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Limit states criteria for structural evaluation of existing buildings

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Difficulties have been encountered in applying the requirements in the National Building Code (NBC) and referenced CSA standards to the structural evaluation and upgrading of existing buildings in Canada. The Associate Committee of the NBC has therefore initiated an effort to provide guidelines on the application of Part 4 of the NBC to existing buildings, allowing alternative requirements where needed. As an initial step towards fulfilling this goal, this paper reviews all aspects of limit states criteria for structural evaluation and proposes minimum load factors based on a life safety criterion. The proposed load factors allow more flexibility in practice but require more professional judgment.

Key words: existing buildings, structural evaluation, criteria.

Des difficultés ont été rencontrées dans l'application des exigences du Code national du bâtiment du Canada (CNBC) et des normes de l'ACNOR auxquelles le CNBC fait référence, à l'évaluation des structures et à l'amélioration des bâtiments existants au Canada. Le Comité associé du CNBC a donc entrepris des efforts en vue d'élaborer des lignes directrices concernant l'application de la partie 4 du CNBC aux bâtiments existants, prévoyant même des exigences de rechange le cas échéant. Comme première mesure en vue de la réalisation de cet objectif, cet article examine tous les aspects des critères aux etats limites d'évaluation des constructions et propose des coefficients de charge minimums basés sur un critère de sécurité. Les coefficients de charge proposés permettent en pratique une plus grande flexibilité mais exigent un meilleur jugement professionnel.

Mots clés : bâtiments existants, évaluation des structures, critère.

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1. Introduction

Renovation of existing buildings is a growing activity in the construction industry. Although the National Building Code (NBC) includes renovation within its scope, the structural requirements contained in Part 4 of the NBC and referenced CSA standards (NBC/CSA criteria) were written for the design of new buildings (or new additions). When applied to existing buildings they create difficulties:

- Many requirements are in the nature of specifying certain arrangements or percentages of materials (such as reinforcing in masonry) which are economical to implement during the construction process but uneconomical to comply with for existing buildings built before the criteria existed. In such cases alternative criteria are needed.
- Many older buildings consist of structural systems, components, or materials which the NBC/CSA standards do not address. When properly connected together, however, these old systems can be made to work effectively. Information on structural properties of such systems are lacking, making evaluation difficult.
- Many old buildings, despite lack of code compliance, have performed satisfactorily over the years without distress or failure. In addition, since the building exists, some structural parameters such as dead load or strength can be measured. The NBC/CSA design criteria take no account of this information.

To help overcome difficulties in applying the NBC to existing buildings, a task group to the NBC Part 4 committee has been set up to develop guidelines that will identify methods and criteria to be used for the structural evaluation and upgrading of existing buildings.

As an initial step towards fulfilling this goal, this paper discusses the differences between the requirements for existing buildings and those for new buildings and proposes minimum load factors for structural evaluation of existing buildings based on risk to life. All aspects of criteria for structural evaluation are reviewed, including format, loads and load factors, analysis, resistances and resistance factors, measurements, load testing, and past performance.

2. Requirements for existing versus new buildings

Structural criteria such as load and resistance factors are based on fundamental requirements of human safety, human comfort, building function, and economics. The NBC/CSA criteria address human safety first and foremost, but they also address function, comfort, and damage (serviceability limit states) and are based on economical solutions or methods to achieve these basic requirements. The fundamental requirements for existing buildings are the same as for new buildings, but there are differences between the two situations that affect the resulting criteria:

- *Economics*: In the design of a new building, it costs very little to provide a degree of structural safety that experience shows is very high. In other words, very little is gained in shaving safety factors in specific situations in order to save money and this means that the generic criteria contained in the NBC/CSA, which conservatively cover all situations, are suitable for practice. For structural evaluation, however, the difference in cost between meeting or not meeting a criterion, in other words the cost if upgrading is required, can be large. The economics of upgrading therefore puts much greater pressure to determine criteria for each situation based on the fun-

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damental requirements of life safety, comfort, function, and economics.

• *Heritage*: Heritage puts value on preserving existing materials and, depending on the artefacts affected, the less intervention the better. Besides, it makes sense to use materials already there rather than to support them (as an artefact) by an entirely new structure. Many unreinforced masonry buildings, for example, can be made safe against earthquakes by properly tieing the masonry to the wood floors. The heritage criterion is therefore minimum destruction of heritage value of existing materials and systems, either as a result of the renovation process or as a result of expected future building damage.

• *Uncertainties*: Uncertainties in loads and resistances at the design stage are reflected in load and resistance factors. At the evaluation stage, uncertainties in loads and resistances can be either greater than at the design stage (e.g., hidden component or details, deterioration) or less (properties measured, load tests or satisfactory past performance). The range is therefore broader and the incentive to find out what exists should be high. In fact, structural criteria should not be applied until key components and details of the structure have been clearly identified.

