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LIMIT STATES DESIGN OF ANTENNA TOWERS

**by
Yohanna M. Farid Wahba**

**A Thesis
submitted to the Faculty of Graduate Studies and Research
through the Department of Civil and Environmental Engineering
in Partial Fulfillment
of the Requirements for the degree of
Master of Applied Science
at the University of Windsor**

**Windsor, Ontario, Canada
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ABSTRACT

The Canadian standard CAN/CSA-S37-M86 "Antennas, Towers and Antenna Supporting Structures" follows a quasi-limit states approach in which the member forces determined for specified loads are multiplied by a unified factor and compared against factored resistances given in CAN3-S16.1-M84. This results in a design basically the same as a working stress design with a factor of safety of 5/3. Due to the non-linear behaviour of the structure even under service loads, the non linearity in the load effect due to ice accretion on the towers, and the direct interaction between ice and wind loads, the load factors specified in CAN/CSA-S16.1-M89 "Limit States Design of Steel Structures" cannot be directly applied to antenna-supporting structures.

In this study, a forty-one different towers (representing different heights and designed for different ice classes and wind pressures) were analyzed under specified loads and then under a set of factored loads. From the comparison of the design forces in different parts of the towers with those calculated according to the existing standard, new proposed partial load factors for dead load, wind load and ice thickness are determined. A comparison between the design forces according to CAN/CSA-S37-M86 and forces obtained using the proposed load factors is included for all the towers. In addition, based on the analysis of climatic data, the wind pressure that can be applied to iced towers was determined and a change in the ice map of Canada given in CAN/CSA-S37-M86 is proposed. The proposed specified ice thicknesses were also verified under service loads.

To My Beloved Wife, Dalia

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NOMENCLATURE

A_f	face area of flat members, bare.
A_{fr}	face area of round members, bare.
A_g	metallic area of the guys.
A_h	accumulation of ice on horizontal surface.
A_i	face area for radial ice.
A_{mr}	maximum equivalent radial ice thickness with probability of 1/40.
A_r	equivalent radial ice thickness.
A_v	accumulation of ice on vertical surface.
a	the horizontal projection of the guy.
C	chord length of the guy.
C_a	acceleration factor for wind on antennas.
C_D	drag coefficient.
C_{df}	drag factors for flat members.
C_{dr}	drag factor for round members.
C_e	height factor.
C_g	gust factor.
C_L	lift coefficient.
c	influence factor.
cQ_m	mean of the load effect cQ .
D	dead load.
d	diameter of the guy.
d_o	total drag acting parallel to y-axis.
E_g	modulus of elasticity of the guy.
F.S.	factor of safety.
H	horizontal component of the tension in the guys.
I	ice load.

K	the slope of the secant of the reaction-deflection curve for the guy.
k	correction factor depending on size of the conductor.
L	approximate length of the guys.
L_o	approximate length of the guys at normal temperature.
L_t	unstressed length of the guy at temperature t °C.
l	Z-component of the total lift l_t .
l_t	total lift on the guy.
l_h	X-component of the total lift l_t .
p	design wind pressure independent of the drag factor of the object.
p_i	the maximum hourly wind pressure under iced conditions.
Q	load intensity.
Q_c	code-specified load intensity.
q	reference velocity wind pressure.
R	nominal resistance.
R_m	mean of the resistance R .
r	radius of the conductor.
T_i	initial tension in the guy.
t	temperature in °c.
V_D	the coefficient of variation of the dead load.
V_E	the coefficient of variation of structural analysis.
V_Q	coefficient of variation of the load effect.
V_R	coefficient of variation of the resistance.
V_W	the coefficient of variation of the wind load.
v	wind velocity.
W	wind load.
W_i	wind load on ice tower.
W_o	wind load on bare tower.
W_r	the resultant of all the forces acting on the guy.
w	total weight of the guy.
α	load factor.

α_d	load factor for dead loads.
α_i	factor for radial ice thickness.
α_w	load factor for wind loads.
β	safety index.
γ	importance factor.
Δ	deflected distance in direction of the wind.
Δ_g	stretch of the guys.
θ	the true angle between the guy chord and the wind.
κ	wind direction with respect to Y-axis.
ϕ	resistance factor (performance factor).
ψ	load combination factor.
ω	angle of inclination of the guy chord to the horizontal.

CHAPTER I

INTRODUCTION

1.1 General

The telecommunications industry plays a vital role in our lives that includes television, radio, phones, car phones, etc. As these applications increase and diversify, the demand for taller and more reliable antenna towers and antenna-supporting structures also increases.

Antenna towers are of two kinds: self supporting towers and guyed towers. Self supporting towers are normally used for low to medium heights up to 100 metres. These towers usually have a square or a triangular cross section with a large cross section at the bottom which gets smaller towards the top. An example is shown in Fig. 1.1.

A guyed tower consists of a mast which is usually of a constant triangular cross section (some square cross section masts are built in Canada but they are more popular in Europe) and one or more levels of guy cables which provide lateral support to the structure and are anchored at the ground level. For triangular mast legs, 60° schifflerized angles, solid steel rounds, or pipes are used, but for legs of a square mast and for the

horizontals and the diagonals of both types, structural steel angles are used. Examples of guyed towers are shown in Fig. 1.2 and 1.3.

The two most common types of guy wires used are guy strand and bridge strand, which are shown in Fig. 1.4.

A guyed tower is an example of a structure with non-linear behaviour even under working conditions. This non-linear behaviour is the result of:

- change in the stiffness of the guy cable with the change in the guy tension.
- the non-linear force-deformation relationship for the structure.
- the large displacements occurring in the structure even under normal design loads.

Also there is significant interaction between the loads imposed on the tower due to wind and ice; an increase in ice thickness results in an increase in the weight imposed on the tower and in the horizontal force acting on the tower due to the increase in the surface area exposed to wind.

1.2 Need For The Investigation

In recent years, most structural standards in Canada have moved toward the unified limit states design philosophy with common safety and serviceability criteria applied to all materials and types of structures. However, the current Antenna Supporting Structures Standard CAN/CSA-S37-M86 still follows a quasi-limit states approach in which the member forces determined for specified loads are multiplied by a unified factor of 1.50 and compared against factored resistances given in CAN3-S16.1-M84, "Steel

Structures for Buildings (Limit States Design)". This approach results in a design basically the same as that following the working stress design specified by CSA S37-1976. It is therefore important to find appropriate partial load factors that could be applied to the loads, thus resulting in a truly limit states based design for future revisions of the Antenna Supporting Structures Standard.

1.3 OBJECTIVES

In order to change the current specification to a truly limit states based design specification, the following factors should be determined:

- a load factor for dead loads α_d
- a load factor for wind loads α_w
- a factor for ice thickness α_i
- a load combination factor ψ to be applied for the case of combined wind and ice loads.

Further, it is reasonable that these factors be based on the analysis of different existing towers designed according to the current standard (CAN/CSA-S37-M86) thus providing a link between both.

CHAPTER II

HISTORICAL BACKGROUND

2.1 General

The advancement in knowledge and technology has always resulted in an improvement in the specifications and the underlying philosophy through which various structures are designed. Since antenna towers are usually made of steel, the progress in the specifications concerning the design of steel structures generally, and the antenna towers specifically, should be studied. In this chapter, the progress in the steel structures for buildings standard as it shifted from working stress design (CSA-S16-1969) to the limit states design (CSA-S16.1-1974), are summarized and the progress in the standards regarding antenna supporting structures CSA S37 follows.

2.2 Developments in Steel Structures for Buildings Standards

2.2.1 Working Stress Design (WSD)

In Canada, steel structures designed up to the mid-seventies mainly followed the working stress design philosophy WSD (sometimes called the allowable stress design).

The last Canadian specification based on WSD was CSA-S16-1969, where the actual stresses, computed by linear elastic theory for the working loads specified by the National Building Code of Canada (NBCC), were demonstrated to be less than or equal to allowable stresses given by the Structural Steel Specification. These allowable stresses were determined by dividing the maximum stress defining an appropriate limit state by a factor of safety, $F.S. > 1.0$.

- Since the 1950's, this traditional approach has given way to a method involving several load and resistance factors in recognition of the fact that one single factor of safety does not provide uniform reliability and the resulting designs are not uniformly economical.

2.2.2 Limit states design (LSD)

2.2.2.1 Introduction

All structures have two basic functional requirements, namely serviceability during the useful life of the structure and safety from collapse under severe loading conditions. Limit states define the various types of failure and unserviceability that is to be avoided. Those concerning safety are called the *ultimate limit states* which include collapse, overturning, buckling, etc. Those concerning the unserviceability are called the *serviceability limit states* and for antenna towers these include vibration, excessive deformation, tilt, or twist for the antennas.

2.2.2.2 Design Criteria

The general format for the limit states design can be expressed in the following form :

Factored resistance $\geq$ Effect of factored loads

$$\phi R \geq \gamma \sum c_i \alpha_i Q_i \quad (2.1)$$

where :

ϕR = factored nominal resistance

ϕ = resistance factor, $\phi < 1.0$, accounting for the uncertainties in determining the resistance.

R = nominal resistance of the structural element, determined for nominal material and sectional properties.

γ = importance factor, accounting for the importance of the structure.

$\sum c_i \alpha_i Q_i$ = factored load effect (axial force, shear, moment,.. etc.) for any load combination

c = influence factor by which the factored intensity αQ is transformed into a load effect.

α = load factor, generally $\alpha > 1.0$ to account for the uncertainties of the loads.

Q = load intensity.

Instead of the traditional single factor of safety, limit states design uses partial safety factors:

- load factors, α_i , to be applied to the loads taking into consideration the

variability of the load i and the load pattern, i.e., the uncertainties in defining that load.

- a load combination factor ψ , to be applied to loads other than dead loads to take into account the reduced probability of loads from different sources occurring simultaneously.

- an importance factor γ , to be applied to the loads to take into account the consequences of collapse as related to the use of the building, human safety or economic loss.

All the above factors are a function only of the types of loads and the degree of importance of the structure, regardless of the material of construction. The factor concerning the material of construction is the performance factor, ϕ , which takes into account the variability of material properties, dimensions, workmanship, type of failure, and uncertainty in prediction of resistance. Again this performance factor is only a function of the type of material used regardless of the applied loads or the type of the structure.

There is nothing novel about Eq. (2.1) if only one load factor is used for all load types, and a linear relationship exists between load and stress. Then the factor of safety would be :

$$F.S. = \frac{\alpha}{\phi} \quad (2.2)$$

and it is taken to be equal to 5/3 as in CSA S16-1969.

2.2.2.3 Probabilistic Modelling

Many schemes which have been proposed for the modelling of the structural design process have used the concepts of the theory of probability; these vary from a complex fully probabilistic approach to very simple schemes using only the minimal statistical properties of the means and standard deviations.

