# Evolution of design standards and recorded failures of guyed towers in Canada

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The information contained herein constitutes the foundations for a companion paper in which a set of guidelines for the upgrading of existing guyed towers is developed, following a rational approach. Large increases in strength requirements for guyed towers have been introduced by successive revisions of the CAN-CSA-S37 Standard "Antennas, Towers and Antenna-Supporting Structures" (S37). Up to now, there has been a perception among tower owners that tower failures were few and, in consequence, that the added strength requirements are not needed, and further, that existing towers should not be forced to comply with the latest edition of the S37 standard. This paper demonstrates that the failure rate for guyed towers designed to earlier versions of S37 is generally unacceptably high. General comments on standard developments and various design philosophies are presented. The evolution of the Canadian standard for the design of guyed towers is also examined, as deficiencies in earlier versions of S37 are partly accountable for the high observed failure rate. Other international standards for the design and analysis of guyed towers are also reviewed for their approach towards the upgrading of existing towers. Since guyed telecommunication towers are often reviewed for addition of new antennas, upgrading to the strength levels of the most recent edition of the standard — as required in many cases — can be very expensive. Yet, in many cases, much of this expense is unjustified for a variety of reasons. This paper proposes the need for the development of upgrading guidelines and further development work on S37.

*Key words*: guyed towers, failure, failure rate, working stress design, limit state design, environmental loading, existing structures, strength upgrading.

L'information présentée ici servira pour un article connexe dans lequel une série de directives pour hausser la résistance des tours haubannées existantes sera élaborée selon une approche rationnelle. Les exigences de résistance des tours haubannées ont été largement haussées suite aux révisions successives de la norme CAN/CSA-S37 « Antennas, Towers and Antenna-Supporting Structures » (S37). Les propriétaires de tours demeurent, à ce jour, sous l'impression que les pertes totales de tours sont des évènements rares, et qu'en fait toute hausse de résistance pour les tours déjà en place est superflue, voire que ces tours ne devraient pas être tenues de rencontrer les exigences de la plus récente édition de S37. Cet article démontre que le taux de ruine pour les tours haubannées calculées selon des versions antérieures du S37 est généralement inacceptablement élevé. Des commentaires sur le développement des normes en général, et sur les diverses philosophies de design sont présentés. L'évolution de la norme canadienne pour le calcul des tours haubannées est aussi revue puisque ce sont des lacunes au sein des premières éditions de cette norme qui sont en partie responsable du haut taux de ruine observé. La manière dont d'autres normes internationales traitent le problème des tours haubannées existantes est également examinée. Puisque des antennes supplémentaires sont souvent ajoutées aux tours haubannées existantes, la hausse de la résistance jusqu'au niveau requis par la plus récente édition de la norme s'avère souvent une opération dispendieuse. Pourtant, dans plusieurs cas, et pour diverses raisons, une telle dépense n'est pas justifiée. Cet article propose qu'il existe un besoin de développer des directives concernant la hausse de résistance des structures existantes, de même que certains autres aspects de la norme S37.

*Mots clés*: tour haubannée, ruine, taux de ruine, design aux contraintes admissibles, design aux états limites, charges environnementales, structure existante, hausse du niveau de résistance.

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## 1. Introduction

A large number of guyed towers have been constructed throughout Canada, over the past 50 years or so, in parallel with the strong emergence of the telecommunications industry. These especially light and slender structures are particularly sensitive to the environmental loads to which they are subjected. In consequence, the considerable improvements in the knowledge of member behaviour, overall structural response, and environmental loads, reflected in subsequent editions of the national standards for towers and antenna-supporting structures by the Canadian Standards Association, have generally resulted in significant increases in the strength requirements of these structures.

The continuous updating of the design wind and ice loadings, considering the latest environmental data as well as major changes in the way these loads were applied on the towers, although properly correcting deficiencies in knowledge and ensuring more reliable engineered structures, rapidly enlarged the gap between new and existing towers' strength requirements, more so than for other engineering structures during the same period. As antenna loads on many structures are changed, increased in size and (or) in number (possibly several times during the service life of the structure), leading to the review and upgrading of existing towers, the financial significance of this gap becomes obvious. Existing towers thus continued for a long time to be analyzed to their original design editions of the Canadian standard, based upon the perception among tower owners that tower failures were few and that, in consequence, the added strength requirements were unnecessary. The latest edition of the Canadian tower standard — CAN-CSA-S37-M86, "Antennas, Towers and Antenna-Supporting Structures" (S37) (CSA 1986) — practically suppressed this "double standard," but did not propose a rational procedure with regard to existing towers.