• *Past performance*: Satisfactory past performance provides information not available at the design stage. It provides direct information on the serviceability of the building, and this usually (but not always) means that serviceability criteria for design need not be applied. It provides indirect information on the safety of the building, information whose value depends on a number of factors such as age and type of loading.

Evaluation criteria should therefore be more situation-specific (less generic) than design criteria, allowing the evaluator to take into account the consequences of failure (life safety, function, damage, etc.) in specific situations and to incorporate all information that can be obtained, including satisfactory past performance.

In general, the minimum criteria for evaluation will be based on life safety, whose measure is described by the probability of death or injury for persons exposed to structural failures. Certain buildings such as hospitals must also remain functional after a disaster such as an earthquake, and therefore damage control is an added fundamental requirement for these buildings. Finally, additional structural protection against damage beyond that required for life safety should be provided for specific failures if the reduction in expected loss (including heritage) due to future damage is justified by renovation costs.

3. Criteria for evaluation

Review of existing criteria

Recommended criteria for structural evaluation consist in adjustments to criteria contained in design standards. Evaluation criteria recommended by the Institution of Structural Engineers (ISE 1980) include adjustments of load and resistance factors on the basis of reduced uncertainty (dead load, analysis, load testing). Evaluation criteria recommended by Comité Euro-International du Béton (CEB 1983) contain similar adjustments, including a decrease in load factors for dead load and earthquake, and more conservative resistance factors for damaged and partially repaired structures. The Czech standard on evaluation of building structures (CSN 1986) contains a reduction in the load factor if the maximum applied load during the life of the structure was greater than the design load.

The most significant relaxation from design criteria, however, is a decrease in seismic load. This relaxation arises out of severe economic pressures when applying current design criteria to existing buildings. The latest U.S. criteria (FEMA 1989) recommend an across-the-board decrease in earthquake load of 33% for flexible buildings (in the medium period range) and 15% for stiff buildings (of short natural period), but increased earthquake loading is required (by means of a reduction in the ductility factor) where detailing for ductility is inadequate or uncertain. For seismic evaluation, the New Zealand National Society for Earthquake Engineering (NZNSEE 1985) recommends a reduction factor of the code earthquake load ranging from 0.4 to 1.0, depending on occupancy classification and post-disaster consequences. Earlier U.S. criteria for seismic evaluation (FEMA 1986) contain reductions similar to NZNSEE (1985) to as low as 0.25. For heritage unreinforced masonry buildings, Agbabian, Barnes, and Kariotis (ABK 1986) recommend a decrease in seismic zone if the consequences of the specific failure are not life threatening. The Vancouver Building Code (1985) allows a relaxation of NBC earthquake requirements when renovation cost is less than 75% of the assessed building value (which can be low for an old building) provided public safety is ensured, but does not specify how much.

Evaluation criteria for bridges have recently been introduced, which allow a relaxation from design criteria based on the probable consequences of failure. The CSA standard code for design of highway bridges (CSA 1990*a*) allows reductions in load factors for bridge evaluation as a function of element behaviour, system behaviour (redundancy), and degree of inspection, based on the concept that these factors provide warning which reduces the likelihood of death or injury. The Ontario Highway Bridge Design Code (OHBDC 1983) provides a 10% reduction in load factor for bridge evaluation, provided the bridge is posted for the load restriction and is inspected every 5 years. This code goes as far as stating that a concrete bridge requires no evaluation if it is inspected regularly and shows no signs of distress. The draft for the next edition of the code (OHBDC 1990) also decreases the live load factor if redundancy is present. Both the CSA and OHBDC bridge codes allow a reduction in load factor (higher failure risk) for controlled vehicle passage where normal traffic is kept off the bridge. Both also allow increased resistance for specific components such as concrete deck slabs and connectors in steel construction, but specify decreased resistance where deterioration is evident. Recommendations for bridge evaluation to the American Association of State Highway Transportation Officials (AASHTO) by Moses and Verma (1987) vary the live load factor as a function of loading control and traffic volume, and vary the resistance factors as a function of deterioration and redundancy.

The CSA standard on antenna towers (CSA 1990*b*) recently introduced a reduction in load factor of up to 20% for evaluation of existing towers. The reduction depends on life safety and the consequences of service disruption.