Figure 2.1 shows the resistance, R , and the load effects cQ as two independent random variables that can be represented by two distribution curves. This independence is not entirely true, but for steel structures generally, where the mass of the steel skeleton is usually a small fraction of the total mass of the building, it is a satisfactory assumption. R and cQ in Fig. 2.1 refer to the resistance and the forces in the individual members in the structure and not to the structure as a whole.

Figure 2.2 illustrates the scheme used in the WSD where code-specified values Q_c , which are chosen in an arbitrary manner to lie above the mean, are compared to a code-specified resistance R_c divided by a certain factor of safety F.S. This scheme does not account for the uniformity of the reliability of the structure, but it insures that according to the specified loads the reliability is just adequate, and in other cases it is more than adequate.

Figure 2.3 represents the basis for the development of the LSD code in Canada where the two distributions for R and cQ are combined into one curve $[\ln (R/cQ)]$. Thus a limit state is reached when $\ln (R/cQ) = 0$. The safety index β , relates the distance between the mean $[\ln (R/cQ)]_m$ and the zero line which represents the failure. β is approximately given by

$$\beta = \frac{\ln \frac{R_m}{cQ_m}}{\sqrt{V_R^2 + V_Q^2}} \quad (2.3)$$

where :

R_m is the mean of the resistance R

cQ_m is the mean of the load effect cQ

V_R is the coefficient of variation of the resistance

V_Q is the coefficient of variation of the load effect

In the above, it should be noted that the coefficient of variation is equal to the ratio of standard deviation to the mean.

2.3 Developments in Antenna Towers Standards

The standard regulating the design of antenna towers and antenna supporting structures is CSA S37. In this section, a comparative study is done on the design criteria and the different load factors and load conditions used in the last three editions of this standard.

2.3.1 CSA S37-1976

The 1976 edition of the standard fully adopts the Working Stress Design (WSD). The loads which were used in the analysis were the specified loads and the resulting forces in the members were compared to the allowable stresses as defined in the CSA Standard S16-1969, "Steel Structures for Buildings".

Wind loads were defined for three wind zones as follows :

Wind Zone A : reference wind pressure = 10.0 psf

Wind Zone B : reference wind pressure = 13.0 psf

Wind zone C : reference wind pressure = 18.0 psf

The Ice loads were specified for five classes with thicknesses as follows :

Ice Class I : Ice thickness = 0.5 in

Ice Class II : Ice Thickness = 1.0 in

Ice Class III: Ice Thickness = 1.5 in

Ice Class IV : Ice Thickness = 2.0 in

Ice Class V : Ice Thickness = open

The two cases of loading considered were :

(a) Dead load + wind load

(b) Dead load + 0.5 wind load + ice load

It is clear from the above that this version of the standards consistently followed the Working Stress Design method as it was still the basis for CSA S16-1969.

2.3.2 CSA-S37-M1981

After the introduction of limit states design for steel structures in Canada in 1974, CSA-S37-M1981 was the first edition of "Antenna Towers and Antenna Supporting Structures", written in SI units and adopted limit states design procedures. Otherwise it was the same as S37-1976 with no major changes, and coexisted with it.

Loads were approximately the same as in S37-1976 but were expressed in SI units.

Reference wind pressures were specified as follows :

Wind Zone A : 450 Pa

Wind Zone B : 600 Pa

Wind Zone C : 850 Pa

Radial ice thickness was specified as follows :

Ice class I : 10 mm

Ice class II : 25 mm

Ice class III : 40 mm

Ice class IV : 50 mm

Ice class V : open

The cases of loading specified were :

Dead load + wind load

Dead load + 0.5 wind load + ice load

and for the limit states design a unified load factor of 1.5 was applied to all types of loads as follows :

$1.5(\text{dead load} + \text{wind load})$

$1.5(\text{dead load} + 0.5 \text{ wind load} + \text{ice load})$

Due to the non linear-behaviour of guyed towers and because this unified load factor was applied to the loads, the resulting design was completely different from that of Working Stress Design. It should be noted that serious convergence problems were experienced by designers while running the available computer programs for the above loading conditions because the increase in the weight of the guys changed the properties

of lateral supports of the towers.

2.3.3 CAN/CSA-S37-M86

The reference velocity pressure, q , is determined to be the one in 30 years return hourly wind pressure for the site or may be determined from the National Building Code of Canada (NBCC) with special consideration given to local topography, weather or other conditions. As an alternative, the value of " q " may be obtained from the wind map with one of the wind classes A, B or C, as shown in the wind map (Fig. 2.4).

The same ice loading is to be considered as the previous standards, using the ice map (Fig. 2.5).

The same load cases as in the S37-1976 are to be taken with no load factors to be applied. In addition, two other load cases were added for the overturning of the self supporting towers. The load cases are :

Dead load + wind load on bare tower

Dead load + ice load + wind load on iced towers

0.7 dead load + wind load on bare tower

0.7(dead load + ice load) + Wind on iced towers

A factor, α , is to be applied to the forces in the individual members. This factor is 1.5, except in the case of the cantilever span of a guyed tower where $(L/r) > 120$, this factor is 2.0.

It should be noted here that the application of this unified factor to the forces in the members (not to the applied loads) would give exactly the same design as if using

Working Stress Design with a factor of safety of 5/3 since

$$F.S. = \frac{\alpha}{\phi} = \frac{1.5}{0.9} = \frac{5}{3} \quad (2.4)$$

2.4 Steel Buildings vs Antenna Towers

As shown in section 2.2, LSD was adopted in the steel structures standard for buildings since the mid-seventies but until now it has not been introduced into the antenna towers standard. The reasons for this are discussed below.

2.4.1 Loads

For most steel structures wind load is considered a secondary load but for antenna towers it is the most important primary load. That is why for every single tower the reference wind pressure is not determined directly from the values provided in the NBCC for 30-year return period but by reference to the local topography at the tower site.

For steel buildings, ice accretion is of no significance to the design forces but for antenna towers ice has a significant effect on the vertical and horizontal forces on the tower due to the increase in the weight of the members and increase in the area exposed to wind. Thus the influence factor, c , that transforms the load intensity into a load effect is not a linear factor, because an increase in the ice thickness would result in a quadratic increase in the weight with a linear increase in the wind force.

2.4.2 Analysis

Due to the non-linear behaviour of guyed towers, the principle of superposition is not valid; thus, the structural behaviour of guyed towers must be analyzed for all loads simultaneously.

2.5 The Need For Partial Load Factors

From the above discussion it can be concluded that a unified load factor applied to the loads, as in S37-M1981, would lead to a completely different design (in addition to some convergence problems in the analysis). If a single factor is to be applied to the forces in the members, it would lead to a working stress design with no consideration given to the degree of uncertainty associated with different load types. For the following reasons, it is necessary to carry out a study to find the appropriate partial load factors:

- more consistent safety for different combination of loads, as the degree of uncertainty in calculating each load can be contained in the factor.
- savings in weight can be made for previously over-designed towers.
- less probability of under-designed towers.
- uniformity of the same LSD philosophy of all structural specifications.
- ability to predict the behaviour of the structure under severe loading conditions or near failure.
- facility to input new information on loads and load combinations as such information becomes available.

CHAPTER III

THEORETICAL MODELLING

3.1 General

The two most common and acceptable models used for the analysis of guyed towers under static and quasi-static loads as specified in the "Recommendations For Guyed Masts" published by the International Association For Shell and Spatial Structures [1981] are :

- an elastic beam-column to represent the shaft, with non-linear elastic supports at the guy attachment points, representing the guys.
- a finite element representation of the guyed mast using elastic beam-column elements for the shaft and cable elements for the guys.

The first model is the one most widely used in the industry as it is simpler for modelling and less time-consuming and the results are close to the second approach within a tolerance accepted by the standards.

3.2 Program Used

Throughout this research the towers were analyzed by a family of programs GUYMAST by Weisman Consultants Inc., Downsview, Ontario, Canada. [GUYMAST 1987] These programs were specifically developed for antenna towers. These programs are :

- MASTLOD, a program used for the preparation of the loads to be used by the analysis programs for both self supporting towers and guyed towers.

- DISHFORC, a program that calculates the loads generated on the towers due to parabolic dishes and conical horn antennas under different wind and ice conditions.

- GUYMAST, a program that executes the analysis of guyed towers using the output of MASTLOD.

- MAST, a program that executes the analysis of self supporting towers using the output of MASTLOD.

- OUTRIG, a program to determine the forces in the outriggers (torsion resistors) based on the output from the GUYMAST.

For each guyed tower, the loads on the mast based on the geometry, type of ladder, transmission lines, and material properties are prepared by MASTLOD; the loads due to the parabolic dishes are prepared by DISHFORC. Other loads from other types of mountings such as ice guards, yagi antennas, etc. are calculated by tables provided by the manufacturers. Then all loads which are applied to the mast, gathered from different programs, are combined and the geometrical and material properties of the guys are added

to form an input for GUYMAST which performs the structural analysis.

For self supporting towers, the same procedure for load preparation is used but the towers are analyzed using MAST.

3.3 MASTLOD

This program is used to calculate the loads imposed on the mast due to its own weight, wind load and ice loads.

3.3.1 Loads

The type of mountings, ladders, antennas, and ice guards have a significant effect on the loads generated on the tower. These mountings are taken into consideration as follows:

1- Ladders : Towers are usually provided with ladders to facilitate access to the antennas. These induce a vertical load and moments in both x and y axes which are determined by the distance between the centre of the tower to the ladder and the azimuth as shown in Fig. 3.1.

2- Transmission Lines : There is a wide range of transmission lines used in the towers. The forces and the moments imposed on the towers due to transmission lines are determined by the distances and the azimuth as shown in Fig. 3.2.

3- Ice guards : These are platforms usually fixed right above the parabolic dishes and antennas to protect them from melting ice from the tower (Fig. 3.3).

4- Antennas : The loads generated from the different types of antennas (other than

the parabolic dishes which are calculated by DISHFORC) have to be calculated based on the bare weight and iced weight and the exposed surface area for bare and iced cases. Examples of types of antennas commonly used are shown in Figs. 3.4 and 3.5.

3.3.2 Modelling Restrictions

The towers considered are restricted to the following:

- the mast may have only one of the two cross-section types: a square or an equilateral triangle.
- all faces in any one panel must be of identical configuration.
- materials of all similar members in any one panel must be identical.
- only seven standard types of panels may be used. (Fig. 3.6)
- within any one region which could be as small as one panel, the panel heights and slopes of the legs must be constant.

3.3.3 Treatment of Loads

Generally the loads are treated according to CAN/CSA-S37-M85. In addition the following assumptions are used:

- Wind is always horizontal and in the same direction at all elevations on the tower.
- Ice is built up uniformly on all members of the tower and around each member.
- Adjoining and intersecting members may have reduced ice and wind loads

due to overlap, depending on the proximity to other members and obstructions.