In this paper, the general history of standard developments and design philosophies, in light of their impact on the analysis and design of guyed towers, will be summarized. Then the evolution of the Canadian Standard S37, with particular

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emphasis on its treatment of existing towers, will be examined. The problems inherent to the treatment of existing towers will become evident from this overview.

To determine the actual performance of the existing inventory of towers, the authors have undertaken a comprehensive survey of tower failures throughout the industry. Results from this survey, as well as the estimated actual corresponding failure rate, will be presented. Also, other international standards will be reviewed, to determine how the special problems presented by existing towers have been treated elsewhere. Finally, further work required on the S37 code will be identified.

In a subsequent paper (Bruneau *et al.* 1989), a set of guidelines for the upgrading of existing guyed towers, as well as the methodology adopted in their development, will be presented.

## 2. General information on standard developments

This section contains information on the development of standards and the concepts of working stress design and limit states design, with which the reader may already be familiar. Nevertheless, considering the nature of the current edition of CAN-CSA-S37-M86 and the impact of various design philosophies on the design of guyed towers, the inclusion of this information as part of this paper was deemed worthwhile.

Most engineering design standards regulating analysis, design, and construction of structures appeared only after large numbers of typical structures had been designed and constructed. In fact, many successful structures and engineering landmarks constructed in the past century were designed when formal design standards did not exist; rather, their design was guided largely by a combination of the technical knowledge, training, and experience of the designer. However, structural failures were numerous by today's standards. As formal structural design standards started to appear, they ensured protection for the owners against deficient designs and provided an increased level of safety for the population.

For newer types of structure, subjected to harsh environmental conditions, such as telecommunication towers, the demand for the establishment of design standards occurred after a short period of recorded performance history. However, conservative solutions addressing the risks of underestimating the severity of the ultimate loading conditions, or emphasizing consideration for the unknowns in the true structural behavior, were not always adopted.

The first editions of most standards regulating the field of civil engineering consisted of collections of mainly empirical rules believed adequate to provide the necessary structural strength and ability to perform successfully in service. The levels of protection against failure were often inconsistent among the various design rules within a given standard; for most structures, however, the recorded history of satisfactory behaviour justified their continued use. Where service problems or failures occurred, the standards were modified to prevent the repetition of such incidents.

Standard modifications developed in response to better information on the nature and intensity of the applied loads, the response of the structure to these loads, the behavior of the materials and structural members in the particular service environment, or combinations of these factors. While the theoretical models for the behaviour of structures are now fairly sophisticated and comprehensive, much of the engineer's knowledge of the loads, materials, and structure behavior is still empirical. As a result, most standard developments were driven by the particular characteristics of different classes of structures, e.g., towers, dams, bridges, buildings, etc. The outcome was a series of standards specializing in particular structures with many inconsistencies between their respective philosophies and safety requirements.

Recent developments in the field of structural engineering have also attempted to produce uniformity in the level of protection provided by the various design rules within all standards. This is achieved through the probabilistic calibration of the specified loads and resistances to achieve uniform safety levels or classes throughout. The practical implementation of this process leads to the development of "limit states design" (LSD) standards.

Basically, structural standards fall into two main types: those based on working stress design (WSD) or factors of safety, and those based on limit states design (LSD) or load factors. Both standard types provide for a margin between the strength of the structure and the load to which it may be subjected, to assure a reasonable level of safety against failure. The difference between the two types arises from the manner in which the margin is applied against the loads and resistance.

In WSD, only the member strength is factored downwards to establish safe, or working, stress levels. All load effects, from the weight of the structure to wind, ice, and other loads, are treated with equal importance. This relationship between the loads and the resistance can be expressed as

$$[1] \qquad \sum_{i=1}^{n} L_i \leq R/\mathrm{FS}$$

that is, sum of all service loads ≤ member strength/factor of safety; where, typically, FS is 1.5 or larger.

In LSD, the uncertainties associated with each of the applied loads and the resistance are directly attached to each term by the use of load factors, $\alpha_i$, and performance factors, $\phi$. This relationship between the loads and the resistance can be expressed as

$$[2] \qquad \sum_{i=1}^{n} \alpha_i L_i \leq \phi R$$

that is, sum of all factored loads ≤ performance factor × member strength.

