Recommended guidelines for upgrading existing towers

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A companion paper reviewed the history of the development of the Canadian standard for towers, and demonstrated that the failure rate for guyed telecommunications towers designed to earlier editions of the CAN-CSA-S37 Standard "Antennas, Towers and Antenna-Supporting Structures" (S37) is generally unacceptably high. These towers are often reviewed for addition of new antennas, and upgrading to the strength levels of the most recent edition of the standard — as required in many cases — can be very expensive, due to the sizeable increases in strength requirements for guyed towers which have been introduced by successive revisions of S37. Clearly, there is a need to match strength requirements to the significance of the structure and the economic consequences of its failure. Uniformly high strength requirements will occasionally result in reinforcement costs that exceed the costs of the risk of failure. In answer to this, a rational approach has been used to develop a set of guidelines for the upgrading of existing towers. This methodology relies on the classification of towers in various reliability classes, having increased probabilities of failure attached, and correspondingly lower load factors for use in the analysis. Results from this research are presented herein, together with the recommended proposed guidelines.

Key words: guyed towers, reliability classes, probability of failure, safety index, upgrading cost, failure rate, environmental loads, guidelines.

L'évolution de la norme canadienne pour le calcul des tours fut revue dans un article connexe, où il fut également démontré que le taux de ruine pour les tours de télécommunication haubannées calculées selon les éditions antérieures de la norme CAN-CSA-S37 « Antennas, Towers and Antenna-Supporting Structures » (S37) est généralement inacceptablement élevé. Des antennes supplémentaires sont souvent ajoutées aux tours haubannées existantes, et la hausse de résistance requise pour satisfaire les exigences de la plus récente édition de la norme s'avère souvent une opération dispendieuse, principalement à cause des fortes augmentations du niveau de résistance de base spécifié par la norme S37 au cours des éditions successives. De toute évidence, les exigences de résistance devraient tenir compte de l'importance de la structure et des conséquences économiques de sa ruine. Des exigences de résistance uniformément élevées résulteront parfois en des coûts de renforcement dépassant le coût du risque de ruine. Un ensemble de directives pour la hausse de la résistance des tours haubannées existantes a été developpé selon une approche rationelle. La méthodologie adoptée consiste à établir diverses classes de sécurité, chacune ayant un risque de ruine plus élevée que la précédente et un facteur de charge proportionnellement réduit à utiliser lors de l'analyse structurale. Les résultats de cette recherche, de même que l'ensemble des directives recommandées, sont présentés dans cet article.

Mots clés : tours haubannées, classes de sécurité, risque de ruine, indice de sécurité, coût de la hausse de résistance, taux de ruine, charges environnementales, directives.

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1. Introduction

The introduction of successively more rational and updated national Canadian standards for towers and antenna-supporting structures by the Canadian Standards Association, as well as the evolution of analysis techniques over the last 30 years or so, have resulted in ever-increasing strength requirements for these structures. Increasing the load capacity, or strength, of a structure increases its reliability or, in other words reduces the probability of its failure. Thus, in general, structures designed to earlier standards would be less reliable, or more likely to fail, than those designed to the latest issue of the standard.

The antenna loads on many existing structures are changed, increased in size, and (or) in number — possibly several times during the service life of the structure. The Canadian standard, S37, and the regulations of the Department of Communications (DOC) call for a review of the adequacy of an existing structure to accommodate contemplated load changes. The standard governing the design of towers in Canada is CSA-S37, "Antennas, Towers and Antenna-Supporting Structures." Previously, existing structures could be reviewed to their original design edition. In effect, this allowed the maintenance of the reliability levels inherent in the original design edition. On the other hand, the current issue, S37-M86 (CSA 1986), stipulates in a note to Clause 3.1 that

> An existing structure should not be modified or have loads added to it without having the physical condition and loading of the structure verified by inspection and the design confirmed by an Engineer to determine that the requirements of this Standard are met. A structure designed to an earlier version of this Standard may be checked to the requirements of that earlier edition for the modified condition. If a structure requires strengthening as a result of a modification, it should be strengthened to meet the latest edition of this standard.

While this provision is rational, in that all new towers should be constructed to the latest standard, it remains the case that if no reinforcement is required under the provision of the previous standard, nothing needs to be done. However, finding that a minimal amount of reinforcement is required under the provision of the earlier standard now triggers a major strength upgrade to M86, or even replacement where strengthening is not feasible. Since many existing structures, particularly those designed to 1965 and earlier versions, would be found inadequate without any additional loads under the provisions of S37-M86, significant expenses are expected to occur simply to upgrade to the new standard when adding equipment onto existing towers.

Apart from its cost impact, this requirement of S37-M86 creates a significant step function in the reliability levels for

NOTE: Written discussion of this paper is welcomed and will be received by the Editor until February 28, 1990 (address inside front cover).

existing structures. While this step function is not as significant for self-supporting towers, guyed towers are very sensitive to it. A more rational approach to the upgrading of existing towers would allow a conscious decision to maintain the same level of reliability that existed prior to any modification or, where warranted, to improve the reliability level in one or two steps to that implicit in the current S37-M86.

This paper presents a set of guidelines for the upgrading of existing towers developed following standard structural reliability analysis procedures. The methodology adopted relies on the classification of towers in various reliability classes, having increased probabilities of failure attached, and correspondingly lower load factors for use in the analysis. Results from this work are presented herein, together with the recommended proposed guidelines.

2. Determination of structural reliability

To be able to assess the effect on reliability of adopting various load factors, a reliability analysis was conducted; results are described in the following sections. The methodology followed in the determination of structural reliability of guyed towers consisted in the following steps:

1. Determination of reliability classes along with approximate targets for notional reliabilities, and description of the conditions for which the corresponding risks are deemed acceptable.
2. Calibration analysis to determine the notional reliability currently provided by the S37-M86 recommended load factors.
3. Study of the variability of notional reliabilities as a function of load factors and load combinations.
4. Determination of the load factors that would provide notional reliabilities close to the target values, based on the results of steps 2 and 3, as well as professional judgment.

This section presents the principal steps of this methodology. The cost impacts from the recommended load factors will be studied in Sect. 5.1.