- Eccentricity of the member with respect to the tower centre line will produce a bending moment about the centre line due to gravity and ice loads, as well as a torsional moment due to wind.

3.4 DISHFORC

This program calculates the loads imposed on the tower due to the parabolic (microwave) dishes or conical antennas mounted on the towers. A combination of the following types may be used (Fig. 3.7):

- Standard dishes
- Standard dishes with radome
- High performance dishes
- Grid dishes
- Conical horns

The program uses the properties for the dead loads and the wind surface area based on the values provided by Andrew Antennas Corporation in their catalogues.

3.5 GUYMAST

This program is used to perform the analysis of guyed towers through the use of three separate models. These are:

- 1- A set of guys connected at the same elevation on the mast, used independently for each guy level (Fig. 3.8).

2- A continuous beam model using the stiffness method (including beam-column interaction) used to analyze the bending of the mast in the two orthogonal vertical planes (Fig. 3.9).

3- A shaft subjected to torsional loading for the analysis of the tower twist (Fig. 3.10).

First, with an arbitrary set of initial displacements, the guy model is used to obtain the guy stiffness and guy loads to be applied to the mast models to calculate a new set of mast displacements which are then fed back into the guy model to obtain a better approximation of guy stiffness and loads. The process is repeated until displacements calculated in the beam model match those used by the guy model in determining the support stiffness within a specified tolerance.

3.5.1 Mast Model

The mast model consists of a continuous beam-column having elastic supports capable of providing one reactive force and one reactive moment. The reactive forces and moments are determined in the guy model on the basis of displacements and rotations calculated in the corresponding mast models. All loads on the beam model are assumed to be acting in the same plane as the displacements and rotations, and must pass through the centroid of the cross-section.

The base of the tower is always assumed to be constrained from displacements only but all other supports are assumed to be free to rotate and displace as determined by the guy sets.

For axial shortening calculations, the mast is subdivided according to guy attachment levels, loads, and discontinuity in mast properties. The displacements are then accumulated up the tower.

For the calculation of the twist, the tower is modelled as a shaft subjected to torsional moments and resisted by torsional supports determined by the guy sets as shown in Fig. 3.10. Mast bending and torsion are assumed to act independently of each other, which is acceptable for small deformations of the towers.

For the analysis, the following assumptions are used:

- Loading may be concentrated, uniformly or linearly distributed, but must always be parallel or normal to the ground.
- Mast properties for bending and torsion are assumed to be constant between guy levels.
- For beam-column interaction the axial load is taken to be the average for each span.
- Mast and its elements remain in the elastic region for all loads used.
- Web member strains are ignored.

3.5.2 Guy Model

The guy model uses a parabolic approximation for the shape of the hanging guy as derived by *Odley [Odely 1966]* using the following assumptions :

- Wind is blowing parallel to the ground and of uniform velocity for the length of the guy.

- The guy curve is a parabola for all loading conditions.
- Drag and lift coefficients for a guy are as indicated by *Diehl [Diehl 1936]*.

As shown in Fig. 3.11 the approximate length of the guy L , is given by :

$$L = a \left(\sec \omega + \frac{w^2}{24H^2 \sec^3 \omega} \right) \quad (3.1)$$

where :

L is the approximate length in metres

a is the horizontal projection of the guy in metres

ω is the angle of inclination of the guy chord to the horizontal

w is the total weight of the guy in kN.

H is the horizontal component of the tension in the guys in kN.

and H is computed from :

$$H = \frac{\cos \omega}{2} [w \sin \omega + \sqrt{4T_i^2 - w^2 \cos^2 \omega}] \quad (3.2)$$

where T_i is the initial tension in the guy, kN.

The stretch in the guy is calculated from :

$$\Delta_g = \frac{Ha}{A_g E_g} \left(\sec^2 \omega + \frac{w^2}{12H^2} \right) \quad (3.3)$$

where :

Δ_g stretch of the guys (m)

A_g metallic area of the guys (mm²)

E_g modulus of elasticity of the guy (GPa)

Therefore the unstressed length of the guy at normal temperature is :

$$L_o = L - \Delta_g \quad (3.4)$$

and at a temperature t °C, the unstressed length, L_t , is

$$L_t = L_o [1.0 + \alpha_t (t - t_o)] \quad (3.5)$$

where: α_t = coefficient of thermal expansion

3.5.2.1 Wind Loads on The Guys

For the case of wind blowing on the guy at an angle, Fig. 3.12 shows the forces acting on the guy. The projection of these forces on a horizontal plane makes an angle of κ with the direction of the wind. In Fig. 3.12,

W the total weight acting vertically down,

d_o the total drag acting parallel to y-axis,

l_1 the total lift parallel to BD (projection of the guy on xz axis) and normal to d_o ,

θ is the true angle between the guy chord and the wind.

The total drag, d_o and the total lift, l_1 are calculated from the following formulas :

$$d_o = 4.622 (C d v^2) * C_D * 10^{-6} \text{ kN} \quad (3.6a)$$

$$l_1 = 4.622 (C d v^2) * C_L * 10^{-6} \text{ kN} \quad (3.6b)$$

where :

C is the chord length of the guy (m)

d diameter of the guy (mm)

v wind velocity (km/h)

C_D , drag coefficient

C_L , lift coefficient

The values for C_D , C_L are from the curves given by *Diehl [Diehl 1936]* for θ varying from 0° to 90° .

In Fig. 3.13, the tower has deflected a distance Δ , in the direction of the wind at the point of guy attachment. Thus the guy lies in a new plane that contains the resultant W_r of all the forces acting on the guy. This new guy plane is presented by the triangle B_1O_1A in the figure. In Fig. 3.13, W_r is given by:

$$W_r = \sqrt{L_h^2 + d_o^2 + (W - L)^2} \quad (3.7)$$

where :

L_h is the component of the total lift L , in the x-direction

L is the component of the total lift L , in the z-direction

For any particular value of Δ and temperature, t , the value of H_1 is determined by trial as follows :

1- Assume a value for H_1

2- Compute

$$L_1 = a_1 \left(\sec \omega_1 + \frac{W_r^2}{24H_1^2 \sec^3 \omega_1} \right) \quad (3.6)$$

3- Compute

$$\Delta'_g = \frac{H_1 a_1}{A_g E_g} \left(\sec^2 \omega_1 + \frac{W_r^2}{12H_1^2} \right) \quad (3.9)$$

4- Compute

$$\hat{L}_t = L_1 - \Delta'_g \quad (3.10)$$

5- Compare the value of L'_t given by Eq. (3.10) with that given by Eq. (3.5). Since the unstressed length of the guy is invariant, both values should be equal. If the values do not agree, assume a new value for H_1 and repeat the calculations until satisfactory agreement is reached.

Having obtained a value for H_1 , forces at the tower, anchorage and the guy tension may be found. The above procedure is repeated for all guys at the same attachment point from which the resultant force, R , for a specified guy level at a certain deflection, Δ , may be calculated.

Fig. 3.14 shows the relation between the resultant of the forces, R , and the deflection, Δ . Assuming that the change in R is linear for two successive values of Δ , then

$$R = K\Delta + Q \quad (3.11)$$

where K is the slope of the secant of the reaction deflection curve for the guy set between the calculated deflection and at a deflection slightly greater than that.

3.5.2.2 Ice Loads On The Guys

Ice loading on a guy not only increases the weight of the guy, but also its effective area subject to wind pressure.

The ice accretion on the guys is assumed to be uniform and the diameter of the guys, d , in Eq. (3.6) is taken to be the diameter with ice, and the total weight, W , is the total weight of the guy plus the weight of ice.

3.6 MAST

Program MAST is for the analysis of self supporting towers using a space truss model based on the stiffness method. The program uses the output from MASTLOD as part of the input, thus the same modelling limitations explained in section 3.3 are valid. For the analysis, the following assumptions are used for the treatment of the loads :

- All loads used internally in MAST must be joint loads at panel points on the legs.
- A load applied between panel points is distributed to the panel points in inverse proportion to its distance from the two nearest panel points.
- A load applied on an antenna above the top of the tower is brought to the top of the tower along with the statically equivalent moments.
- A load applied within the region defined by the tower faces at that elevation is distributed to the legs and faces in inverse proportion to its distance from the legs and faces of the tower.
- Bending moments are converted into equivalent horizontal force couples applied at the top and bottom of the nearest panel and acting in a vertical plane.
- Torsional moments are converted into a couple of equivalent horizontal forces one face width apart acting in the horizontal plane at the same elevation as the torsional moment.
- A torsional moment outside the tower faces acts on the legs defining the face corresponding to the influence zone in which the moment acts.

- A torsional moment acting between influence zones is distributed to the adjoining influence zones in inverse proportion to its angular distance from each zone.

CHAPTER IV

METHODOLOGY

4.1 General

The process of developing a code from the working stress design to a limit states design is usually done by calibrating the new criteria to certain structural elements in the current design specifications thus establishing a link between the two design methods. This procedure of calibration as described by *Allen [Allen 1975]* has been followed by the Canadian Standards while shifting to LSD in CSA S16.1-1974; a similar approach was also used by *Galambos [Galambos 1976]* for finding the appropriate load and resistance factors for the AISC-LRFD in the U.S.A. A brief explanation of both methods and that used throughout this study is given in this chapter.

4.2 Probabilistic Study

The methods used to derive the load and resistance factors use essentially the same two "second-moment" statistical properties of the mean and the standard deviation.

4.2.1 CSA LSD Code

The basis for the LSD code in Canada is shown in Fig. 2.3 where the uniformity of reliability was insured by adjusting the value of the safety index β . For the determination of the load factors α , and the resistance factors ϕ , first values for α and ϕ are assumed, and then these values are adjusted until:

- (i) β is reasonably constant for the whole domain
- (ii) there is some matching over at least part of the domain between the old and the new standards.

This is illustrated schematically in Fig. 4.1.

4.2.2 AISC-LRFD Code

For the AISC-LRFD code, the load and resistance factors were determined by deciding first a common safety index β on the basis of calibration to standard members, and then applying this β to all structural elements through the linearization of the expression for β in Eq. 2.3.

The following expressions for the load factors for dead load and wind load were obtained:

$$\alpha_D = 1 + 0.55\beta\sqrt{V_E^2 + V_D^2} \quad (4.1)$$

$$\alpha_W = 1 + 0.55\beta V_W \quad (4.2)$$

where :

V_E is the coefficient of variation of structural analysis

V_D is the coefficient of variation of the dead load

V_w is the coefficient of variation of the wind load

Based on statistical information combined with experience and from rough knowledge of actual failure rates, the values of the coefficients of variation were determined from which the load and resistance factors were calculated.

