	370	290	15
Noranda	305	48.25	79.02	350	270	18
Percé	5	48.53	64.22	720	560	28
Pincourt	25	45.38	73.98	420	330	30
Plessisville	145	46.22	71.78	350	270	27
Port-Cartier	20	50.02	66.87	540	420	25
Puvinrituq	5	60.03	77.28	600	470	10
Québec area						
Ancienne-Lorette	35	46.78	71.38	410	320	30
Lévis	50	46.8	71.18	410	320	30
Québec	120	46.8	71.23	410	320	30
Sillery	10	46.77	71.25	410	320	30
Ste-Foy	115	46.78	71.28	410	320	30
Richmond	150	45.67	72.15	320	250	26
Rimouski	30	48.43	68.55	520	400	27
Rivière-du-Loup	55	47.83	69.53	500	390	25

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Roberval	100	48.52	72.23	350	270	25
Rock-Island	160	45.04	72.10	350	270	20
Rosemère	25	45.63	73.80	400	310	32
Rouyn	300	48.23	79.02	350	270	18
Saguenay	10	48.43	71.07	360	280	25
Saguenay (Bagotville)	5	48.35	70.88	380	290	25
Saguenay (Jonquière)	135	48.42	71.22	350	270	25
Saguenay (Kénogami)	140	48.42	71.25	350	270	25
Saint-Eustache	35	45.56	73.90	370	290	32
Saint-Jean-sur-Richelieu	35	45.31	73.26	410	320	30
Salaberry-de-Valleyfield	50	45.25	74.13	420	330	32
Schefferville	550	54.8	66.83	420	330	15
Senneterre	310	48.38	77.23	320	250	18
Sept-Îles	5	50.2	66.38	540	420	25
Shawinigan	60	46.55	72.75	350	270	30
Shawville	170	45.6	76.48	350	270	25
Sherbrooke	185	45.42	71.90	320	250	20
Sorel	10	46.03	73.12	430	330	30
Ste-Agathe-des-Monts	360	46.05	74.28	350	270	25
St-Félicien	105	48.65	72.45	350	270	25
St-Georges-de-Cacouna	35	47.92	69.50	500	390	25

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
St-Hubert	25	45.5	73.42	420	330	32
St-Hubert-de-Rivière-du-Loup	310	47.82	69.15	400	310	25
St-Hyacinthe	35	45.62	72.95	350	270	30
St-Jérôme	95	45.78	74.00	370	290	30
St-Jovite	230	46.12	74.60	330	250	25
St-Lazare-Hudson	60	45.4	74.14	420	330	30
St-Nicolas	65	46.7	71.40	420	330	30
Sutton	185	45.1	72.62	410	320	26
Tadoussac	65	48.15	69.72	520	400	25
Témiscaming	240	46.72	79.10	320	250	22
Terrebonne	20	45.69	73.63	400	310	30
Thetford Mines	330	46.08	71.30	350	270	25
Thurso	50	45.6	75.25	400	310	30
Trois-Rivières	25	46.35	72.55	430	330	30
Val-d’Or	310	48.1	77.78	320	250	18
Varennes	15	45.68	73.43	400	310	30
Verchères	15	45.78	73.35	430	330	30
Victoriaville	125	46.05	71.97	350	270	28
Ville-Marie	200	47.33	79.43	400	310	22
Wakefield	120	45.64	75.93	340	270	25
Waterloo	205	45.35	72.52	350	270	26
Windsor	150	45.57	72.00	320	250	25

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
New Brunswick						
Alma	5	45.6	64.95	480	370	27
Bathurst	10	47.6	65.65	480	370	26
Campbellton	30	48	66.67	450	350	25
Edmundston	160	47.37	68.33	380	290	25
Fredericton	15	45.95	66.65	380	290	25
Gagetown	20	45.79	66.16	400	310	25
Grand Falls	115	47.05	67.73	380	290	24
Miramichi	5	47.03	65.47	410	320	28
Moncton	20	46.1	64.78	500	390	32
Oromocto	20	45.84	66.48	390	300	25
Sackville	15	45.88	64.37	490	380	32
Saint Andrews	35	45.07	67.05	450	350	26
Saint George	35	45.13	66.82	450	350	26
Saint John	5	45.27	66.05	530	410	26
Shippagan	5	47.73	64.70	630	480	28
St. Stephen	20	45.2	67.28	420	330	25
Woodstock	60	46.16	67.60	370	290	23
Nova Scotia						
Amherst	25	45.83	64.20	480	370	30
Antigonish	10	45.62	62.00	540	420	30
Bridgewater	10	44.38	64.52	550	430	30

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Canso	5	45.33	61.00	610	480	30
Dartmouth	10	44.67	63.57	580	450	31
Debert	45	45.43	63.47	480	370	27
Digby	35	44.62	65.77	550	430	22
Greenwood (CFB)	28	44.98	64.90	540	420	22
Halifax	55	44.65	63.60	580	450	31
Kentville	25	45.08	64.50	540	420	26
Liverpool	20	44.03	64.72	610	480	30
Lockeport	5	43.7	65.12	600	470	30
Louisburg	5	45.92	59.98	650	500	32
Lunenburg	25	44.38	64.32	610	480	30
New Glasgow	30	45.58	62.65	550	430	30
North Sydney	20	46.22	60.25	590	460	32
Pictou	25	45.68	62.72	550	430	30
Port Hawkesbury	40	45.62	61.35	740	570	30
Springhill	185	45.65	64.05	480	370	28
Stewiacke	25	45.13	63.35	500	390	28
Sydney	5	46.15	60.18	590	460	32
Tatamagouche	25	45.72	63.30	550	430	30
Truro	25	45.37	63.27	480	370	29
Wolfville	35	45.08	64.37	540	420	27

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Yarmouth	10	43.83	66.12	560	430	25
Prince Edward Island						
Charlottetown	5	46.23	63.13	560	430	30
Souris	5	46.35	62.25	580	450	30
Summerside	10	46.4	63.78	600	470	30
Tignish	10	46.95	64.03	660	510	30
Newfoundland and Labrador						
Argentia	15	47.3	53.98	750	580	35
Bonavista	15	48.65	53.12	840	650	40
Buchans	255	48.82	56.87	600	470	24
Cape Harrison	5	54.78	57.95	600	470	35
Cape Race	5	46.65	53.07	1050	810	45
Channel-Port aux Basques	5	47.57	59.15	780	600	32
Corner Brook	35	48.95	57.95	550	430	24
Gander	125	48.98	54.59	600	470	30
Grand Bank	5	47.1	55.77	740	570	35
Grand Falls	60	48.93	55.67	600	470	28
Happy Valley-Goose Bay	15	53.32	60.37	420	330	20
Labrador City	550	52.95	66.92	400	310	15
St. Anthony	10	51.37	55.58	870	670	30
St. John’s	65	47.57	52.72	780	600	42
Stephenville	25	48.55	58.58	580	450	24

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Twin Falls	425	53.5	64.53	400	310	17
Wabana	75	47.63	52.95	750	580	40
Wabush	550	52.92	66.87	400	310	15
Yukon						
Aishihik	920	61.6	137.52	380	290	10
Dawson	330	64.07	139.42	310	240	10
Destruction Bay	815	61.25	138.80	600	470	10
Faro	670	62.23	133.36	350	270	10
Haines Junction	600	60.78	137.54	340	260	10
Snag	595	62.4	140.37	310	240	10
Teslin	690	60.17	132.72	340	260	10
Watson Lake	685	60.13	128.71	350	270	10
Whitehorse	655	60.72	135.05	380	290	10
Northwest Territories						
Aklavik	5	68.23	135.01	480	370	10
Echo Bay / Port Radium	195	66.05	117.88	530	410	10
Fort Good Hope	100	66.29	128.63	440	340	10
Fort McPherson	25	67.43	134.88	400	310	10
Fort Providence	150	61.35	117.65	350	270	10
Fort Resolution	160	61.19	113.67	390	300	10
Fort Simpson	120	61.93	121.35	390	300	10
Fort Smith	205	60	111.88	390	300	10

(Continued)

Table E.1 (Continued)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Hay River	45	60.85	115.73	350	270	10
Inuvik	45	68.35	133.72	480	370	10
Mould Bay	5	76.23	119.33	580	450	10
Norman Wells	65	65.28	126.85	440	340	10
Rae-Edzo	160	62.83	116.05	470	360	10
Tungsten	1340	61.95	128.27	440	340	10
Ulukhaqtuuq / Holman	10	70.73	117.75	860	660	10
Wrigley	80	63.23	123.47	390	300	10
Yellowknife	160	62.48	114.35	470	360	10
Nunavut						
Alert	5	82.48	62.25	750	580	10
Arctic Bay	15	73.03	85.17	550	430	10
Arviat	5	61.12	94.05	580	450	21
Baker Lake	5	64.32	96.02	540	420	15
Eureka	5	79.98	85.95	550	430	10
Igluligaarjuk / Chesterfield Inlet	10	63.33	90.70	560	430	21
Iqaluit	45	63.73	68.50	580	450	10
Iqaluktuuttiaq / Cambridge Bay	15	69.12	105.03	540	420	10
Isachsen	10	78.78	103.53	600	470	10
Kangiqiniq / Rankin Inlet	10	62.82	92.08	600	470	22
Kanngiqtugaapik / Clyde River	5	70.45	68.57	720	560	11

(Continued)

Table E.1 (Concluded)

Location	Elevation (m)	Latitude	Longitude	1/50 wind pressure (Pa)*	1/10 (Pa)	Reference ice thickness (mm)†
Kugluktuk	10	67.83	115.08	460	360	10
Nottingham Island	30	63.1	78.00	780	600	14
Resolute	25	74.68	94.90	690	540	10
Resolution Island	5	61.3	64.88	1230	950	20
Salliq / Coral Harbour	15	64.13	83.17	690	540	16

Annex F (informative)
Corrosion protection of guy anchorages

Note: *This Annex is not a mandatory part of this Standard.*

F.1 General

F.1.1 Steel anchor shafts in contact with soil

In recent years, there has been a serious increase in reports of corrosion of galvanized steel anchor shafts that are in direct contact with the soil. The effect of acid rain might be one of the major causes of this increase. While the majority of the problems have occurred at sites where the installation is about 20 years old or more, there have recently been a few cases where the installation is less than 10 years old. It appears that most of the serious problems have occurred at AM antenna tower sites. There have also been cases of increased corrosion rates at sites adjacent to underground pipelines utilities. This can be due, in part, to the type of soils where these sites are located. The contribution of currents in the ground and in the radial ground system can be significant, but is unknown. Given similar conditions, the corrosion rate should be similar regardless of the tower type.

F.1.2 Corrosion mechanisms and hazards

F.1.2.1 Mechanisms

There are a number of corrosion mechanisms that can attack the anchor shaft. These are
a) galvanic corrosion;
b) stray current;
c) bacteria; and
d) soil corrosion.

F.1.2.2 Hazards

In general, the corrosion hazard increases
a) as the following factors decrease: soil resistivity, pH, and redox potential (which is a measure of microbiological corrosion potential); and
b) as the following factors increase: salt content, organic content, oxygen differential or transfer, and moisture content fluctuation.

F.2 Corrosion protection

F.2.1 General

Although all anchorage steel located below grade should be hot dip galvanized, Clause 8.5.2 requires that anchorage steel located below grade and not encased in concrete have corrosion protection in addition to galvanizing. Where galvanizing is not practical, adequate corrosion protection should be considered in the design.

F.2.2 Methods of protection

There are several methods that can be used to provide additional corrosion protection:
a) Encasement of the steel in a reinforced concrete collar with proper reinforcing at the interface between the collar and the main body of concrete to help prevent cracking. The concrete collar must provide a minimum coverage of 75 mm on all sides of the shaft or bolts.

b) Cathodic protection can be achieved by connecting a magnesium anode to the galvanized anchor shaft. This will effectively transfer the corrosion action from the anchor shaft to the anode. The anode should be sized to suit the exposed area of steel and the corrosion rate of the soil. This requirement should be established upon completion of the installation, by experts in this field, based on actual on-site measurement of potential differences. A life expectancy of about 20 years should be considered as a minimum. The effectiveness of the anode can be periodically checked. This should be done on a five-year basis in the early life of the anode and at more frequent intervals as it nears the end of its life expectancy.
c) Taping or painting of the anchor shaft with anti-corrosion products meeting the criteria of Clause F.2.3 is also effective in providing protection. The danger with this method is that the coating may be damaged in the installation or backfilling operations. If this happens, the corrosion will be concentrated at this location and could cause a serious problem. Cathodic protection might also be necessary when employing this method.
d) Providing a corrosion allowance by increasing the wall or member thickness over that required by the design, or by designing steel anchorages with a reduced yield strength. This will allow for a sacrificial thickness for corrosion while maintaining a minimum steel area for the design loads over the design life of the structure.

F.2.3 Characteristics of protective coatings

The additional corrosion protection system must remain
a) crack-free and not become brittle or fluid over the anticipated service temperature range; and
b) chemically stable, non-reactive with adjacent materials, and impervious to moisture.

F.3 Inspections

Inspection of existing installations is very important. Care must be exercised in excavating around the shaft that the structural capacity of the anchor is not so reduced that a failure might occur. Securing of the guys to an alternative anchorage is recommended if there is any question about the condition of the anchor shaft or the anchor block.

An inspection of all of the anchors, not just a random sample, is necessary. Experience has shown a wide variation in the corrosion action across a tower site.

Annex G (informative) ***Reliability classes***

Note: *This Annex is not a mandatory part of this Standard.*

G.1 General

Recent editions of this Standard have included successively more rational design criteria, as well as improved analysis techniques. A review of other standards shows that taking into account the consequences of failure of the structure when establishing its required reliability is commonly permitted. It has therefore been considered appropriate to introduce this concept into this Standard.

G.2 Reliability

The average reliability shown in Table G.1 were established based on recorded failure rates of structures under various loadings. Failure rates were assessed statistically and from these statistics the importance factors required to achieve the average reliabilities were determined for the selected reliability classes. Appropriate applications of the three reliability classes have been defined in Table 3.

Table G.1
Average reliability
(See Clause G.2.)

Reliability class	Average reliability (30 year)	Importance factor
I	0.9999	1.0
II	0.9990	0.9
III	0.9900	0.8

From Table G.1, it can be seen that a tower designed to Class II is approximately ten times more likely to fail and a tower designed to Class III approximately 100 times more likely to fail than a tower designed to Class I.

G.3 Classification

The reliability class for a tower should be selected by the owner, after consultation with the engineer, in accordance with the criteria set out in Clause 6.3. The owner should obtain written acknowledgement from all third parties with services on the tower that they agree to share the higher risk of failure consequent to the lower reliability classification.

Annex H (informative)
Dynamic response of guyed towers to wind turbulence

Note: *This Annex is not a mandatory part of this Standard.*

H.1 General

H.1.1 Dynamic analysis

Guyed towers are susceptible to dynamic excitation due to wind turbulence. The dynamic response of a guyed tower can be determined from a full dynamic analysis that accounts for the physical properties of the mast including attachments, the stiffness, mass, and drag characteristics of the guys, and the statistical properties of the turbulent wind. A dynamic analysis should include all significant vibration modes of the guyed tower and should account for structural and aerodynamic damping along the mast and the guys.

H.1.2 Patch load method

In lieu of a full dynamic analysis, the dynamic response of a guyed tower conforming to the criteria given in Clause H.2 can be determined using the patch load analysis method outlined in Clause H.3. In this method, the steady and fluctuating components of the guyed tower response are calculated separately and then combined to obtain the peak design response. The fluctuating response component is approximated with the use of a series of static load patterns applied in succession to the mast.

H.2 Criteria for use of the patch load method

H.2.1 Applicability

The patch load method is intended for use with guyed towers that are designed in accordance with conventional practice. The method assumes a fairly uniform distribution of mass and wind drag characteristics along the mast. For guyed towers with unusual structural configurations or attachments, a full dynamic analysis should be undertaken in accordance with the guidelines of Clause H.1.1.

H.2.2 Restriction

Where the economic consequences or physical hazards resulting from the failure of a guyed tower are high, a full dynamic analysis should be undertaken in accordance with the guidelines of Clause H.1.1.

H.2.3 Criteria to be satisfied

In addition to guidelines in Clauses H.2.1 and H.2.2, the following criteria should be satisfied in order to use the patch load method:

a) the height of the cantilevered section above the top guy is less than one-half of the spacing between the top two guy support levels on the mast;

b) the stiffness parameter, β_s, is less than 1.0, where β_s is defined in accordance with Clause H.2.4.1; and

c) the inertial resistance parameter, Q, is less than 1.0, where Q is defined in accordance with Clause H.2.5.

When these three criteria are not met, a full dynamic analysis should be undertaken in accordance with the guidelines of Clause H.1.1.

H.2.4 Calculation of parameters

H.2.4.1 Stiffness parameter, β_s

The stiffness parameter is calculated using the following formula:

$$\beta_s = \frac{4\left(\frac{E_m I_m}{\bar{L}_s^2}\right)}{\frac{1}{n_g}\sum_{i=1}^{n_g} K_{G_i} H_{G_i}}$$

where

E_m = Young's modulus for the mast legs

I_m = the bending moment of inertia of the mast

$\bar{L}_s$ = the average span length between guy support levels on the mast

n_g = the number of guy support levels on the mast

K_{G_i} = the resultant horizontal guy stiffness at the *i*th support level as provided for in Clause H.2.4.2

H_{G_i} = the height of the *i*th guy support level above the base of the mast

H.2.4.2 Resultant guy stiffness, K_G

The resultant horizontal stiffness of all guys at a support level is determined by the following formula:

$$K_G = \sum_{j=1}^{m_g} \cos^2 \alpha_j \left(\frac{1}{k_{e_j}} + \frac{1}{k_{g_j}}\right)^{-1}$$

where

m_g = the number of guys attached to the mast at that support level

α_j = the horizontal angle between the wind direction and the vertical plane of the *j*th guy

k_{e_j} = the horizontal elastic stiffness of the *j*th guy in its own vertical plane as defined in Clause H.2.4.3

k_{g_j} = the horizontal gravity stiffness of the *j*th guy in its own vertical plane as defined in Clause H.2.4.4

H.2.4.3 Elastic guy stiffness, k_e

The horizontal elastic stiffness of an individual guy in its own vertical plane is given by

$$k_e = \frac{A_G E_G \cos^2 \theta_G}{L_c}$$

where

A_G = the cross-sectional steel area of the guy

E_G = Young's modulus for the guy

θG = the vertical angle formed between a horizontal plane and the straight chord line joining the two ends of the guy in the undeflected position of the mast

L_c = the length of the straight chord line joining the two ends of the guy in the undeflected position of the mast

H.2.4.4 Gravity guy stiffness, k_g

The horizontal gravity stiffness of an individual guy in its own vertical plane is determined by the following formula:

$$k_g = \frac{12\bar{T}^3}{w_G^2 L_c^3}$$

where

$\bar{T}$ = the average initial tension in the guy

w_G = the weight of the guy per unit length

H.2.5 Inertial resistance factor, *Q*

The inertial resistance factor, Q, is determined by the following formula:

$$Q = \frac{1}{30}\left(\frac{H_m \bar{V}_H}{\bar{D}_m}\right)^{0.33}\left(\frac{\bar{m}}{H_m \overline{C_d A}}\right)^{0.5}$$

where

H_m = the height of the mast including any cantilever

$\bar{V}_H$ = the mean hourly wind speed at the top of the tower, m/s, as provided for in Clause H.2.6

$\bar{D}_m$ = the average face width of the mast, m

$\bar{m}$ = the average mass of the mast per unit length including attachments, kg/m

$\overline{C_d A}$ = the average effective drag area of the mast and attachments per unit length, m^2/m. Calculation of the product $C_d A$ is discussed in Clauses 5.8 and 5.9

H.2.6 Mean hourly wind speed, $\bar{V}_H$

The mean hourly wind speed at the top of the tower, in m/s, may be calculated using the following formula:

$$\bar{V}_H = 1.245\sqrt{C_{eH} q}$$

where

C_{eH} = the height factor as provided for in Clause 5.5, calculated at the top of the tower including any cantilever

q = the reference velocity pressure as provided for in Clause 5.4, Pa

H.3 Dynamic response analysis using the patch load method

H.3.1 Design dynamic response in the mast, $\hat{r}$

H.3.1.1 Calculation of design dynamic response

The design dynamic response at any location along the mast is calculated according to the following formula:

$$\hat{r} + \bar{r} \pm \tilde{r}_{PL} \lambda_B \lambda_R \lambda_{TL} g$$

Alternatively, the design response may be conservatively taken as

$\hat{r} = \bar{r} \pm 3.8\tilde{r}_{PL}$ for the mast below the top guys

and

$\hat{r} = \bar{r} \pm 5.5\tilde{r}_{PL}$ for the cantilever above the top guys

where

$\hat{r}$ = the design response in the mast

$\bar{r}$ = the mean response component as provided for in Clause H.3.1.2

$\tilde{r}_{PL}$ = the resultant patch load response as provided for in Clause H.3.1.3

λ_B = the background scaling factor as provided for in Clause H.3.1.4

λ_R = the resonant amplification factor as provided for in Clause H.3.1.5

λ_{TL} = the turbulent length scale factor as provided for in Clause H.3.1.6

g = a peak factor to be taken as 4.0

The sign of the fluctuating response term, $\tilde{r}_{PL}\lambda_B\lambda_R\lambda_{TL}g$, at each location along the mast should be chosen so as to produce the most severe effect at that location. The patch load calculations must be repeated for each wind direction considered (see Clause 6.5).