3. Proposed reliability classes

Crucial to the determination of reduced load factors is the definition of reliability classes. Three classes of reliability are proposed:

- *Class I*: This class would include important installations which are to have a reliability equivalent to that currently provided by S37-M86. It is believed that this class would provide adequate safety for structures constructed in urban areas, or where the structure abuts a residential area. This class would include broadcast sites where the tower failure is likely to result in either loss of life, unacceptable loss of service, or unacceptable economic losses.
- *Class II*: This class would include noncritical existing installation located in rural or remote areas where the likelihood of loss of life if the tower collapsed would be negligible. This class would also include sites where loss of service and (or) economic consequences of a failure are not considered critical. The notional probability of failure for towers in this class would be expected to be one order of magnitude larger than for Class I.
- *Class III*: This class would include nonessential existing installations located in rural or remote areas where all the consequences of failure are tolerable and higher risk of failures can be tolerated during the life of the structure. This would include sites where loss of the tower, equipment attached, and service provided would be of little consequence. The notional probability of failure for towers in this class would be expected to be one order of magnitude larger than for Class II, thus two orders of magnitude larger than for Class I.

It is proposed that the target notional reliabilities over 30 years be set at $(1 - 10^{-4})$, $(1 - 10^{-3})$, and $(1 - 10^{-2})$ for classes I, II, and III respectively.

TABLE 1. Relationship between p_f and β — sample list

β	p_f ($\times 10^{-4}$)	β	p_f ($\times 10^{-4}$)
0	5000	3.1	9.67
1	1600	3.2	6.87
2	227	3.3	4.83
2.1	179	3.4	3.37
2.2	139	3.5	2.33
2.3	107	3.6	1.59
2.4	82	3.8	0.72
2.5	62.1	4.0	0.317
2.6	46.6	4.2	0.134
2.7	34.7	4.4	0.054
2.8	25.5	4.6	0.021
2.9	18.7	4.8	0.0079
3.0	13.5	5.0	0.0030

4. Calculation of reliability factors

4.1. Determination of variables and methodology

The impact on structural reliability of using reduced load factors was assessed by calculating the safety indices obtained when using various load factors. The reliability levels were established at the member level and, as such, the governing performance functions were determined from those of a single member in compression under concentric axial load. The first-order second moment (FOSM) method (in which two measures, the mean value and the standard deviation of probability density functions, are considered) was used for this study (CSA 1981; Ellingwood *et al.* 1980; Nowak and Lind 1979). This method uses a linear approximation of the performance functions at the most likely failure point. Reliability is measured by a safety index, β, which can be translated into notional probabilities of failure, p_f, according to the relationship:

$$[1] \quad p_f = \Phi(-\beta)$$

where $\Phi(.)$ is the standard normal cumulative probability. Some of these values are presented in Table 1 for reference in the following discussion. Standard normal cumulative probability tables are available from most mathematical handbooks.

Although more sophisticated methods are also available, the FOSM method provides sufficiently accurate results. However, it is worth noting that, even if the more accurate methods were used, the calculated probabilities of failure cannot be directly related to actual failure rates in service for two major reasons:

- simplifications in the assumptions made, particularly in the structural analysis of complex indeterminate structures; and
- the fact that the theory does not include failures due to gross human error.

Therefore, the safety index, β, can be considered only as a relative measure of safety. Nevertheless, it is useful for comparing design rules for different load combinations. The performance function, Z, used for the calculation of safety index, β, was

$$[2] \quad Z = R - S = R - E(D + G + W + I)$$

TABLE 2. Probabilistic assumptions for 1/30 probability of exceedance in any given year

	Ratio of mean over specified values	Coefficient of variation
Dead load, D	1.0	0.07
Guy load, G	1.0	0.07
Wind load, W	0.8	0.25
Ice load, I	1.0	0.35
Analysis error, E	1.0	0.07
Resistance, R	1.08	0.12
B	1.00	0.02
P	1.03	0.05
M	1.05	0.10
F	1.00	0.05

where S is the function of the applied forces on the member; D is the deal load contribution to the member compression; G is the guy load contribution to the member compression; W is the wind load contribution to the member compression; I is the ice load contribution to the member compression; E is a factor to express the impact of approximations in structural analysis; and R is the structural resistance of the member ($R = BFMP$, where B is the compression resistance of a member expression specified by CAN3-S16.1-M84; M is a factor for the material properties variability; F is a factor for the geometric property variability; and P is a factor representing the accuracy of the model).

The safety index is obtained from the ratio of the mean to standard deviation of the performance function, that is,

$$[3] \quad \beta = \mu_Z/\sigma_Z$$

The statistical values used for the subsequent analyses are tabulated in Table 2. All values used were based on existing information (Allen 1975; CSA 1984; Ellingwood *et al.* 1980), except for ice (Chaine and Skeates 1974). For the ice loading case, values were determined based on judgment, following the assumption that the uncertainty in the determination of ice loading is greater than that for wind loading. Sensitivity analyses were performed to determine the effect of this assumption.

The presence of the variable B accounts for a slight additional variability for the resistance equation, depending on the slenderness ratios. While strength expressions used in earlier steel design standards lacked uniformity in their safety index throughout the range of allowable slenderness ratios for compression members, from Fig. 1 it appears that the design equations of the current standard, CAN3-CSA-S16.1-M84, corrected this variability. Therefore, B was assigned a low coefficient of variation, independent of the slenderness ratio.

Finally, although the structural resistance equations for members in concentric axial compression currently in the code were developed for use in the design of other than steel angles, the reliability results obtained using the above indicated statistical parameters are expected to be equally adequate for the case of steel angles, if not conservative (Madugula and Mohan 1987). The recommendation in S37-M86 to use these equations appears reasonable.

4.2 Results from reliability analysis

The safety indices were calculated for the indicated performance function for designs performed using arbitrarily selected load factors of 1.5, 1.3, and 1.1 applied to the equations of