Proposed format for evaluation

The 1990 NBC/CSA limit states format provides an excellent basis for evaluation criteria. The basic criterion is

[1*a*] factored resistance ≥ effect of factored loads

where

[1*b*] factored loads $= \alpha_D D + \psi\gamma[\alpha_L L + \alpha_Q Q + \alpha_T T]$

where α refers to the load factors for dead load (D), occupancy or snow load (L), wind or earthquake load (Q), and deformation forces (T); ψ is a load combination factor, and γ an importance factor which takes into account the consequences of failure as related to the use and occupancy of the building. The factored resistance is a function of $\phi_i f_i$ and member geometry, where ϕ_i is the resistance factor for each material resistance, f_i.

The NBC specifies loads and load factors. The CSA standards specify resistances and resistance factors and acceptable methods of analysis. The NBC load factors are 1.25 or 0.85 for dead load, 1.5 for variable loads (occupancy, snow, and wind), 1.0 for earthquake, and 1.25 for deformation forces. The NBC importance factor, γ, is equal to 1.0 for most buildings, 0.8 for buildings of low human occupancy, and greater than 1.0 for post-disaster facilities (1.3 and 1.5 applied to the earthquake load, approximately 1.25 applied to the wind load). Except for cladding, where the 10-year wind is applied instead of the 30-year wind, the NBC adjusts structural criteria for the consequences of failure only according to use and occupancy classification.

Of special concern are the NBC/CSA earthquake criteria, because they cause the greatest difficulties for the evaluation and upgrading of existing buildings. The 1990 NBC specifies structural earthquake loads (Q in [1*b*]) on the basis of a base shear force, V:

$$[2a] \quad V = 0.6V_e/R$$

$$[2b] \quad V_e = vSIFW$$

where V_e is the elastic base shear force, v is the specified ground velocity ratio representing rock motion with a probability of exceedence of 10% in 50 years, S is the dynamic response factor (maximum value 3 for low-period buildings), I is the importance factor (1.0, 1.3, 1.5), F is the soil factor (maximum value 2 for soft deep soil), W is the weight of the building, and R is a force modification factor that depends on ductility (energy dissipation) and on the structural system (redundancy). R ranges from 1 for brittle structures such as unreinforced masonry to 4 for ductile redundant structures such as ductile moment-resisting space frames. The factor 0.6 in [2*a*] is a calibration factor determined to provide the same general level of safety (component sizes) in the 1990 NBC as in the 1985 NBC.

For earthquake, the NBC specifies a criterion for damage control which limits the lateral displacement between storeys (elastic displacement times R) due to the design earthquake to $0.02h$ for most buildings and $0.01h$ for post-disaster buildings, where h is the storey height. Also non-loadbearing components must themselves be anchored to prevent them from falling. The anchorage force, V_p, is obtained from

$$[2c] \quad V_p = vS_pW_p$$

where W_p is the weight of the component and S_p is a coefficient which varies widely ($0.7 < S_p < 15$) and depends on a number of factors, including dynamic amplification, the consequences of failure and relative cost.

The same format, [1] and [2], will be applied to evaluation of existing buildings. For more consistent reliability, however, the load factors for dead, variable, and earthquake loads will be adjusted individually rather than to apply an importance factor, γ, according to [1*b*].

4. Loads and load factors

Loads

In most cases the loads for evaluation will be determined by the NBC procedures. For preliminary evaluations it may be expedient to simplify, conservatively, the NBC loads, especially for earthquake. Experience at the site may show, however, that the building has not been, and is unlikely to be, subjected to the NBC load. This should be verified by a site-specific investigation, taking into account the history of experience, future alterations to the building or site, and control of use and occupancy loads. Guidelines for determining site-specific loads are needed, preferably contained in the commentaries to Part 4 of the NBC.

For earthquake loads, the R (ductility) factor should be determined by inspection of details and by consulting the appropriate CSA standard. If the details of the components are unknown or are found to be seriously deficient compared to current standards, the value of R should be taken equal to 1.

Load factors

As discussed above, there are two basic structural safety concepts for determining load factors:

(1) structural safety, or the probability of structural failure. The probability of failure or its converse, the reliability (reliability = 1 − probability of failure), can be calculated as shown in Appendix 2. The measure of structural safety generally used is the reliability index, β, defined in terms of expected loads and resistances and their uncertainties. The reliability index provides the basis for the NBC/CSA load and resistance factors; it is approximately 3 (safe) for "well-behaved" failures, 3.5 (safer) for sudden failures and 4.5 (safest) for connectors.

(2) life safety, or the probability of death or injury for persons exposed to structural hazards. This probability is equal to the probability of failure times the likelihood of death or injury given failure. An extension to the life safety concept is the concept of hazard reduction; here hazardous structures within an inventory of structures are upgraded so as to obtain the greatest reduction in life risk for the funds available. This concept has been applied to seismic upgrading within a municipality such as Los Angeles.