The performance factor, usually ranging from 0.85 to 0.90, accounts for the effects that may result in a given member's actual strength being less than its theoretical strength, as well as the significance of the member failure. The load factors account for the effects that may result in the actual loads being higher than the specified or service loads, and are dependent on the type of load — the greater the variability in the load, the larger the load factor; e.g., 1.25 is typically used for own weight of the structure, while 1.5 is typically used for wind or ice load.

LSD standards recognize that member strengths and structural loads have statistical distributions that vary in scatter with the particular strength property and applied load characteristics. Accordingly, based on these statistical distributions, the performance and load factors are calibrated to achieve adequately low notional probabilities of failure, where the actual load exceeds the actual strength. The target probabilities of failure, usually in the range of $10^{-3}$ – $10^{-5}$, are selected to reflect significance of the class of structures, economic impact of a loss, and level of risk to human life.

Another difference between WSD and LSD arises from the level of load against which the margin between load and resistance is established. In LSD design, the various loads are factored upwards *before* being applied to the structure. In WSD,

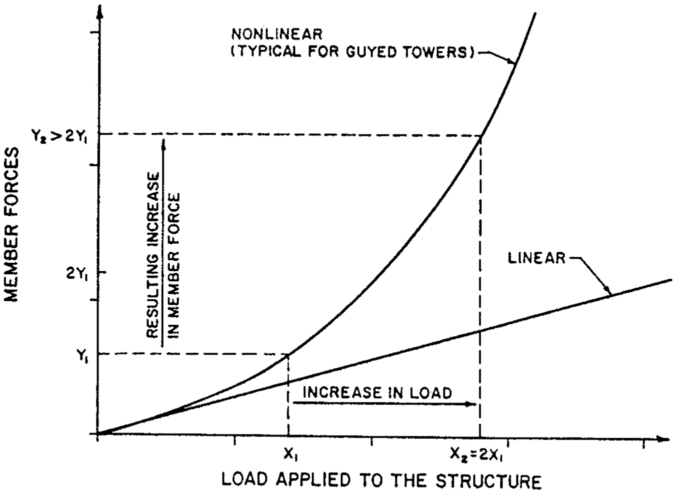


FIG. 1. Comparison of the effect of linear and nonlinear relationships between load applied to the structure and resulting member forces.

it is the member strength that is factored downwards, while the overall loads are not factored. Therefore, the factor of safety is applied only *after* the effect of the unfactored loads has been distributed throughout the structure.

In the case of guyed towers, which are largely nonlinear in behaviour, this philosophically different approach is of a much greater consequence than it is for linear types of structure. As illustrated in Fig. 1, for a nonlinear relationship between the load applied to the structure and the resulting member forces, increases in the overall loads applied to the structure will produce much larger increases in the resulting member forces than if the relationship were linear.

Clearly, a design method in which load factors are applied to the member forces obtained *after* analyses using unfactored loads are performed cannot be referred to as a true LSD method, as this approach is in fact closer to a WSD method in philosophy, no matter how much is borrowed from the terminology and format of the LSD design method.

## 3. Evolution of the Canadian Standard S37

The first Canadian standard governing the design of antenna-supporting structures was CSA Standard C22.4-111, "Specifications for Antenna Towers and Antenna-Supporting Structures," issued in 1954. That was later replaced with the standard prepared by CSA Committee S37, which started with CSA Standard Specification S37-1965, "Specifications for Antenna Towers and Antenna-Supporting Structures" (CSA 1965). That first S37 standard improved on the design assumptions adopted in 1954; as well, it updated the wind and ice load requirements.

While S37-1965 was a significant improvement over C22.4-111, it assumed that ice would neither accumulate nor remain on the guy cables at the peak design wind conditions as a result of the combined effect of the wind and the sway of the guys. As well, S37-1965's specified ice accretion values were based on limited environmental records, which later proved to be insufficient in some parts of the country.

Committee S37 continued working on updating the tower standard to improve its provisions in keeping with improvements in the knowledge of member behavior, overall structural response, and environmental loads. Subsequently, S37-1976 (CSA 1976) was issued incorporating a new load condition specifying ice load on the guys combined with one half of the peak wind load and the member strength requirements of the CSA standard for the design of steel structures for buildings, S16. These two provisions significantly increased the basic strength requirements for towers. S37-1976, however, continued to allow existing structures to be reviewed to the requirements of earlier tower standards as directed by the authority having jurisdiction.