H.3.1.2 Mean response component, $\bar{r}$

The mean response component is the response of the tower due to the mean wind load. The mean wind load is calculated in accordance with Clause 5.8, except that the design wind pressure, P, is replaced by the mean wind pressure, $\bar{P}$, which is determined by the following formula:

$$\bar{P} = qC_e$$

where q is defined in Clause 5.4 and C_e is defined in Clause 5.5. Calculation of the mean response component should include second-order effects arising from the deflection of the mast and displacement of the guy attachment points.

H.3.1.3 Resultant patch load response, $\tilde{r}_{PL}$

H.3.1.3.1 Calculation of resultant patch load response

A series of patch load cases are applied successively to the mast. The resultant patch load response at each location along the mast is calculated by the following formula:

$$\tilde{r}_{PL} = \sqrt{\sum_{i=1}^{n_{LC}} r_{PL_i}^2}$$

where

n_{LC} = the number of patch load cases that are required

r_{PL_i} = the response at the location in question due to the ith patch load

H.3.1.3.2 Required patch load cases

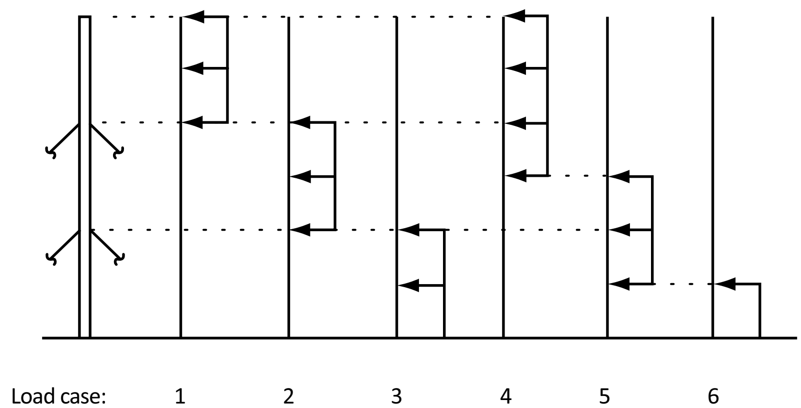
Separate arrangements of patch wind loading are required as described by the following distributions:

a) between each set of adjacent guy support levels;
b) from the base of the mast to the lowest guy support level;
c) from the top guy level to the top of the cantilever (if applicable);
d) from midpoint to midpoint of each set of adjacent spans;
e) from the base of the mast to the midpoint of the lowest span; and
f) from the midpoint of the span immediately below the top guy to the top of the mast, including any cantilever.

For example, the required patch loads for a guyed tower with two guy support levels and a cantilevered top section are shown in Figure H.1.

Note: *The patch loads should be applied to the tower in its deflected position under mean wind loads. Therefore, the response due to each patch load case may be calculated as the difference between the load effect resulting from the patch load combined with the mean wind load and the load effect due to the mean wind load acting alone. Mean wind loads are defined in Clause H.3.1.2.*

Figure H.1
Required patch load cases for two-level mast
(See Clause H.3.1.3.2.)



H.3.1.3.3 Patch wind load

The wind load for individual patch load cases is determined in accordance with Clause 5.8, except that the design wind pressure, P, is replaced by the patch load wind pressure, P_{PL}, which is determined by the following formula:

$$P_{PL} = 2qi_o\sqrt{C_e}$$

where

q = the reference velocity pressure as provided for in Clause 5.4

i_o = the intensity of turbulence

= 0.18 for open country sites

= 0.27 for forest and urban sites

C_e = the height factor as provided for in Clause 5.5 and calculated at the mid-height of the load patch in question

H.3.1.4 Background scaling factor, λ_B

The background scaling factor, λ_B, is determined as follows:

a) for negative bending moments at guy support levels: $\lambda_B = 0.70$
b) for positive mid-span moments: $\lambda_B = 0.69\beta_s^{0.07}$
c) for shear force: $\lambda_B = 0.70$
d) for deflections: $\lambda_B = 0.84\beta_s^{0.05}$

where

β_s = the stiffness parameter as provided for in Clause H.2.4.1

Note: *Positive bending moments are those which induce compressive stresses on the windward face of the mast; negative bending moments induce tensile stresses on the windward face.*

H.3.1.5 Resonant amplification factor, λ_R

The resonant amplification factor, λ_R, is determined as follows:

a) for bending moments: $\lambda_R = 0.34Q + 0.91$

b) for shear force: $\lambda_R = 0.30Q + 0.92$

c) for deflections: $\lambda_R = 0.16Q + 0.95$

d) for cantilever response (moment, shear, and deflection): $\lambda_R = 1.75$

where

Q = the inertial resistance parameter as provided for in Clause H.2.5

H.3.1.6 Turbulent length scale factor, λ_{TL}

The turbulent length scale factor, λ_{TL}, is determined as follows:

a) for negative bending moments at guy support levels: $\lambda_{TL} = 1.0$

b) for positive mid-span moments: $\lambda_{TL} = 0.58\bar{L}_s^{\,0.135}$

c) for shear force: $\lambda_{TL} = 1.31\bar{L}_s^{\,-0.066}$

d) for deflections: $\lambda_{TL} = 1.97\bar{L}_s^{\,-0.165}$

where

$\bar{L}_s$ = the average span length of the mast as defined in Clause H.2.4.1

H.3.2 Design dynamic guy tensions

Design guy tensions should be determined assuming that the deflected shape of the mast corresponds with the peak dynamic deflections calculated in accordance with Clause H.3.1.1.

H.4 Bibliography

Davenport, A.G., and Sparling, B.F. 1991. Dynamic Gust Response Factors for Guyed Towers. *Proceedings, 8th International Conference on Wind Engineering*. University of Western Ontario. London, Canada.

Sparling, B.F., Smith, B.W., and Davenport, A.G. 1993. Simplified Dynamic Analysis Methods for Guyed Masts in Turbulent Winds. *International Association for Shell and Spatial Structures, Meeting of the Working Group on Masts and Towers.* Prague.

Annex J (informative) **Serviceability limit states**

Note: *This Annex is not a mandatory part of this Standard.*

J.1 General

Limit states in this Standard are defined as being either ultimate (those that will cause some type of structural failure) or serviceability (those that will cause an unacceptable reduction in the level of service). For directionally sensitive antennas, such as those used for microwave or satellite telecommunications, the serviceability limit state typically is reached when the signal level is degraded below the acceptable level by loss of alignment, caused by twisting or tilting of the antenna. The degree of deflection that causes unacceptable signal loss is dependent on the system specifications and the length of time over which that loss of alignment can be tolerated is dependent on the system reliability requirements.

J.2 Deflection limit

For any particular antenna, the maximum deflection that will still provide acceptable system performance must be established by the owner. This allowable angular deflection depends on system characteristics such as the permissible signal loss, antenna diameter, signal frequency, and potential interference. The deflection is expressed as the allowable twist and allowable tilt, which might or might not be equal. When a structure carries more than one antenna, the allowable angular deflections should be specified for each antenna to permit the governing case to be determined.

J.3 Serviceability wind pressure

The required operational availability of the radio system is also established by the owner as part of the design specification. From this, the total length of time in a year that the signal may be degraded can be established. In addition to signal degradation due to deflection of the structure, some factors that can result in lack of performance are fading due to geographical and atmospheric conditions, equipment failure, power outage, cross-system interference, and human error. The system designer must then establish a signal loss budget accounting for all of these factors, to determine how much of the total time below specified performance may be allocated to degradation of the signal due to wind effects.

For example, by the system specification, the operational availability of each link of a radio system is required to be 99.993%. This means that over a period of one year it must function at specified performance or better for 99.993% of the time.

Number of hours in a year = 365.25 ×24 = 8766

Maximum annual hours of signal degradation = (1.0 − 0.99993) × 8766 = 0.614 h

Total losses due to other factors = 0.390 h

Maximum annual signal loss due to wind effects = 0.614 − 0.390 = 0.224 h

Once this permissible annual duration of deflections exceeding the serviceability limit has been established, Figure 8 may be used to determine what proportion of the site-specific wind pressure, q_h, exceeds that annual limit. From Figure 8, the serviceability factor, τ, for an annual duration of 0.224 h is approximately 0.6.

Figure 8 has been established based on statistical data on the relationship between the once-in-10-year return wind pressure and annual duration of occurrence of various wind pressures, as discussed in Annex E. This ratio, which is defined as the serviceability factor, τ, is applied to the design wind pressure, P, to obtain the serviceability limit state wind pressure.

If the deflections of a particular antenna installation at various wind pressures are known, so that the ratio of wind pressure at the limiting deflection to the design wind pressure may be established, Figure 8 may be used to determine the system availability.

J.4 Allocation of allowable deflection

The deflection limit is the total allowable off-axis antenna movement due to deflections from all sources. For antennas on pipe mounts, the deflection is typically allocated as 10% due to movement of the mount and antenna, and 90% due to movement of the supporting structure. For antennas with integral mounts (for example, many types of satellite antennas), the antenna and its support are typically rated by the manufacturer, according to selected wind speeds and signal degradations. If such an antenna is mounted on a structure that also has a significant deflection, such as a configuration of beams on the roof of a building, the combined deflection must be considered when determining the operational limit of the complete installation.

J.5 Additional information

Additional information on allowable angular deflection values can be found in ANSI/TIA-222-H.

Formulas for conversion of wind speeds to pressures are provided in Annex E.

Annex K (informative) ***Commentary on Clause 7***

Note: *This Annex is not a mandatory part of this Standard.*

K.0 Introduction to Annex K

Clause 7 of this Standard deals with members' capacity and the effective slenderness ratio to be used. Therefore, this Annex covers most of the commonly used bracing configurations in latticed structures. In this Annex, the clauses are numbered according to the clause to which they refer in the Standard (e.g., Clause K.7.1.5 of this Annex refers to Clause 7.1.5 of the Standard).

K.7.1.5 Member shapes

Schifflerized 60° angles are weaker in torsional-flexural buckling and stronger in flexural buckling than regular 90° steel angles; hence, it is reasonable to assume that the design of 60° angles should account for torsional-flexural buckling. However, the results of experiments show that the design strength in accordance with CSA S16, 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° steel angles.

For regular 90° steel angles, strengths in torsional buckling and in local plate buckling are likely to be very close to each other. If this holds true for 60° steel angles as well, the use of an effective yield stress based on the susceptibility of the individual plate elements to local buckling can be assumed to closely approximate the torsional buckling portion of torsional-flexural buckling.

Based upon test results of 60° steel angles at the University of Windsor by Adluri (1990) and Madugula and Adluri (1994), the Technical Committee decided that torsional-flexural buckling need not be explicitly accounted for in the design of 60° steel angles if compression resistances are obtained using the basic compression strength equations and the effective yield stress.

If the designer wishes to check 60° angles by accounting for torsional-flexural buckling explicitly, reference can be made to Adluri and Madugula (1991).

K.7.1.6 Minimum Charpy V-notch value

Resistance to brittle fracture in members subject to tension due to axial forces or moments is an important consideration when specifying the minimum physical properties of the steel to be used at a particular site. This is of significance for tall towers using high tensile grades of steel (F_y greater than 350 MPa), and thick members or solid rounds with thicknesses greater than 100 mm. This is usually done by specifying a Charpy V-notch value; however, a method for determining the minimum value has not been readily available. BS ISO 5950-1 gives a formula using the thickness and yield of the steel, the two critical values.

The minimum Charpy V-notch (CVN) value may be determined from the following:

$$CVN = \frac{F_y t}{710c}$$

where

F_y = yield strength, MPa

t = thickness of an angle or plate, the wall thickness of a tube, the flanges of rolled section or the diameter of a solid round

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

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

For the extremely cold temperatures that occur in some regions of Canada, it might not be possible to obtain steel with the required values. In these cases, a modified yield may be employed based on a slower rate of loading, if applicable. CSA S6 provides guidance in selecting this value.

K.7.1.7 Normal framing eccentricity

Using "normal framing eccentricity" is an accepted method to account for eccentric bending when designing members in lattice structures. Some standards require the use of the very conservative method of combined flexure and axial forces for only a slight deviation from the definition. This section includes leg members based on good industry practice. Studies at the University of Windsor (Adluri 1990) have confirmed that the adjustment presented here is still conservative.

K.7.1.8 Secondary bracing members

In earlier editions of this Standard, the axial force in secondary bracing members has been taken as 2% of the force in the member being supported. This edition has adopted a variable axial force, based on the slenderness ratio (L/r) of the member being supported and provides, for commonly used bracing patterns, a simple method for determining the required minimum resistance of the different supporting members connected at the same panel point.

K.7.2.1.1 Unbraced length, *L*

When modelling 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 to determine 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 can be taken between the node points.

K.7.2.2 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 by eccentricity at the joints. This is the traditional value and has provided satisfactory results over the years. The effective length factor of $K = 1.1$ for angle leg members in square towers with staggered bracing is used based on ASCE 10 recommendations driven from full scale testing of towers.

K.7.2.3 Bracing members

The clauses use the same concepts used in the 2001 edition of this Standard; however, the Technical Committee has brought in some of the material from ANSI/TIA-222-G and extended the type of bracing patterns used in order to address some of the questions raised in the past.

K.7.2.3.2 Cross bracing (tension/compression)

The Technical Committee has reviewed the requirements for cross bracing in the previous edition 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 a new approach more suitable for our applications. The formula adjusts the effective length for the out-of-plane buckling depending on the cross-over point providing support. This formula is based on the work of Picard and Beaulieu (1987).

K.7.2.5 Effective yield stress

As a large number of steel angles used in towers are above the maximum width-thickness ratio imposed by CSA S16, 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 that in ASCE 10.

This section has been expanded to include tubular round members and polygonal tubular members.

Table K.1 provides fillet radii of metric size angles.

Table K.1
Angle member fillet radius, mm
(See Clause K.7.2.5.)

Equal leg angles

Section size	200 × 200	150 × 150	125 × 125	100 × 100	90 × 90	75 × 75	65 × 65	55 × 55	45 × 45	35 × 35	25 × 25
Fillet radius	16	13	13	10	10	8	6	6	6	5	3

Unequal leg angles

Section size	200 × 150	200 × 100	150 × 100	125 × 90	125 × 75	100 × 90	100 × 75	90 × 75	90 × 65	80 × 60	75 × 50	65 × 50	65 × 35	45 × 30
Fillet radius	16	13	13	11	11	10	10	10	8	8	8	6	5	5

K.7.2.6 Compressive resistance

This edition of the Standard has adopted a single compression formula as given in CSA S16 (see Loov, 1996). The distinction in compressive resistances of large solid round members between stress-relieved and non-stress-relieved members has been eliminated (see Sennah *et al* 2009).

K.7.3.8 Link plates

This edition of the Standard has introduced design requirements for link plates and pins. The resistance formulas are based on ANSI/TIA-222-H as well as ASCE 48.

K.7.4 Flexural members

In earlier editions of this Standard, the resistances of tubular members only subjected to flexural loads were presented. This Standard has updated the flexural members resistances of round and polygonal tubular members and allowed the use of the plastic section modulus for compact tubular sections with lower *D/t* ratios. Furthermore, the flexural resistance of solid round members and angles were introduced.

The formulas for evaluating the resistances of members subjected to combined flexural and compression forces were introduced. Although latticed structures are typically evaluated based on axial forces only, the standards presented these formulas to enable evaluation of members subjected to large eccentric forces that are beyond the normal framing eccentricities limits or other similar cases.

Equal leg angles subjected to combined axial and bending forces can be checked using Clauses 4.8.1.2 and 4.7.4 of ANSI/TIA-222-G, with a resistance factor of 0.90. Unequal leg angle members subjected to combined axial and bending forces can be checked using Clause H2 and F10 of ANSI/AISC 360, with a resistance factor of 0.90.

K.7.5.3 Anchor rods

This Clause has been introduced to include the current thinking on this subject. The resistances of anchor rods have been specified for various commonly used installations (see ANSI/TIA-222-H).

K.7.5.7 Splices

This Clause presents the current thinking on this subject. The minimum required tensile strength of the splice has been reduced from 50% to 33%, bringing this edition of the Standard in line with other tower standards. This edition also allows the lower value permitted for 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 effect of the eccentricity between the splice plate group and the leg angles on the bolts and the flexural stresses resulting in the splice plates need to be considered (see Sakla *et al* 1999).

A good source for guidance on the design of flanges for tubular members can be found in ANSI/TIA-222-H.

K.7.5.8 U-bolts

This Clause presents guidance on the determination of U-bolt capacity under different loading scenarios. The capacities have been adopted from ANSI/TIA-222-H which were based on a destructive testing program of a significant number of U-bolted connections.

Annex L (informative)
Geotechnical site investigations

Note: *This Annex is not a mandatory part of this Standard.*

L.1 General

A geotechnical report should be provided for all sites and should include, as a minimum, the following information in accordance with recommendations of the *Canadian Foundation Engineering Manual*.