To comply with the NBC and yet allow the flexibility needed for structural evaluation and upgrading, the concept of life safety is applied in Appendix 3 to determine proposed minimum load factors.

The proposed load factors for structural evaluation are contained in Table 1. To apply Table 1, the evaluator determines for each potential failure an adjustment, Δ, to the NBC/CSA reliability index, β, by evaluating three contributory factors: inspection/performance, system behaviour, and risk category. Inspection/performance takes into account the information provided by the structure, whereas system behaviour and risk category take into account the consequences of the specific failure.

The load factors in Table 1 for earthquake are similar to those proposed elsewhere (NZNSEE 1985, FEMA 1986). See Appendix 3 for further discussion of Table 1 and the notes to Table 1.

Additional protection beyond that recommended in Table 1 may be required for damage control. The evaluator should estimate this based on economic and heritage considerations (repair cost vs. loss expectation). Once it is decided that a

TABLE 1(*a*). Proposed minimum load factors for structural evaluation and upgrading of existing buildings

Reliability index adjustment[a] $\Delta = \Delta_1 + \Delta_2 + \Delta_3$	Load factor: Dead α_D	Variable α_L or α_Q	Earthquake α_Q	Load combination factor[b] ψ
−0.4	1.35	1.70	1.40	0.70
0.0	1.25	1.50	1.00	0.70
0.25	1.20	1.40	0.80	0.70
0.5	1.16	1.30	0.63	0.75
0.75	1.12	1.20	0.50	0.75
1.00	1.08	1.10	0.40	0.80
1.25	1.05	1.05	0.33	0.80

[a]See Table 1(*b*).
[b]See Sect. A3.4 in Appendix 3.

TABLE 1(*b*). Reliability index adjustment contributory factors

Assessment factor	Δ_i
Inspection performance, Δ_1	
No inspection or drawings (a penalty)	−0.4
Inspected for identification/location	0.0
Satisfactory performance[a] or dead load measured[b]	0.25
System behaviour, Δ_2	
Failure leads to collapse, likely to injure people	0.0
In between	0.25[c]
Failure local only or very unlikely to injure people	0.50[c]
Rick category for failure, Δ_3	
Very high	Use NBC
High (n = 100−1000)[d]	0.0
Normal (n = 10−99)[d]	0.25[e]
Low (n = 0−9)[d]	0.50[e]

[a]Applies only to dead and variable load factors, age 50 years or more, without structural deterioration.
[b]Applies only to dead load factor.
[c]Reduce by 0.25 for earthquake loading where $R \geq 3$.
[d]The parameter n is determined as the maximum number of people exposed to failure times the weekly hours of normal occupancy/40 if this ratio is less than 1.0 (NZNSEE 1985).
[e]Reduce by 0.25 for assembly occupancy loads or wood structures.

specific upgrading is required, then the upgrading should generally be designed according to the NBC criteria.

Analysis to determine load effects

For indeterminate structures (except for some floor systems, most building structures are indeterminate), better analytical methods mean reduced uncertainty. The CSA bridge standard (CSA 1990*a*) has taken this into account in the evaluation of load factors, but such a procedure is not justified for building structures because there is a lack of meaningful test data comparing measured with calculated member forces. Instead, it is recommended to follow traditional practice by using simple conservative procedures (statics for determinate structures), and to use less conservative rational procedures to determine a better estimate.

5. Resistances and resistance factors

Resistances

Resistances depend on material properties and dimensions which, for an existing building, can be measured. From such measurements the nominal properties should be determined corresponding to a lower fractile of test results, where the test results are adjusted to correspond to the conditions assumed for the appropriate CSA structural design.

Some components, such as concrete decks, have considerably more resistance than indicated by the design criteria, so alternative resistance criteria may be recommended for their evaluation (OHBDC 1983). Other components or systems are either not addressed in the NBC/CSA standards (systems no longer used for new buildings) or disallowed through minimum dimensions or reinforcing. Examples include reinforced concrete beams with less than the minimum required stirrups (CSA 1990*a*), rivets (CSA 1990*a*), and unreinforced masonry buildings with wood floors and roofs (FEMA 1989). It is expected that the NBC guidelines or CSA standards will either provide or reference alternative criteria for such cases. A project is underway to adopt the *U.S. Handbook on Seismic Evaluation of Existing Buildings* (FEMA 1989) to the NBC/CSA format. This will provide alternative criteria useful for the seismic evaluation of buildings in Canada. Further research, however, is needed to develop criteria for seismic evaluation of thick stone masonry buildings (Allen *et al.* 1989).