Since no regulatory body in Canada has jurisdiction over tower design standards and structural adequacy, most owners decided to direct that structural adequacy of existing towers be maintained to their respective original design standards. New towers, however, were consistently designed to the current issue of S37. This had the effect of substantially improving the reliability of new towers, but of generally not increasing the reliability of existing structures.

Intermediate revisions to S37-1976, and conversion to Système International (SI) units published as CSA-S37-M81 (CSA 1981), introduced another significance change with respect to the determination of the environmental load. For existing towers, if analyzed according to S37-M81, site-specific determination of design wind loading could now be used, instead of the maps provided in the standard based on information such as that provided by the Supplement to the National Building Code of Canada. However, the determination of site-specific winds was limited to special structures which justified the added costs of an ad hoc analysis of available meteorological records, site topography, etc. No routine source for this service had yet been developed.

The major modification to the subsequent edition of the standard, CAN-CSA-S37-M86, consisted in the introduction of a new method to calculate the wind pressure on a tower. The shape factor coefficient, used in the estimation of the design wind pressure in S37-1976, was replaced by a drag factor, which is a function of the tower shape, member shape, solidity ratio (a ratio expressed by the percentage of a tower-face filled by material, typically equivalent to the "shadow" area produced by the tower face), and wind direction. This mainly resulted in an increased design wind pressure for towers with very low and very high solidity ratios, and a significantly increased design wind load for towers with high solidity ratios approaching unity. The added wind loads and the approach to calculating them were based on wind tunnel measurements by various researchers. The effect of these changes was to substantially increase the design loads for areas with heavy ice loading and congested towers.

Also, this latest edition, in an attempt to be compatible with the current limit state design version of the steel structures standard, CSA-S16.1-1974 (CSA 1974), departed from the allowable stress format used in earlier versions, and introduced a magnification factor to be applied to all calculated element forces before proceeding with the element design. This approach is not to be mistaken for a true limit states design procedure, where the load factor would be applied to the overall structural loads before the analysis, not to the member loads resulting from the analysis.

Earlier attempts to develop S37-1976 as a true LSD standard were not successful. Guyed towers being largely nonlinear in their behavior, application of the recommended Canadian limit states design load factors to the loads prior to the analysis led to tower designs which, in the view of the committee, were unacceptably heavy in relation to typical designs existing at the time. Therefore, the 1.5 load factor currently used in S37-M86 was adopted as an interim measure, awaiting further development in the research for the determination of load factor adequate for use in nonlinear analyses, and calibrated with S37-1976 to ensure minimal disturbances to the implicit reliability and safety factors in effect at the time.

By the early 1980s, the Atmospheric Environment Service (AES) of the Canadian Department of the Environment developed a routine, cost-efficient and fast service for the estimation of site-specific wind data. AES provides wind data which include the effect of terrain roughness and site exposure. This is now a routine procedure used for most tower analyses in Canada. Unfortunately, AES at this time is not able to provide site-specific information for ice accretion.

Up to the introduction of S37-M86, the S37 standard continued to allow existing structures to be reviewed and upgraded to their original design edition, even in cases where the existing structure was modified or had loads added to it. Currently, under S37-M86, an existing structure can still be analyzed to the requirements of its original design edition. However, if any strengthening is required as a result of the modified conditions, the strengthening should meet the latest editions of the standard.

This feature had a very significant effect. Existing towers which would barely meet the original design code requirements could be left without reinforcement, thus remaining at the lower reliability level implicit in that code. However, existing towers requiring even minor amounts of reinforcement would now have to be reinforced to the full-strength requirements of M86, thus achieving a substantial increase in reliability implicit in the new standard, but, in many cases, at a very significant cost premium. This was independent of the significance of the structure or the consequences of its failure.

## 4. Canadian tower failures

Up to now, there has been a perception among tower owners that tower failures were few and, in consequence, that the added strength requirements are not needed, and further, that existing towers should not be forced to comply with the latest edition of the S37 standard. It was decided that this perception should be tested against an analysis of actual failure rates.