L.2 Normal soil sites

For soil sites, the geotechnical report should include
a) the ultimate and serviceability bearing resistances;
b) the coefficients of lateral at rest, active and passive soil pressure;
c) the angle of internal friction for coarse grained soils;
d) the soil cohesion for fine grained soils;
e) the water table level for the worst expected condition or recommended design level, whichever governs;
f) the density of soil to at least 4 m below grade but to a greater depth if required for the foundation or anchor design. Values should be provided for both dry and submerged conditions;
g) the depth of seasonal frost penetration;
h) the ultimate end bearing and skin friction (tension and compression) capacities for driven piles and poured caissons;
i) the soil resistivity and corrosive nature of the soil; and
j) the site category for seismic response in accordance with Clause 4.1.8.4 of the *NBC*.

L.3 Corrosive soil sites

For corrosive soils sites, in addition to the above information, the geotechnical report should include
a) the soil resistivity (soil resistivity is an important indicator of corrosiveness). Generally accepted levels of corrosive are shown in Table L.1;
b) the soil pH (soils that are acidic (pH < 4.5) or alkaline (pH > 9) are generally associated with high corrosion rates of carbon steel);
c) chloride and sulfate content (soluble salts such as chloride and sulfate accelerate corrosion);
d) stray current (presence of underground utilities might be an indicator that stray current from those utilities is present in the ground);
e) fluctuation in the groundwater table; and
f) anaerobic bacterial action.

Table L.1
Effect of resistivity on corrosion
(See Clause L.3.)

Aggressiveness	Resistivity (Ohm-cm)
Very corrosive	< 700
Corrosive	700–2000
Moderately corrosive	1200–5000
Mildly corrosive	5000–10 000
Non-corrosive	> 10 000

L.4 Rock sites

For rock sites, the geotechnical report should include

a) the type and condition of rock, including fracture frequency and spacing;
b) the depth to sound rock, including thickness of weathered rock. If significant overburden is present, the information shown under Clause L.2 should also be provided for the overburden;
c) the rock density;
d) the ultimate and serviceability bearing resistances;
e) the unconfined compressive strength;
f) the water table level for the worst expected condition or the recommended design level, whichever governs;
g) the recommended anchoring procedure;
h) the rock quality designation (RQD); and
i) the site category for seismic response according to Clause 4.1.8.4 of the *NBC*.

L.5 Permafrost sites

For permafrost sites, the geotechnical report should include

a) the lithology, ice content (per National Research Council Canada classification system), and temperature of permafrost;
b) the thickness of the active layer and depth to base of permafrost;
c) the recommended type of foundation;
d) the recommended foundation resting level;
e) the analysis and recommendation on material sensitivity and construction impact related to mechanical and thermal disturbance;
f) the ultimate and serviceability bearing resistances of the recommended foundation for creep (long term) and instantaneous loading;
g) any other applicable information shown under Clause L.2;
h) comments related to climate change and the effect on foundation performance; and
i) the site category for seismic response according to Section 4.1.8.4 of the *NBC*.

Annex M (informative) ***Earthquake-resistant design of telecommunication structures***

Note: *This Annex is not a mandatory part of this Standard.*

M.1 General

It is generally recognized that, in latticed communication towers, wind effects and combinations of wind and ice effects are more likely to govern the design (stability, strength, and serviceability) than are earthquake effects. Although we have reports of towers failing due to extreme wind and/or ice, we have no such reports in connection with earthquakes in Canada. A 1999 survey (Schiff 1999) of earthquake performance of communication towers summarizes documented reports of only 16 instances of tower damage related to seven important earthquakes since 1949, none of which having been a direct threat to life safety. It was concluded from this survey that broadcast towers and large building-supported microwave towers are the most vulnerable types.

These conclusions are confirmed in an update of this survey prepared during the revision of this Annex. The main sources of information are listed in references and comprise earthquake damage reconnaissance reports prepared by various acknowledged expert groups and conference presentations. The period covered is from the 17 August 1999 Kocaeli (Turkey) earthquake until the devastating Sendai earthquake and tsunami of 11 March 2011. Information was gathered on 20 strong earthquakes with moment magnitude $M_w \geq 6.3$ and affecting large populated areas. Devastation has been widespread in several recent earthquakes and if damage resulting from tsunamis are excluded, the effects on telecommunications structures can be summarized as follows:

a) In almost all instances, there were severe disruptions in the cellular telecommunications network and emergency (portable) structures were quickly put in place to restore service.

b) Most outages were due to power disruptions; electric power grids are inevitably disabled by strong earthquakes and power backup systems do not remain operational very long (batteries get discharged and emergency power generators run out of fuel), when they are not themselves damaged by the strong shaking if they are not properly anchored to their racks or supports.

c) When building collapse is widespread, telecommunications shelters are also destroyed and rooftop installations are automatically disabled. This was the case in Chile (27 February 2010, 8.8 M_w), China (Sichuan, 12 May 2008, 7.8 M_w), Kashmir (8 October 2005, 7.6 M_w), Haïti (12 January 2010, 7.0 M_w). While only minor damage was reported on several ground structures in Bam (26 December 2003, 6.6 M_w), several rooftop structures were heavily damaged.

d) Numerous failures of towers on ground (especially cellular towers) were also reported in Sichuan, Kashmir, and Haïti, which is explained by the lack of stringent earthquake-resistant design standards in these regions, and by the very large peak horizontal ground accelerations measured or estimated, in the order of 1.0 g, 0.7 g, and 0.5 g, respectively. The Sichuan earthquake destroyed more than 2800 cellular towers and 16 500 wireless stations. In Kashmir, one broadcast TV tower collapsed and several structures were damaged but still standing.

e) Some ground lattice structures remained standing in devastated areas such as in Gujurat (Bhuj, 26 January 2001, 7.6 M_w) and in Léogâne (Haïti).

f) Several cellular towers were severely damaged (one monopole collapsed) in Christchurch, New Zealand (22 February 2011, 6.3 M_w) where the ground accelerations were exceeding 1.8 g in several areas.

g) It is noteworthy that no structural damage to telecommunications structures on ground has been reported during earthquakes when the peak horizontal ground accelerations were less than 0.7 g.

The 1994 edition of this Standard introduced this Annex as devoted to seismic analysis of towers. Since then, and largely due to documented damages observed after the Kobe earthquake of January 1995 (American Society of Engineers, 1998), there has been an increased interest in North America in establishing earthquake-resistant design guidelines specifically for communication towers. The Federal Emergency Management Administration (FEMA, 1998 and 2000) of the United States had also increased the effort of its National Earthquake Hazard Reduction Program (NEHRP) on risk reduction for so-called non-structural components of buildings and lifeline structures. This increased awareness of seismic risk has, in turn, encouraged the American Electrical and Telecommunication Industries Association (TIA) to formulate mandatory seismic provisions for the first time in ANSI/TIA-222-G in 2006. The American Society of Civil Engineers (ASCE) published, in 2002, *Dynamic Response of Lattice Towers and Guyed Masts*, which devotes a special chapter to seismic input and response.

CAN/CSA-S832 explicitly addresses the design of restraints for equipment installed on building rooftops. Complementary information about seismic design and performance of telecommunication towers on building rooftops can be found in references listed in this Annex.

The 2013 edition of this Standard is a milestone in the evolution of our Canadian practice as it is the first to introduce mandatory earthquake-resistant design procedures for all post-critical telecommunications structures located in regions of moderate to high seismicity.

M.2 Seismicity and earthquake-resistance performance levels

M.2.1 General

Earthquake-resistant design precautions vary depending on the seismicity of the tower location and the performance level defined by the owner for the tower. Geotechnical considerations are an important factor and are addressed in Clause M.6. The discussion and recommendations presented in Clauses M.2 and M.3 are for structures on firm ground.

Uniform seismic hazard maps and corresponding earthquake data for Canada are prescribed by the *National Building Code of Canada.* As introduced in the 2005 edition of the *NBC*, the design earthquake selected corresponds to a probability of exceedance of not more than 2% in 50 years, which is mathematically equivalent to a minimum return period of about 2475 years and a yearly probability of exceedance of 0.0004. The selection of this level of hazard is aimed at providing an acceptable level of life safety for buildings; properly designed structures should resist moderate earthquakes without significant damage and major earthquakes without collapse despite significant permanent deformations in ductile systems. As for buildings, life safety is the first and foremost concern. However, it is not the only performance objective appropriate to telecommunication towers. Depending on the tower's economical value and the function of the structure, the owner should decide on the appropriate performance level for the following three concerns: life safety, interrupted serviceability, and continuous serviceability. However, post-critical telecommunication structures, as defined in Clause 3, needs to achieve the continuous serviceability performance level. This implies that post-critical structures equipped with directional antennas cannot be designed to undergo large permanent deformations (i.e., R = 1 is used for ultimate limit states design). On the other hand, post-critical structures with non-directional telecommunication systems can be designed for limited ductility.

By design, post-critical telecommunication facilities should not have any significant structural stiffness irregularity.

M.2.2 Life safety — Performance level 1 (PL1)

The tower should not collapse in a failure mode that can be a direct threat to life safety. This should apply to all towers located in areas of human occupancy, with special attention paid to towers supported by buildings. This category can also be extended to cover cases where property protection is important and/or failures in domino effects are to be prevented (as in arrays of closely-spaced towers, for example). Life safety is the minimum performance level for all towers located in medium to high seismic zones (see Table M.1).

M.2.3 Interrupted serviceability — Performance level 2 (PL2)

The tower should not sustain damage that will make it unserviceable after the earthquake has occurred, but it is not required to be serviceable during the strong motion. Existing post-critical installations should achieve this performance level.

M.2.4 Continuous serviceability — Performance level 3 (PL3)

The tower should remain fully serviceable during and after the earthquake. This performance level is the most stringent of all three and can be required of towers that are essential structures in a telecommunication network or that are used in the automatic control of electrical utility systems. Note that under large amplitude ground motions, this objective can prove impossible to achieve. In that case, interrupted serviceability will be the next best feasible performance objective. When the tower site is not known, or for generic designs for multiple sites, the purchaser or owner should be assured that the appropriate seismic performance level is provided.

M.2.5 Performance objectives related to seismic hazard

All structures located in high seismicity areas should achieve the minimum life safety performance level (PL1). High seismicity is defined where peak horizontal ground accelerations are in the order of 35% *g* (gravitational acceleration) or more at soil Class C sites, and 5% damped spectral acceleration response values of *S*(0.2), as defined in Sentence 4.1.8.4. of the *NBC*, do not exceed 0.80 *g*.

Those towers located in low seismicity areas need no seismic design precaution. Low seismicity is defined where peak horizontal ground accelerations at soil Class C sites are less than 20% *g*, and 5% damped spectral acceleration response values of *S*(0.2), as defined in Sentence 4.1.8.4 of the *NBC* do not exceed 0.35 *g*.

In moderate seismicity areas, that is, where peak horizontal ground accelerations are in the range of 20 to 40% *g* at soil Class C sites, and the values of *S*(0.2) are in the range of 0.35 to 0.80 *g*, the effects of earthquake loads should be considered in the design of all towers and antenna structures supported on buildings and towers of continuous serviceability (PL3). A seismic design check should be made on towers of interrupted serviceability (PL2) when there is important asymmetry in mass distribution or stiffness irregularity, or for all guyed masts taller than 150 m.

Table M.1 summarizes the above recommendations and indicates the type of analysis (static or dynamic) that is appropriate whenever a seismic design check is necessary, with structural type and height restrictions. Specific analysis procedures are addressed in more details in Clause M.3.

Table M.1
Seismic design check recommendations
(See Clauses 5.12.5.1, 6.6.2, 6.6.3, M.2.2, M.2.5, and M.3.1.)

Seismicity level	Type of structure	Life safety (PL1) and interrupted serviceability (PL2)	Continuous serviceability (PL3)
Low $S_a(0.2)$ $\leq 0.35g$	**All**	No seismic check necessary	Simple static check using the simplified static force procedure (Clause 5.12.5.2)
Moderate $0.35g$ $< S_a(0.2)$ $\leq 0.80\,g$ **and High** $S_a(0.2)$ $> 0.80\,g$	**Building-supported**	**Static check** (See 4.1.8.18 of *NBC* and CAN/CSA-S832)	**Dynamic check** (See 4.1.8.18 of *NBC* and CAN/CSA-S832)
	All ground structures	Static check using the equivalent static force procedure (Clause 5.12.5.3) for all ground structures with $h \leq 50$ m and no significant stiffness or mass irregularity	Static check using the equivalent static force procedure (Clause 5.12.5.3) for all ground structures with $h \leq 50$ m and no significant stiffness or mass irregularity
	Free standing lattice towers	Static check using the equivalent modal analysis procedure for 50 m $< h \leq 150$ m and no significant stiffness or mass irregularity	Static check using the equivalent modal analysis procedure for 50 m $< h \leq 150$ m and no significant stiffness or mass irregularity
		Linear dynamic analysis for $h > 150$ m and shorter towers with significant stiffness or mass irregularity	Linear dynamic analysis for $h > 150$ m and shorter towers with significant stiffness or mass irregularity
Moderate $0.35g$ $< S_a(0.2)$ $\leq 0.80\,g$	Guyed masts	Dynamic check using the simplified seismic analysis procedure for 50 m $< h \leq 150$ m	Dynamic check using linearized seismic analysis procedure for $h > 50$ m
		Dynamic check using linearized seismic analysis procedure for $h > 150$ m	
High $S_a(0.2)$ $> 0.80\,g$	Guyed masts	Dynamic check using the simplified seismic analysis procedure for 50 m $< h \leq 150$ m	Nonlinear dynamic analysis for 50 m $< h \leq 350$ m
		Dynamic check using the linearized seismic analysis procedure for 150 m $< h \leq 350$ m	Nonlinear dynamic analysis with consideration of vertical accelerations and phase lag of ground motion at supports for $h > 350$ m
		Nonlinear dynamic analysis with consideration of vertical accelerations for $h > 350$ m	

M.3 Seismic analysis procedures

M.3.1 General

Structural dynamics principles (Chopra, 1995; Clough and Penzien, 1993) dictate that the seismic sensitivity of a structure is influenced by the coincidence between its dominant natural frequencies and the frequency content of the ground motion. Past earthquake records have typical dominant frequencies in the range of 0.1 to 10 Hz, with a concentration in the 0.3 to 3 Hz range for horizontal motion, while vertical motion involves the higher frequency band.

The first step in the assessment of tower sensitivity to earthquakes is the evaluation of its dominant natural frequencies. This can be done approximately using simplified or empirical expressions available in the literature, as summarized in the American Society of Civil Engineers *Guide* (2002), or in other standards [e.g., AS 3995 or Eurocode 8, Part 3 (EN 1998-3). A more precise evaluation might become necessary only for those towers that will require a detailed dynamic analysis procedure as described below.

The main recommendations of this Clause with regard to the degree of sophistication of seismic analysis are summarized in Table M.1, which lists the minimum requirements specified in Clause 5.12.5.

Dynamic analysis is mandatory for all structures taller than 150 m that are required to perform at PL3.

M.3.2 Self-supporting lattice towers

Self-supporting lattice towers behave as geometrically linear structures and their dynamic response is simple to evaluate using modal superposition with response spectra. In the Canadian industry, short towers with a maximum height of about 50 m are typically structures of high fundamental frequencies and are not significantly affected by earthquakes.

For short lattice towers which are essentially rigid with $T_1 \leq 0.2$ s, Clause 5.12 allows the use of a robust simplified static force procedure (see Clause 5.12.5.2), which assumes that the tower motion will follow the ground so the tower will be subjected to a uniform acceleration equal to the ground acceleration. The force calculated in Clause 5.12.1 can then be compared to the wind design force on the tower or, in effect, to the maximum horizontal load resulting from any other combined effects on the structure to ascertain whether seismic effects deserve more attention. Because the method is very simple, no reduction of the seismic load demand, V, is permitted. This horizontal force V is transmitted to the foundation (base shear) and the overturning moment is obtained by applying the force at the centre of mass of the tower. Since the tower is assumed rigid and moving with the ground, serviceability is not an issue.

For short lattice towers ($h \leq 50$ m) that may have a period $T_1 > 0.2$ s but where the ground shaking is not severe, an equivalent static force procedure is acceptable, as stated in Clause 5.12.5.3. The seismic force V is calculated using Clause 5.12.2. Although force reduction factors, R, larger than unity are allowed for life safety checks, post-critical structures that require continuous serviceability cannot allow permanent deformations. The lateral force V is distributed along the tower height according to an assumed mode shape corresponding to the deflected shape the tower would take if its weight was applied in the horizontal direction. In essence, this method is similar to Method 1 of ANSI/TIA-222 G (Clause 2.7.7).

For lattice towers in the 50 to 150 m range, proportioning for bending and torsional rigidity (serviceability criteria) most often results in structures with natural frequencies lying in the earthquake sensitive range. However, the effects of wind loads or combined wind and ice loads tend to be more critical than earthquake effects, essentially because these climatic loads are amplified with load factors in the analysis, whereas the load factor applicable to the design earthquake is unity. These towers should therefore remain elastic during the design earthquake and should not be of concern unless continuous serviceability is required. For towers without significant irregularities in stiffness or in mass distribution several methods have been proposed in the literature that define a total lateral base shear force that is vertically distributed to be equivalent to a truncated modal analysis procedure.

Khedr (1998) and Khedr and McClure (1999 and 2000) have proposed simple linear expressions (functions of tower mass, fundamental axial or lateral frequency, and corresponding peak ground acceleration) to estimate the base shear and the vertical reaction of these towers. However, base overturning moment is a better indicator of seismic response than base shear for these towers. A simple predictor is suggested in McClure *et al* (2000). Approximate static methods for the prediction of member forces have also been proposed by Gálvez (1995) and Khedr (1998). These methods have not been calibrated to predict displacements or twists and tilts, and are applicable to regular towers for which the lowest three lateral modes and the fundamental axial mode of vibration dominate the response. A procedure similar to these methods is presented as Method 2 in ANSI/TIA-222-G (Clause

2.7.8). In accordance with Clause 5.12.5.4, all of the above are acceptable as a static check, keeping in mind that post-critical structures are assigned a value of *R* equal to unity.

Detailed dynamic analysis of self-supporting lattice structures is recommended for 50 m and taller towers (it is mandatory above 150 m) for PL3 in seismically active zones, and for irregular towers and towers supported by buildings for PL2 and PL3. In the latter case, the earthquake resistance of the supporting building must comply with the requirements of the *NBC*.

Dynamic analysis should be preceded by a frequency analysis where the lowest natural frequencies and mode shapes of the tower are calculated in order to confirm whether a detailed analysis is necessary. Such an analysis should be based on the response spectrum method with enough modes to account for the participation of at least 90% of the effective mass of the tower in the horizontal direction: inclusion of the lowest two axial modes should suffice in the vertical direction (Khedr, 1998). Modal viscous damping ratios should be set between 1 and 3% of critical damping, as suggested in ASCE *Dynamic Response of Lattice Towers and Guyed Masts*, to account for internal structural damping. Aerodynamic damping can be neglected as the amplitudes and projected areas are likely to be too small to introduce significant drag in still air. Since a detailed three-dimensional model of the tower is routinely created for static analysis, one can use the same detailed model to study seismic effects. The tower mass (including its ancillary components and antennas) can be lumped along the main legs, preferably at nodes with adequate lateral support to avoid the effects of spurious localized modes of vibration. Eccentric effects of large antenna masses can be modelled with rigid extensions from the main legs. Linear dynamic analyses can be performed using commercially available software.