There is also the possibility of taking into account the resistance of non-structural components, such as masonry infill, to certain types of loads. Each case should be evaluated taking into account the structural behaviour of interacting components and the likelihood of future alterations to non-structural components.

Resistance factors

Certain components such as bolts and welds are "over-designed" (β = 4.5) in the CSA standards to provide a much-improved structure at little extra cost. This criterion can be overly restrictive for evaluation. The CSA bridge standard (CSA 1990*a*) has therefore introduced a resistance modification factor for bridge components, which reduces the reliability of such components to that necessary to life safety. Some of the resistance modification factors are shown in Table 2.

Deterioration is an important factor affecting the resistance, both at the time of evaluation as well as in the future life of the building. Deterioration not only weakens materials, it decreases cross sections and destroys bond (with loss of structural ductility), and introduces greater uncertainty. Deterioration can be taken into account by a resistance modification factor less than 1.0. Recommendations are given in Table 2 for bridges.

In summary, it is more practical to apply a resistance modification factor (usually 1.0) and to determine resistance as a function of nominal properties based on measurements than to adjust the resistance factors contained in the CSA design standards. Resistance modification factors need to be developed for buildings.

Load tests to determine resistance

Load tests can be used to determine the minimum resistance of a portion of the structure (proof test), usually a floor. From a reliability perspective, the proof test alters the assumed probability curve for resistance (see Fig. A1) by truncating it below the proof load. This increases the reliability index (Fujino and Lind 1977) but not significantly unless the proof load is well above the factored load level. Generally, it is best to follow current load testing criteria (CSA 1984) which require satisfactory performance under test loads that correspond to the factored loads for gradual (bending) failure, and 1.1 times the factored loads for brittle (shear) failure.

TABLE 2. Resistance modification factor for evaluation of bridges

Component or condition	Resistance modification factor
Steel bolts (CSA 1990)	1.5
Steel welds (CSA 1990)	1.3
R/C compression members (CSA 1990)	1.2
R/C shear (less than minimum stirrups) (CSA 1990)	0.84
Deterioration	
AASHTO recommendation (Moses and Verma 1987)	0.8–1.0
OHBDC (1990) draft	$C^{(t-1)}$[a]

[a]In this expression, t is the years between inspections and C is the annual rate of deterioration, assumed to be 0.96 if no data are available.

Sometimes load tests can be used to improve structural analysis by determining member forces more accurately, taking into account structural behaviour that is neglected by conventional analysis. This has been used to advantage, for example, for evaluating truss bridges (Nowak and Tharmabala, 1988).

6. Past performance

Satisfactory performance of an existing building over the years provides useful information not available at the design stage.

In many cases, satisfactory performance eliminates the need to apply code serviceability criteria for structural evaluation. Unacceptable deformation, vibration, or local damage will usually be evident to the users within a period of 10 years from construction. Examples where serviceability checks may be required include change of use (related, for example, to human activities such as aerobics or to the installation of new equipment) or alteration of components affecting stiffness or damping.

Satisfactory performance, in combination with a complete inspection, also indicates whether deformation forces (T) have had an adverse effect within 10 years from construction. This is why the load factor for deformation forces has not been included in Table 1.

Satisfactory performance also provides evidence for safety, provided structural deterioration has not taken place. It is better than a load test in that it tests the whole structure, not just part of it, to the real (site-specific) loads that occur. If the building is old, say 100 years, then this provides evidence of satisfactory safety for dead and variable loads, but not for earthquake unless the building has been subjected to earthquake ground motions equivalent to the design earthquake (average return period 500 years). Relying solely on past performance, however, will generally not provide a level of reliability satisfactory for human safety, even if earthquake and deterioration are not factors. Structural evaluation must also be used to identify and, if necessary, upgrade major deficiencies.

So far, no rational method has been developed for incorporating satisfactory past performance into evaluation criteria. The Institution of Structural Engineers (ISE 1980) recommends that a structure may be considered adequate if it is in "good" condition with future deterioration unlikely, and if the ratio of collapse load to apparent distress load is sufficiently greater than the ratio of future maximum load to past maximum load. This is a good principle, but numerical estimates can presently be based only on judgement. The Ontario bridge code (OHBDC 1983) states that a concrete bridge need not be evaluated if it is inspected regularly and shows no signs of distress, but this statement is based on many load tests of typical concrete bridges. Bayesian reliability methods for incorporating successful experience in evaluation criteria are a promising approach (Hall 1988), but attempts so far have not been successful for collecting sufficient information to quantify successful performance. This requires more research. In the meantime "satisfactory experience" is included in Table 1 as a factor, albeit a small one, for reducing load factors.