For the purpose of this analysis, a survey was conducted to collect as much information as possible with respect to documented tower failures. There is currently no Canadian registry of tower performance. Therefore, every major Canadian telecommunications tower owner or designer was contacted to gather information about recorded tower failures. Sufficiently complete documentation with respect to tower location, height, original design standard edition, number of guy levels, failure date, and cause and description of failure had to be available for a reported failure to be considered in this survey. As such, a number of failures that were insufficiently documented had to be disregarded.

Results of the survey are summarized in Table 1. All failures reported are cases of total collapse. Failure is thus defined as total loss of a tower.

In addition to the data in Table 1, eight other guyed tower failures were reported over the same period, due to a variety of causes. These included small aircraft collisions, vehicle collisions, sabotage, fatigue of defective turnbuckle, corrosion of buried steel shaft, use of inappropriate material during erection, and many more. These failures are attributable to causes that cannot be designed for by the use of increased load factors. In the following discussions of tower reliability, these causes are assumed invariable, and are likely to continue to cause a relatively high failure rate in guyed masts as compared to sturdier structures.

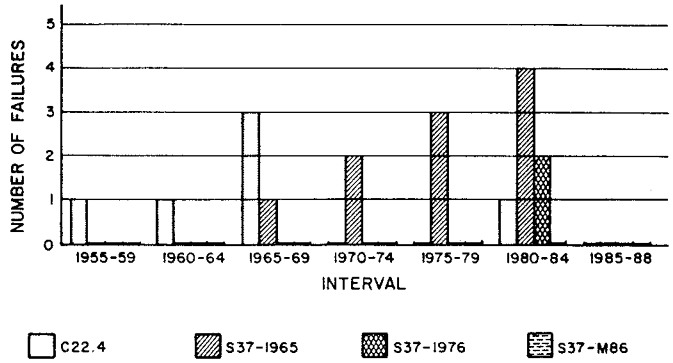


FIG. 2. Reported failures of guyed towers taller than 75 m in Canada between 1955 and 1988.

Of the 19 reported tower failures, only the 18 guyed towers taller than 75 m are considered further. The absence of more reported failures for towers of less than 75 m may be attributed to three hypotheses: (a) these towers are less sensitive to overloads; (b) the failure of these towers is of lesser interest and, therefore, not always documented; or (c) the tower industry may tend to remember the failures of taller structures only.

Owing to the lack of data, it is not possible to analyze the relative significance of these hypotheses, and the study can focus only on guyed towers above 75 m in height. The histogram for the 18 reported tower failures as a function of time and design standards used is presented in Fig. 2. From this information, rates of failure of guyed towers were determined as indicated below. It should be realized that the accuracy of the following analyses is governed by the limitations inherent to the information available. These limitations will be indicated hereafter, where appropriate.

It is noteworthy that four towers collapsed in March, 1983, at Baldy Mountain, Brandon, and Carlyle, as a result of excessive and rare ice loading under light wind conditions.

## 5. Rate of tower failure

The rate of tower failure is defined by the ratio of the average annual number of tower failures divided by the total number of existing towers.

For guyed towers taller than 75 m, the average annual number of tower failures can be approximated as uniform over a given period of service (which varies, depending on the design standard used). From Fig. 2, an average of 0.33 towers designed according to C22.4 were seen failing every year from 1954 to 1969 inclusive. One failed after 1979. The annual number of failures for towers designed to C22.4, over the period from 1954 to 1988, averages approximately 0.18 tower per year. Towers designed according to S37-1965 have an annual average number of failures of 0.43 tower per year from 1965 to 1988. Towers designed according to S37-1976 have an average annual number of failures of 0.17 from 1976 to 1988. Therefore, a currently expected average annual number of failures would be approximately 0.77 per year. Although not meaningful in itself until expressed as a rate of failure, this is a large number of annual failures. Further, the reported tower failures, although based on fairly comprehensive surveys, do not necessarily cover all the Canadian tower failures that occurred. The actual failure rates could, in fact, be higher than reported here.