4.2.3 Application to Guyed Towers

The two methods described above are applicable to a linear and determinate structure which is not the case with guyed towers. But based on the probabilistic approach, the second method could be applied to verify the load factors. Using the same values for V_B , V_D and V_w as those used by Allen [Allen 1975] (which are 0.07, 0.07, 0.25 respectively) and assuming that V_I is equal to V_w , taking a value for β of 3.4 (which lies in the middle of the usual range of 3.0 to 3.8) and substituting in Eq. 4.1 and 4.2, the corresponding values for the load factors are predicted to be:

$$\alpha_D = 1.19$$

$$\alpha_w = 1.47$$

$$\alpha_I = 1.47$$

However, these values cannot be used directly for guyed towers due to the non-linear response of this type of structure since the principle of superposition is not valid. For this study, a rather simple but justified method is used to find partial load factors based on the study of different existing towers.

4.3 Towers For The Study

A wide range of towers has been considered in this study so as to provide an envelope to permit the determination of the partial load factors.

These are existing towers, the data for which were available from various manufacturers and consultants so as to be representative of the trend of different designers. Care has been taken to ensure confidentiality of all the information regarding the designer, manufacturer, owner and location of the tower.

The matrix of towers used was:

- Range of height of towers - 27 to 342 metres
- Reference wind pressure - 300 to 1000 Pa
- Radial ice thickness - 10 to 50 mm
- Member type - angles and solid rounds

Table 4.1 shows the matrix of the towers included in the investigation. The towers are designated as follows:

-The first character is a letter from A to F representing different height groups as follows:

- Group A 0 - 150 ft. (0 - 45.8 m)
- Group B 150 - 250 ft. (45.8 - 76.2 m)
- Group C 250 - 400 ft. (76.2 - 122.0 m)
- Group D 400 - 700 ft. (122.0 - 213.5 m)
- Group E 700 - 1000 ft. (213.5 - 305.0 m)
- Group F > 1000 ft. (> 305.0 m)

-The second character is a roman number I,II,III,IV representing the different ice classes as specified by CAN/CSA-S37-M86.

-The third character is the serial number of the tower in a certain height group and ice thickness.

As an example, tower BIII means a tower whose height is between 150-250 ft (Group B) and lies in ice class II with ice thickness 25 mm and it is the first tower in that group.

The profiles of these towers are shown in Figs. 4.2 to 4.38.

4.4 Towers Under Specified Loads

The towers were first analyzed under the specified loads according to CAN/CSA-S37-M86 as described in the following.

4.4.1 Dead Loads

The dead load (D), was taken the total of:

- The weight of the structure including mast and guys.
- The weight of all supplementary equipment like antennas, dishes, ice guards, etc.

4.4.2 Wind Load

The wind pressure on the tower was calculated from the following formula :

$$p = q C_e C_g C_a \quad (4.3)$$

where :

p = design wind pressure independent of the drag factor of the object.

q = reference velocity pressure and taken as

-once in 30 years return hourly pressure for the site given in the Supplement to the National Building Code of Canada (NBCC).

or obtained from the wind map giving three wind classes as in Fig. 2.4.

In both cases "q" has to be modified according to the local topography, weather or other conditions.

For the present study q was taken as the wind pressure for which the tower was originally designed.

C_e = height factor which is given by :

$$C_e = \left(\frac{H_x}{10} \right)^{0.2}$$

where H_x is the height in metres

C_g = gust factor, usually taken as 2.0

C_a = acceleration factor for wind on antennas or structures located on or near the roof surface of buildings, which is taken as 1.0 unless stated otherwise.

4.4.3 Ice Load

The ice load (I) considered was the load produced by ice formed radially on all exposed surfaces of all members of the towers including the guys and the supplementary equipment. The density of the ice was taken as 900 kg/m³. The radial ice thickness used in the present study was the same thickness for which the tower was originally designed.

4.4.4 Load Combinations

The following load combinations were considered :

$$D + W_o \quad (4.5)$$

$$D + I + W_i \quad (4.6)$$

where W_o (Wind load for bare condition) is taken as

$$W_o = P(C_{df}A_f + C_{dr}A_{fr}) \quad (4.7)$$

and W_i (wind load for iced condition) is taken as

$$W_i = 0.5 P(C_{df}A_f + C_{dr}A_{fr} + C_{di}A_i) \quad (4.8)$$

where :

C_{df} = drag factor for flat members

C_{dr} = drag factor for round members

A_f = face area of flat members, bare

A_{fr} = face area of round members, bare

A_i = face area for radial ice

For each load condition, three wind directions were considered as shown in Fig. 4.39.

Thus a total of six loading conditions were considered for each of the towers analyzed.

4.5 Towers Under Factored Wind Loads

To find the appropriate load factor for wind, α_w , which would result in design similar to the existing towers, the bare towers were analyzed under the following factored loads:

4.5.1 Dead Load

To analyze the towers under factored loads, a load factor for dead loads, α_D had to be determined first. The following factors were used:

$\alpha_D = 1.25$ for the weight of the mast and for the weight of all supplementary equipment.

$\alpha_D = 1.0$ for the self weight of the guys.

These are further discussed in detail in Section 5.2

4.5.2 Wind Load

A wind load factor α_w , of 1.4 to 2.0 at increments of 0.1 was applied to the specified reference velocity pressure for which the towers were designed. After the analysis of the first three towers, the wind load factor was limited to only three values: 1.4, 1.5 and 1.6. This will be further discussed in Section 5.3.

4.5.3 Load Combinations

Only one load combination of the bare towers with wind was considered as follows:

$$\alpha_D D + \alpha_w W_o$$

and the wind was considered to be blowing at three different angles to the tower as shown in Fig. 4.39. Thus three load conditions for each tower were considered.

4.6 Towers Under Factored Ice Thicknesses

To find the appropriate ice thickness factor, α_i , iced towers subjected to wind were considered with the following loadings

4.6.1 Dead Load

The dead load factor, α_D , was taken the same as in the previous case of $\alpha_D = 1.25$ for the weight of the mast and all supplementary equipment, and $\alpha_D = 1.0$ for the self weight of the guys.

4.6.2 Wind Load

The wind load taken for iced tower, W_i , was taken as half the design wind pressure, p , (as specified in the current standard) multiplied by a load factor for wind, α_w , of 1.5. This will be further discussed in Section 5.3.

4.6.3 Ice Load

The following ice thicknesses were considered in the analysis:

- For towers designed for ice class I with specified thickness of 10 mm, the radial ice thickness was taken as 14 mm, 16 mm and 18 mm, corresponding to α_i of 1.4, 1.6, and 1.8 respectively.
- For towers designed for ice class II with specified thickness of 25 mm, the radial ice thickness taken was 30 mm, 32 mm, 34 mm and 35 mm, corresponding to α_i of 1.2, 1.28, 1.36, and 1.4 respectively.
- For towers designed for ice class III with specified ice thickness of 40 mm, the radial ice thickness taken was 48 mm, 50 mm, and 52 mm. corresponding to α_i of 1.2, 1.25, and 1.3 respectively.
- For towers designed for ice class IV with specified ice thickness of 50 mm, the radial ice thickness taken was 56 mm, 58 mm, and 60 mm. corresponding to α_i of 1.12, 1.16, and 1.2 respectively.

4.6.4 Load Combinations

The only load combination considered for this set of analysis was the case of wind blowing on iced tower:

$$\alpha_D D + \alpha_I I + \alpha_W W_i$$

with α_D , α_I , α_W specified as above and the wind blowing from three wind directions of $0^\circ, 90^\circ, 180^\circ$, as shown in Fig. 4.39. Thus three loading conditions were considered for each tower.

4.7 Analysis of Climatic Data

In LSD, safety and serviceability are controlled not only through the use of partial load and resistance factors but also by defining specified loads and material properties statistically in terms of probability level for material properties and return period for wind or ice loads. Thus this study has been carried out to determine

- appropriate radial ice thickness that could be used as the specified load for different ice classes, and
- the wind pressure to be applied in case of iced towers.

The information regarding the ice accretion is from the Ice Accretion Handbook by Environment Canada [*Environment Canada 1974*].

4.7.1 Climatic Data

For the towers in the field, the radial ice thickness is the amount of ice accretion around the members or the guys. In the handbook mentioned above, radial ice thickness

was defined as the probable equivalent ice accretion around a 25 mm conductor (which may be considered an average size for the structural elements of a tower) and computed from the following formula developed by *Châiné and Castonguay [Châiné and Castonguay 1974]*. :

$$A_r = \sqrt{\frac{k \cdot r}{2} (\sqrt{A_h^2 + A_v^2}) + r^2} - r \quad (5.4)$$

where :

k = is correction factor depending on size of the conductor

A_v = accumulation of ice on vertical surface (mm)

A_h = accumulation of ice on horizontal surface (mm)

r = radius of the conductor (mm)

A_r = equivalent radial thickness (mm)

Also, this Handbook provided statistical analysis based on the actual data gathered from most Canadian weather stations for ice storms for at least nine to fifteen years of record. Using Gumbel Extreme Value Distribution, equivalent radial ice thicknesses were calculated for 2, 5, 10, 30 and 40-year return periods. In addition, the maximum wind speed during an ice storm (defined as the maximum wind speed observed during an ice storm and also for that part of the 24 hour period following the storm during which the temperature remains below 0 °C.) was also recorded. From this data the value of the maximum wind speed that would occur with the maximum radial ice thickness was predicted.

4.7.2 Specified Radial Ice Thickness

Similar to the return period for wind loads, the radial ice thickness should be related to a specified return period. Thus from the values recorded and predicted in the Handbook, it was found that the maximum radial ice thickness with a probability of occurrence $1/40$ would give a reasonable value of ice loading (column 2 in Tables 4.2 to 4.7).

Note that because the degree of uncertainty of calculating the ice thickness is greater than that of the wind load, a 40 year return period was used for ice compared to only 30 years for the wind.

4.7.3 Wind Load During An Ice Storm

It is of extreme importance for the study of the loads on antenna towers to study the wind speed at maximum ice. It has been observed that the maximum wind speed seldom occurs with the maximum ice accretion. Therefore the factor that could be applied to the maximum wind pressure (30-year return period) to compute the wind pressure at the maximum ice accretion had to be determined and in this study it was determined as follows:

- For each of the available Canadian weather stations the maximum wind speed at maximum ice accretion was determined from the Handbook (column 4 in Tables 4.2 to 4.7).
- For the same stations the maximum wind speed in 30-year return period (column 5 in Tables 4.2 to 4.7) was calculated from the maximum wind pressure provided by the

NBCC [NBCC 1990] using the following formula :

$$q(\text{kPa}) = 50 * 10^{-6} v^2$$

where v is the wind speed in km/h.

-The ratio between the maximum wind speed predicted to occur with the maximum ice thickness in 40-year to the maximum wind speed in 30-year is calculated (column 6 Tables 4.2 to 4.7).