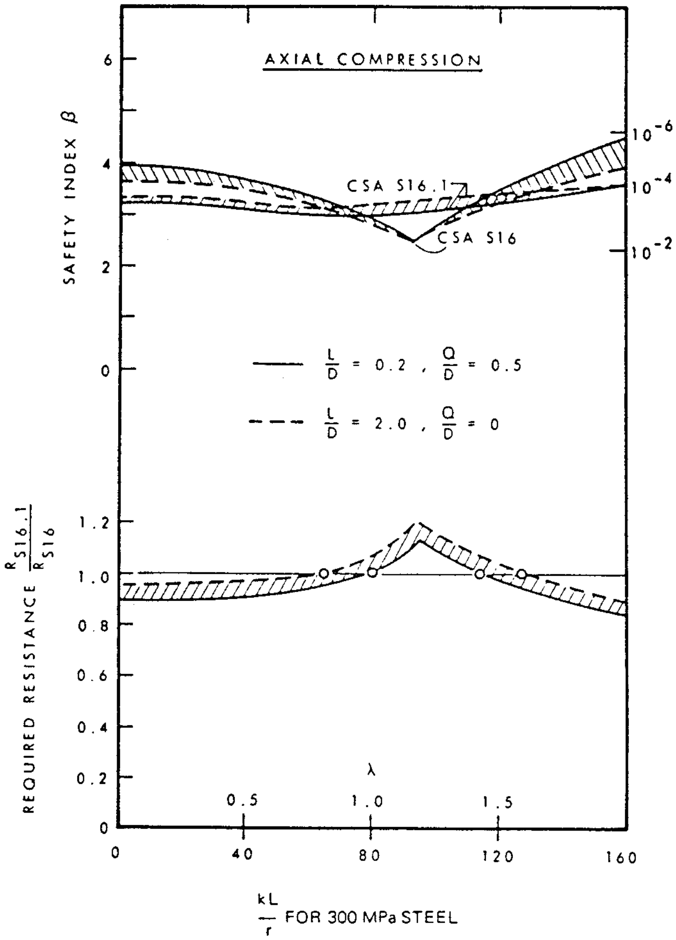

FIG. 1. Comparison of S16-69 and S16.1-M84 — axial compression. (Reproduced from Commentary to CSA-S16.1-M84.)

S37-M86. Results are presented in Fig. 2 for the dead–guy–wind load combination, and in Fig. 3 for the dead–guy–(half-wind)–ice load combination, in terms of normalized values D_n, G_n, W_n, and I_n. These represent portions of the total loads due to each load effect; for example, D_n = D/P, where P is the total design axial load on the member. As such, Fig. 2 illustrates the effect of various proportions of wind contribution to dead plus guy loads, $W_n/(D_n + G_n)$, on the final safety index resulting from those designs. Figure 3 illustrates the effect of various proportions of ice to wind ratios, $I_n/0.5W_n$, for a specific ratio of ice and wind to dead and guy load set at $(0.5W_n + I_n)/(D_n + G_n) = 0.67$. In all cases the normalized contribution from guy load was taken as twice that from dead load. This is reasonably representative of proportions found in existing towers. Also, the sum of wind and ice loads were taken to be 40% of the total design load, which is a representative upper limit on the contribution of these environmental loads to the total design force acting on individual elements. For all practical purposes, variations in safety indices produced by different G_n/D_n ratios were insignificant. Safety indices increased in situations where wind and ice contribution is less significant.

The curves of relative safety indices in the first load case studied (dead plus guy plus wind) were relatively flat, with almost constant values of safety index across the range of values of the ratio $W_n/(D_n + G_n)$. For the current edition of the S37-M86 standard (i.e., load factor of 1.5), the safety index varies from 3.3 to 3.6 ($p_f = 5.4 \times 10^{-4}$ to 1.4×10^{-4}). The

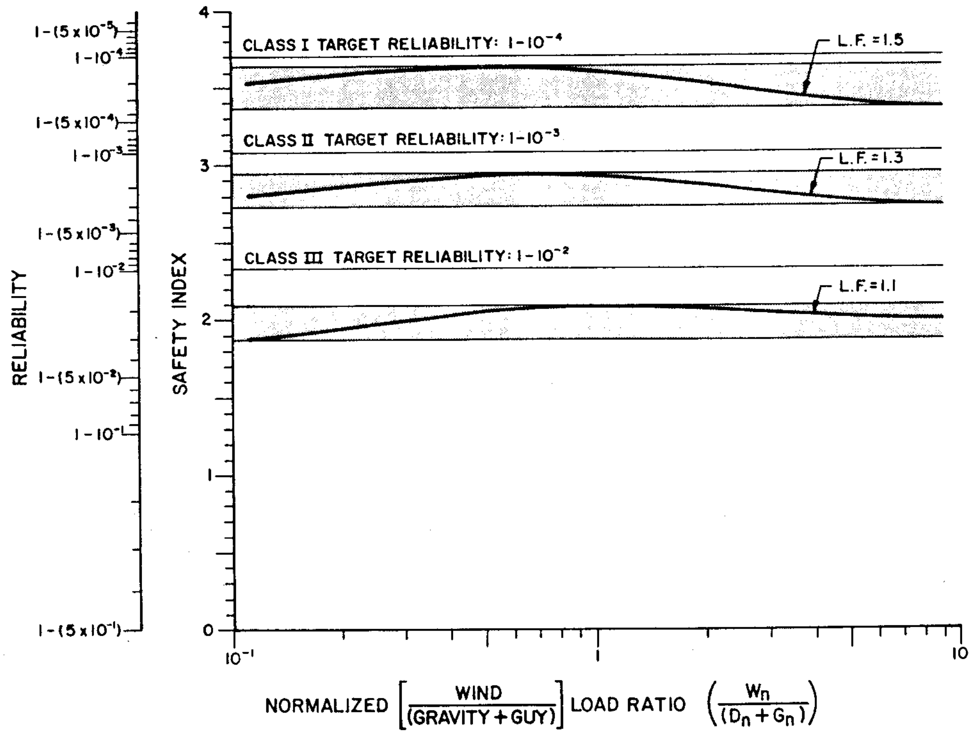

FIG. 2. Relative safety indices.

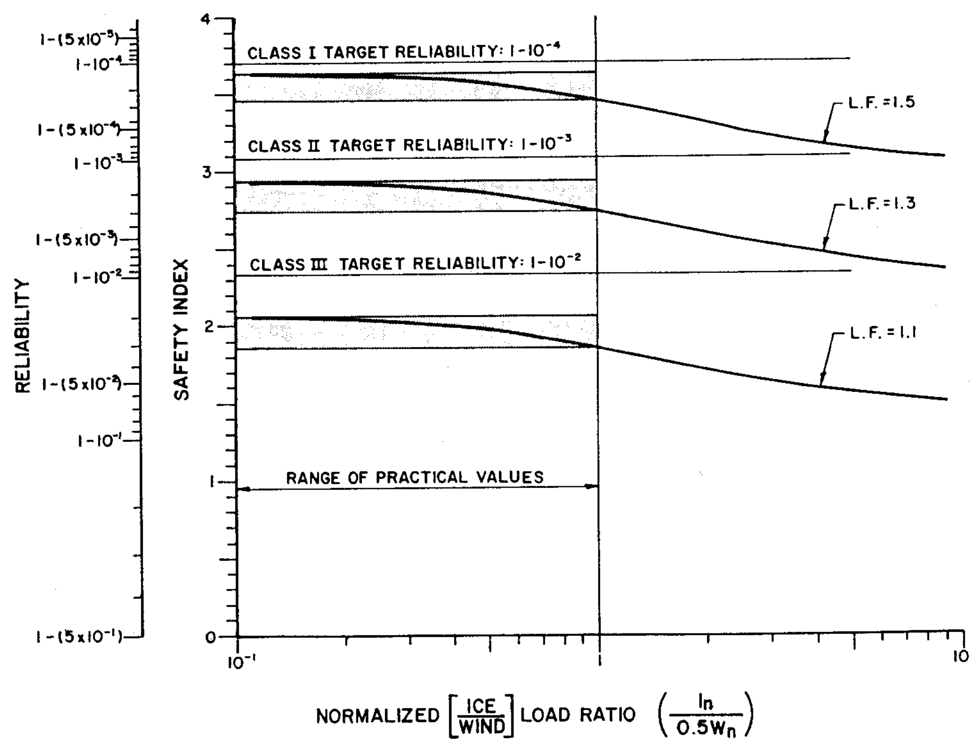

FIG. 3. Relative safety indices.