To determine the rate of failure, some knowledge must exist about the number of towers in existence. Because a complete registry of tower structures (as opposed to transmitting anten-

TABLE 1. Summary of reported tower failures*

| Location | Date of collapse | Height (m) | Number of guy levels | Design code | Reported by§ |
|---|---|---|---|---|---|
| St. John's† (Nfld.) | Mar. 1958 | 094 | 7 | C22.4 | MH |
| Antigonish (N.S.) | 1962 | 091 | 4 | C22.4 | Abroyd |
| Rossfield (N.S.) | 1965 | 125 | 4 | C22.4 | MH |
| Winnipeg (Man.) | Jul. 1966 | 110 | 5 | S37-1965 | MH |
| Trois-Rivières† (Que.) | Dec. 1967 | 330 | 10 | C22.4 | MH |
| Quebec City† (Que.) | Mar. 1968 | 076 | 2 | C22.4 | MH |
| Essex‡ (Ont.) | Apr. 1970 | 046 | Self-support | S37-1965 | MH |
| Mt. Megantic (Que.) | 1972 | 094 | 3 | S37-1965 | L&R |
| Swan Hills (Alta.) | Nov. 1972 | 091 | 8 | S37-1965 | MH |
| Grenfield (Sask.) | 1976 | 125 | 6 | S37-1965 | MH |
| Grand Banks† (Nfld.) | 1979 | 107 | Unknown | S37-1965 | L&R |
| Upsolquitch (N.B.) | Jan. 1979 | 213 | 8 | S37-1965 | MH |
| Saglek (Lab.) | Oct. 1980 | 092 | 5 | S37-1976 | MH |
| Baldy Mtn. A† (Man.) | Mar. 1983 | 146 | Unknown | S37-1965 | L&R |
| Baldy Mtn. B† (Man.) | Mar. 1983 | 155 | 7 | S37-1965 | MH |
| Brandon† (Man.) | 1983 | 411 | 9 | S37-1965 | L&R |
| Carlyle† (Sask.) | Mar. 1983 | 207 | Unknown | C22.4 | L&R |
| Mt. Scio‡ (Nfld.) | 1983 | 209 | Unknown | S37-1976 | L&R |
| Warmley (Sask.) | 1983 | 107 | 6 | S37-1965 | L&R |

*Only those failures reported to be wind overload, ice overload, or wind and ice overload.
†Known to be a broadcast tower.
‡Assumed to be a broadcast tower.
§Sources: MH, Morrison Hershfield Limited; Abroyd, Abroyd Communications Limited; L&R, LeBlanc and Royle.

nas) has never been assembled in Canada, it is not possible to determine the number of towers that existed in service at different times in the past. Therefore, although it is expected that the increase in reported failures with time (seen in Fig. 2) may be a consequence of the corresponding increase in the number of existing towers, there is no way to verify this possibility. Rate of failure will thus be calculated assuming that the aforementioned average annual number of failures of 0.77 tower per year can be related only to the number of towers currently in existence.

The number of towers in service can be estimated from various sources, all partly incomplete. A database owned by Morrison Hershfield Limited, Consulting Engineers (MH), containing over 3000 towers, was scanned (Morrison Hershfield Limited 1988), and results are summarized in Table 2. Towers of all types (broadcasting, microwaves, etc.) are included in MH's database; therefore, the results from MH's database have been interpreted as cumulative for all types of towers. In addition, the Canadian Department of Communication[1] (DOC) maintains a registry of transmitting broadcast antennas and their elevations. This database was used to infer the number of broadcast towers and their heights as summarized in Table 3.

Only towers higher than 75 m were considered in the analysis. Other than the 937 broadcasting towers in the DOC database, the total number of existing towers, of all types, in the height range of interest is not known. Considering that 60% of all the failures reported in Table 1 occurred for broadcasting towers, it could be assumed that this proportion is representative of the actual population of towers in service. Therefore, the estimated total number of all towers in service is estimated to be 937/0.6. Further, since this study is concerned only with guyed

[1]Canadian Department of Communication. 1988. Broadcasting database. Special information request.

TABLE 2. Number of existing towers over 75 m high, as obtained from MH towers database (total of 3091 entries)

| Height range (m) | Self-support | N/A* | Guyed towers | All |
|---|---|---|---|---|
| 75–120 | 52 | 187 | 425 | 664 |
| 121–199 | 5 | 59 | 177 | 241 |
| 200–274 | 0 | 14 | 22 | 36 |
| ≥275 | 1 | 8 | 18 | 27 |
| Total | 58 | 268 | 642 | 968 |

*Information on type not available.