-The relation between this ratio and the wind speed is shown in Fig.4.40. From that figure, it can be concluded that this ratio is not a function of the wind speed.

-The relation between this ratio and the radial ice thickness is shown in Fig. 4.41, which also shows that it is not a function of the ice thickness.

-The relation between the number of the stations and this ratio is shown in Figs. 4.42a and 4.42b.

4.7.4 Results

From Fig. 4.42b it can be shown that a ratio of 0.77 would cover about 95% of the stations.

-Choosing this value, the reference wind pressure for iced towers can be computed as:

$$\begin{aligned} p_i &= (0.77)^2 p \\ &= 0.6 p \end{aligned}$$

where p_i = the maximum hourly wind pressure under iced conditions

p = maximum hourly wind pressure (30-year return period)

Comparing the values of the maximum radial ice accretion (40-year return period)

based on climatic observations shown in column 2, Tables 4.2 to 4.7, to the values specified in the map of the current standard CAN/CSA S37-M86 (Fig. 2.4), it was noticed that in some of the stations the recorded ice thickness is much greater than that specified by the standard. Thus a new map based on these values is recommended, as shown in Fig.4.43. The ice thicknesses as specified by the proposed map are shown in column 3 of Tables 4.2 to 4.7 for comparison. It should be noted that these proposed changes to the ice map agree to a great extent with that provided by the Ontario Highway Bridge Design Code [OHBDC 1983] as shown in Fig. 4.44.

-This new map is proposed based on new thicknesses for different ice classes as follows:

$A_{mr} = 0 - 10 \text{ mm}$	Ice Class I	Specified Thickness = 10 mm
$A_{mr} = 11 - 20 \text{ mm}$	Ice Class II	Specified Thickness = 20 mm
$A_{mr} = 21 - 30 \text{ mm}$	Ice Class III	Specified Thickness = 30 mm
$A_{mr} = 31 - 40 \text{ mm}$	Ice Class IV	Specified Thickness = 40 mm
$A_{mr} = > 40 \text{ mm}$	Ice Class V	Specified Thickness = open

where:

A_{mr} = maximum equivalent radial ice thickness with probability of occurrence 1/40.

CHAPTER V

RESULTS & DISCUSSION

5.1 General

As mentioned on the outset, the aim of this study is to determine the necessary partial load factors so as to change the current standard to a new LSD based standard.

These factors are :

- Load factor for dead load α_D
- Load factor for wind load α_W
- Ice thickness factor α_I

Also from the climatic data analysis described in Section 4.7, two other parameters are determined:

- Radial ice thickness depending on the location of the tower
- Design wind pressure for iced towers

The results of the analysis from which these values can be concluded are presented in this chapter.

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5.2 Load Factor for Dead Load

The load factor for dead load had to be determined before analyzing the towers under the set of the proposed wind load factors and ice thicknesses.

Following the concept of the limit states design described, the load factor is determined by the degree of uncertainty in calculating the load. Thus a dead load factor of 1.25 for the own weight of the mast and the weight of all supplementary equipment was used. This is the same dead load factor used by nearly all the standards to take into account any inaccuracy in calculating the loads, say, due to neglecting the weight of the gusset plates, welds, bolts, brackets for transmission lines, etc.

For the guys, a dead load factor of 1.0 is taken which is acceptable as their weight is to a great extent exactly determined.

5.3 Load Factor for Wind Load

From the analysis of the towers under specified loads it was possible to separate parts of a tower having maximum forces under bare conditions and the parts of a tower having maximum forces under iced conditions. For determination of the wind load factor, only those parts of a tower for which the bare condition was critical were included.

The following were considered in the comparison:

- maximum force in the mast legs for approximately every six metres and at each guy level.
- total face shear at each guy level and at points of application of concentrated loads.
- maximum guy forces at each guy level.

From this study it was found that :

- for towers designed for ice Class I, wind load without ice gave higher design forces in almost all parts of the mast and the guys, except for the very bottom part of the legs which are affected by iced condition.

- for towers designed for class III or IV ice loads, the tower is mainly affected by the iced conditions except at the top where some parts of the mast are affected by wind without ice.

- towers designed for class II ice loading lay between the above two cases depending on the value of the wind pressure.

The towers were analyzed again under factored loads as described in section 4.5, for a wind load factor from 1.4 to 2.0 in increments of 0.1 and the forces determined for each wind load factor. These forces were then compared with the forces under specified loads and their ratios determined. This process was repeated for towers BII, CII, and BIII.

From the above analysis it was found that this ratio exceeds 1.5 (which is equal to the current specification) for more than 92 % of the parts of the towers for a wind load factor of 1.6. Therefore for the rest of the towers the analysis was limited to only three wind load factors of 1.4, 1.5 and 1.6. Figures 5.1 to 5.3 show the effect of wind load factors on the design forces in members for wind load factors of 1.4, 1.5 and 1.6.

As explained in section 2.4, a ratio of 1.5 of forces due to factored load to that due to specified load would result in the same design as the current standard and from Fig. 5.4, it can be concluded that 63% of the members have a ratio of 1.45 to 1.55 and

also 84% of the members have a ratio greater than 1.45.

Therefore a wind load factor of 1.5 was considered appropriate. Also it should be noted here that it is the same load factor used by CAN/CSA-S16.1-M84.

5.4 Ice Thickness Factor

For antenna towers, the effect of ice and wind loads are coupled since increasing the ice thickness does not only increase the weight on the tower but would also increase the projected area for the wind. That is the reason for the complexity in finding an ice thickness factor.

For each ice class, a set of computer runs were made using different factors for ice thickness as explained in section 4.6 and following the same method for wind load factor. However, for the ice thickness factor only those parts of tower for which the iced condition was critical were included.

For each ice thickness, the ratio between the forces due to factored loads were compared to those due to specified loads as discussed below.

5.4.1 Ice Class I

For towers designed for ice class I with a specified ice thickness of 10 mm, the ice thickness was varied from 14 to 18 mm and Figs. 5.5 and 5.6 show the relation between the frequency of the members and the ratio between the forces due to factored loads and that due to specified loads.

As shown in Fig. 5.6, the ice thickness found appropriate was 18 mm, i.e. an ice thickness

factor of 1.8.

5.4.2 Ice Class II

The same procedure was repeated but for different ice thicknesses varying from 30 to 35 mm. From Figs. 5.7 and 5.8 it can be concluded that an ice thickness of 35 mm would result in a tower which is close to the present design, i.e. an ice thickness factor of 1.4.

5.4.3 Ice Class III

The same procedure was repeated for the towers designed for ice class III and the ice thickness was varied from 48 to 52 mm. As shown in Fig. 5.9 and 5.10 an ice thickness of 52 mm would give a tower close to the present design, i.e. an ice thickness factor of 1.3.

5.4.4 Ice Class IV

For the towers designed for class IV ice the ice thickness was varied from 56 to 60 mm and as shown in Figs. 5.11 and 5.12 the ice thickness that would result in a tower close to the present design is 60 mm, i.e. a load factor for ice thickness of 1.2.

5.5 Examining The Ice Thickness Factor

As shown in section 5.4, it was found that the ice thickness factor that could be applied to the existing specified ice thicknesses of 10, 25, 40, and 50 which results in

towers that are not dramatically different from the towers designed according to CAN/CSA-S37-M86 varies according to the ice class. However this cannot be justified in a LSD, as the load factor should be a function only of the degree of uncertainty of calculating the loads and not of the value of that load.

Upon examining the specified ice thicknesses according to the climatic data discussed in Section 4.6, new ice thicknesses were specified according to a 40-year return period thus resulting in a consistent degree of reliability irrespective of the value of the ice thickness.

As the degree of uncertainty in figuring the ice thickness, based on climatic data, could be considered equal to that in figuring the wind pressure, a factor of 1.5 to be applied to the radial ice thickness based on 40 years return period was considered appropriate.

5.6 Verifying the Proposed Load Factors for Guyed Towers

From the output of all the computer runs applied to all the guyed towers available and based upon climatic data analysis the following partial load factors are proposed:

- Dead load factor α_D : 1.25 for the mast and all supplementary equipment.
1.0 for the guys.
- Wind load factor α_w : 1.50
- Ice thickness factor α_i : 1.50
- Design wind pressure for iced condition equals to 0.6 that of bare condition
- Ice classes are specified according to section 4.7

Having determined the above partial load factors, the available towers were analyzed again with the proposed factors and the present code to compare both designs. The envelope of maximum leg forces and face shears of the towers and their variation with the elevation have been plotted for the two design methods in Figures 5.13 to 5.49. Also, the guy forces were determined and at every guy level the forces in the guys are recorded for both design methods, as shown in Tables 5.1 to 5.6.

5.7 Verifying The Proposed Factors For Self Supporting Towers

The standard for antenna towers include both types: guyed and self supporting towers. These proposed factors should apply to both cases as they have the same types of applied loading. Thus the proposed load factors (in addition to a dead load factor, α_d , of 0.85 for the cases of reversal of stresses) were used on four different self supporting towers, one from each ice class, and again compared to the existing standard. The envelopes of the maximum tension and compression in the legs, diagonals, and horizontals for each tower are shown in Figures 5.50 to 5.61.

5.8 Verifying the Service Loads

It is very important for antenna towers to be checked against rotations and deflections, as excessive rotations or deflection of the antennas would alter the function of the whole tower. Thus four guyed towers, one from each ice class, were examined under the proposed service loads. The deflections according to the specified loads in CAN/CSA S37-M86 and that according to the proposed service loads are shown in Figs.

5.62 to 5.65.

5.9 Comparison Between Both Designs

From Figs. 5.13 to 5.61 that compare both design forces for guyed and self supporting towers it can be noticed that:

- for towers designed for ice classes I and II, regardless of the height group, the proposed partial load factors result in approximately the same leg forces but with a minor increase in the face shear that may not change the sizes of the web members used.
- for towers designed for ice classes III and IV, the proposed load factors would result in higher values for some parts of the legs which may have been under-designed for extremely high values of ice accretion and wind pressure.

From Tables 5.1 to 5.6, it can be seen that for towers designed for ice classes I and II the maximum guy tensions under the proposed load factors are slightly lower; for towers designed for ice classes III and IV, they are higher in some guy levels.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

1) In order to change the existing standard CAN/CSA-S37-M86 to a truly limit states design, (i.e. such that towers designed would not be dramatically different from the current design but have a more uniform degree of safety against collapse), the following load factors should be used:

- Load factor for dead load α_d
 - = 1.25 for the mast and mountings.
 - = 1.0 for the guys
 - = 0.85 for the cases of overturning and stress reversal.
- Load factor for wind load $\alpha_w = 1.5$
- Ice thickness factor for ice loads $\alpha_i = 1.5$
- A wind pressure factor of 0.6 to be applied to the wind pressure in case of iced towers.

2) A better ice thickness specification should be used based on the climatic data and the statistical extrapolation for 40 years return period; thus the ice map of Canada should be changed to a new map.