lowest reliability occurs when the wind load constitutes 100% of the design load. Reductions in the load factor seem to produce uniform decreases in the safety index. When a 1.3 load factor is used, the safety index ranges from 2.7 to 2.9 ($p_f = 4.0 \times 10^{-3}$ to 1.6×10^{-3}). This is equivalent to a tenfold increase in the probability of failure. Finally, a 1.1 load factor would further reduce the safety index, now ranging from 1.8 to 2.1 ($p_f = 3.9 \times 10^{-2}$ to 1.9×10^{-2}), i.e., another order of magnitude increase in the probability of failure. For this last load factor, the lowest reliability would occur if the members were designed to resist the dead and guy loads acting alone. Table 3 summarizes the notional reliability obtained for the first load combination.

TABLE 3. Notional reliability for the combinations of dead, guy, and wind loads

Load factor	Min. safety index	Reliability (30 years)	Reliability (Annual)
1.5	3.3	$1-5.4\times10^{-4}$	$1-2.0\times10^{-5}$
1.3	2.7	$1-4.0\times10^{-3}$	$1-1.0\times10^{-4}$
1.1	1.8	$1-3.9\times10^{-2}$	$1-1.3\times10^{-3}$

TABLE 4. Notional reliability for the combinations of dead, guy, half-wind and ice loads — reliability when $I_N/(0.5\ W_N) = \infty$

Load factor	Min. safety index	Reliability (30 years)	Reliability (Annual)
1.5	2.9	$1-1.7\times10^{-3}$	$1-4.0\times10^{-5}$
1.3	2.2	$1-1.25\times10^{-2}$	$1-4.0\times10^{-4}$
1.1	1.4	$1-8.0\times10^{-2}$	$1-2.8\times10^{-3}$

TABLE 5. Notional reliability for the combinations of dead, guy, half-wind and ice loads — reliability when $I_N = 0.5W_N$

Load factor	Safety index	Reliability (30 years)	Reliability (Annual)
1.5	3.4	$1-3.0\times10^{-4}$	$1-1.0\times10^{-5}$
1.3	2.7	$1-3.0\times10^{-3}$	$1-1.0\times10^{-4}$
1.1	1.9	$1-3.2\times10^{-2}$	$1-1.1\times10^{-3}$

The relative safety indices curves in the second load case studied (dead plus guy plus half-wind plus ice) displayed a larger sensitivity. Since the ice loads were assigned a much larger variability than the wind loads (mainly due to the larger "ignorance factor" with respect ot the predicted ice loadings), reliability is seen to be steadily decreasing as the design load contribution due to wind is progressively replaced by that due to ice. Again, the reductions in the safety index seem to be uniformly related to the load factor level. However, the reliability bounds, for a given load factor, are spread farther apart than in the dead plus guy plus wind load combination. For the 1.5 load factor, as the percentage effect of the ice load increases from 0 to 100% of the total ice and wind load contribution, the safety index varies from 3.6 to 2.9 ($p_f = 1.4 \times 10^{-4}$ to 1.6×10^{-3}), a tenfold decrease without changing the load factor. For the load factor of 1.3, this variation ranges from 2.9 to 2.2 ($p_f = 1.6 \times 10^{-3}$ to 1.3×10^{-2}), and for the 1.1 load factor, it ranges from 2.1 to 1.4 ($p_f = 1.9 \times 10^{-2}$ to 8.0×10^{-2}). The notional reliabilities obtained are summarized in Table 4.

Since the load case governing the design of tower members, using S37-M86, would always include a wind load contribution, Table 5 presents the reliability results that would be obtained should the ratio of normalized ice to half the normalized wind load be limited to about 1, the maximum ratio expected in practical situations. It is interesting to note that the resulting reliability is not much different from that which has been obtained for the first load case, despite the larger uncertainties in the ice load. However, it should be emphasized that reliability against rime ice loading (in-cloud icing) cannot be calculated at this time.

To best meet the target reliabilities for the practical range of the various load proportions, load factors of 1.5, 1.35, and 1.2 were visually estimated from Figs. 2 and 3. This approach was judged appropriate as, contrary to a true LSD approach, the shape of the safety indices curves cannot be controlled in this case. The target 10-fold and 100-fold increases in the probability of failure have been approximately achieved by reducing the load factors from 1.5 to 1.35 and 1.5 to 1.2 respectively. These reductions correspond to 90 and 80% respectively of the load factor used in the current edition of S37. The load factors necessary to achieve the reliability levels previously described for each reliability class are summarized in Table 6.

The preceding reliability analysis was carried out to arrive at the notional probabilities of failure and recommended load factors. This analysis is strongly dependent on the actual statistical variability of ice loads, which is unknown at this time. Further, it should be emphasized that all load factors reviewed in this study are applicable to the member forces obtained after analysis, in accordance with the current S37-M86 requirements, and, as such, are not true LSD load factors. If these load factors were to be used in a true LSD approach, the resulting safety indices would be substantially higher, due to the highly nonlinear behaviour of guyed structures. To maintain similar safety indices when using a true LSD approach, a different calibration is required to arrive at the requisite load factors. Finally, this reliability study was based on member strengths only, and overall tower system reliability was not considered.

5. Review of cost impacts

The decision to adopt reduced load factors in some circumstances, resulting in reliability levels lower than those currently in place in S37-M86, cannot be justified unless it results in significant savings in the expected upgrading costs. These savings, incurred at the time of upgrading, then have to be weighed against the replacement cost due to the expected larger collapse rate consequent on the decision to use less reliable towers. The following two sections attempt to determine both these costs.

5.1. *Upgrading cost as affected by load factors*

If only one element in the current edition of S37 differed from its preceding edition, cost impacts could be assessed fairly straightforwardly. However, in the current situation, a significant number of changes took place. In particular, the revisions to the estimation of environmental loads significantly complicated the variations in cost determination. A detailed review and discussion of the evolution of the S37 standard is presented in Magued *et al.* (1989).