TABLE 3. Number of broadcasting towers over 75 m high, as obtained from the broadcasting database of the Canadian Department of Communications (total of 4477 entries)

| Height range (m) | All |
|---|---|
| 75–120 | 599 |
| 121–199 | 256 |
| 200–274 | 44 |
| ≥275 | 38 |
| Total | 937 |

towers, and that these towers comprise approximately 90% of all towers taller than 75 m (Table 2), the actual number of towers to be used in the rate-of-failure calculation should be adjusted accordingly [i.e., (937 × 0.9)/0.6]. The approximate rate of failure of broadcasting guyed towers is thus

$$\frac{0.77 \text{ tower/year} \times 0.60}{937 \text{ broadcasting towers} \times 0.90} = 0.055\ \%/\text{year}$$

## 6. Discussion of results

### 6.1. Observations

A survey of the major participants in the telecommunications industry revealed that 18 guyed towers taller than 75 m collapsed over the period between 1958 and 1988. Considering the relatively small numbers of existing towers, this translates into a failure rate which by far exceeds that of all other types of structure in Canada. Many of these collapses were due to excessive environmental loads, which exceeded those values believed to be maxima when the towers were designed. This has been corrected, to some extent, by an increase in knowledge about the statistical variability of the extreme values of parameters, specified load combinations, analysis techniques, and member strength models, all of which have resulted in increased strength requirements for towers. Unfortunately, the prediction of ice accumulation has not evolved at the same rate as wind predictions and, as of today, still carries a large degree of uncertainty. This problem is compounded by the fact that the first edition of the S37 standard did not consider the inclusion of ice accumulations on the guys for design, which was subsequently found to be an omission of serious consequence. These aforementioned conditions resulted in many towers being designed with substandard reliability, which is largely reflected in the recorded numerous failures of such structures. Should the industry elect not to upgrade the current reliability of these towers, they are expected to continue failing at the rate reported above.

Further, the lack of complete documentation of tower failures is a serious handicap to the development of economical design standards. A collaborative effort, involving the Canadian telecommunications industry, should be implemented, to record and report on any partial or complete tower failures. This effort would involve

- investigation and documentation of the conditions leading to collapse;
- monitoring of the rate of failure;
- dissemination of information on tower failure; and
- periodic review of the adequacy of the provisions of the Canadian S37 standard with respect to existing towers.

Finally, most existing towers analyzed to the latest edition of the standard will require reinforcement, and the generally unacceptably high estimated failure rate observed seems to support S37-M86's approach with respect to the upgrading of existing towers. Nevertheless, because of economic constraints, it is neither practical nor rational to uniformly upgrade the inventory of existing guyed towers to meet S37-M86 requirements. Clearly, there is a need to develop a rational procedure for the upgrading of those structures, which would take into account the significance of the structure and the consequences of its failure to the owner. At this point, a review of the way in which other international standards deal with existing towers is worthwhile. Two standards were reviewed; the findings are summarized following.

### 6.2. Review of other international standards

#### 6.2.1. Electronic Industries Association EIA-222-D

The EIA-222-D "Structural Standards for Steel Antenna Towers and Antenna Supporting Structures" (EIA 1984) is not a limit states design standard. It is very similar to the CSA-S37 standard in its content, general approach, and the way it treats existing structures. In that respect, it states

> The Committee does not intend that existing structures be analysed for each revision of the standard; however, structural analysis of existing structures should be performed by qualified professional Engineers using the last edition of this standard when:
>
> - There is a change in antennas, transmission lines, and/or appurtenances (quantity, size, location, or type),
> - There is a change in operational requirements (twist and sway),
> - There is a need to increase wind or ice loading.

Similar to CSA-S37-M86, no intermediate upgrading levels are possible.

#### 6.2.2. IASS recommendations for guyed masts

The International Association for Shell and Spatial Structures (IASS 1981) code is a true limit states design standard with particular recommendations to "distinguish between three reliability classes of guyed mast structures, according to the risk of loss of life and to the risk for other losses, economic or social, which would ensue if failure occurred." These classes are described as follows:

- *Class I*: All structures constructed in urban built-up areas, or where loss of life could occur if they collapsed, as well as all structures where loss of the service provided causes unacceptable danger to life, inestimable economic loss, or unacceptable loss of service.
- *Class II*: All structures where the likelihood of loss of life

if they collapsed would be negligible and adequate warning arrangements are incorporated to ensure that the general public are not unduly endangered, as well as structures where the economic consequences justify a reduced reliability, and structures where loss of the service provided is not critical and alternative means of communication can be provided.

- *Class III*: Structures where all the consequences of failure are tolerable.