M.3.3 Monopoles

Monopoles are typically lightweight cantilever structures with fundamental natural frequencies well above the earthquake sensitive frequency range, and therefore need not be checked for seismic effects. Monopoles with large antenna masses near the top could be sensitive: this possibility can be checked with a quick estimate of their fundamental sway frequency using a prismatic cantilever beam model with lumped mass at the top.

For transverse vibrations, the following approximate expression is obtained:

$$f_{t1} = \frac{1}{2\pi}\sqrt{\frac{3EI_0}{M_{TOT}L^3}}$$

where

EI_0 = flexural rigidity of the monopole at the base

M_{TOT} = lumped antenna masses plus the mass of the monopole and its ancillary components

L = elevation of the centroid of the monopole

If the seismic sensitivity of the monopole is confirmed (with a fundamental frequency lying between 0.3 and 3 Hz), a response spectrum analysis (Chopra, 1995; Clough and Penzien, 1993) using a simple lumped parameter model (elastic spring-mass-viscous dashpot) should suffice to estimate the magnitude of the response to base accelerations. Maximum horizontal displacements should also be checked to evaluate whether additional P-Delta effects need to be accounted for in the evaluation of the overturning moment at the base. Tubular structures are easily amenable to dynamic analysis in commercial software.

M.3.4 Guyed masts

Masts with regular geometry and mass distribution, with heights ranging from 150 to 300 m, typically used by the Canadian industry have their fundamental transverse frequencies within the earthquake sensitive range. However, a limited number of numerical simulations by Amiri (1997) and Dietrich (1999) have shown that seismic effects are not likely to govern the design for PL1 in areas with moderate seismic hazard. The mast itself is relatively lightweight (design wind forces will be greater than gravity forces) and since its mass is more or less linearly distributed over its height, the lateral inertia forces generated by seismic excitations of this distributed mass will not be as significant as the wind forces.

As for short rigid lattice towers, short and regular guyed masts ($h \leq 50$ m) can be checked statically with the robust simplified static force procedure of Clause 5.12.5.2, and the considerations discussed in the second paragraph of Clause M.3.2 also apply.

Few simplified seismic analysis procedures have been proposed for guyed masts. Masts with heights ranging from 50 to 150 m are not strongly nonlinear and equivalent static methods based on models of flexible beams on elastic linear supports can be reasonably accurate for PL1 checks. Such methods are not accurate enough to predict displacements, twists, and tilts; therefore, nonlinear dynamic analysis is the recommended method for all guyed masts taller than 50 m which require PL3.

Ghafari and McClure (2011 and 2012a) have recently proposed a simplified linearized dynamic analysis procedure for masts with height ranging from 150 to 350 m. The method has been calibrated for PL1 checks on elastic structures only, without considerations of vertical acceleration effects.

The most potentially important seismic effects in guyed masts appear to be induced by cable-mast interactions. They can occur when vertical ground motion is combined with the usual horizontal motion, provided that there is frequency coincidence between the input dominant frequencies and the frequencies of dominant strongly coupled cable and mast modes, if such modes exist. Numerical studies have shown that dynamic amplifications in the guy tensions are more likely to be significant in the top and bottom clusters of multi-level guyed masts, and that relatively slack cables (unusual in Canadian practice) with initial tension below 5% of their ultimate tensile strength are vulnerable. Likewise, earthquake effects on the mast itself appear to be significant only in the top cantilevered portion of tall masts and in the first span near the base. It is noted that, in their studies, both Guevara and McClure (1993) and Amiri (1997) have modelled the seismic input as a prescribed acceleration at the ground level. Work by Dietrich (1999) used more rigorous models with simulations based on prescribed components of ground displacements along the three orthogonal directions at each support and with consistent arrival times. This study was later expanded by Faridafshin and McClure (2008), which shows the effect of phase lag of the ground motion for structures with large footprint. Structures at sites with softer soils are more prone to these effects because of their reduced shear wave velocity compared to stiffer soil. As a general guideline, it is recommended to account for effects of vertical accelerations and asynchronous support motions for tall masts with heights taller than 350 m.

All of the above-mentioned studies have shown that detailed nonlinear seismic analyses are far more complex than response spectrum analyses, and not always necessary when checking only PL1 requirements. A frequency analysis, as suggested in Clause M.3.1, for the initial configuration can help to identify the seismic sensitivity of the tower and potential interaction effects due to clustered frequencies. This information will help the designer decide whether it is really necessary to proceed with a more detailed nonlinear time step analysis. A rational simplified expression has been proposed in the Australian Standard AS 3995 to estimate the mast fundamental lateral frequency of guyed towers. Other expressions proposed in the literature [Amiri, 1997; Wahba, 1999; Eurocode 8, Part 3 (EN 1998-3) are empirical formulas that should be used with care as their calibration with real tower performance is incomplete.

Multi-level guyed masts exceeding 150 m in height can exhibit important geometric nonlinearities. Numerical studies reported by Guevara and McClure (1993, 1994), Khala (1994), Mossavi Nejad (1996), Amiri (1997), Madugula *et al* (1998), Wahba (1999), Meshmesha (2005), Hensley and Plaut (2007), and Ghafari Oskoei and McClure (2010, 2011) have all indicated that a reliable dynamic analysis of a guyed tower must allow for geometric nonlinearities and interactions between the mast and the guys. These dynamic interactions can be properly assessed only if the inertia properties of both the mast and the guys are correctly modelled. Several examples are provided in the open literature and more information on the seismic response and modelling of tall guyed towers and seismic input is provided in ASCE *Dynamic Response of Lattice Towers and Guyed Masts*.

M.4 Antennas and ancillary components

Antennas and tower ancillary components should be properly attached to the primary structure. Special considerations should be given to connecting brackets in antenna mounts supporting heavy payloads, especially if torsional effects are anticipated. The inertia effects of the antennas and other attachments can be accounted for by using the lumped mass approach discussed in Clause M.3.2. However, detailed simulations on self-supporting lattice towers done by Khedr (1998) have shown that these inertia effects are not significant unless the additional masses are in the order of more than 5% of the total mass of the primary structure. This observation cannot be extrapolated to tall masts, where the particular location of additional lumped masses with respect to guy clusters also becomes a factor. It should be noted that such masts are not typically carrying heavy additional payloads.

With reference to the survey of telecommunications structures damaged in past earthquakes, damage to antenna mounts and slippage of microwave antennas on their mounts were observed in the Sichuan 2008 earthquake, while in the Chile 2010 earthquake, several structures had antennas falling from their support. As a simple rule, antenna mounts and connections should be designed to resist reversed cycling effects of vertical forces equivalent to the antenna weight, with proper consideration of loading eccentricities.

This Annex does not specifically address the seismic design of attachments and mechanical and electrical components attached to the primary structure. Guidance can be found in ASCE *Methods of Achieving Improved Seismic Performance of Communications Systems* and CAN/CSA-S832. Care should be taken to anchor telecommunication shelters on their base and secure their content with adequate restraints. Precautions are necessary, especially for towers with performance objectives PL2 and PL3 in high seismic hazard zones.

M.5 Additions to existing towers

Additions and alterations to existing towers should have at least the same seismic performance as new towers. The addition or modification should not decrease the seismic resistance of any structural element of the existing tower, unless the reduced resistance is equal to or exceeds that required for new towers.

Existing towers should be verified for compliance with the recommendations of this Annex, in keeping with their performance objective: the minimum life safety objective PL1 should be achieved in all cases.

M.6 Geotechnical considerations

It is essential to obtain a site-specific geotechnical report for all towers 50 m or taller in active seismic areas and in areas with moderate risk if the structure is designed to perform above PL1, the minimum life safety level (see Clause M.2.4).

The geotechnical report should provide the dynamic soil and rock properties (e.g., V_{s30}, the average shear wave velocity down to 30 m depth) in sufficient detail and will define the Site Class in accordance with Section 4.1.8.4 of the *NBC*.

Subsurface investigations should be conducted to sufficient depths to define the engineering properties of all geological units that contribute to site seismic response and potential ground failure.

Special consideration should be given to towers located near an active fault and where there is a risk of soil settlement, liquefaction, foundation sliding, or pile failure due to ground motions.

Several useful considerations for earthquake-resistant design of tower foundations are provided in ANSI/TIA-222-G seismic provisions and in ASCE *Dynamic Response of Lattice Towers and Guyed Masts*. Key issues that are addressed include the need for a sufficient reserve capacity for short-term dynamic loads, the phenomenon of the rocking foundation due to seismic overturning moments in tall self-supporting towers, dynamic pullout forces on guy anchors, and soil-structure interactions.

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Annex N (normative)
Tower dynamic effects and fatigue

Note: *This Annex is a mandatory part of this Standard.*

N.1 General

Free standing or guyed tubular structures (poles) with completely round or semi round shape radomes as well as shrouded tripoles are susceptible to fatigue resulting from natural wind gusts (along wind vibrations) or more seriously from cross wind vibrations (vortex shedding or galloping). Wind induced vibrations can be random or can occur with repetitive constant amplitudes. Cantilevered antenna support pipe mounts on towers and rooftops can equally be subjectile to fatigue.

N.2 Wind dynamic Effects

N.2.1 General

Monopoles with external attachments exceeding 10% of the effective projected area of the pole at the top third of the pole height shall be checked against the dynamic effects of wind gusts in accordance with Clause N.2.2 only.

Monopoles with external attachments less than 10% of the effective projected area of the pole at the top third of the pole height and circular or semicircular shrouded monopoles with shroud height to diameter ratio exceeding 3 shall be checked against the dynamic effects of wind gusts as shown in Clause N.2.2 and vortex shedding as shown in Clause N.2.3.

Shrouded tripoles with moment connections (Verndeel type truss) and height to width of shroud exceeding 3 shall be checked against the dynamic effects of wind gusts as shown in Clause N.2.2 and galloping as shown in Clause N.2.4.

Latticed shrouded tripoles shall be checked against galloping in accordance with Clause N.2.4.

N.2.2 Wind gusts

The response of the structure to the static and fluctuating wind components (natural wind gusts) shall be assessed in accordance with the following formula:

$$q_{wg} = \sqrt{\frac{0.005}{\zeta_s}}\ \frac{q_{10}}{2}$$ **Equation N.1**

where

q_{10} = 10-year return period mean hourly wind pressure at 10 m above ground level, $300 \le q_{10} \le 390$

ζ_s = structural damping ratio, when tower specific experimentally determined values are unavailable, the structural damping ratio ζ_s of 0.3% is to be used

The structure is analyzed using q_{wg} determined from Equation N.1 using the appropriate height factor, C_e, and a gust factor C_g = 1.0 and the applicable importance factor, I, for the structure using a wind load factor, α_W of 1.0 and without any dead loads.

The stresses resulting from the analysis are further compared against the constant amplitude threshold stress range, F_{srt}, in accordance with Clause N.3.

N.2.3 Vortex shedding

The critical wind speed, V_{crit} (in m/s), when this resonant condition occurs may be predicted by the expression

$$V_{cr} = \frac{1}{S} f_n D_H$$ **Equation N.2**

where

S = the Strouhal number

f_n = the natural frequency of the structure, Hz

D_H = the diameter (or width) of the structure, m

For circular, near-circular, and cylinders, the Strouhal number is a function of Reynolds number, *Re*, and estimated by the following:

If $Re < 2 \times 10^5$ then $S = 1/6$ **Equation N.3a**

If $Re > 2.5 \times 10^5$ then $S = 1/5$ **Equation N.3b**

where

$$Re = 67\,000 V_{cr} D_H$$ **Equation N.4**

Linear interpolation may be used for the intermediate region between the cases covered by Equations N.3a and N.3b. For cantilevered structures, the critical wind velocity is typically defined at the top of the structure (denoted here as V_{cr}).

For vortex shedding response, the analysis method shown in Annex A of CSA S6 or Eurocode 1, Part 1-4 may be used.

When the suspect mode of vibration under vortex shedding is limited to the fundamental mode of sway only, the dynamic response may be approximated by applying a uniformly distributed static wind force, F_{vs} (in N/m), acting over the top third of the structure.

$$F_{vs} = \frac{C_1}{\sqrt{\lambda}\sqrt{\zeta_s - \zeta_a}} q_H D_H$$ **Equation N.5**

where

λ = the aspect ratio of the structure (H/D)

ζ_s = the structural (mechanical) damping ratio, expressed as a fraction of critical damping

ζ_a = the negative aerodynamic damping associated with vortex shedding

q_H = the wind pressure (in Pa) corresponding to V_{cr}

D_H = the diameter (in m) averaged over the top third of the structure

Where the actual structural damping is unknown, or when tower specific experimentally determined values are unavailable, the structural damping ratio ζ_s of 0.30% shall be used.

For smooth cylinders undergoing vortex-induced vibrations, negative aeroelastic damping might develop that can potentially lead to unstable response. This aerodynamic damping ratio, ζ_a, for the fundamental sway mode, may be estimated using the expression:

$$\zeta_a = C_2 \frac{\rho_a D_H^2}{m_H}$$ **Equation N.6**

where

ρ_a = the air density (approximately 1.27 kg/m^3)

m_H = mass per unit length (in kg/m), averaged over the top one-third of the structure

For most situations, the constants C_1 and C_2 may be approximated as follows:

$$C_1 = 3 \text{ if } \lambda > 16$$ **Equation N.7a**

or

$$C_1 = 0.75\lambda^{0.5} \text{ if } \lambda < 16$$

$$C_2 = 0.6$$ **Equation N.7b**

For slender structures in conditions likely to generate low levels of turbulence, the vortex shedding response can be significantly enhanced. If V_{Hc} < 10 m/s and λ > 12, then the following values shall be used:

$$C_1 = 6$$ **Equation N.7c**

$$C_2 = 1.2$$ **Equation N.7d**

Stresses resulting from vortex shedding oscillation amplitudes are compared to the fatigue strength of the connection in accordance with Clause N.3.

The number of cycles expected during the 50 year design life of the structure may be used to determine the Fatigue stress range, F_{Sr} for the connection. The number of cycles, N, that the structure is subjected to V_{cr} is estimated from the following:

Equation N.8

$$N = 3.15 \times 10^7\, T\, f_n \left(\frac{2}{V_o}\right)\left(\frac{V_{cr}}{V_o}\right) e^{-\left(\frac{V_{cr}}{V_o}\right)^2}$$

where

T = expected design lifetime of structure in years, 50 years for new structure (years)

f_n = natural frequency of structure at resonant mode (Hz)

V_{cr} = critical wind speed at which resonant conditions are predicted (m/s)

V_o = 20% of the design wind speed at the height of the cross section where vortex shedding occurs

= $0.2 \times 1.25 \times \sqrt{qC_e}$ (m/s)

The likelihood of large amplitude oscillations due to vortex shedding may also be assessed on the basis of the Scruton number, S_c:

$$S_c = \frac{4\pi m_H \zeta_s}{\rho_a D_H^2}$$ **Equation N.9**

In general, Scruton number values in excess of 10 signify low vibration amplitudes.

N.2.4 Galloping

Shrouded triangular structures becomes dynamically unstable and starts galloping when (see Figure N.1)

$$C_3 = \frac{dC_L}{d\alpha} + C_D < 0$$ **Equation N.10**

where

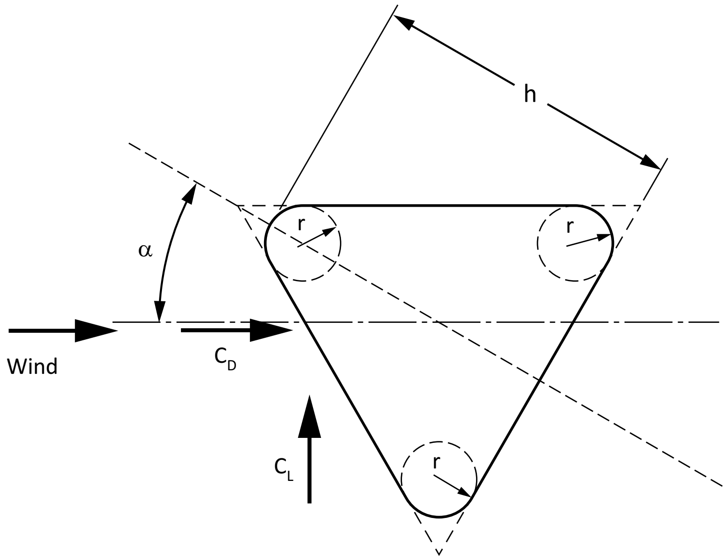
$\frac{dC_L}{d\alpha}$ = the change of the static lift coefficient with respect to the change in the angle of attack from wind

C_L = the static lift coefficient

C_D = the static drag coefficient

α = the angle of attack from wind

Figure N.1
Galloping
(See Clause N.2.4.)



Shrouded tri-poles, where the ratio of the length to width of the shroud exceeds 3, shall be investigated for galloping susceptibility. The onset wind speed at which galloping will start, V_{OG} shall be determined from the following:

$$V_{OG} = \frac{2\,S_C}{C_3}\,f_n\,D_H$$ **Equation N.11**

where

S_C = Scruton number as per Equation N.9

C_3 = –1.0 for sharp corners triangular shaped structures
–1.5 for rounded corners with ratio r/h = 0.04
–1.75 for rounded corners with ratio r/h = 0.08
–2.0 for rounded corners with ratio r/h = 0.12

f_n = natural frequency of the structure

D_H = face width of the structure

Wind tunnel tests can be used to determine accurate values of C_3 for different shaped structures.

Galloping oscillation starts at the onset wind velocity for galloping, V_{OG} and normally the amplitudes increase rapidly with increasing wind velocity. Structures shall be designed with a V_{OG} to be greater than the design wind speed, 1/50 hourly mean wind speed, at the mid height of the top section of shroud where galloping occurs.