6.2 Recommendations For Further Study

1) Reliability analysis should be carried out for the towers in which the reliability of the whole structure could be studied under the proposed factored loads and compared to the reliability of the towers designed according to CAN/CSA-S37-M86.

2) The effect of the variation of the ice thickness with the size of the members should be studied.

3) Towers should be analyzed under dynamic loading from which quasi-static load combinations could be added to consider any dynamic effect of wind.

ICE CLASSES	HEIGHT GROUPS					
	Height in Feet					
	A <150	B 150-20	C 250-400	D 400-700	E 700-1000	F >1000
I (10 mm)	1	2	3			
II (25 mm)	3	4	3	1	1	1
III (40 mm)	2	2	2	1		1
IV (50 mm)	2	4	4			
TOTAL	8	12	12	2	1	2

Table 4.1 : Classification of Towers

Site		Max ice thick. 1/40 (mm)	Ice thick. by proposed map	Max Hourly Wind Speed (km/h)		Wind Speed Ratio
				Max ice	1/30	
Fort Nelson	B.C.	3	10	69	43	0.62
Kimberley	B.C.	10	10	76	23	0.30
Penticton	B.C.	2	10	102	55	0.54
Smith River	B.C.	3	10	71	32	0.45
Calgary	ALTA.	10	10	96	55	0.57
Edmonton	ALTA.	3	10	89	63	0.71
Grande Prairie	ALTA.	4	10	94	52	0.55
Lac La Biche	ALTA.	3	10	86	50	0.58
Lethbridge	ALTA.	2	10	123	58	0.47
Medicine Hat	ALTA.	6	10	99	48	0.48
Peace River	ALTA.	5	10	76	52	0.68
Rocky Mtn. House	ALTA.	5	10	80	42	0.53
Vermillion	ALTA.	8	10	75	45	0.60
Wagner	ALTA.	7	10	82	43	0.52
White Court	ALTA.	4	10	88	42	0.48
Broadview	SASK.	9	10	80	53	0.66
Estevan	SASK.	5	10	101	55	0.54
Hudson Bay	SASK.	5	10	82	48	0.59
Moose Jaw	SASK.	5	10	93	50	0.54
North Battleford	SASK.	10	10	111	35	0.32
Prince Albert	SASK.	6	10	82	45	0.55
Regina A.	SASK.	6	10	88	55	0.63

**Table 4.2 : Climatic Data For Stations Located in Ice class I (10 mm)
Zone According to CAN/CSA-S37-M86**

Site		Max ice thick. 1/40 (mm)	Ice thick. by proposed map	Max Hourly Wind Speed (km/h)		Wind Speed Ratio
				Max ice	1/30	
Saskatoon	SASK.	5	10	53	94	0.56
Swift Current	SASK.	9	10	69	106	0.65
Yorkton	SASK.	10	10	58	86	0.67
Brandon	MAN.	13	20	60	95	0.63
Dauphin	MAN.	33	20	52	86	0.60
Gimli A.	MAN.	18	20	52	86	0.60
The Pas	MAN.	8	10	43	93	0.46
Portage La Prairie	MAN.	11	20	64	93	0.69
Rivers	MAN.	13	20	63	93	0.68
Winnipeg	MAN.	15	20	56	92	0.61
Armstrong	ONT.	15	20	29	71	0.41
Graham	ONT.	27	30	34	71	0.48
Kapuskasing	ONT.	19	20	31	75	0.41
Kenora	ONT.	15	20	35	69	0.51
Landsdowne House	ONT.	12	20	42	76	0.55
Nakina	ONT.	14	20	32	69	0.46
Sault Ste Marie	ONT.	33	30	40	86	0.47
Sioux Lookout	ONT.	13	20	39	71	0.55
Thunder Bay	ONT.	26	30	52	76	0.68
Timmins	ONT.	20	20	37	80	0.46
Trout Lake	ONT.	11	20	48	88	0.55
White River	ONT.	12	20	34	69	0.49

**Table 4.2 cont. : Climatic Data For Stations Located in Ice class I (10 mm)
Zone According to CAN/CSA-S37-M86**

Site		Max ice thick. 40 yrs (mm)	Ice thick. by proposed map	Max Hourly Wind Speed (km/h)		Wind Speed Ratio
				Max ice	1/30	
Abbotsford	B.C.	21	20	50	105	0.48
Comox	B.C.	9	20	47	108	0.44
Fort St. John	B.C.	7	20	63	85	0.74
Nanimo	B.C.	6	20	23	108	0.21
Prince George	B.C.	8	20	40	77	0.52
Prince Rupert	B.C.	1	20	48	100	0.48
Quesnel	B.C.	8	20	19	76	0.25
Smithers	B.C.	3	20	34	86	0.40
Terrace A.	B.C.	21	20	39	81	0.48
Vancouver	B.C.	7	20	32	108	0.30
Earlton	ONT.	17	20	40	89	0.45
Gore Bay	ONT.	20	30	43	85	0.51
Killaloe	ONT.	16	30	47	76	0.52
London	ONT.	28	30	58	98	0.59
Moosonee	ONT.	15	20	37	69	0.54
Mourt Forest	ONT.	25	30	63	86	0.73
North Bay	ONT.	26	30	64	79	0.81
Ottawa	ONT.	25	30	47	86	0.55
Somcoe	ONT.	30	30	42	88	0.48
Sudbury	ONT.	22	30	55	89	0.62
Toronto	ONT.	27	30	56	98	0.57

**Table 4.3 : Climatic Data For Stations Located in Ice class II (25 mm)
Zone According to CAN/CSA-S37-M86**

Site		Max ice thick. 1/40 (mm)	Ice thick. by proposed map	Max Hourly Wind Speed (km/h)		Wind Speed Ratio
				Max ice	1/30	
Trenton	ONT.	23	30	45	93	0.48
Warton	ONT.	18	30	74	93	0.80
Windsor	ONT.	24	30	61	85	0.72
Maniwaki	P.Q.	15	30	24	75	0.32
Montreal	P.Q.	47	30	84	86	0.95
Nitchequon	P.Q.	12	20	60	82	0.73
Roberval	P.Q.	23	30	52	72	0.72
St. Hubert	P.Q.	23	30	90	86	1.0
Schefferville	P.Q.	17	20	77	88	0.88
Val D'or	P.Q.	18	20	42	76	0.55
Dawson A.	Y.T.	3	10	47	69	0.68
Teslin	Y.T.	4	10	43	71	0.61
Watson Lake	Y.T.	5	10	39	69	0.57
Whitehorse A.	Y.T.	9	10	40	82	0.49
Fort Resolution	N.W.T.	8	10	77	85	0.90
Fort Simpson	N.W.T.	5	10	43	86	0.50
Fort Smith	N.W.T.	5	10	55	86	0.64
Hay River	N.W.T.	4	10	43	80	0.54
Norman Wells	N.W.T.	2	10	72	108	0.67
Yellowknife	N.W.T.	4	10	53	93	0.57

**Table 4.3 Cont. : Climatic Data For Stations Located in Ice class II (25 mm)
Zone According to CAN/CSA-S37-M86**

Site		Max ice thick. 1/40 (mm)	Ice thick. by proposed map	Max Hourly Wind Speed (km/h)		Wind Speed Ratio
				Max ice	1/30	
Churchhill	MAN.	21	30	68	109	0.62
Mont Joli	P.Q.	22	30	64	118	0.54
Qubec A.	P.Q.	35	30	61	98	0.62
Sept Iles	P.Q.	33	30	69	130	0.53
Sherbrooke	P.Q.	12	30	45	72	0.63
Campellton	N.B.	13	30	27	98	0.28
Chatham	N.B.	46	30	47	86	0.55
Fredricton	N.B.	14	30	47	86	0.55
St. John	N.B.	31	30	72	98	0.73
Greenwood	N.S.	14	30	52	98	0.53
Yarmouth A.	N.S.	21	30	61	101	0.60
Goose	NFLD.	17	30	58	92	0.63
Cambridge Bay	N.W.T.	16	30	48	82	0.59
Chesterfield	N.W.T.	17	30	58	102	0.57
Coral Harbor	N.W.T.	26	30	69	155	0.45
Frobisher Bay	N.W.T.	10	30	55	117	0.47
Inuvik	N.W.T.	8	20	29	105	0.28

**Table 4.4 : Climatic Data For Stations Located in Ice class III (40 mm)
Zone According to CAN/CSA-S37-M86**

Site		Max ice thick. 1/40 (mm)	Ice thick. by proposed map	Max Hourly Wind Speed (km/h)		Wind Speed Ratio
				Max ice	1/30	
Moncton	N.B.	29	40	79	108	0.73
Halifax	N.S.	30	40	69	102	0.68
Sydney A.	N.S.	36	40	76	105	0.72
Truro	N.S.	17	40	39	98	0.40
Charlottetown	P.E.I.	31	40	103	105	0.98
Summerside	P.E.I.	24	40	90	112	0.80
Bonavista	NFLD.	39	40	61	112	0.54
Buchans	NFLD.	20	40	66	105	0.63
Cape Race	NFLD.	43	40	93	139	0.67
St. Anthony	NFLD.	23	40	60	123	0.49
St. John's	NFLD.	29	40	68	121	0.56

**Table 4.5 : Climatic Data For Stations Located in Ice class IV (50 mm)
Zone According to CAN/CSA-S37-M86**

Tower	Guy Level (m)	Maximum Forces (kN)		Ratio
		Specified Loads	Factored Loads	
AII1	41.2	26.2	36.9	1.41
	33.5	20.1	28.5	1.42
	22.9	20.4	28.0	1.37
	10.7	17.3	21.8	1.26
AII1	41.9	17.3	28.9	1.67
	26.7	17.3	28.5	1.65
	11.4	12.9	20.0	1.55
AII2	44.2	22.2	32.9	1.48
	29.7	18.2	27.6	1.52
	14.5	14.2	20.5	1.44
AII3	22.1	10.7	16.9	1.58
	9.9	10.7	16.0	1.50
AIII1	37.3	58.3	95.2	1.63
	23.6	41.8	70.3	1.68
	9.9	28.5	44.9	1.58
AIII2	38.9	53.8	84.9	1.58
	24.4	49.4	86.7	1.76
	9.9	32.5	53.8	1.66
AIV1	30.1	38.2	67.2	1.76
	14.9	37.8	68.5	1.81
AIV2	37.3	49.4	81.8	1.66
	23.6	56.5	96.5	1.71
	9.9	30.2	49.4	1.64
BI1	54.6	51.1	72.9	1.43
	36.3	38.2	45.8	1.20
	18.0	24.0	27.1	1.13
BI2	57.2	43.1	69.8	1.62
	41.9	31.6	40.9	1.29
	26.7	25.8	35.1	1.36
	11.4	13.8	17.8	1.29