For the above classifications, the risk to life and loss of service can be easily evaluated. However, the economic consequences of a failure need to be related to additional guidelines. The IAS code provides a rule-of-thumb equation to estimate the "economic reliability" requirements as follows:

If the total economic consequences of failure in the design service period of $N$ years are a multiple, $r$, of the cost of replacement of the mast, the reliability class to be chosen should provide annual reliability at least as great as

$$[3] \quad 1 - 1/(10rN)$$

The notional (very approximate) annual reliability required against collapse is then set at $(1 - 10^{-4})$ for Class I, $(1 - 10^{-3})$ for Class II, and $(1 - 10^{-2})$ for Class III. These notional annual probabilities cannot be taken as absolute reliability and should be used only in the intended context, that is, to help determine a reliability class for economic considerations as related by the proposed rule-of-thumb.

Once the desired design reliability class has been determined by the above procedure, the load factors to use in design can now be determined.

Some of the IASS load combinations would then be equivalent to

For Class I:

$$[4] \quad 1.3D + 1.56W < \phi R$$

$$[5] \quad 1.0D + 0.77(1.56W + 1.3T) < \phi R$$

$$[6] \quad 1.0D + 0.77(1.56W + 1.56I + 1.3T) < \phi R$$

For Class II:

$$[7] \quad 1.15D + 1.38W < \phi R$$

$$[8] \quad 0.9D + 0.77(1.38W + 1.15T) < \phi R$$

$$[9] \quad 0.9D + 0.77(1.38W + 1.38I + 1.15T) < \phi R$$

For Class III:

$$[10] \quad 1.0D + 1.20W < \phi R$$

$$[11] \quad 0.8D + 0.77(1.20W + 1.0T) < \phi R$$

$$[12] \quad 0.8D + 0.77(1.20W + 1.20I + 1.0T) < \phi R$$

where $D$ is the dead load, guy loads, and 110% of the equipment load; $W$ is the wind load with a probability of 1/50 of being exceeded in any one year; $I$ is the ice load with a probability of 1/50 of being exceeded in any one year; $T$ is the temperature load with a probability of 1/50 of being exceeded in any one year; $R$ is the structural nominal resistance; and $\phi$ is the member performance factor, which varies between 0.75 and 1.11 for members and 0.42 and 0.55 for guys.

Direct comparison with Canadian practice is not possible, as the load factors used in Canadian standards have essentially been derived for the design of buildings and bridges rather than towers, and because, further, S37-M86 itself is not true LSD.

It has been suggested (Davenport[2]) that the reliability afforded by the IASS Class II is intended as the target for tower structures, and that classes I and III respectively provide superior and inferior levels of reliability. This appears reasonable in light of the severity of the Class I load condition, in the context of guyed towers.

The authors tried to determine if probabilistic studies have been conducted to support the IASS selection of load factors. It appears, from contact with members of the Working Group 4 of IASS (Davenport[2]; Mogensen[3]; Stottrup-Anderson[4]), that the load factors were based on experience with, at best, some rough calibration to existing practice.

Given the European experience of IASS, a similar approach could be developed for the Canadian S37 standard, which would take into account the significance of the existing towers and the consequence of their failure. This will be the subject of a subsequent paper by the authors.

## 7. Conclusions and recommendations

Due to deficiencies in knowledge of the earlier editions of the Canadian tower standards, principally with respect to the characteristics of the environmental loads and how they were to be applied, the level of reliability of towers designed to those earlier editions is significantly less than that currently provided by S37-M86. Although no towers designed to S37-M86 are yet reported to have failed, this does not preclude the possibility that other inadequacies of the current standard could be revealed by future developments.

A new "ice-only" load case appears desirable, as some towers have been reported to collapse under excessive and rare ice loading under conditions of light or no wind. This new load case could govern the design in some regions where significant in-cloud icing under low wind may occur.

There is also a need for the establishment of a Canadian registry of tower failures, in order to keep on record thoroughly documented, accurate, and easily accessible information concerning tower failures. The lack of this information is currently a serious handicap to the development of economical design standards.

Further, S37-M86 currently requires existing towers, when upgraded, to meet the requirements of the latest edition of the standard, irrespective of the significance of the tower or the consequences of its failure. A rational set of guidelines must be developed to account for those essential considerations in the upgrading of existing towers.

Finally, a development effort should be undertaken to implement a true limit states design S37 tower standard.

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