7. Summary and recommendations

This paper is an initial step toward the development of guidelines on the application of NBC/CSA structural requirements to the evaluation of existing buildings.

The paper reviews criteria for the structural evaluation and upgrading of existing buildings in Canada. All aspects are considered, including format (limit states), loads and load factors, analysis, resistances and resistance factors, measurements, load testing, and past performance.

Minimum load factors for evaluation and upgrading of existing buildings are proposed in Table 1. They are derived on the basis of a life safety criterion. The proposed load factors for structural evaluation allow more flexibility than the existing NBC load factors, but require more professional judgment. Further discussion is therefore encouraged before the proposed load factors are adopted for practice.

Much still remains to be done. Guidelines are required specifically for seismic evaluation and upgrading of existing buildings. Research is needed to develop resistance modification factors, evaluation criteria for thick stone masonry, and to better take into account the satisfactory past performance of old buildings.

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Appendix 1. List of symbols

- d subscript, refer to design
- e subscript, refer to evaluation
- L likelihood ratio — the likelihood of death or injury if failure takes place
- m resistance margin ($m = r - s$)
- n number of people at risk to failure
- P_f probability of failure
- r resistance
- R force modification (ductility) factor for earthquake
- s effect of loads
- V coefficient of variation
- α load factor
- β reliability index
- Δ reliability index adjustment for evaluation ($\Delta = \beta_d - \beta_e$)
- $\bar{}$ bar denotes the mean value

Appendix 2. Structural reliability theory simplified

The probability of failure of a structural component (in bending, shear, compression, stress, etc.) is equal to the probability that the load effect, s, is greater than the resistance, r. Both load effect and resistance are uncertain and their uncertainty can be determined by tests and other data. Figure A1 shows the probability curves for load effect and resistance and the normal curves (solid) approximating the actual curves (dashed), where the tails overlap in the region of failure, that is, where the load effect is greater than the resistance.

The probability of failure can be accurately determined from the probability curve for the resistance margin, $m = r - s$, shown in Fig. A2, by determining the area of the probability curve for $m = r - s$ less than zero. Based on the assumption that r and s are independent normal random variables, the probability curve for $m = r - s$ is also a normal curve with mean $\bar{m} = \bar{r} - \bar{s}$ and standard deviation $\sigma_m^2 = \sigma_r^2 + \sigma_s^2$ (see, for example, Thoft-Christensen and Baker 1982). The area of the probability curve below $m = 0$ in Fig. A2 is determined (from a table of the normal curve) by the number of standard deviations between zero and the mean $\bar{m}$, i.e.,

$$\text{[A1]} \quad \beta = \frac{\bar{m}}{\sigma_m} = \frac{\bar{r} - \bar{s}}{\sqrt{\sigma_r^2 + \sigma_s^2}}$$

where β is called the reliability index. The greater is β, the greater is the reliability and the smaller is the probability of failure (reliability = 1 − probability of failure).

A better model for structural reliability is to replace r and s by their natural logarithms, $\ln r$ and $\ln s$, and to fit normal curves where the tails overlap in the same way as shown in Fig. A1. In this case, [A1] becomes

$$\text{[A2]} \quad \beta = \frac{\overline{\ln r} - \overline{\ln s}}{\sqrt{\sigma_{\ln r}^2 + \sigma_{\ln s}^2}}$$

This equation can be transformed mathematically into the following more useful form (Thoft-Christensen and Baker 1982):

$$\text{[A3]} \quad \beta = \frac{\ln\left[(\bar{r}/\bar{s})\sqrt{(1 + V_s^2)/(1 + V_r^2)}\right]}{\sqrt{\ln[(1 + V_r^2)(1 + V_s^2)]}}$$

where V refers to the coefficient of variation (equal to the standard deviation divided by the mean).

Most loads are determined by multiplying together a number of parameters, X_i (see, for example, eq. [2*b*] for earthquake loads), each of which may be assumed to be an independent random parameter. If the logarithm of each parameter X_i is represented by a normal curve in the upper tail (see Fig. A1), the probability curve for $\ln s$ is also normal with the coefficient of variation determined from (Thoft-Christensen and Baker 1982):

$$\text{[A4]} \quad 1 + V_s^2 = (1 + V_{X1}^2)(1 + V_{X2}^2)(1 + V_{X3}^2) \quad \ldots$$

Equation [A4] can be used to estimate the load uncertainty, V_s, from the uncertainties, V_i, for each contributing parameter X_i.