Table 5.1 : Maximum Tension in Guys for Towers AI1 to BI2

Tower	Guy Level (m)	Maximum Forces (kN)		Ratio
		Specified Loads	Factored Loads	
BII1	70.1	31.6	48.9	1.55
	51.8	27.6	44.0	1.59
	33.5	27.6	43.6	1.58
	15.2	19.1	29.4	1.54
BII2	56.4	37.4	62.3	1.67
	35.8	27.1	46.3	1.71
	16.0	19.1	30.2	1.58
BII3	72.4	40.5	62.7	1.55
	53.3	31.1	50.7	1.63
	34.3	25.3	41.4	1.64
	16.0	17.8	26.7	1.50
BII4	58.7	26.2	41.4	1.58
	44.2	21.8	33.8	1.55
	29.0	21.3	33.8	1.59
	13.7	15.6	22.2	1.42
BIII1	48.2	44.0	73.8	1.68
	32.0	41.8	70.3	1.68
	14.9	30.7	49.4	1.61
BIII2	56.4	43.6	82.7	1.90
	41.2	45.8	90.7	1.98
	25.9	46.2	94.3	2.04
	10.7	21.3	41.4	1.94
BIV1	64.1	115.6	215.2	1.86
	48.8	114.3	197.0	1.72
	33.6	96.9	163.6	1.69
	15.3	61.8	108.1	1.75
BIV2	44.2	51.6	95.6	1.85
	28.2	40.9	76.4	1.87
	11.4	19.1	30.9	1.62

Table 5.2 : Maximum Tension in Guys for Towers BII1 to BIV2

Tower	Guy Level (m)	Maximum Forces (kN)		Ratio
		Specified Loads	Factored Loads	
BIV3	67.8	111.2	185.4	1.67
	54.1	113.8	195.2	1.72
	40.4	88.0	150.3	1.71
	26.7	77.4	137.4	1.78
	13.0	49.4	85.8	1.74
BIV4	54.1	128.1	225.0	1.76
	34.3	98.7	176.5	1.79
	14.5	64.0	107.6	1.68
CI1	100.7	42.2	55.1	1.31
	80.5	72.5	90.7	1.25
	60.4	28.9	37.4	1.29
	40.3	56.0	69.4	1.24
	20.1	52.5	60.9	1.16
CI2	72.9	25.3	39.6	1.57
	54.6	17.3	30.2	1.75
	36.3	17.3	31.6	1.83
	18.0	8.9	16.0	1.80
CI3	73.2	71.1	99.2	1.40
	57.9	30.7	42.7	1.39
	44.22	30.2	42.7	1.41
	29.0	29.3	41.4	1.41
	13.7	19.6	27.1	1.38
CI11	72.9	51.1	85.8	1.68
	57.6	35.6	58.7	1.65
	39.3	32.0	54.7	1.71
	20.3	27.1	46.7	1.72
CI12	80.8	42.7	61.8	1.45
	68.6	44.0	63.1	1.43
	52.6	41.8	58.7	1.41
	34.3	22.2	31.1	1.40
	14.5	33.3	43.6	1.31

Table 5.3 : Maximum Tension in Guys for Towers BIV3 to CI12

Tower	Guy Level (m)	Maximum Forces (kN)		Ratio
		Specified Loads	Factored Loads	
CII3	96.8	76.7	36.9	1.41
	83.1	68.9	28.5	1.42
	67.9	40.9	28.0	1.37
	51.1	33.4	21.8	1.26
	32.8	34.2	28.9	1.67
	14.5	20.9	28.5	1.65
CIII1	112.1	152.1	250.8	1.65
	95.3	129.4	208.5	1.61
	74.7	116.1	190.8	1.64
	55.7	96.5	166.3	1.72
	35.8	71.6	137.4	1.92
	16.0	56.9	106.3	1.87
CIII2	98.4	82.7	135.6	1.64
	83.9	74.3	117.4	1.58
	66.3	60.9	93.4	1.53
	48.8	44.9	68.0	1.52
	31.1	44.5	68.9	1.55
	13.7	38.2	54.7	1.43
CIV1	72.8	38.2	221.9	1.76
	52.4	37.8	205.9	1.81
	33.0	49.4	163.2	1.66
	17.8	56.5	112.9	1.71
CIV2	78.8	162.7	282.8	1.74
	54.4	120.1	203.2	1.69
	36.1	112.1	197.9	1.77
	20.8	110.7	195.6	1.77
CIV3	87.9	144.5	243.7	1.69
	61.5	179.6	313.9	1.75
	36.1	121.8	206.8	1.70
	17.8	79.6	128.5	1.61
CIV4	102.7	115.3	200.0	1.73
	81.3	92.8	155.6	1.68
	60.7	74.4	117.8	1.58
	41.7	70.4	109.8	1.56
	20.3	55.9	84.9	1.52

Table 5.4 : Maximum Tension in Guys for Towers CII3 to CIV4

Tower	Guy Level (m)	Maximum Forces (kN)		Ratio
		Specified Loads	Factored Loads	
DIII	129.6	67.8	114.3	1.69
	107.5	62.5	103.6	1.66
	85.4	57.6	91.1	1.58
	61.4	40.0	62.2	1.56
	41.1	39.6	63.6	1.61
	19.1	35.2	56.0	1.59
DIII	115.9	111.6	175.7	1.57
	96.1	84.9	138.3	1.63
	76.3	68.5	108.1	1.58
	56.4	64.0	96.9	1.52
	36.6	47.1	72.0	1.53
	16.8	42.7	64.9	1.52
EIII	205.9	161.4	251.7	1.56
	175.4	145.0	237.5	1.64
	144.9	117.4	195.2	1.66
	114.4	99.2	163.6	1.65
	83.9	90.7	151.6	1.67
	53.4	68.9	109.4	1.59
FIII	22.9	53.8	77.8	1.45
	342.5	271.9	473.1	1.74
	293.7	261.4	423.8	1.62
	244.3	218.7	327.3	1.50
	194.6	211.6	311.7	1.47
	145.2	153.6	232.6	1.51
FIII	93.9	94.6	153.4	1.62
	44.5	139.9	193.6	1.38
	307.3	407.0	779.1	1.91
	244.2	473.9	843.1	1.78
	178.9	334.4	565.2	1.69
	113.6	291.7	472.2	1.62
FIII	48.3	153.1	246.4	1.61

Table 5.5 : Maximum Tension in Guys for Towers DIII to FIII

Fig. 1.1 : A Self Supporting Antenna Tower

Fig. 1.2 : A Guyed Tower with Three Guy Levels

Fig. 1.3 : A Guyed Tower

31 Wire Galvanized Bridge Strand

103 Wire Galvanized Bridge Strand

Fig. 1.4 : Guy Strand and Bridge Strand

Fig. 2.1: Representation of Load Effect and Resistance as Random Variables

Fig. 2.2 : Probabilistic Representation of Working Stress Design

Fig. 2.3 : Definition of the "Safety Index", β

Class	Reference velocity pressure (q), Pa
A	450
B	600
C	850

Fig. 2.4: Wind Map of Canada as Specified in CAN/CSA-S37-M86

Class	Nominal ice thickness, mm
I	10
II	25
III	40
IV	50

Fig. 2.5: Ice Map of Canada as Specified in CAN/CSA-S37-M86

Fig. 3.1: Ladder Position and Orientation

Fig. 3.2: Transmission Lines-Position and Orientation

Fig. 3.3: An Ice Guard Above a High Performance Dish

Fig. 3.4: Examples of Yagi Antennas

Fig. 3.5: Examples of Omnidirectional Antennas

Fig. 3.6: Standard Types of Panels Used

Fig. 3.7: Examples of Microwave Dishes

Fig. 3.8: Typical Guy Model

Fig. 3.9: Mast Model

Fig. 3.10: Twist Model

Fig. 3.11: Elevation of the Guy in Normal Position

Fig. 3.12: Loads on the Guy

Fig. 3.13: Geometry of the Guy in Deflected Position

Fig. 3.14: R-Δ Curve for Guys at One Level

Fig. 4.1 : Schematic Representation of Calibration in LSD

Fig. 4.2 : Profile Of Tower A11

Fig. 4.3 : Profile Of Tower AII1

Fig. 4.4 : Profile Of Tower A112

Fig. 4.5 : Profile Of Tower AII3

1- 1HAY 2PANELS K72314743 @ 41m T.L 1 5/8"
2- 1PANEL K723147 @ 30m T.L 1 5/8"

Face Width = 610 mm (2.0')
Panel Height = 762 mm (2.5')

TOWER AIII1

WIND : $q = 862 \text{ Pa. (18.0 psf)}$ ICE : Class III 40mm (1.6")

Fig. 4.6 : Profile Of Tower AIII1

Fig. 4.7 : Profile Of Tower AIII2

ANTENNAS & LINES:

1- SRL-360 @ 31.1m T.L. 7/8"

2- SRL-210C @ 36.6m T.L. 7/8"

Face Width = 610 mm (2.0')

Panel Height = 381 mm (1.25')

TOWER AIV1

WIND : $q = 600$ Pa. (12.55 psf) ICE : Class IV 50mm (2.0")

Fig. 4.8 : Profile Of Tower AIV1

Fig. 4.9 : Profile Of Tower AIV2

Fig. 4.10 : Profile Of Tower BI1

Fig. 4.11 : Profile Of Tower B12

ANTENNAS & LINES:
1- ASP 600 @ 61m T.L. 7/8"

Face Width = 610 mm (2.0')
Panel Height = 762 mm (2.5')

TOWER BII2

WIND : $q = 600 \text{ Pa. (12.54 psf)}$ ICE : Class II 25mm (1.0")

Fig. 4.13 : Profile Of Tower BII2

Fig. 4.14 : Profile Of Tower BII3

Fig. 4.15 : Profile Of Tower BII4

1- KP4-15 GRID DISH @ 52.4m T.L 7/8"
2- SRL-210C-4 @ 53.3m T.L 7/8"

Face Width = 610 mm (2.0')
Panel Height = 381 mm (1.25')

Horizontal member: 1/2" S.R., 2' x 2' x 1/4", 1/2" S.R. REGULAR, 1/2" S.R.

Diagonal member: 1/2" S.R., 2' x 2' x 1/4", 1/2" S.R. REGULAR, 1/2" S.R.

Leg member: 1/2" S.R., 2' x 2' x 1/4", 1/2" S.R. REGULAR, 1/2" S.R.