A practical approach to this problem was to select a number of actual towers which were recently upgraded to meet the new S37-M86, assuming that they are representative of the population, and extrapolate overall costs accordingly. These towers were selected to cover the major combination of specified ice and wind loading zones, and simultaneously to cover most major Canadian geographical regions. The towers selected, along with their typical design parameters, are listed in Table 7. For comparison with the earlier standard, the wind pressure on flat surfaces, p, at 9.1 m derived from the reference velocity pressure, q, is shown for the 1986 values.

For all of these towers, additional equipment was to be added and reanalysis by an engineer had been required. To determine the actual expense incurred due to the change of code edition, this equipment-related expense had to be deducted. This was achieved by analyzing the tower to the requirements of the

TABLE 6. Recommended load factors and corresponding notional reliabilities

Reliability class	Load factor	Average reliability (30 years)	Average reliability (Annual)
I	1.5	$1-1\times10^{-4}$	$1-3.3\times10^{-6}$
II	1.35	$1-1\times10^{-3}$	$1-3.3\times10^{-5}$
III	1.2	$1-1\times10^{-2}$	$1-3.3\times10^{-4}$

TABLE 7. Selected towers for upgrade costs study

			Original design loads		S37-M86 loads		
Tower	Location	Height (m)	Wind (kPa) p	Ice (mm)	Wind (kPa) q	Wind (kPa) p	Ice (mm)
A	Ont.	290	1.44	13	0.45	1.35	25
B	N.B.	110	1.92	25	0.71	2.14	50
C	N.S.	167	1.92	25	0.60	1.80	50
D	Ont.	152	1.44	13	0.34	1.02	10
E	Ont.	118	1.44	13	0.29	0.86	13
F	Nfld.	95	1.92	38	0.86	2.59	50
G	Ont.	152	1.44	13	0.48	1.44	25
H	Man.	116	1.92	13	0.45	1.35	10
I	N.B.	138	1.92	25	0.62	1.87	38
J	Que.	149	1.44	25	0.60	1.80	3.8

S37-1965 (CSA 1965) and estimating the cost of upgrade required. The tower was then reanalyzed using S37-M86 (CSA 1986), and the new upgrading costs were estimated. Further, the equivalent load factor to S37-M86 for which no upgrading costs would be incurred was also calculated. This equivalent load factor will be referred to as the zero-1986-cost factor. As the impact of added equipment on the existing tower and required reinforcement increases, this zero-1986-cost factor decreases.

The acceptance of reduced load factors (say, 1.35 and 1.2 for classes II and III described previously) for analyses based on the latest edition of S37 would also result in reduced upgrading costs. The variation of upgrading costs as a function of the load factors used in the structural analysis is strongly case-dependent and is expected to be largely irregular, consisting of numerous steps. For example, past a given load factor, some guys may need to be replaced, which triggers a major increase in costs. Unfortunately, the actual value of the load factor where such sudden jumps in upgrading costs would occur cannot be determined in general terms. Therefore, for the purpose of this study, a linear variation of cost in function of upgrading level, from the zero-1986-cost point to the actual upgrading cost to meet the requirements of CSA-S37-M86 (load factor of 1.5), is appropriate, and should be seen as a representative average cost for a number of similar structures. From this straight line, costs can be estimated for various load factors, and the upgrading costs estimated using S37-1965 can be attributed an equivalent load factor. This is illustrated in Fig. 4. Using this method, the upgrading costs for the cases identified above are summarized in Table 8. Negative values in Table 8 indicate that, for these particular cases, upgrading costs are less if S37-M86 is used instead of S37-1965.

The upgrading costs shown are only budgetary in nature and do not include any additional expenses incurred to maintain normal operations during the structural upgrading. These additional costs may be significant and, in some circumstances, may even exceed the upgrading costs themselves. However, these case-dependent costs were deliberately omitted, so that the comparison could address only the effect of code changes on structural adequacy.

5.2. Replacement costs at current failure rate

A survey of the major participants in the telecommunications industry revealed that guyed towers taller than 75 m have collapsed at the rate of 0.77 tower/year, or 0.055%/year, over the period between 1958 and 1988 (Magued *et al.* 1989). The regular failure of towers throughout the country adds a significant cost to the industry. Contrary to planned expenses, emergency situations, such as those created by the unexpected and sudden loss of a broadcasting tower, generate expenses far in excess of those otherwise dictated by competitive bidding; not to mention loss of revenue, which may greatly exceed all other expenses from physical losses. Therefore, in maintaining low reliability levels of existing towers, there is an implicit cost currently being borne by the telecommunications industry for the replacement of towers collapsing at semi-regular intervals. A particular owner may enjoy many consecutive years of adequate structural performance before finding himself in a crisis situation. Further, these trouble-free intervals may help to create a misperception of the actual structural reliability.

If towers continue being upgraded only to meet the recommendations of S37-1965, the cost to maintain the inventory of existing towers as collapses occur can be translated into an equivalent annual cost. Such an approximate annual replacement budgetary cost has been estimated. In Fig. 5, the cost of a new broadcasting tower is expressed in terms of height in metres. The lower curve is representative of the cost of a totally new tower. Should the new tower actually be a replacement for a collapsed existing tower, these costs should be increased by 25%, to allow for removal of the collapsed tower debris, as shown by the middle curve in Fig. 5. Should the tower be a replacement for an existing tower that did not collapse, and for which a careful and systematic removal must be accomplished, the original costs could be doubled, as indicated by the higher curve in Fig. 5. From this figure, for the range of tower heights from 75 to 195 m (a range covering the bulk of existing towers taller than 75 m), the cost of replacing a collapsed tower by a new tower ranges from \$75 000 to \$400 000. An average of \$225 000, for the fabrication and erection of the tower structure, is reasonable for the purposes of this discussion. As previously mentioned, expenses arising from "emergency replacements" were not included, but are worth contemplating.

• Should a facility be of sufficient importance to require prompt replacement, an additional premium could be attached to the cost of the tower for most work orders whose urgency does not respect usual bidding procedures, and where fabricator and contractor are to deliver against tight deadlines.