N.3 Fatigue

N.3.1 General

Poles and shrouded tripoles components including welded connections, bolts, anchor rods, and porthole openings shall be proportioned so that their fatigue capacities are equal to or greater than the effects of the stress fluctuations as calculated in accordance with Clause N.3.2.

$$\alpha_F \ \Delta F \leq \phi \ F_{sr}$$ **Equation N.12**

where

α_F = 1.0

ΔF = wind load induced stress range as per Clauses N.2.2 and N.2.3 if applicable

ϕ = 1.0

F_{sr} = fatigue resistance stress range for that specific connection as per Clause N.3.2

N.3.2 Fatigue resistance

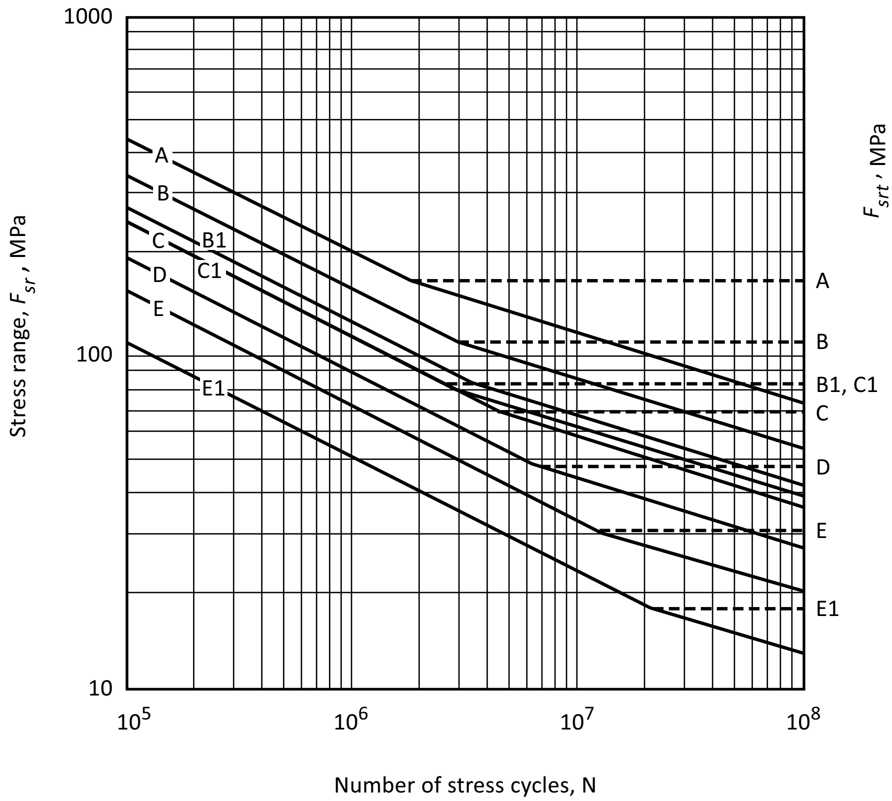
The nominal fatigue resistance stress range, F_{sr} ,shall be calculated as shown in Figure N.2 and as described in Clause N.3.3, using ordinary elastic analysis and the principles of mechanics of materials and then multiplied by a stress concentration factor. The stress concentration factor shall have a value of 1.0 unless otherwise noted in Table N.3.

The nominal fatigue resistance stress range, F_{sr}, shall be the constant amplitude threshold range, F_{srt}, for the fatigue detail category of the member or detail. The constant amplitude threshold stress range shall be as given in Table N.1 for steel members and details.

Table N.1
Constant amplitude threshold stress ranges for various fatigue detail categories (A to E1)
(See Clause N.3.2.)

Detail category	Constant amplitude threshold stress range, Fsrt, MPa
A	165
B	110
B1	83
C	69
C1	83
D	48
E	31
E1	18

Figure N.2
Stress range vs. number of cycles for various detail categories (A to E1)
(See Clause N.3.2)



For fatigue details 4.4 to 4.7 and 5.1, shown on Table N.2, the fatigue capacity shall be determined based on the infinite life stress concentration factor value – K_I, where

$$K_I = [(1.76 + 0.072t_T) - 4.76 \times 0.22^{K_F}]K_F$$

where

t_T = thickness of tube (mm)

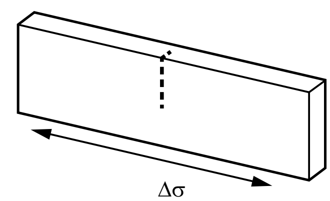
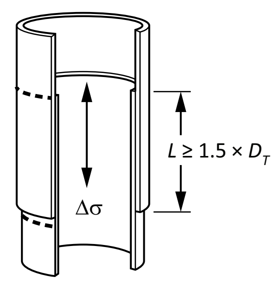
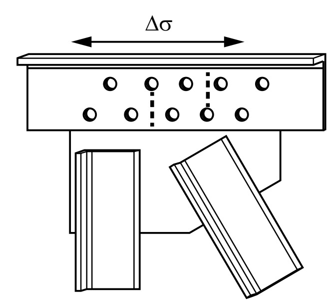
K_F = shall be calculated from Table N.3 for the respective details

N.3.3 Fatigue detail categories

The fatigue detail categories for telecommunication towers components and details shall be as shown in Table N.2.

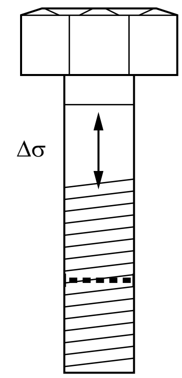
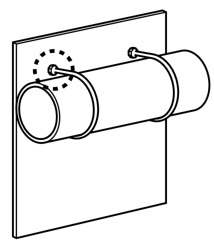
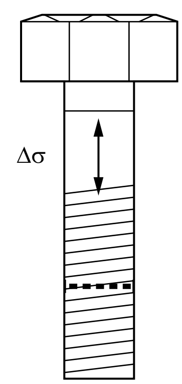
Table N.2 identifies the potential location of cracking for each fatigue detail. “Longitudinal” implies that the direction of applied stress is parallel to the longitudinal axis of the detail and “Transverse” implies that the direction of applied stress is perpendicular to the longitudinal axis of the detail.

Table N.2
Fatigue details for telecommunication towers
(See Clause N.3.3.)

Description	Fatigue detail category	Potential crack location	Example
Section 1 — Plain material			
1.1 – With rolled or cleaned surfaces. Flame-cut edges with smoothness of 1000 μ-in or less	A	Away from all welds or structural connections	
1.2 – Slip-joint splice where L is greater than or equal to 1.5 diameters	B	In a section at the edge of tube splice	
Section 2 — Mechanical fasteners			
2.1 – Net section of fully-tightened, high-strength (ASTM A325, A490) bolted connections	B	In the net section originating at the side of the hole	Bolted joints 
2.2 – Anchor bolts or other fully-tensioned high strength bolts in tension not subject to bending or prying forces; stress range based on the tensile stress area	C	At the root of the threads extending into the tensile stress area	Anchor bolts Flanged connections bolts

(Continued)

Table N.2 (Continued)

Description	Fatigue detail category	Potential crack location	Example
			
2.3 – Connection of members or attachment of antenna mounts, etc. with clamps or U-bolts Anchor bolts or other fasteners in tension; stress range based on the tensile stress area. Misalignments of less than 1:40 with firm contact between the anchor bolts, washers and base plate	D	At the root of the threads extending into the tensile stress area	 
Section 3 — Holes and cutouts			
3.1 – Net section of unreinforced holes and cutouts	A	In tube wall at edge of unreinforced hole	Wire outlet holes, drainage holes, hand holes

(Continued)

Table N.2 (Continued)

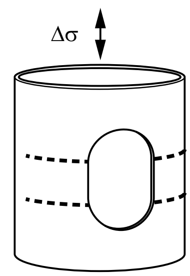
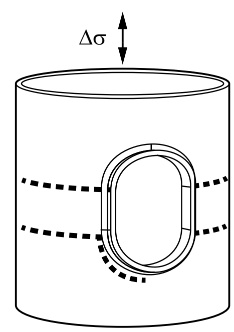
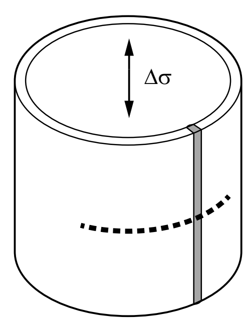
Description	Fatigue detail category	Potential crack location	Example
			
3.2 – Reinforced holes and cutouts:			
a) At root of reinforcement-to-tube weld	B	In tube wall and hole reinforcement from root of reinforcement-to-tube weld	
b) At toe of reinforcement-to-tube weld	D	In tube wall and hole reinforcement from toe of reinforcement-to-tube weld	Reinforced hand holes 
Section 4 — Groove welded connections			
4.1 – Tubes with continuous full or partial penetration groove welds parallel to the direction of the applied stress	B1	In the weld away from the weld termination	Longitudinal seam weld 

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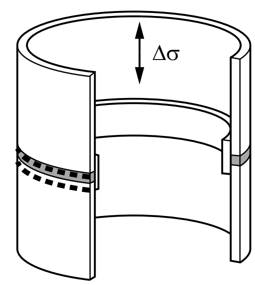
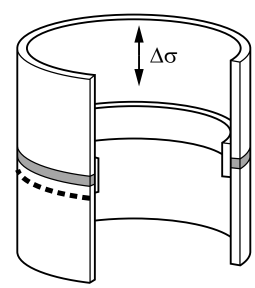
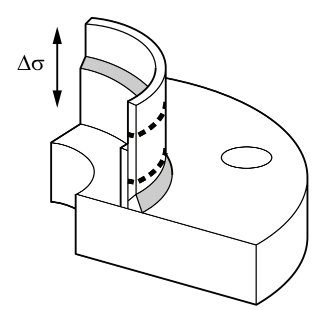
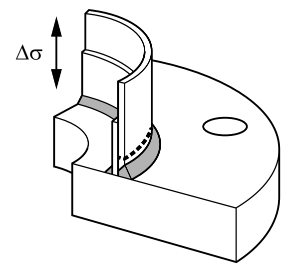
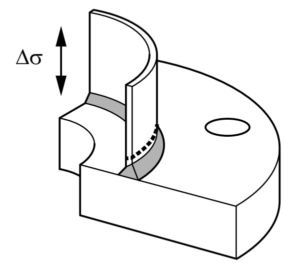
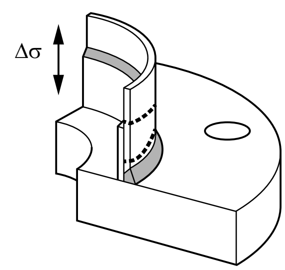
Table N.2 (Continued)

Description	Fatigue detail category	Potential crack location	Example
4.2 – Full penetration groove-welded splices with welds ground to provide a smooth transition between members (with or without backing ring)	D	In weld through the throat or along the fusion boundary	Column butt-splices 
4.3 – Full penetration groove-welded splices with weld reinforcement not removed (with or without backing ring)	E	In tube wall along weld toe	Column butt-splices 
4.4 – Full penetration groove-welded tube to transverse plate connections with backing ring attached to the plate with a full penetration weld, or with a continuous fillet weld around interior face of backing ring, and the backing ring welded to the tube with a continuous fillet weld at top face of backing ring	$K_I \leq 3.2 \rightarrow$ C $3.2 < K_I \leq 5.1 \rightarrow$ D $5.1 < K_I \leq 7.2 \rightarrow$ E	In tube wall along groove weld toe or backing ring top weld toe	Column to base plate connection 
4.5 – Full penetration groove-welded tube to transverse plate connections with backing ring attached to the plate with a full penetration weld, or with a continuous fillet weld around interior	$K_I \leq 3.2 \rightarrow$ C $3.2 < K_I \leq 5.1 \rightarrow$ D $5.1 < K_I \leq 7.2 \rightarrow$ E	In tube wall along groove weld toe	Column to base plate connection

(Continued)

Table N.2 (Continued)

Description	Fatigue detail category	Potential crack location	Example
face of backing ring, and the backing ring not welded to the tube at top face of backing ring			
4.6 – Full penetration groove welded tube to transverse plate connections welded from both sides with back gouging (without backing ring)	$K_I \leq 3.2 \rightarrow$ C $3.2 < K_I \leq 5.1 \rightarrow$ D $5.1 < K_I \leq 7.2 \rightarrow$ E	In tube wall along groove weld toe	Column to base plate connection 
4.7 – Full penetration groove-welded tube to transverse plate connections with backing ring not attached to the base plate, and the backing ring welded to the tube with a continuous fillet weld at the top face of backing ring	$K_I \leq 3.2 \rightarrow$ C $3.2 < K_I \leq 5.1 \rightarrow$ D $5.1 < K_I \leq 7.2 \rightarrow$ E	In tube wall along groove weld toe or backing ting top weld toe	Column to base plate connection 
Section 5 — Fillet welded connections			
5.1 – Fillet welded tube to transverse plate connection	$K_I \leq 4.0 \rightarrow$ D $4.0 < K_I \leq 6.5 \rightarrow$ E $6.5 < K_I \leq 7.2 \rightarrow$ E1	In tube wall along fillet weld toe	Column to base plate connection

(Continued)

Table N.2 (Continued)

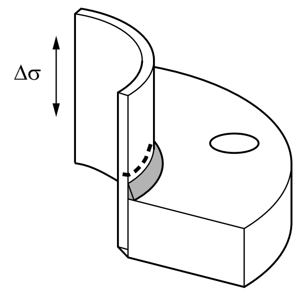
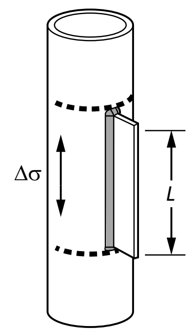
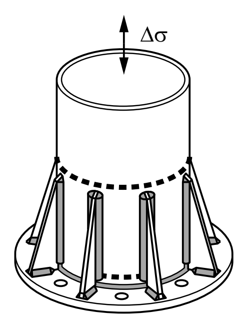
Description	Fatigue detail category	Potential crack location	Example
			
Section 6 — Attachments			
6.1 – Longitudinal attachments with partial or full penetration groove welds, or fillet welds, in which the main member is subjected to longitudinal loading: $L < 51$ mm $51 \text{ mm} \le L$	 C D	In the main member at the toe of the weld at the termination of attachment	
6.2 – Tube to transverse plate connections stiffened by longitudinal attachments with partial or full penetration groove welds, or fillet welds in which the tube is subjected to longitudinal loading and the welds are wrapped around the attachment termination	$K_I \le 5.5 \rightarrow$ D	In tube wall at the toe of the attachment to tube weld at the termination of the attachment In tube wall at the toe of tube to transverse plate weld	Longitudinal stiffened base plate 

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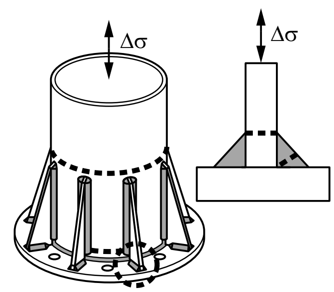
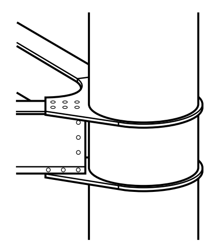
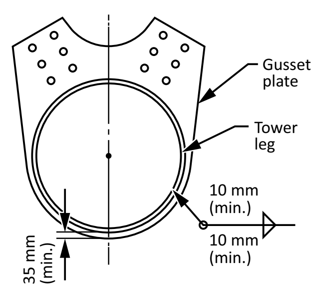
Table N.2 (Continued)

Description	Fatigue detail category	Potential crack location	Example
6.3 – Transverse load bearing partial joint penetration groove welded or fillet welded attachments where *t* ≤ 13 mm and the main member is subjected to minimal axial and/or flexural loads [where *t* > 13 mm (see Notes 1 to 5)]	C		Longitudinal stiffened base plate 
6.4 – Fillet weld ring stiffened connection	See Note 6)		 

Notes:

1) *When t > 13 mm F_{srt} shall be the lesser of 69 MPa or the following:*

$$F_{srt} = 69 \times \left[\frac{0.094 + 1.23\frac{H}{t_p}}{t_p^{\frac{1}{6}}} \right] \text{MPa}$$

where

H = the effective weld throat (mm)

t_p = the attachment plate thickness (mm)

2) *The stiffener thickness shall be greater than the tube thickness but not exceed 1.25 times the tube wall thickness.*

(Continued)

Table N.2 (Concluded)

3) *Stiffeners shall be placed symmetrically with respect to bolts and spacing between stiffeners shall not exceed 400 mm. A minimum of 6 stiffeners shall be used.*
4) *Stiffeners shall be tapered with a termination angle of 15° and a minimum height of 300 mm.*
5) *Wrap-around weld at stiffener terminations shall not be ground.*
6) *The ring stiffened connection shall be used for infinite life.*
7) *For fatigue detail category stresses refer to Table N.1 and Figure N.2.*

Table N.3
Fatigue stress concentration factors, K_F
(See Clause N.3.2.)

Section type	Detail (see Table N.2)	Location	Fatigue stress concentration factor for finite life, K_F	
Round	Fillet welded tube to transverse plate connections (Detail: 5.1)	Fillet weld toe on tube wall	$K_F = 2.16 + 207 \times \left(9.08 - 9.24\dfrac{C_{BC}{}^{0.0474}}{N_B{}^{0.0105}}\right) \times (4.54 + 2.05 \times t_T) \times \left(14.6 - \dfrac{D_T{}^{1.15}}{35.3}\right) \times t_{TP}{}^{-2.36}$ Valid for: 5 mm ≤ t_T ≤ 13 mm 203 mm ≤ D_T ≤ 1270 mm 51 mm ≤ t_{TP} ≤ 102 mm 1.25 ≤ C_{BC} ≤ 2.5 $K_F = \left(\dfrac{.011 \times \frac{D_{BC}}{N_B} + 26}{\frac{t_{TP}}{t_T}}\right)^{\left(\frac{1}{7.6-5.1\,C_{BC}}\right)}$ Valid for: 5 mm ≤ t_T ≤ 19 mm 300 mm ≤ D_T ≤ 1830 mm 38 mm ≤ t_{TP} ≤ 102 mm 1.04 ≤ C_{BC} ≤ 1.25	N3-1
	Groove welded tube to transverse plate connections (Details: 4.4 to 4.7)	Groove weld toe on tube wall	$K_F = 1.35 + \left(2.63 - 2.68\dfrac{C_{BC}{}^{0.0674}}{N_B{}^{0.0029}}\right) \times (12.1 + 8.24 \times t_T) \times \left(44.1 - \dfrac{D_T{}^{1.12}}{4.93}\right) \times \left(\dfrac{1.0}{C_{OP}{}^{-0.689} - 0.764}\right) \times t_{TP}{}^{-1.95}$ Valid for: 5 mm ≤ t_T ≤ 16 mm 203 mm ≤ D_T ≤ 1270 mm 51 mm ≤ t_{TP} ≤ 102 mm 1.25 ≤ C_{BC} ≤ 2.5 0.3 ≤ C_{OP} ≤ 0.9	N3-2

(Continued)

Table N.3 (Continued)

Section type	Detail (see Table N.2)	Location	Fatigue stress concentration factor for finite life, K_F	
	Fillet welded tube to transverse plate connections stiffened by longitudinal attach-ments (Detail: 6.2)	Weld toe on tube wall at the end of attachment	$K_F = \left(\frac{\frac{t_{ST}}{t_T^{0.334}} - 1.98}{1.73\frac{t_{ST}}{t_T^{0.707}}}\right) \times \left(\frac{0.160 + \frac{h_{ST}}{29.4}}{1.0 + \frac{h_{ST}}{22.7}}\right) \times \left(0.519 + \frac{D_T}{N_{ST}^{1.60}\, t_T^{1.42}} + \frac{11.5}{t_T^{0.797}} + \frac{359}{t_{ST}^{2.91}}\right)$ Valid for: 6 mm ≤ t_{ST} ≤ 22 mm 6 ≤ N_{ST} ≤ 12 8 mm ≤ t_T ≤ 16 mm 610 mm ≤ D_T ≤ 1270 mm	N3-3
	Fillet welded tube to transverse plate connections stiffened by longitudinal attach-ments (Detail: 6.2)	Fillet weld toe on tube wall	$K_F = \left[\left(9.84 - \frac{D_T}{46.3} + \frac{D_T^{1.03}}{5.71 \times N_{ST}^{0.914}}\right) \times \left(\frac{1.0}{h_{ST} + 167}\right) \times \left(1.0 + \frac{25.0}{t_{ST}^{0.631}}\right) \times \left(2.26 + \frac{t_{TP}}{117}\right) \times x\ K_F \text{ as per Equation N3-1}\right]$ Valid for: 305 mm ≤ h_{ST} ≤ 1067 mm 6 mm ≤ t_{ST} ≤ 22 6 ≤ N_{ST} ≤ 12 610 mm ≤ D_T ≤ 1270 mm	N3-4
Multisided	As above (Details: 4.4 to 4.7, 6.2)	As above	$\left[\left(0.186 - \frac{t_T}{227}\right) \times \left(1.97 + \frac{D_T^{2.44}}{2.22 \times 10^6}\right) \times e^{\left(\frac{0.028}{R-1.01}\right)} + 0.923\right]$ Valid for: 203 mm ≤ D_T ≤ 1270 mm 25 mm ≤ r_b ≤ 102 6 ≤ N_{ST} ≤ 12	N3-5

where

C_{BC} = $\frac{D_{BC}}{D_T}$

C_{OP} = $\frac{D_{OP}}{D_T}$

D_{BC} = diameter of circle through the fasteners in the transverse plate, mm

D_{OP} = diameter of concentric opening in the fasteners in the transverse plate, mm

D_T = external diameter of tube or outer opposite to flat distance of multi-sided at transverse plate, mm

h_{ST} = height of longitudinal attachment (stiffener)

N_B = number of fasteners in the transverse plate

N_S = number of sides

(Continued)

Table N.3 (Concluded)

N_{ST} = number of longitudinal attachments (stiffeners)

$$R = \frac{Cos\left(\frac{\pi}{N_s}\right)}{1.0 - \frac{r_b}{R_n}\left(1.0 - \cos\left(\frac{\pi}{N_s}\right)\right)}$$

r_b = internal bend radius of the corners

R_n = radius of inner inscribed circle

t_{ST} = thickness of longitudinal attachments (stiffeners), mm

t_T = thickness of tube, mm

t_{TP} = thickness of transverse plate, mm

N.4 References

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Annex P (informative)
Fabrication tolerances for tubular pole structures

Note: *This Annex is not a mandatory part of this Standard.*

P.1 General

There is currently no published standard for fabrication tolerances of tubular pole-type structures. It has been left to the purchaser to negotiate these requirements with the fabricator. There is a need to provide some guidelines for these negotiations. This Annex sets out some of the critical values that apply to these structures. Where values have not been given, standard tolerances for structural steel as provided by CSA S16 and the CISC *Handbook of Steel Construction* should be used.