Dimensions: EL = 175', EL = 158.8', EL = 105', EL = 48.8'

Loads: 0.6 KIPS, 75 KIPS, 21 KIPS, 34 KIPS @ 39', 26 KIPS, 130' RADIUS FLAT

Connections: 6" TORSION RESISTOR, 1/2" 1x7 GUY STRAND (GRP & GRP) LT. = 2.6 KIPS, 7/8" 1x7 GUY STRAND (GRP & GRP) LT. = 2.0 KIPS

WIND : $q = 600$ Pa. (12.5 psf) ICE : Class III 40mm (1.6")

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Fig. 4.17 : Profile Of Tower BIII2

Fig. 4.18 : Profile Of Tower BIV1

Fig. 4.19 : Profile Of Tower BIV2

Fig. 4.20 : Profile Of Tower BIV3

Fig. 4.21 : Profile Of Tower BIV4

Fig. 4.22 : Profile Of Tower CII

Fig. 4.23 : Profile Of Tower C12

Fig. 4.24 : Profile Of Tower CI3

A = 1" REGULAR
 AX = 1" X-BRACED
 B = 1" REGULAR

ANTENNAS & LINES:

- 1- SRL-480 @ 76.7m T.L. (4) LDF7-S0A
- 2- SRL-480 @ 76.7m T.L. (2) LDF7-S0A
- 3- SRL-480 @ 76.7m T.L. (2) LDF7-S0A
- 4- 8' HP DISH @ 64.9m T.L. LDFSP-S0A
- 5- 8' HP DISH @ 64.9m T.L. LDFSP-S0A
- 6- 8' HP DISH @ 43.9m T.L. LDFSP-S0A
- 7- 8' HP DISH @ 34.7m T.L. LDFSP-S0A

Face Width = 762 mm (2.5')
 Panel Height = 762 mm (2.5')

TOWER CII1

WIND : $q = 431 \text{ Pa. (9.0 psf)}$ ICE : Class II 25mm (1.0")

Fig. 4.25 : Profile Of Tower CII1

Fig. 4.26 : Profile Of Tower CH2

Fig. 4.27 : Profile Of Tower CII3

Fig. 4.29 : Profile Of Tower CH12

A = 7 2 | x 2 | x 2
B = 7 2 | x 2 | x 2
C = T T | 1 | x 1
D = T T | 1 | x 1
E = T T | 1 | x 1 (X-BRACING)

ANTENNAS & LINES:

- 1- U11X-12X-59CR @ 38.1m T.L (2) EWP-52
2- U11X-12X-59CR @ 23.2m T.L (2) EWP-52
3- U11X-12X-59CR @ 78.9m T.L (2) EWP-52
4- U11X-12X-59CR @ 64.0m T.L (2) EWP-52

Face Width = 1219 mm (4.0')
Panel Height = 1015 mm (3.33')

TOWER CIV1

WIND : $q = 722$ Pa. (15.1 psf) ICE : Class IV 50mm (2.0")

Fig. 4.30 : Profile Of Tower CIV1

Fig. 4.31 : Profile Of Tower CIV2

Fig. 4.32 : Profile Of Tower CIV3

ANTENNAS & LINES:
1- N.A.

Face Width = 914 mm (3.0')
Panel Height = 762 mm (2.5')

TOWER CIV4

WIND : $q = 500 \text{ Pa. (10.5 psf)}$ ICE : Class IV 50mm (2.0")

Fig. 4.33 : Profile Of Tower CIV4

Fig. 4.34 : Profile Of Tower DII1

Fig. 4.35 : Profile Of Tower DIII1

ANTENNAS & LINES:

1- K5332189 2BAYS/2PANELS @ TOP43m T.L. 3"φ

Face Width = 1524 mm (5.0')

Panel Height = 1524 mm (5.0')

TOWER EII1

WIND : q = 450 Pa. (9.4 psf) ICE : Class II 25mm (1.0")

Fig. 4.36 : Profile Of Tower EII1

Fig. 4.37 : Profile Of Tower FII1

Fig. 4.38 : Profile Of Tower FIII1

Fig. 4.39 : Wind Directions Considered

Fig. 4.40 : Relation Between Wind during Ice Storms and Maximum Ice

Fig. 4.41 : Relation Between Wind during Ice Storms and Maximum Wind

Fig. 4.42a : Ratio of (Wind Speed at Maximum Ice / Maximum Wind Speed) vs Percentage of Stations

Fig. 4.42b : Ratio of (Wind Speed at Maximum Ice / Maximum Wind Speed) vs Cumulative Percentage of Stations

Ice Class	Ice Thickness
I	10 mm
II	20 mm
III	30 mm
IV	40 mm
V	open

Fig. 4.43 : Proposed Ice Map of Canada

Fig. 4.44 : Ice Map of Canada as Specified by OHBDC-1983

Fig. 5.1 : Effect of Wind Load Factor on the Design Forces in Members
($\alpha_w = 1.40$)

Fig. 5.2 : Effect of Wind Load Factor on the Design Forces in Members
($\alpha_w = 1.50$)

Fig. 5.3 : Effect of Wind Load Factor on the Design Forces in Members
 $(\alpha_w = 1.60)$

Fig. 5.4 : Ratio of (Force due to Factored Load / Force due to Specified Load)
vs Percentage of Members Exceeding $(\alpha_w = 1.50)$

**Fig. 5.5 : Effect of Ice Thickness Factor on the Design Forces in Members
(Ice Class I $\alpha_1 = 1.80$)**

**Fig. 5.6 : Ratio of (Force due to Factored Load / Force due to Specified Load)
vs Percentage of Members Exceeding (Ice Class I $\alpha_1 = 1.80$)**

Fig. 5.7 : Effect of Ice Thickness Factor on the Design Forces in Members
(Ice Class II $\alpha_1 = 1.40$)

Fig. 5.8 : Ratio of (Force due to Factored Load / Force due to Specified Load)
vs Percentage of Members Exceeding (Ice Class II $\alpha_1 = 1.40$)

Fig. 5.9 : Effect of Ice Thickness Factor on the Design Forces in Members
(Ice Class III $\alpha_1 = 1.30$)

Fig. 5.10 : Ratio of (Force due to Factored Load / Force due to Specified Load)
vs Percentage of Members Exceeding (Ice Class III $\alpha_1 = 1.30$)

**Fig. 5.11 : Effect of Ice Thickness Factor on the Design Forces in Members
(Ice Class IV $\alpha_1 = 1.20$)**

**Fig. 5.12 : Effect of Ice Thickness Factor on the Design Forces in Members
(Ice Class IV $\alpha_1 = 1.20$)**

Fig. 5.13 : Maximum Leg Forces and Face Shear for Tower A11

Fig. 5.14 : Maximum Leg Forces and Face Shear for Tower A111

Fig. 5.15 : Maximum Leg Forces and Face Shear for Tower A112

Fig. 5.16 : Maximum Leg Forces and Face Shear for Tower A113

Fig. 5.17 : Maximum Leg Forces and Face Shear for Tower AIII1

Fig. 5.18 : Maximum Leg Forces and Face Shear for Tower AIII2

Fig. 5.19 : Maximum Leg Forces and Face Shear for Tower AIV1

Fig. 5.20 : Maximum Leg Forces and Face Shear for Tower AIV2

Fig. 5.21 : Maximum Leg Forces and Face Shear for Tower B11

Fig. 5.22 : Maximum Leg Forces and Face Shear for Tower B12

Fig. 5.23 : Maximum Leg Forces and Face Shear for Tower B111

Fig. 5.24 : Maximum Leg Forces and Face Shear for Tower B112

Fig. 5.25 : Maximum Leg Forces and Face Shear for Tower BII3

Fig. 5.26 : Maximum Leg Forces and Face Shear for Tower B114

Fig. 5.27 : Maximum Leg Forces and Face Shear for Tower BIII1

Fig. 5.28 : Maximum Leg Forces and Face Shear for Tower B1112

Fig. 5.29 : Maximum Leg Forces and Face Shear for Tower BIV1

Fig. 5.30 : Maximum Leg Forces and Face Shear for Tower BIV2

Fig. 5.31 : Maximum Leg Forces and Face Shear for Tower Biv3

Fig. 5.32 : Maximum Leg Forces and Face Shear for Tower BIV4

Fig. 5.33 : Maximum Leg Forces and Face Shear for Tower C11

Fig. 5.34 : Maximum Leg Forces and Face Shear for Tower C12

Fig. 5.35 : Maximum Leg Forces and Face Shear for Tower CI3

Fig. 5.36 : Maximum Leg Forces and Face Shear for Tower CII1

Fig. 5.37 : Maximum Leg Forces and Face Shear for Tower CII2

Fig. 5.38 : Maximum Leg Forces and Face Shear for Tower CH3

Fig. 5.39 : Maximum Leg Forces and Face Shear for Tower CIII1

Fig. 5.40 : Maximum Leg Forces and Face Shear for Tower CIII2

Fig. 5.41 : Maximum Leg Forces and Face Shear for Tower CIV1

Fig. 5.42 : Maximum Leg Forces and Face Shear for Tower CIV2

Fig. 5.43 : Maximum Leg Forces and Face Shear for Tower CIV3

Fig. 5.44 : Maximum Leg Forces and Face Shear for Tower CIV4

Fig. 5.45 : Maximum Leg Forces and Face Shear for Tower DIII

Fig. 5.46 : Maximum Leg Forces and Face Shear for Tower DIII1

Fig. 5.47 : Maximum Leg Forces and Face Shear for Tower EIII

Fig. 5.48 : Maximum Leg Forces and Face Shear for Tower FH1

Fig. 5.49 : Maximum Leg Forces and Face Shear for Tower FIII1

**Fig. 5.50 : Maximum Compressive and Tensile Forces
for Legs of Tower M11**

**Fig. 5.51 : Maximum Compressive and Tensile Forces
for Diagonals of Tower M11**

**Fig. 5.52 : Maximum Compressive and Tensile Forces
for Horizontals of Tower M11**

**Fig. 5.53 : Maximum Compressive and Tensile Forces
for Legs of Tower M111**

**Fig. 5.54 : Maximum Compressive and Tensile Forces
for Diagonals of Tower MII1**

**Fig. 5.55 : Maximum Compressive and Tensile Forces
for Horizontals of Tower MIII**

**Fig. 5.56 : Maximum Compressive and Tensile Forces
for Legs of Tower MIII1**

**Fig. 5.57 : Maximum Compressive and Tensile Forces
for Diagonals of Tower MIII1**

**Fig. 5.58 : Maximum Compressive and Tensile Forces
for Horizontals of Tower MIII1**

**Fig. 5.59 : Maximum Compressive and Tensile Forces
for Legs of Tower MIV1**

**Fig. 5.60 : Maximum Compressive and Tensile Forces
for Diagonals of Tower MIV1**

**Fig. 5.61 : Maximum Compressive and Tensile Forces
for Horizontals of Tower MIV1**

**Fig. 5.62 : Deflections for a Tower
Ice Class I**

**Fig. 5.63 : Deflections For a Tower
Ice Class II**

**Fig. 5.64 : Deflections for a Tower
Ice Class III**

**Fig. 5.65 : Deflections For a Tower
Ice Class IV**

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