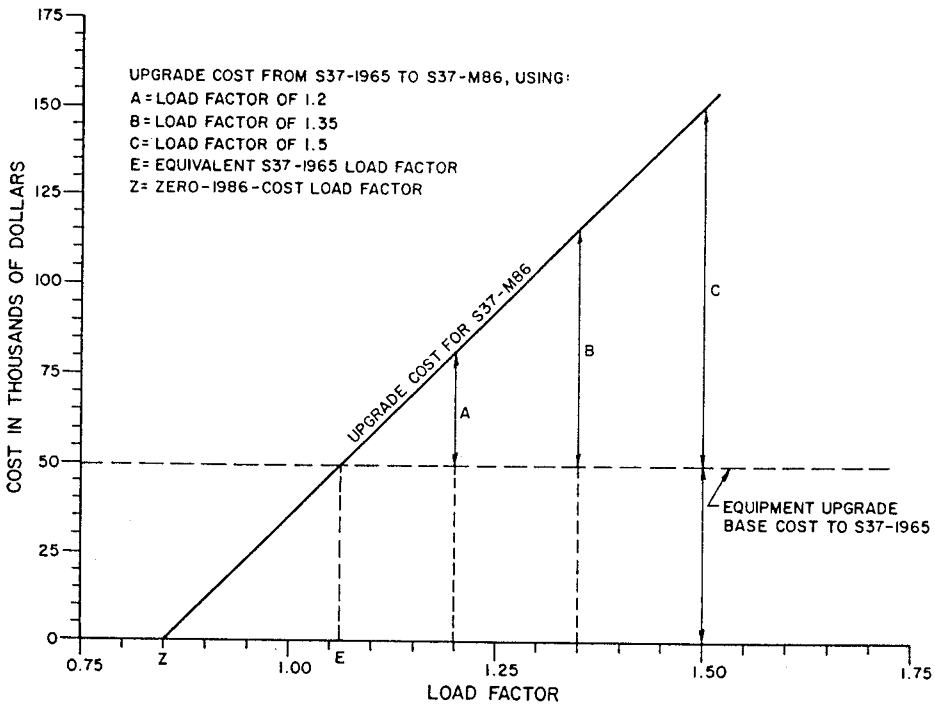

FIG. 4. Average structure upgrading costs as a function of load factors.

TABLE 8. Upgrading costs

Tower	Cost to upgrade to S37-1965 ($)	Cost to upgrade to S37-M86 ($)	Zero-1986-cost load factor	Upgrading costs from S37-1965 to S37-M86 using load factor 1.2	1.35	1.5	Equivalent S37-1965 load factor
A	38 000	115 000	0.84	24 000	50 000	76 000	1.06
B	10 000	140 000	0.72	76 000	103 000	140 000	0.78
C	0	190 000	0.80	110 000	150 000	190 000	0.80
D	65 000	43 000	1.5	—	—	−22 000	>1.5
E	48 000	20 000	1.5	—	—	−28 000	>1.5
F	0	150 000	0.42	108 000	127 000	150 000	0.42
G	0	130 000	0.46	92 000	111 000	130 000	0.46
H	10 000	0	1.5	—	—	−10 000	>1.5
I	0	130 000	0.42	94 000	112 000	130 000	0.42
J	35 000	190 000	0.52	97 000	126 000	155 000	0.70
Avg.*				60 100	78 100	96 100	

*Average is for the ten cases studied, using $0 for cases D, E, and H.

• The cost to replace antennas and transmission feeds lost in the collapse could be a major item.

• Incapacity to broadcast for an extended period of time translates into very costly losses in revenues. These losses are difficult to establish accurately, as they vary depending on the number of stations on a given tower and the size of the audience reached by each station. According to Statistics Canada (1987), weekly revenues for FM and TV stations average from $100 000 to $230 000 per station, for moderate to large population areas respectively. Even for a minimal one week of down-time, and assuming an equal number of FM and TV antennas on each tower, losses in revenues could easily range from $100 000 to $2 750 000, depending on the number of stations affected by the collapse and the size of the audience affected.

• Finally, liability compensations attached to personal injury or loss of life can be quite large.

Although some of these costs may be absorbed by the insurers in the short term, they are eventually transferred to the industry through increased premiums. Of course, self-insured owners would have to absorb these costs in their entirety. Therefore, the collapse of a single tower costs, on average to the industry, anywhere from a few hundreds of thousands to several million dollars per event, depending on the importance of the loss, and significantly higher if injuries or loss of life occurs. Thus, based on the average failure rate noted, the annual replacement cost to the broadcast industry can be expected to average from $300 000 to $1 500 000 per year (liability for loss of life excluded).

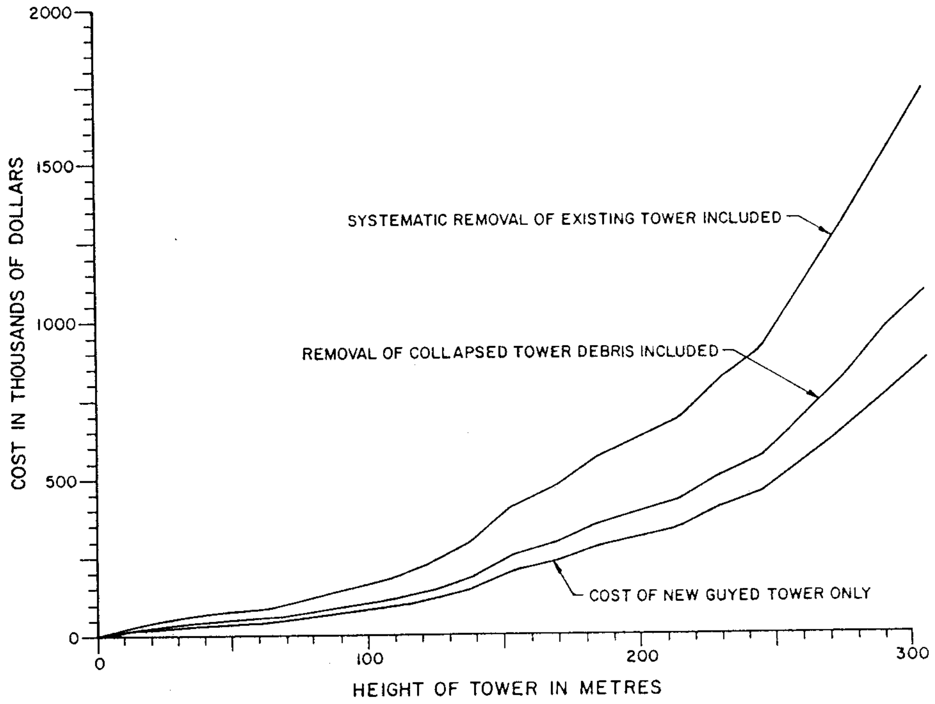

FIG. 5. Replacement costs of guyed towers.

6. Discussion of results

6.1. Replacement costs

The reasons for the generally unacceptably high rate of failure of guyed towers have been presented elsewhere (Magued *et al.* 1989). Should the industry elect not to upgrade the current reliability of these towers, they are expected to continue failing at the reported rate. The maintenance of existing structures at reduced reliability levels indicates acceptance of the eventual cost to replace failed structures. The current S37-M86 position to leave existing towers unaltered until such is required due to modification of existing conditions is reasonable. However, the relatively high risk of failure associated with reduced load factors necessitates a review of the adequacy of all sites which would meet the definition of classes I and II.