P.2 Pole sections

The values for pole sections are

a) length: ± 12 mm;
b) diameter (except at slip joint): ± 6 mm;
c) straightness: ± 6 mm in 3 m; and
d) twist: ± 5° in 12 m.

P.3 Slip joints

The values for slip joints are

a) inside diameter of outside section: + 3 mm, –2 mm;
b) outside diameter of inside section: + 2 mm, –3 mm; and
c) squareness of measured diameter: ±6 mm.

P.4 Assembled poles

The values for assembled poles are

a) length: + 300 mm, –150 mm;
b) diameter: ± 6 mm;
c) straightness: ± 60 mm in 30 m;
d) pre-camber: 25 mm in 150 mm of camber;
e) Splice length: + 10%, – 0%; and
f) twist: ± 3° in 12 m.

P.5 Welding

Tolerances should be in accordance with CSA W59.

Note: *Poles with slip fit splices are susceptible to creep or shortening at the joints due to self-weight and load effects. Attention should be paid to the design of linear attachments spanning the joints.*

Annex Q (informative)
Properties of Schifflerized (60°) steel angles

Note: *This Annex is not a mandatory part of this Standard.*

Q.1 General

The following formulas are provided to assist the designer with determining the geometrical properties of 60° steel angles formed from 90° angles, the process known as Schifflerizing. These formulas are based on the work of Adluri and Madugula at the University of Windsor.

If it is necessary to calculate the warping constant, C_w, see Adluri and Madugula (1991b). For other properties and greater detail, see also Adluri and Madugula (1991a).

Q.2 Geometric properties

The important geometric properties of the Schifflerized angle section are given below. Details of derivation are available in Adluri and Madugula (1991b):

a) $A = 2t(a + b - t / 2)$, where A is the cross-sectional area

b) $\bar{u}_c = \frac{2(a - t/2)^2 + 4b(a - t/2) + \sqrt{6}b^2}{4\sqrt{2}(a + b - t/2)} + \frac{t}{\sqrt{2}}$

c) The distance of the shear centre from the centroid is given by

$\bar{u}_s = \frac{(\sqrt{3} - 1)(3c^2b^2 + \sqrt{2}cb^3)}{4c^3 + 12c^2b + 6\sqrt{2}cb^2 + 2b^3} - \frac{t}{\sqrt{2}} + \bar{u}_c$

where $c = a - t / 2$

d) $W = a + b - t - c / 2$

e) $I_u = 2(I_{u1} + I_{u2})$

where

$$I_{u1} = \frac{3bt^3}{48} + \frac{b^3t}{48} + bt\left(\frac{b}{4} + \frac{a - t/2}{\sqrt{2}}\right)^2$$

and

$$I_{u2} = \frac{(a - t/2)^3}{24} + \frac{t(a - t/2)^3}{6}$$

f) $I_z = 2(I_{z1} + I_{z2}) - A(\bar{u}_c - t / \sqrt{2})^2$

where

$$I_{z1} = \frac{bt^3}{48} + \frac{3b^3t}{48} + bt\left(\frac{a - t/2}{\sqrt{2}} + \sqrt{3}\frac{b}{4}\right)^2$$

and

$$I_{z2} = \frac{(a - t/2)t^3}{24} + \frac{t(a - t/2)^3}{6}$$

g) $I_{pc} = I_u + I_z$

h) $I_x = 0.25 \times I_u + 0.75 \times I_z$

i) $\bar{I}_y = 0.25 \times I_z + 0.75 \times I_u$

j) $I_{ps} = I_{pc} + A\bar{u}_s^2$

k) $J = \frac{2}{3}(a + b - t/2)t^3$

l) $\bar{r}_x = \left(\frac{I_x}{A}\right)^{\frac{1}{2}}$

m) $\bar{r}_y = \left(\frac{I_y}{A}\right)^{\frac{1}{2}}$

n) $r_u = \left(\frac{I_u}{A}\right)^{\frac{1}{2}}$

o) $r_z = \left(\frac{I_z}{A}\right)^{\frac{1}{2}}$

where

a = length from the heel to the bend line of Schifflerized angle leg

b = length from bend line to toe of Schifflerized angle leg

c = fillet radius

C_w = warping constant

I_{ps} = polar moment of inertia about shear centre

I_u = maximum moment of inertia (about u - u axis)

I_z = minimum moment of inertia (about z - z axis)

I_x = moment of inertia (about x-x axis)

I_y = moment of inertia (about y-y axis)

J = Saint-Venant's torsion constant

r_x = radius of gyration for out of the plane of the face buckling

r_y = radius of gyration for in the plane of the face buckling

r_z = minimum radius of gyration (about z-z axis)

r_u = maximum radius of gyration (about u-u axis)

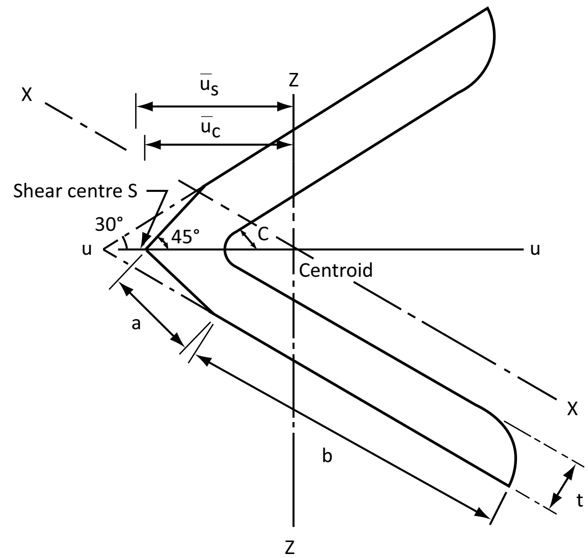
t = thickness of the leg of Schifflerized angle member

$\bar{u}_c$ = distance of the centroid from heel

$\bar{u}_s$ = distance between the centroid and shear centre

w = flat width of Schifflerized angle leg (dimension *b* in Figure Q.1)

Figure Q.1
Typical cross-section of a Schifflerized angle
(See Clause Q.2.)



Q.3 References

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Annex R (informative)
Climber attachment anchorages

Note: *This Annex is not a mandatory part of this Standard.*

R.1 Climber attachment anchorages

This Annex provides designers with examples of suitable climber attachment anchors (refer to Figures R.1 and R.2). Any person using these anchorages should use caution and ensure that attachments are made to sound members that do not exhibit signs of damage and/or excess corrosion.

Only qualified persons (as defined in Clause 16.10) using equipment specified in the CSA Z259 series of standards may climb.

Note: *There are many industry related working documents produced by the Structure, Tower and Antenna Council (STAC) and others that can help a climber decide on proper attachment points and procedures.*

Figure R.1
Designer example all-weld tower
(See Clause R.1.)

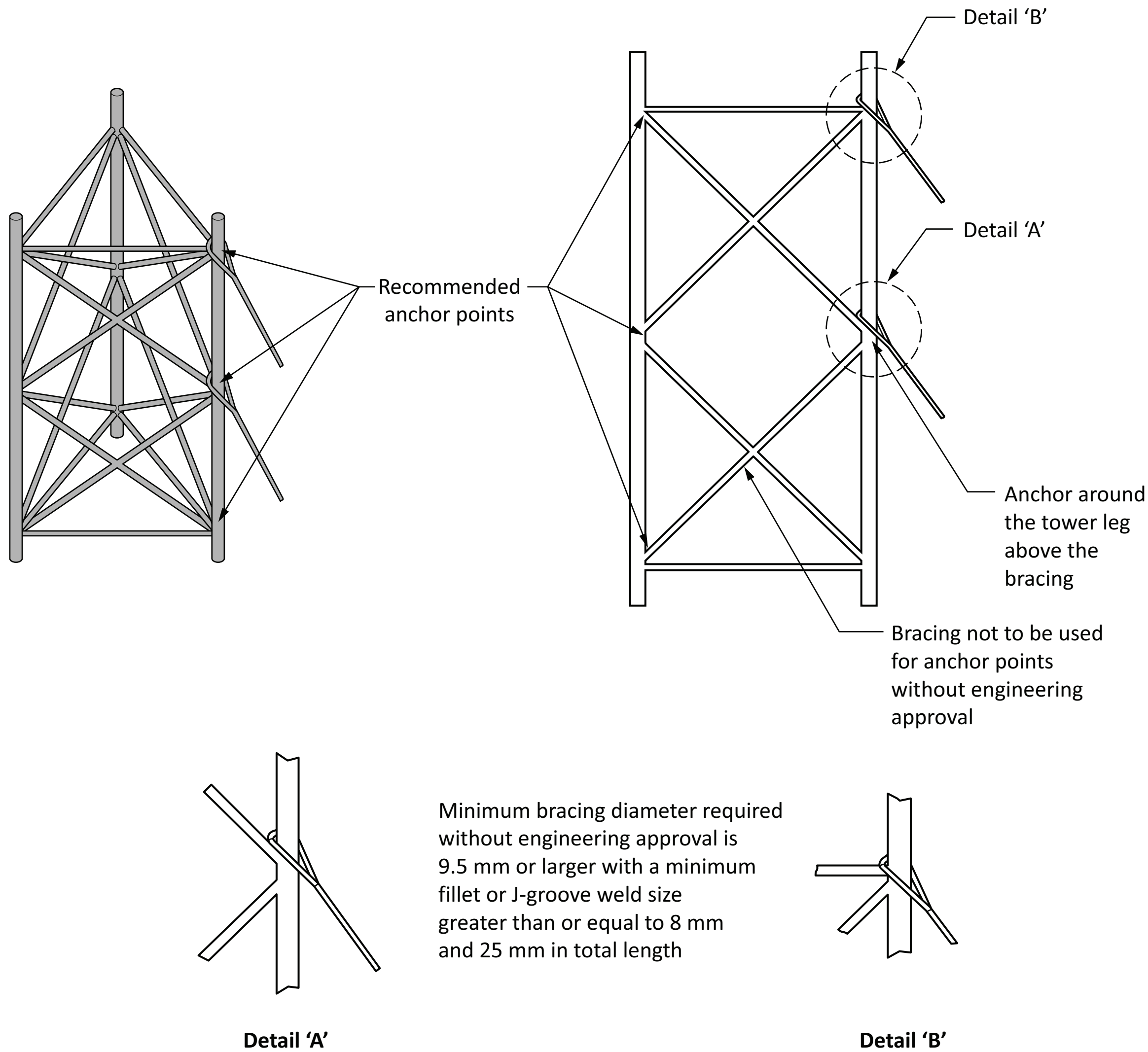
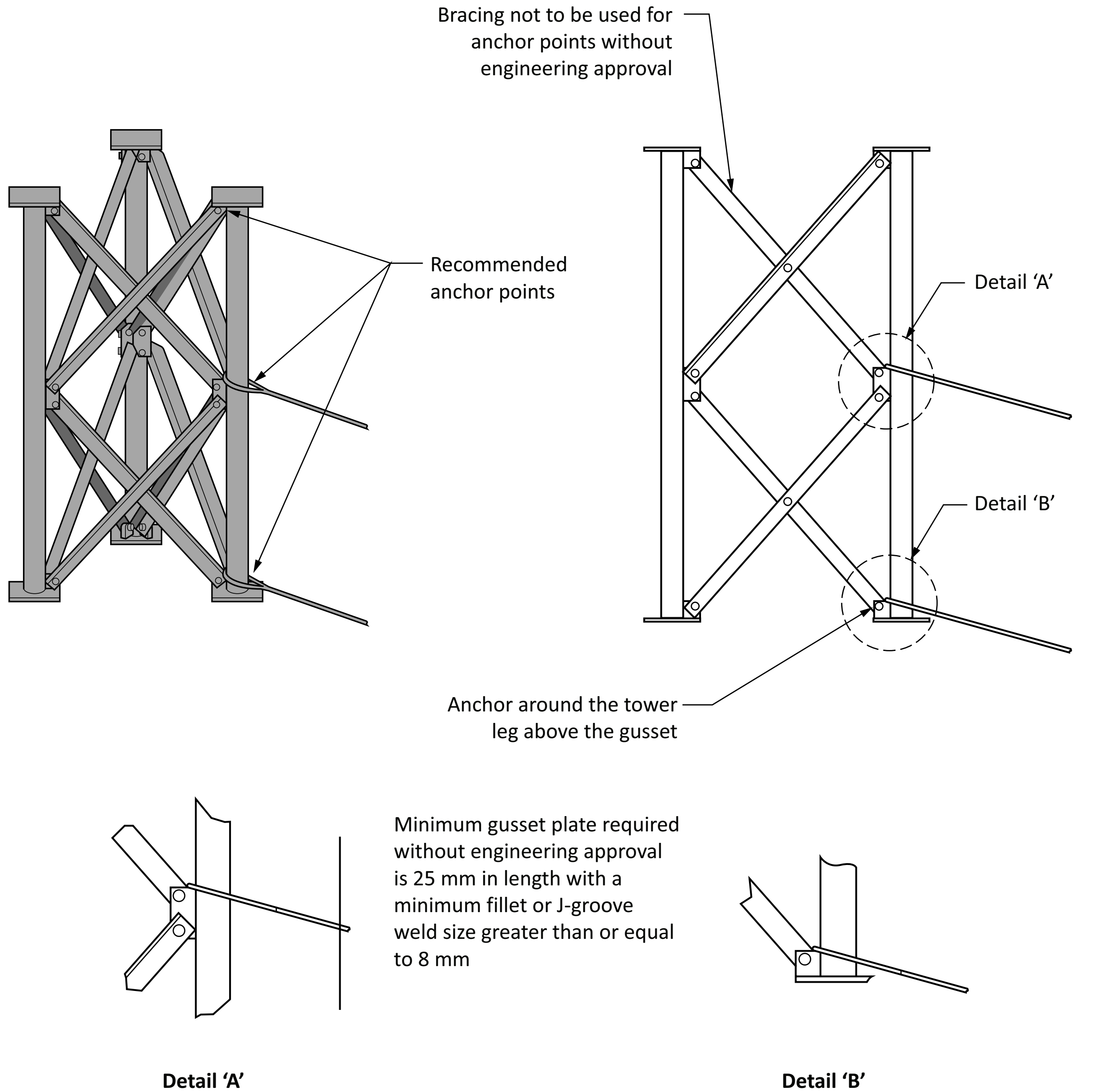


Figure R.2
Designer example knock-down tower
(See Clause R.1.)



Annex S (informative) ***Commentary on CAN/CSA-S37-18***

Note: *This Annex is not a mandatory part of this Standard.*

Introduction

This Commentary sets out the background of CAN/CSA-S37-18, with the reasons and sources of data for its clauses, so that users can better understand their intent. Clauses receiving no comment are considered to be self-explanatory.

This Commentary was originally written for CSA S37-01 by three major contributors to it: the retired Chair, Donald Marshall, P.Eng; the retired Vice-Chair, Albert Nightingale, P.Eng; and the late Roy Dryburgh, P.Eng. It was, unfortunately, too late to include it in CSA S37-01.

The Commentary, with the permission of the original authors, has been updated to suit the new edition of this Standard.

The Technical Committee expects that this Standard will be used primarily by engineers, who can be expected to have sufficient knowledge of structural design to apply the Standard properly without much assistance from this Commentary. There are, however, some aspects of tower design and performance that might not be within the general practice of structural engineering. In these instances, this Commentary might be helpful.

The committee members are from all parts of the country and all major players of the Canadian telecommunication industry.

The public often ask why there is the need to modify a standard that has proven to be satisfactory over decades. The need comes from four sources:

a) Research has uncovered aspects of tower details or behaviour that were not previously appreciated but which have an impact on the safety or performance of the structure. These could lead to either more or less conservative designs, as will be seen in the changes in this edition.
b) The committee has discovered errors in the previous edition that have been corrected in this edition.
c) Building Codes and other related standards on which this Standard might depend for information and procedures or into which this standard is incorporated have changed, thus requiring a complementary or compensatory change in this Standard.
d) Tower design engineers still have to submit their designs for approval by municipal officials. At such times the building officials require compliance with certain clauses of building codes, to which the building officials might not have the experience to approve or accept. Covering these in this Standard will assist in the process.

In this Commentary, the clauses are numbered according to the clause to which they refer in the Standard (e.g., Clause S.1.1 of this Commentary refers to Clause 1.1 of the Standard).

S.1 Scope

S.1.1 General

This Standard applies to the various types of structures, which can be generally described as "communications structures", as they either act as antennas themselves, which transmit or receive signals, or as supports for such antennas. It covers all of the aspects of tower design, including wind loading and atmospheric icing, which are not similarly covered in the *National Building Code of Canada* (*NBC*). It provides guidance for treatment of special elements unique to broadcast and communication towers, including guy cables, tower foundations, and maintenance provisions.

S.1.2 Exclusions

Clause 1.2 exempts antennas and structures, the failure of which would not likely cause harm to persons or significant economic consequences for the owner or the public.