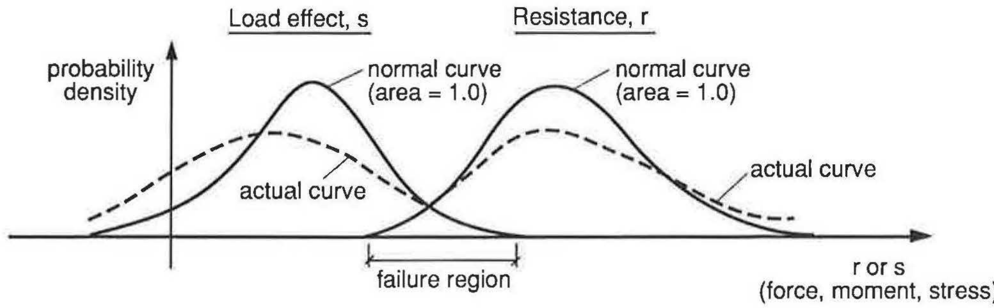

FIG. A1. Probability curves for load effect (s) and resistance (r).

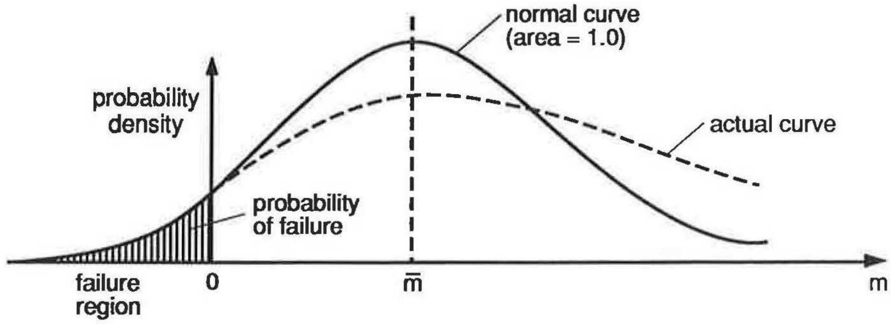

FIG. A2. Probability curve for resistance margin, $m = r - s$.

Appendix 3. Determination of load factors for the evaluation of existing buildings

Existing load factors in the National Building Code are intended for the design of new buildings (or new additions). They are generic in the sense that they apply equally to all structural components, independent of the consequences of a specific component failure. To take into account the consequences of a specific failure plus the information that an existing structure provides, load factors for evaluation and upgrading are developed in the following on the basis of a life safety criterion.

A3.1. Life safety criterion

Risk to life due to structural failure of buildings is presently very low (annual death risk in Canada 2×10^{-7}), most incidences being tornado deaths due to lack of anchorage/reinforcing for key building details. Experience in attempting to use structural reliability theory has shown that a life safety criterion such as 2×10^{-7} cannot be directly used to determine load and resistance factors. Instead, a mixture of past experience and simplified reliability theory will be used, based on the following life safety criterion (CSA 1981):

$$[A5] \quad P_f = \frac{TAK}{L\sqrt{n}}$$

where P_f is the probability of failure determined by reliability theory (Appendix 2), K is a calibration factor based on experience with existing codes, A is a human activity factor which reflects what risk is acceptable in relation to other non-structural hazards associated with the activity (taken as 1 for buildings, 3 for bridges, and 10 for certain work-related activities), $\sqrt{n}$ is a risk aversion factor associated with the number of people, n, exposed to the failure, L is a likelihood ratio (called a warning factor, W, in CSA (1981)) reflecting the likelihood of death or injury if failure takes place ($L = 1$ for no warning or protection), and T is an assumed reference period such as 50 years.

Equation [A5] has been used to determine reliability indices for bridge evaluation (CSA 1990*a*), which, in turn, were used to determine load and resistance factors for structural evaluation. This approach cannot be used for buildings, however, because of the lack of sufficient data on loads and resistances (better data for bridges under traffic loads than for buildings under various loads, especially earthquake). Instead, it will be assumed that the NBC/CSA limit states criteria provide appropriate reliability for design and that for evaluation, only a ratio of the probability of failure for evaluation, P_{fe}, to the probability of failure for design, P_{fd}, will be considered. From [A5],

$$[A6] \quad \frac{P_{fe}}{P_{fd}} = \frac{L_d}{L_e}\sqrt{\frac{n_d}{n_e}}$$

where the subscripts e and d refer to evaluation and design, respectively. In practice, it is difficult to assess the likelihood ratio, L, for specific failures; therefore, categories are needed based on those assessment factors that affect L the most. Table A1 lists the recommended assessment factors, the parameter in [A6] affected, and whether the factor is taken into account in NBC/CSA criteria. Table A2, based on the assumptions given later, recommends the ratios of P_{fe}/P_{fd} for evaluation using the assessment factors recommended in Table A1. Table A2, along with the following reliability assumptions, will be used to determine load factors for the evaluation of existing buildings.