6.2. Economic consequences of upgrading, using reliability classes

When modifications of existing conditions or addition of new loads onto existing towers are requested, a review of structural adequacy by a qualified engineer is mandatory. The practice, until publication of S37-M86, was to upgrade the tower only to satisfy the original code requirements. The current standard allows analysis to the original code; however, should any upgrading be required, reanalysis should be done to the 1986 edition of the standard, and reinforcing should be done to meet these requirements as well. Both practices are faulty to some extent.

Not improving the reliability of existing structures does little to reduce the excessively high reported failure rate. Nevertheless, requiring that all existing structures be upgraded to the current 1986 level would result in excessive costs. In some circumstances, existing towers may have already served beyond their expected service life, or may be of a lesser significance in a network, so that their reliability need not be equivalent to new towers. From this reasoning, intermediate steps seem appropriate.

Three reliability classes were determined in Sect. 3. These allow the owner, in collaboration with the engineer, to determine how reliable the upgraded tower should be. In some circumstances, savings of 70% in the cost to upgrade the tower can be achieved for going to the lower reliability class (Class III), instead of the reliability levels currently required by S37-M86 (Class I). In other cases, the savings are much less significant, and the owner may elect to buy the increased reliability provided by Class I, in view of all the cost attached to a tower collapse. Given the costs to achieve the three upgrading classes, the owner would thus be in a position to decide on an appropriate level of realibility. From this reasoning, it becomes essential that the owner be provided with the upgrading costs necessary to satisfy each reliability class.

The changes in environmental loads specified in the tower standard have had the largest impact on upgrading costs. Towers on sites where environmental loads were reduced when going from S37-1965 to S37-M86 generally do not require any upgrading. In fact, some of these towers would have required upgrading using the environmental loads stipulated in earlier editions of the standard, whereas the current edition allows smaller loads, which may obviate the need for any upgrading. Most existing towers in Northern Ontario and the Prairies may not need to be upgraded to meet the requirements of S37-M86, whereas towers in other areas of the country probably will. The most significant upgradings are expected for towers in regions where ice requirements have increased from 1965 to 1986. The effect of wind is not normally as significant, except where site-specific wind data indicate large changes in design values.

6.3. Applicable standard

Verification of the adequacy of existing towers using obsolete editions of the S37 standard leads to irrational situations. Reasons for this include the changes in wind and ice zones and intensities, which occurred in different editions of the standard. Simple logic alone would dictate that the latest edition of any such map, and that one alone, should be used in any engineering tower analysis. Alternatively, up-to-date site-specific data obtained from the Atmospheric Environment Service (AES) of the Canadian Department of the Environment can be used.

Further, revisions in the analytical equations determining the resistance of axially loaded compression members have also occurred over time. As indicated in Fig. 1, the reliability provided by previous code equations was not uniform. Therefore, it is reasonable to also require design calculations to always be performed using the latest strength equations. Using design equations proven inadequate is not good practice.

Meeting the above two criteria automatically leads to the latest issue of S37 as the only admissible standard, at least as far as environmental loads, strength equations, and analysis techniques are concerned.

6.4. Reliability classes and load factors

Since, in some situations, the upgrading costs to the current edition of the CSA-S37 standard could be fairly prohibitive, lower upgradeability costs could be achieved if a lower reliability against failure could be used. Three classes of realibility for existing structures are proposed.

New structures should all be designed for Class I reliability. This is equivalent to the level currently provided by S37-M86 using a load factor of 1.5. The creation of other classes is intended as a special circumstance to mitigate the cost impact of upgrading existing towers designed to earlier editions of S37.

Structural reliability analyses have indicated that a 10% reduction in the load factor (to bring the load factor down to 1.35) would produce an approximately tenfold increase in the probability of failure. This reduced reliability could be acceptable if the existing tower can meet the criteria set in the Class II reliability. For Class III towers, a further reduction in reliability would be achieved by using a 20% reduction of the load factor (to bring the load factor down to 1.2). At this level, Class III structures would then have a probability of failure approximately 100-fold larger than Class I structures.

The proposed lower reliabilities are recommended as a mitigation to the owners against excessive costs incurred to correct previous codes' admitted deficiencies, and at any time, no new towers should be allowed to be designed to lesser reliability than provided by the 1.5 load factor.

6.5. Adequacy review procedures

In Canada, a broadcaster requesting a licence to operate a transmission facility must complete the form "Data Required Regarding the Structural Adequacy of Antenna-Supporting Structures for Broadcasting Undertakings," and submit it to the Director of the Broadcasting Regulation Branch of the Canadian Department of Communications (DOC). This form must be signed and sealed by an engineer to certify the structural adequacy of the antenna-supporting structures for the station, and must be submitted each time that loading is increased by the installation of additional equipment onto existing towers, as well as for new towers.

In reviewing the adequacy of existing towers considered as Class II or III, the form would be appended with a statement by the owner accepting the increased risk inherent in adopting a lower load factor, and a statement by the engineer that the tower meets the requirements of CAN-CSA-S37-M86 under the reduced load factor.

Although it is the owner who would determine the class of an existing structure in accordance with the limitations expressed in the reliability classes definitions, the engineer's review may indicate that, in some cases, upgrading to a class of superior reliability can be achieved inexpensively. For that reason, consultation with the engineer may be beneficial in the final selection of the most advantageous reliability class for towers under review. Ideally, the structural adequacy review of an existing tower would proceed as follows:

- When there is a need to review the structural adequacy, change the antennas, or upgrade an existing tower, the structure would be analyzed to the requirements of S37-M86, and the existing load factor determined.
- From the results of the analysis, the extent of reinforcement to achieve each of classes I, II, and III would be evident regardless of the designated class of the structure. A preliminary rough estimate of the cost involved in upgrading to each of the classes would then be made. Depending on the extent of reinforcement needed, this step may require one iteration to determine the feasibility of reinforcing the structure.
- The preliminary rough cost estimates would be reported to the owner, who would decide whether or not the extra cost to upgrade to a higher class than the initial designation is indicated. In some cases, it may be possible to upgrade to the next higher class for only a minor additional cost.
- The detailed design of the reinforcement and final cost estimate would be done to the final class decided upon. The Department of Communications' structural adequacy form would then be completed, as described previously, indicating the class of structure decided by the owner.

7. Deficiencies in current knowledge

As of today, there is still a significant number of deficiencies in current knowledge, preventing better control of the reliability of guyed towers.