S.1.3 Unusual designs and antennas

Clause 1.3 is a reminder to engineers that they are to make sure that this Standard adequately covers the particular design being considered, which can be sufficiently different from the typical towers envisioned by the committee to warrant special calculations and provisions. Any departures from this Standard must still provide for the level of safety implicit in this Standard.

S.1.4 Dynamic effects of wind

Although this Standard does not cover many aspects of the dynamic behaviour of wind, it does try to cover the impact of dynamic wind action in the gust factor, C_g (see Clause 5.6).

S.1.5 Other design approaches

Clause 1.5 is a reminder that engineers with suitable experience and expertise may use another approach that is acceptable to the owner of the structure and the regulatory authority, as long as it provides the level of safety implicit in this Standard. Clause 1.5 is something of a complement to Clause 1.3.

S.2 Reference publications

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

S.4 Design requirements

S.4.5 Existing structures

Communication structures that support wireless telecommunication antennas and transmission lines can be subjected to increased effects due to factored loads both on the installation of new and removal of existing antennas and lines. When these changes occur it is important that the structure be re-evaluated for compliance with this Standard. These changes can occur frequently due to expansion in demand from users and rapidly changing technology.

Generally, structures that 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, however, 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 no longer valid and are withdrawn when new Standards are published. When there have been increases in the environmental loads between versions, the structure might not meet the current requirements. Clause 6.3 and Annex G provide reliability classes and importance factors that may be used in evaluating these cases.

S.4.6 Analysis scenarios

This Clause has been added to take into account the fact that structures are being designed with future antennas and lines that are not all necessarily installed as planned for. This situation can result in analyses that do not show overloads with respect to the existing antenna configuration which the tower might be subjected to for many years prior to the future antennas being installed.

Note that this situation would impact guyed structures and latticed self-supporting towers with multiple slopes and not monopoles.

S.5 Loads

Most load provisions are the same as in the 2013 edition of CSA S37. The use of site specific wind pressure profiles (q_h), where available, has been highlighted but the traditional use of reference velocity pressure, q, and a height factor C_e is still available to the engineer.

S.5.1 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 systems, etc.) attached to the structure. An allowance of about 5% of the structure weight should be made for galvanizing and fasteners.

S.5.2 Ice load, *I*

The ice load is taken as the weight of glaze ice (900 kg/m^3) assumed to form uniformly on all the exposed surfaces. The reference ice thickness, t_i, is obtained from Figure 1. Higher values should be used where experience or a meteorological study indicates that likelihood.

The values used for the map are based on ice accretions expected from freezing rain falling through a layer of freezing air, as indicated in Annex E. Rime ice has not been considered because of its variability and lack of reliable data. The Technical Committee assumes that this is reasonable as rime ice is less dense than glaze ice and the projected ice area, because of the way it forms, will have a lower drag factor than the values now employed based on a cylindrical cross-sectional shape of the glaze ice formed.

S.5.3 Design wind pressure, *P*

Clause 5.3 is based on the requirements of the *National Building Code of Canada* (*NBC*), 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 by appropriate modification of the flat terrain values, or, preferably, by retaining a meteorologist's services in identifying site-specific characteristics or wind values.

The requirements of Clause 5.3.3 were originally introduced in the 2001 edition of this Standard 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.

S.5.4 Reference velocity pressure, *q*

The reference velocity pressure, q, must be the 50-year return period mean hourly wind pressure at 10 m above ground level, as appropriate for the site, but not less than 320 Pa. The minimum value may be combined with the appropriate terrain C_e profile (open, rough, or intermediate) as specified in Clause 5.5.1. However, to ensure that a reasonable strength is achieved for short towers, q_h ($q*C_e$) must not be less than 290 Pa (Clause 5.3.2.1).

S.5.5 Height factor, C_e

The formula for C_e gives the wind profile with height above grade that will develop over flat terrain. Variations in terrain roughness and slope, both upwind and downwind, will affect the wind profile. A site-specific study by a qualified meteorologist will provide a more appropriate wind profile for the particular location.

S.5.6 Gust effect factor, C_g, and dynamic response

Towers and masts should be designed for a 3 to 5 s gust velocity. Since the reference velocity pressure, q, is that averaged over an hour, it is necessary to increase this value by a factor of between 1.52 and 1.67. The rest of what makes up the C_g of 2.0 covers some of the dynamic effects not explicitly covered in this Standard. The dynamic response to wind for some specific types of monopoles and tripoles will be taken into account through the requirements of normative Annex N.

S.5.8 Wind load, *W*

S.5.8.1 Lattice towers

As ice is considered to form radially on the members, the drag factor for ice area is the same as for round members.

S.5.8.3 Attachments

The appropriate drag factors are to be found in Clause 5.9.4.1.

S.5.9 Drag factor, C_d

The drag factors found in this edition of the Standard are based on the same publications as used by other major national Standards.

S.5.9.1 Lattice towers and masts

The drag factors presented in Clause 5.9.1 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 national Standards, such as various US Building Codes, ANSI/TIA-222-G, and international recommendations such as those of the International Association for Shells and Spatial Structures (IASS) TEG4 (Technical Expert Group for Masts and Towers) and the Eurocode.

Clause 5.9.1 gives drag factors similar to those in the 2001 edition of S37, but introduces reductions for round members under supercritical flow as well as adjustments for drag factors based on the angle between the wind and the direction normal to the tower face.

S.5.9.2 Pole structures

The values given in Table 1 are based on AASHTO Standards, which were derived from wind tunnel tests on round, 8-, 12-, and 16-sided poles. The 18-sided pole values were interpolated between the values for round and 16-sided poles. The 18-sided poles are a new configuration resulting from requirements for larger, stiffer poles and manufacturing limitations.

S.5.9.4 Antennas and other attachments

In the 2001 edition of the Standard, Clause 5.9.4 only dealt with antennas, but because there are now quite a few attachments being placed on towers, mostly to facilitate supporting the antennas, it was felt that this would be the appropriate place to introduce the more varied and more realistic values for attachments that were not explicitly covered in the previous edition of this Standard or where values were given only for linear attachments [Clause 4.9.1.4 of CSA S37-01 (large values of aspect ratio)].

S.5.9.4.1 Discrete attachments

The major manufacturers of parabolic and panel antennas publish drag factors and force coefficients for certain wind directions. The revised Table 2 in Clause 5.9.4.1 gives drag coefficients for attachments having different shapes, different vertical aspect ratios as well as values for shapes with rounded corners. The values in this Table were derived from published data taken from the AS/NZS 1170 Part 2, as well as Chapter 3 of Sachs (1978). The upper bound aspect ratio was revised to be in line with the above references. Interpolation between horizontal values must be used for aspect ratios which fall between the lower and upper limits. Interpolation between vertical values can also be used for rectangular shapes that are not shown, for example rectangles with 1: 1.5 width to depth ratios, etc. The drag factors can be applied to antennas as well as any other structural element and radio equipment.

S.5.9.4.2 Linear attachments

Clause 5.9.4.2 introduces shielding for linear attachments that are wholly within the tower or within 300 mm outside the tower face.

S.5.9.4.5 Effective projected area

Clause 5.9.4.5 brings CAN/CSA-S37 in line with other Standards, which recognize that when the wind blows at an angle to a surface, there will be a drop in pressure acting on that surface proportional to the square of the sine and cosine of that angle. Instead of changing the pressure, an effective projected area, EPA, is calculated, which is the product of the area of the surface in question, the drag coefficient for the shape of the object (C_d), and the sine and cosine of the angle between the wind and the surface, squared. In the 2018 edition, an additional load in the perpendicular direction to the wind flow (90° to the normal wind) was added. The loads from the wind normal to the face as well as 90° to the face are to be considered concurrently in the analyses.

S.5.9.4.6 Clusters of linear attachments

Clause 5.9.4.6 introduces the cluster approach to the treatment of transmission lines (antenna feeders). This approach can significantly reduce the wind load calculated in situations where many lines are installed in close proximity to each other.

S.5.11 Temperature effects, *T*

The ambient temperature that should be assumed for the initial condition for the analysis is set out in Clause 6.1. A decrease in temperature is usually assumed for the cases considering ice on the structure and guy cables. Ice forms at a temperature of between –5 °C and 2 °C. This lower value is usually considered in determining the temperature effect in the structure, especially of guyed masts. Lower temperatures may be considered to see the effect on guy tensions.

S.5.12 Earthquake load and effects, *E*

The 2013 edition of CSA S37 was the first edition to incorporate earthquake load effects. In dealing with earthquake effects, it is important to keep in mind that the loads are inertial, that is they develop from the acceleration of the masses in the structure in response to earth motion as a result of an earthquake. Therefore, non-uniform distribution of mass and large concentrations of it can produce significant local effects even for low accelerations. A detailed treatment is given in Annex M.

S.6 Analysis

In limit states design, the design is tested at the limit against failure, the ultimate limit states, and performance at design loads, serviceability limit state.

The ultimate limit state is the one where the safety factor is incorporated, and in which this safety factor is split into two components: one applied to the calculated loads, a load factor, α, and one applied to the calculated material or system strength or resistance, a resistance factor, ϕ. This allows designers to provide for a more uniform factor of safety, since they can modulate the load and resistance factors based on the level of confidence they might have in the values or methods of calculation, loads expected, and material properties.

S.6.1 Initial condition

Guy stiffness depends in part on the guy's initial tension. It is desirable to have the tension close to the design values most of the time. The Standard, therefore, requires that the initial tensions be set for a temperature close to the average annual temperature for the area. South of 55° N this will be 10 °C, north of that it will be 0 °C.

S.6.3 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 (live loads). The load factor for the self-weight of guy cable systems is taken as 1.0 both because this is known precisely and, more importantly, because the weight of the guy is a major factor in guy stiffness so a factored guy weight would give the wrong stiffness and, therefore, the wrong prediction of tower behaviour. The partial safety factor represented by the load factor is instead included in the reduced resistance factor for guys, ϕ, of 0.6 (see Clause 10.4.4).

Importance factor, γ

The importance factors given in Table 3 allow the designer and the owner, subject to the approval of the regulatory authority, to reduce the safety factor for the structure to suit the minimum reliability class required for the site or use of the structure. Most new towers should be designed for 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 in Table 3.

Load combination factor, ψ

In the absence of a site-specific wind study, a combination factor of 0.5 is applied to the wind loads when there is ice assumed to be on the structure and guys, on the assumption that the probability of the maximum wind load occurring simultaneously with the specified maximum ice load is low. A proper site-specific wind and ice study should yield the most appropriate values for design wind, design ice, and the companion wind to the design ice.

Wind load factor, α_W

The wind load factor of 1.40 was determined after an extensive evaluation of the designs of many satisfactory existing towers. This value has been reduced from that in the 2001 edition because the wind pressures have been increased.

Ice load factor, α_I

The ice load factor is applied to the weight of the ice only. If the load factor were applied to the ice thickness, while the wind load would increase linearly with thickness, the ice weight would increase as the square of the thickness. Benchmark studies indicated that the increases in the load on the guy cables based on factored ice thickness would result in structures much heavier than existing ones that are still considered to be satisfactory.

No load factor is applied to the guy initial tension. The designer may wish to apply factors on guy initial tension in order to get some idea of the impact on the design of errors in setting of initial guy tensions, but this is not currently a requirement of this Standard.

S.6.4 Factored loads for serviceability limit states.

Serviceability states are the ones where the behaviour of the structure or component is assessed at the design load. Therefore most load factors for this limit state are set to 1.

S.6.4.1 Reference velocity pressure for serviceability

Because the serviceability limit state deals with the performance of the system rather than its structural failure, a wind load that will occur with greater frequency is more appropriate. Therefore, the 10-year return wind is often chosen.

S.6.4.3 Serviceability factor

The tower engineer may wish to discuss with the RF engineer the proper serviceability factor, τ, to be used.

S.6.5 Wind direction

Because of the geometries of lattice towers and their guy systems, both their stiffness and strength will change with direction. Also, the variability of attachments and their distribution around the tower will change the loading applied to the tower in different directions. Further, the wind speeds and profiles with height can vary with direction. All these reasons make it essential that all critical wind directions be considered. If each direction listed in Clause 6.5.1 is looked at for each face of the tower, chances are good of hitting on all the controlling cases. With the current availability of affordable software that is capable of doing many analyses with a minimum of effort in very short time, there is no good reason not to look at every possible wind direction.

S.6.6 Earthquake direction

All effects from earthquakes have to do with the effects of acceleration of the mass of the tower. A few directions analyzed independently is, therefore, enough, unless there are large concentrations of eccentric mass or irregularities in the structure, in which case an analysis of the structure with inertial forces in different directions simultaneously might be necessary.

S.6.7 Displacement effects

Very slender self-supporting towers and poles with heavy loads or large wind sail areas at the top and all guyed towers will undergo significant deflections. In these cases the induced additional loads due to the large deflections must be considered.

S.6.8 Cantilever factor

This factor is meant to take care of the extra stresses induced in slender cantilevers of guyed towers due to the whipping effect from the rotation at the interface with the tower at the top guy level.

S.7 Structural steel

S.7.1 General

CSA S16 is the basic Standard for the design of steel structures in Canada. Structures required to conform to CSA S37-13 are typically latticed self-supporting towers and guyed masts made up of angles, solid round bars, and tubular members, or tubular pole type structures. CSA S16, which is applicable primarily to buildings, does not, in some areas, adequately address the use of these types of members. Clause 7 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 can appear more liberal than standard structural practice, they are based on full-scale testing to failure, and have been codified in such Standards as ASCE 10 and Eurocode EN 1993-3-2.

Annex K provides a commentary on Clause 7 and addresses the changes from the previous edition of the Standard.

S.8 Corrosion protection

S.8.5 Anchorages

Clause 8.5 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.

S.9 Other structural materials

Clause 9 covers requirements for structures made of materials other than structural steel, such as aluminium or concrete, and references appropriate CSA Standards.

S.10 Guy assemblies

S.10.1 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 are therefore given extensive coverage. This hardware is usually used for running rigging (cranes, etc.), as opposed to standing rigging as in bridges, for example; thus, the necessity for the different factors laid out in Clause 10. For taller, heavier masts, the guy assemblies are similar to those used in bridges. The division between light and heavy is usually at 32 mm diameter.

S.10.2.2 Pre-stretching

Constructional stretch occurs due to the bedding down of the wires in the strand as it is loaded. The amount of stretch will depend on the cable's construction and the number of layers of wire in the strand. Once the construction stretch has occurred, the cable should only stretch elastically. It is usually not necessary to pre-stretch strand that is less than 32 mm in diameter or cables of short length, especially if a mechanism is provided to make sufficient adjustment. Long, heavy guy assemblies will require more take-up and, most likely, pre-stretching. The decision on this operation is best left to the design engineer in consultation with the strand manufacturer.

S.10.3.1 Clips

Clipping is an acceptable method of securing loops in wire rope or strand of smaller diameter cables. The effective breaking strength of the cable will be reduced due to the crushing effect of the clips and potential slipping at higher loads. This was a common termination used before the mechanical splice and the preformed guy grip were introduced. These have proven to be more economical and much more convenient.

S.10.3.4 Sockets

For strand diameters larger than 32 mm, strand sockets are the only practical end fittings. Most strand sockets use poured hot zinc or lead to wedge in the strand wires; however, more recently, resin has been used, because it does not require the heating of lead or zinc, thus making it much more convenient. The designer must ensure that there is proper treatment applied to prevent the resin's deterioration due to ultra-violet light. It is important that the strand lines up properly with the socket to avoid unequal distribution of the load to the wires of the strand.

S.10.3.5 Thimbles

When a loop is used as the end attachment, it is important that the strand be properly supported using a heavy-duty thimble having the appropriate smooth curvature. A little "brooming" that can occur where the strand goes around the thimble is acceptable. Near the rated breaking strength of the cable even a heavy-duty thimble will start to show some deformation. This can be a useful indication that the cable has been severely overloaded approaching its breaking strength.

S.10.3.7 Turnbuckles

In the late 1960s there were four turnbuckle failures on major guyed masts in Canada. 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 (as when the jaw of the turnbuckle is connected directly to the anchor plate) that introduced bending for which the turnbuckle was not designed. The second was the coarse grain structure of the metal due to the forging operation that caused low ductility, particularly at low temperatures. To reduce this risk, 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.

S.10.3.9 Initial tension tags

The designer specifies the initial tension, at the referenced design temperature, on which the design is based. Usually this will be about 10% of the rated breaking strength of the cable. The designer can choose an initial tension, however, of between 8% and 15% of the breaking strength. This may be in order to control the stiffness of a guy level and, therefore, the deflections at the design load, or it may be a result of variation in slope of the cables around the tower. Design drawings and erection diagrams will normally show these design initial tensions along with their values for variation in ambient temperature. Unfortunately, these are often misplaced and not available to the inspectors or crews doing subsequent maintenance work on the mast. Without the design information, the inspector may rely on Clause 10.4.5 and set the tensions to 10% of the breaking strength, thereby inadvertently affecting the behaviour of the structure. To avoid such situations, Clause 10.3.9 was introduced, requiring the attachment of permanent tags at the anchor end of the guy assembly providing the necessary data. As older installations might not have such tags attached, in any future inspection or maintenance operation the owner/consultant should specify the supply and installation of the necessary tags. It is the responsibility of the owner/consultant to obtain and supply the required information.

S.10.4 Design of guys

S.10.4.2 Efficiency factors for guy assemblies

The efficiency factor was introduced to facilitate adjustment of the minimum breaking strength of the cable based on characteristics of the terminations used, because some terminations could fail or slip before the cable itself breaks.

S.10.4.4 Factored resistance

In limit states design, the major part of the safety factor is applied to the load as a load factor, usually more than 1. The other part is applied to the resistance as a resistance factor, which is usually less than 1.

The resistance factor for guy cables and guy hardware was selected so as to give the same ratio of resistance to effects of load as was the case in the allowable stress design approach in previous versions, which had a safety factor on guys of 2.5:

a) The factor of 2.5 has three components:
   i) 1.67 a main safety factor, with respect to the yield point, for tension members;
   ii) 1.30 an additional safety factor in going from the yield point to the ultimate tensile strength (UTS/YP ratio);
   iii) 1.15 a further safety factor for redundancy, meant to ensure that the guy assembly does not fail before the mast fails.
b) 1.67 x 1.3 x 1.15 = 2.5.

The resistance factor is the inverse of the safety factor, so (1/1.11)* x (1/1.3) x (1/1.15) = 0.6

* *Because the major part of the safety factor was already applied to the wind loads, the 1.67 is divided by the load factor, 1.5, to get 1.67/1.5=1.11.*

S.10.4.5 Initial tensions

While the initial tension at the anchor end of the guy cable is usually specified as 10% of the rated breaking strength of the cable at the design temperature, it is sometimes necessary to select another value between 8% and 15% of the breaking strength to achieve the desired guy set stiffness relationships between guy levels. A stiffer guy set will attract more load and a less stiff one less load to that guy level. In special circumstances these values can be exceeded, but in such cases additional studies need to be carried out to ensure that other potentially detrimental effects are avoided. Low initial tensions may result in low frequency, high amplitude vibrations of the guy cable at moderate wind speeds, “galloping”, while high initial tensions may result in high frequency, low amplitude Aeolian vibrations, “singing”. See also Clause 12.3.1, Item b).