The probability characteristics of glazing ice, used as a load condition, remain mostly unknown. AES has set the correction of this deficiency as one of its goals for the coming three years. Further, knowledge about rime ice (in-cloud icing) is also fairly limited. Increased knowledge about this environmental condition would greatly enhance the reliability of the S37 standard as a whole.

Development of a true limit states design (LSD) format for guyed towers is required. Although the current version of S37 borrows heavily from the terminology and format of the LSD design method, it is, in fact, closer to the philosophy of a working stress design method (Magued *et al.* 1989). Because of the design problems germane to guyed towers, this should be seen as a long-term goal, which should progress through the contribution of many qualified experts. This development will require a review of the statistical parameters applicable to the behaviour of this class of structure as a system. (The analysis presented herein focused on the individual tower truss element, as is appropriate in light of the S37-M86 philosophy.) Future work is also required to address the reliability of the guy elements and their related hardware and anchors, as well as that of the overall guyed system. Some work in these areas has been carried out for various European tower standards. A review of that work would be helpful in the development of the Canadian standard.

The lack of complete documentation of tower failures has been discussed by the authors in another paper (Magued *et al.* 1989). That discussion indicated the need for better documentation of all future events through an industry-wide collaborative effort. This information is most pertinent for the continued monitoring of the performance of these structures and the development of economical design standards.

Finally, the resistance of existing towers to excessive and rare ice loading conditions under light- or no-wind conditions should be assessed. This load condition is currently not a required design load combination in S37-M86, and resistance to this state of load is only indirectly achieved by the inherent capacity imparted to the structure from the other load conditions. A third load combination of mostly ice loading may govern the design in some regions where significant in-cloud icing under low wind may occur. Four towers collapsed in March, 1983, at Baldy Mountain, Brandon, and Carlyle, as a result of excessive loading of this type.

8. Summary of guidelines

In light of the studies conducted by the authors and described in this paper, the authors recommend the following:

1. That the CSA-S37 Committee adopt a revision to S37-M86 incorporating the following guidelines for the structural adequacy of tower structures:

(a) Adopt the lastest issue of S37 as the only applicable standard.

(b) Establish three significant reliability classes for the structures.

• *Class I*: This class would include important installations which are to have a reliability equivalent to that currently provided by S37-M86. It is believed that this class would provide reliability levels for structures constructed in urban areas, or abutting a residential area, equal to those inherent in structures designed to the National Building Code. This class would include broadcast sites where tower failure is likely to result in loss of life, unacceptable loss of service, or unacceptable economic losses.

• *Class II*: This class would include noncritical existing installations located in rural or remote areas, where the likelihood of loss of life, if a tower collapsed, would be negligible. This class also includes sites where loss of service and (or) economic consequences of a failure are not considered critical. The probability of failure for towers in Class II is roughly 10 times larger than for towers in Class I.

• *Class III*: This class would include nonessential existing installations located in rural or remote areas, where all the consequences of failure are tolerable, and unusually high risk of failures can be tolerated during the life of the structure. This would include sites where loss of the tower, equipment attached, and service provided would be of little consequence. Such sites would need to incorporate warning arrangements similar to Class II. The probability of failure for towers in Class III is roughly 100 times larger than for towers in Class I.

(c) The load factors to apply to the equations of S37-M86, and to be used in the adequacy review of existing towers, are proposed as follows: Class I: load factor = 1.50; Class II: load factor = 1.35; and Class III: load factor = 1.20.

(d) All new structures are to be considered as Class I.

(e) All existing structures would be classified, by their respective owners, into one of the three classes determined above. Class I structures would then be reviewed at an early date for adequacy. Classes II and III structures would be reviewed only when a change to the existing conditions is contemplated.

(f) The DOC Structural Adequacy Form for classes II and III structures is to be appended with a statement, signed by both the owner and the engineer.

2. Work should proceed on the development of a true LSD Canadian standard for towers.

3. A central Canadian registry is needed to document, investigate, and disseminate information on tower failures.

4. That the CSA-S37 Committee review the need to add a third load combination to the mandated load combinations, comprising a heavier ice load and a further reduced wind load.

9. Conclusions

Structural reliability studies were conducted to develop rational guidelines applicable to the current edition of the S37 standard. Three reliability classes are defined, with increasing greater probabilities of failure corresponding to reductions in the load factors to be used in the analysis. The upgrading to various reliability classes can have a most advantageous impact on upgrading cost if the increased probability of failure can be tolerated and the availability of upgrading costs to those three reliability levels can be most useful to owners. It is hoped that this set of rational guidelines will lead to more consistent practices with respect to the analysis of existing structures.

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ALLEN, D. E. 1975. Limit states design. Canadian Journal of Civil Engineering, **2**: 36–49.

CHAINE, P. M., and SKEATES, P. 1974. Wind and ice loading criteria selection — industrial meteorology — study III. Atmospheric Environment Canada, Toronto, Ont.

CSA. 1965. CSA-S37-1965 — Antennas, towers and antenna-supporting structures. Canadian Standards Association, Rexdale, Ont.

——— 1976. CSA-S37-1976 — Antennas, towers and antenna-supporting structures. Canadian Standards Association, Rexdale, Ont.

——— 1981. Special publication – S408-1981 — Guideline for the development of limit states design. Canadian Standards Association, Rexdale, Ont.

——— 1984. CAN3-S16.1-M84 — Steel structures for buildings (limit states design). Canadian Standards Association, Rexdale, Ont.

——— 1986. CAN-CSA-S37-M86 — Antennas, towers and antenna-supporting structures. Canadian Standards Association, Rexdale, Ont.

ELLINGWOOD, B., GALAMBOS, T. V., MACGREGOR, J. G., and CORNELL, C. A. 1980. Development of a probability based load criterion for American National Standard A58. National Bureau of Standard, U.S. Department of Commerce, NBS SP 577, Washington, DC.

MADUGULA, M. K. S., and MOHAN, S. 1987. Concentric axially compressed angle columns — comparison of experimental and calculated failure stresses. Proceedings of the 1987 Annual Technical Session of the Structural Stability Research Council, March 24–25, Houston, TX.

MAGUED, M. H., BRUNEAU, M., and DRYBURGH, R. B. 1989. Evolution of design standards and recorded failures of guyed towers in Canada. Canadian Journal of Civil Engineering, **16**: this issue.

NOWAK, A. S. N., and LIND, C. 1979. Practical code calibration procedures. Canadian Journal of Civil Engineering, **6**: 112–119.

Statistics Canada. 1987. 1986 — Analysis of financial statistical data for all private AM, FM & TV stations. Statistics Canada for the Canadian Association of Broadcasters. Ottawa, Ont.