S.10.4.6 Articulation

See Clause S.10.3.7 for commentary on the background on this requirement. Full articulation means that the guy termination can rotate freely in all directions.

S.10.4.7 Take-up devices

Take-up devices are provided to allow the field crew to readily adjust the length of the cable to achieve the specified tension at the ambient temperature and to permit future adjustments that might be required due to the constructional stretch in the cable. See also Clause 11.5, which specifies the amount of adjustment that must be available at the time of the initial installation of the cable to permit future adjustment of guy length and therefore tension. It should be kept in mind, however, that although the main purpose for the adjustability of guy length is to permit adjusting of guy tension, it also serves to permit adjusting of tower verticality and straightness.

S.10.4.8 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 requiring special consideration. Where several cables having different properties, whether of the same or different materials, are used to make up one guy cable, the properties used for calculation of the single guy cable response must be those resulting from the proper combination of the properties of the segments that comprise it.

S.11 Foundations and anchorages

Ultimately, the effects of all wind and gravity loads, no matter how they are generated, must be resisted by the foundations in earth or rock.

These foundations will include blocks of concrete, spread footings, piles, caissons, and rock anchors, depending on the application, soil condition, materials available, and workforce available.

Self-supporting towers, including poles, will require either one block or a number of footings capable of resisting vertical effects of bearing and uplift, horizontal effects, overturning/ moment effects, and torsional effects.

Guyed towers will require at least one footing similar to those required by a self-supporting tower and a number of footings to resist effects at anchors capable of resisting primarily uplift and horizontal effects.

S.11.1.1 References

Clause 11.1.1 references Section 4.2 of the *NBC*, which in turn references the *Canadian Foundation Engineering Manual*. Since the *Canadian Foundation Engineering Manual* is meant to cover foundations for buildings and other non-tower structures, it might not give adequate guidance for aspects particular to tower foundations. Clauses have been added to Clause 10 to assist where other references might not adequately cover the needs of tower engineers with respect to foundation design.

S.11.1.2 Geotechnical site investigations

Clause 11.1.2 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.

Clause 11.1.2 has been expanded considerably to emphasize the importance of correct site-specific soils information to the integrity of the overall structure. It also now includes the following:
a) a requirement to address the soil's sensitivity to earthquake action;
b) a requirement for the geotechnical engineer to comment on the probable failure modes and methods of calculating the resistance; and
c) a requirement, if needed, to address modifications to the resistance factors recommended in Clause 11.2.1.

S.11.2 Design

S.11.2.5 Resistance factors

Previous versions of this Standard, before the 2001 edition, gave only two values for the resistance factor, ϕ: 0.75 for soil or rock and 0.5 for single rock anchors. These were meant to provide the same "safety factors" as were implicit in the working strength design. The Technical Committee undertook a review of these values, as there was some concern that they might be too un-conservative in the case of certain piles in tension, when *Foundation Engineering Manual* procedures were used for their design. The revised load factors ultimately adopted were essentially as listed in Clause 11.2.6.

In the 2013 edition, the committee came to the consensus that a number of resistance factors were overly conservative. The main change was that for pile foundations resisting uplift through friction with the soil. This resistance factor was split between piles that taper to the bottom and all others, the resistance factor for the first going up by 7%, and for the second by 33%. The second change, a small one, was in the case of bearing on soil, where the ratio of long-term load to maximum load exceeded 0.2. The ratio has been removed and it is now assumed that all guyed towers qualify for the reduced resistance factor, which has in any case been increased by 20%, and self-supporting towers do not require the reduction.

S.11.3 Foundations and anchorages in soil

S.11.3.1 Bearing against undisturbed soil

A reminder that in the calculation of lateral and friction resistance, where one expects pressure to develop between the foundation and soil, it is important that the soil is not having to compress much before resisting the action of the foundation. Obviously the reference to undisturbed soil applies to the front face of guy anchors and the sides of pad foundations and piles.

S.11.3.2 Base of foundation below frost line or into permafrost

A reminder that foundations in soils susceptible to frost action, to keep them from being pushed up or from experiencing differential movement, must be founded in the non susceptible part below where the frost can penetrate. Obviously this rule also applies in the active layer in areas of permafrost, except that in such areas, going below the frost line is not going into a frost-free area, but into a permanently frozen area similar to rock. In such situations, it might be possible to take advantage of the permafrost to keep the foundations in place.

S.12 Tower and pole structure installation

S.12.1.1 Construction loads

Following recent events, increased complexity of structural reinforcing and significant design changes or uses of towers from original construction it has become paramount that additional engineering is required when modifying tower loading. The increases in stresses to the structure from construction loads (winches, gin poles, and other construction equipment) or changes to the tower stability must be considered. For guidance on process, procedures, and acceptable practices refer to ANSI/TIA-322 and ANSI/ASSE A10.48 documentation.

S.12.3 Tolerances

S.12.3.1 Guy tensions

a) Initial guy tensions, the main object of Clause 12.3.1, are critical in determining the stiffness of the mast support provided by the guy sets. Considerable overloading of the mast and guys can result from slack guys allowing too much deflection or tight guys attracting too much load to one guy level.
b) There are occasions when it is not possible to set the guy tensions to within the tolerances set out in 12.3.1, Item a), because of variations in guy cable slopes around the tower, for example. In this situation, it was felt best to provide an attainable middle ground.

S.12.3.2 Verticality

The tolerances for verticality and straightness are meant to produce towers that look right, which gives them a good chance of being right. It would take considerably more out-of-straightness than stipulated in Clause 12.3.2 to introduce excessive additional effects that would overload a tower.

It is important to keep in mind that these tolerances are for the tower and components upon completion of construction and under dead load alone. Once live load, even if permanent, as in the case of a pull-off of some sort, for instance in certain radio and military towers, is applied, any deflection due to that is deflection under load not to be considered within construction tolerance.

S.13 Obstruction marking

Clause 13 is provided to alert the owner and/or engineer that obstruction marking, either lighting alone or painting and lighting, might be required for the structure. It also directs the responsible persons to the governing documents. The design engineer must include, in the loading calculations, any lighting equipment to be installed on the structure.

S.14 Bonding and grounding

Clause 14 does not have any direct impact on the design of the structure unless any components of the grounding system are deemed to affect the loading. It is included as a reminder that the supply and installation of these components should be included for all towers. Many tower owners have their own requirements for bonding and grounding, because a good bonding and grounding system is recommended for protection of the equipment installed on the structure and in the equipment shelter at the base from lightning and other stray currents. If nothing is specified, the tower should meet the minimum requirements set out in the referenced documents.

S.15 Insulators and insulation

Insulators or isolators are used in towers and guyed masts for AM radio broadcasting because, in these cases, the tower is the active element and must be kept from being grounded to earth. Other structures and elements, which can re-radiate signals, might need to be electrically shortened by use of break-up insulators. The tower designer will be concerned with the strength requirements; the radio engineer for the project must specify the electrical requirements.

S.15.1.1 Ceramic materials
Insulators used in guy assemblies, particularly the primary insulator at the radiating mast, present a challenge to the designer in trying to conform to the requirements of this Clause. There are basically two types available: the first, generally referred to as a “cage” type, places a ceramic cone in compression, using a metal cage and a metal rod through the cone to transfer the loads; the second type is referred to as an “oil-filled” insulator, which uses a fibreglass loop in tension to place the ceramic cylinder in compression and contain the oil surrounding the loop. The first type has a low flashover rating; so several units may have to be placed in series to achieve the required rating. The second would appear to be a “fail safe” design, but has not always proved to be so. The oil in the oil filled insulator protects it from any arcing that might occur. If the loop fails, however, 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 for both types.

S.15.1.3 Factored resistance
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.

S.16 Ladders, safety devices, platforms, and cages

S.16.1 General
Clause 16 had been prepared for the 2001 edition of S37 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 Supporting Structures” set out in Division II, CSA S37 automatically became part of the Canada Labour Code.

The wording of Clause 16 of S37 was approved as written by Labour Canada before it was published in the *Gazette*, Part I. It was subsequently reviewed after 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 in adding this Clause to the regulations, the Department of Labour is recognizing the special requirements for towers and antenna supporting structures; thus, the new Division II for “Towers, Antennas and Antenna Supporting 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*, CAN/CSA-S37 requirements govern even if they conflict with Division I requirements.

CAN/CSA-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*.

Commentary on the Annexes
The Annex component of this Standard has been increasing steadily over the various editions because it has been found to be helpful to tower designers even though it is not a mandatory part of the Standard.

The material in each Annex speaks for itself and needs no additional commentary. Nevertheless, Annexes C, D, N, and Q have had significant changes made and need some comment.

S. Annex C Measuring guy tensions

S.C.1.2 Direct method

Mention is made of other direct measurement devices that are becoming available, which might prove convenient to the user. A market search will identify these.

S.C.1.3.2, S.C.1.3.3 Pulse technique/Swing technique

The 2001 edition of this Standard provided one formula for both the pulse and the swing method, on the premise that both had to do with the natural frequency of the cable. The 2013 edition of this Standard recognizes, based on further investigation and calculation, that, in fact, the two mechanisms are different: the pulse being the period taken by a wave (pulse) in going up and down the guy cable and the swing being the period of oscillation of the cable as a pendulum. Both depend on the same parameters of the cable of length, mass, and tension, but the period of the pendulum is a little slower than the period of travel of the pulse.

S.C.1.3.6 Form fitting technique

The 2013 edition of this Standard also recognizes that another more rigorous and, therefore, more mathematically-demanding approach can be taken. It is included in this Standard in order that it not be totally excluded from consideration as a proper approach.

S. Annex D Recommendations for condition assessment

In the 2018 edition of this Standard, Annex D has been expanded from earlier versions to show the importance of completing initial construction inspections and recording critical information in accordance with the design drawing and specifications. It keeps prior references to the detailed inspection parameters while adding a few segments that are relevant to current Inspections. It also outlines the importance of completing modification inspections once the existing structures are changed.

Because measurement of verticality, straightness, and twist of the mast might not be readily available elsewhere, the recommended procedure for taking these measurements and the equations for calculating the required values have been added. This is similar to what was done for guy tension measurements in Annex C. These formulations also appear in ANSI/TIA-222-G.

S. Annex L Geotechnical site investigation

Geotechnical site investigation recommendations (Annex L) have been expanded to include recommendations for additional information that should be included in the geotechnical report for corrosive soils. Prior recommendations remain for normal soils, rock sites and permafrost sites. Annex L also includes a new Table L.1 with classification of soil corrosiveness depending on soil resistivity which will guide the user in the choice of the appropriate corrosion protection system

S. Annex N Tower dynamic effects and fatigue

It is not practical to provide a detailed discussion on the subjects of dynamic effects and fatigue in towers in this Standard. This is not because the subjects are unimportant (although major problems or failures related to these subjects are not frequent). It is because the existing design, fabrication, and construction practices are usually adequate to minimize occurrences of such problems. The level of attention given by the designer to these subjects should be commensurate with the height and importance of the tower structure.

S.N.1 General

The significance of these vibrations depends upon the amplitude and the frequency of such oscillations. For many structures, the stresses resulting from the oscillations are very low and there is no need for remedial action. For other structures, the amplitudes might result in significant cyclic stresses in the structure which can lead to fatigue cracks. The fatigue cracks might take years to initially develop and propagate or might develop in a relatively short period of time.

It has been demonstrated that wind induced oscillations result from a complex combination of variables, including effective damping of the structure, soil conditions, and site specific wind flow conditions especially at low wind speed, which are beyond the control of the manufacturer or the designer.

Structures with round or semi-round shapes with length to diameter ratios greater than 3 should be checked against susceptibility of vortex shedding. When oscillations or any signs of fatigue cracks are observed, a vibration mitigation solution is to be implemented. Some of the possible solutions for mitigation include: installation of mass or liquid dampers, installation of helical strakes or additional attachments intended to introduce turbulence, or changing the stiffness, mass, or damping of the structure. The local wind conditions at the site, as well as the risk of injury or loss of life posed by failure of the tower, need to be considered.

S.N.2 Wind dynamic effects

Dynamic effects include the effects of wind gusts and turbulence on guyed structures (see Annex H). This Annex addresses the wind dynamic effects on structures subjected to moment connections such as monopole and tripoles. These dynamic effects include the along wind vibrations as well as cross wind vibrations. In order to accurately model the dynamic behaviour of the structure, it is recommended to take field measurements of the structural damping.

S.N.2.2 Wind gusts

Because of the inherent variability in the velocity and direction, natural wind gusts are the most basic wind phenomena that can induce vibrations in wind-loaded structures. The equivalent static natural wind gust pressure range specified for design was developed with data obtained from an analytical study of the response of cantilevered support structures subject to random gust loads.

S.N.2.3 Vortex shedding

Poles, antennas, towers, and parts of towers having circular cross-sections (and other non-streamlined shapes) are prone to vortex excitation. Crosswind oscillation resulting from vortex shedding can occur in such structures and members, potentially producing significant deflections and stresses. Over a certain range of wind speeds, the flow of air around cylinders and other bluff (non-streamlined) bodies will generate large eddies that are formed and shed at regular intervals from alternating sides of the body. While attached to the body, these eddies disrupt circulation patterns, thereby creating a lift force which acts on the structure at right angles to the direction of the oncoming air flow. As the location of the attached vortex alternates from side to side, the direction of the lift force also reverses direction in a periodic fashion. The critical design condition occurs when the frequency at which these eddies are shed coincides with a natural frequency, f_n, of the structure. It is recommended to check for vortex-frequency coincidence with the natural frequencies of the first two lateral bending modes of a monopole.

In many cases, this critical wind speed is rather small (≤ 5 m/s); therefore, the resulting lift forces will not significantly affect the structure. Also, at wind speeds greater than about 20 m/s, enough natural turbulence is generated to disturb the formation of vortices. On the other hand, when wind speeds are low, temperature gradients can also produce very low turbulence levels, which in turn increase vortex-induced motions, especially in very slender structures.

Guidance for the analysis of vortex excitations can be found in CSA S6 as well as Eurocode 1, Part 1-4. Vortex excitation is generally not a problem for lattice-type towers made of sharp-edged shapes such as structural angles.

The simplified static approach to estimate vortex shedding response was provided in the 1995 edition of the NBC *Users' Guide — Structural Commentaries* (Part 4).

As long as the net damping ratio (structural damping less negative aerodynamic damping) remains positive, displacement amplitudes due to vortex shedding can be assessed and the resulting stresses are to be checked against the fatigue strength of the connection as in accordance with Clause N.3 and compared to number of cycles expected during the 50 year design life of the structure. However, if the negative aerodynamic damping exceeds the available structural damping, large amplitude (up to 1 diameter), unstable motion will occur, leading directly to collapse or to a dramatic acceleration of fatigue effects.

Measures for the mitigation of vortex-shedding vibrations include modification of the natural frequencies of the structure (to alter V_{crit}), the installation of vibration spoilers (such as helical strakes) over the top third of the structure, or the installation of supplemental damping devices (such as tuned-mass dampers). Telecommunication equipment installed at the top of the monopole will typically act as vibration spoilers.

S.N.2.4 Galloping

It has been demonstrated that cross-wind oscillations of triangular shaped structures such as tri-poles with solid shrouds could occur. This could be triggered as a result of the changes in the angle of attack of the incident wind, periodically. The structure becomes dynamically unstable and starts galloping with low-frequency, large-amplitude normal to the flow oscillations.

Galloping depends on the cross-sectional shape and mean wind angle of incidence on the transverse galloping stability (according to the Glauert–Den Hartog criterion for galloping instability). This variation in the angle of attack produces variation in the aerodynamic forces acting on the structure, and therefore produces variation in the response of the structure. The frequency of occurrence of these oscillations tends to be much lower than that of vortex-shedding. Galloping is characterized by large-amplitude vibrations, normal to the direction of wind flow, which can, quite quickly, lead to the failure of the oscillating structure.

Galloping-induced oscillations could occur in flexible, lightly damped structures. The wind tunnel tests of various triangular shapes by Alonso and Meseguer (2006, 2012) sheds some light on galloping for solid triangular structures with and without corner edges. The tests were based on two-dimensional flow thus assuming that the three faces of the structure are fully covered.

Guy cable vibrations are known to occur and can become serious under certain conditions (e.g., wind-induced motions in the presence of large atmospheric icing accretions, rain-induced vibrations, or guy galloping). These vibrations might require special attention when the guy cables are anchored in rock where they can induce fatigue bending stresses in the anchor rods. Methods of design to prevent these phenomena are described in the literature listed in the references. Problems in existing structures can be controlled by the installation of dampers.

S.N.3 Fatigue

There have been very few reported problems associated with fatigue in typical latticed tower structures. Fatigue stresses should be evaluated when significant vibrations are predicted and when fabrication details are such that fatigue life might be critical. Generally, welded details are more critical than bolted details. Abrupt stiffness transitions, for example at splicing pole connections, also exacerbate fatigue effects. For existing towers, a fatigue life assessment can be performed if problems are suspected.

Guy anchorages should also be considered, particularly if vibration problems have been observed. Reference may be made to this section for fatigue design.

Lambert (1995) has reported test results where fatigue failure in both in-line and transverse vibration modes occurred first in an outer wire adjacent to the socket or restraining connection detail, followed by additional outer wires, after which the second layer began to fail. This provides confidence that regular visual inspection will detect early signs of fatigue damage and repair or renewal can be undertaken before guy rupture occurs. For large diameter strands, there is a substantial interval between initial and final failure due to fatigue.

Additional guy cable damping is beneficial to reduce large stress fluctuations in the mast and ensure an acceptable fatigue life. However, structural damping by tuned mass dampers can introduce other problems and should be used with caution (Fischer, 1987). Such dampers should be tuned especially to damp the adequate frequency range of modes as mode transfer can occur.

In the presence of recurrent guy cable galloping, making a small adjustment (usually a reduction) to the cable still-air tension can sometimes be sufficient to solve the problem. This change should not significantly affect the quasi-static behavior, but should de-tune or de-couple the natural frequency of the guy wire from the natural frequency of the shaft or one of its harmonics. On the other hand, any change in the still-air tension during service (owing to inelastic stretch) could introduce resonance where none formerly existed. It is therefore important that periodic checks be made on the still-air tensions in addition to mast verticality, as mentioned in Annex D.

For fatigue strength assessment, because the stress range represents the algebraic difference between the maximum stress and the minimum stress at a given location, only the stresses due to wind load need to be considered (the stresses due to dead load should not be accounted for).

S.N.3.3 Fatigue detail categories

The stress concentration formulas are based on the NCHRP report 176 (2011). For other details that are not listed in Table N.2, the NCHRP report contains guidelines on the finite element model requirements for the determination of the stress concentration factors.

S. Annex Q Properties of Schifflerized (60°) steel angles

Schifflerized angles are standard steel angles that have had each flange of the angle bent inward by 15° at two bend lines some small distance from the heel of the angle [see Figure 13 c) and Figure Q.1] to produce a leg angle that can be used in the framing of a triangular lattice tower resulting in a cross-section, which is an equilateral triangle.

S.Q.2 Geometric properties

The formulas have been reorganized because of the addition of the properties about the x-x and y-y axes, which were left out in the 2001 edition. The radius of gyration about the x-x axis is important for evaluating resistances of Schifflerized angle legs, where the bracing system provides support for the leg only in the plane of the face (warren bracing without horizontals).