TABLE A1. Assessment factors affecting risk to life

Assessment factor	Parameter in eq. [A6]	Factor taken into account by NBC/CSA
Component behaviour	L	Yes
System behaviour	L	No[a]
Inspection	L	No
No. of people at risk	n	Yes[b]
Protection from collapse	L or n	No

[a]Partly, for earthquake only.
[b]Only on the basis of building use and occupancy (importance factor).

TABLE A2. Life risk parameters for structural evaluation and upgrading

Assessment factor	$P_{\text{fe}}/P_{\text{fd}}$ (eq. [A6])	Δ_i (eq. [A7])
Inspection/performance, Δ_1		
No inspection or drawings (a penalty)	0.33	−0.4
Inspected for identification/location	1	0.0
Satisfactory performance[a] or dead load measured[b]	2.5	0.25
System behaviour, Δ_2		
Failure leads to collapse, likely to impact people	1	0.0
In between	2.5	0.25[c]
Failure local or very unlikely to impact people	6	0.50[c]
Risk category for the failure, Δ_3		
Very high (post-disaster or $n > 1000$)[d]	1.0	0.0
High ($n = 100-1000$)[d]	1.0	0.0
Normal ($n = 10-99$)[d]	2.5	0.25[e]
Low ($n = 0-9$)[d]	6	0.50[e]

[a]Applies only to dead and variable load factors, age 50 years or more, without structural deterioration.
[b]Applies only to dead load factor.
[c]Reduce by 0.25 for earthquake loading if R > 3.
[d]The parameter n is determined as the maximum number of people exposed to failure times the weekly hours of normal occupancy/40 if the ratio is less than 1.0 (NZNSEE 1985).
[e]Reduce by 0.25 for assembly occupancy loads or wood structures.

A3.2. Determination of load factors

Load factors can be determined from [A6] by using the log-normal reliability relationship [A3]. Because β and ln P_f are nearly linearly related, the ratio $P_{\text{fe}}/P_{\text{fd}}$ in [A6] corresponds approximately to the difference $\beta_d - \beta_e$ in [A3], where β_d is the NBC/CSA reliability index for design and β_e is the reliability index for evaluation. This difference will be designated the "reliability index adjustment for evaluation", Δ:

$$\text{[A7]} \quad \Delta = \beta_d - \beta_e$$

To take into account the life risk parameters in Table A2, the reliability index adjustment, Δ, is determined from

$$\text{[A8]} \quad \Delta = \sum_{i=1}^{3} \Delta_i$$

where Δ_i is the adjustment for each of the assessment factors defined in Table A2. The corresponding load factors for each value of Δ are obtained from

$$\text{[A9]} \quad \alpha_{ie} = \alpha_{id} \exp\left[-\Delta\sqrt{\ln[(1 + V_r^2)(1 + V_s^2)]}\right]$$

where α_{id} is the NBC load factor and α_{ie} is the load factor for evaluation. Equation [A9] is obtained from [A3] by assuming that the uncertainties in load effect (V_s) and resistance (V_r) are the same for evaluation and design. A correction is described later when this assumption is not valid.

The values of life risk parameters in Table A2 are based on the following assumptions. For the parameter "system behaviour," the maximum reliability index reduction, $\Delta_2 = 0.5$, is the same as that assumed for the CSA bridge standard (CSA 1990*a*). Assuming $\beta_d = 3.5$, $\Delta_2 = 0.5$ corresponds to a ratio, $P_{\text{fe}}/P_{\text{fd}}$, of approximately 6. From [A6], this corresponds to a likelihood ratio, L_e, of approximately 1/6, compared to the generic design assumption, $L_d = 1.0$. For the parameter "risk category," a maximum reliability index reduction, $\Delta_3 = 0.5$, corresponds to a ratio $n_d/n_e = 6^2 = 36$ in [A6]; for Table A2 it is assumed more conservatively that $\Delta_3 = 0.5$ corresponds to a 100-fold decrease in n_e from the generic design assumption n_d = 100 to 1000. For the parameter "inspection/performance", the maximum reliability index reduction, $\Delta_1 = 0.25$, is half of that assumed for the CSA bridge standard (CSA 1990*a*). The reason for this is that systematic periodic bridge inspections which provide warning of failure are not carried out for buildings. Also, a penalty, $\Delta_1 = -0.4$, is applied if key details are not inspected.

Some restrictions to the reductions are recommended in the footnotes to Table A2 and Table 1. Footnote c in Tables A2

TABLE A3. Uncertainty assumptions for estimating load factors

Source	Uncertainty (V_s or V_r)
Load	
Dead	0.1
Variable (occupancy, snow, wind)	0.3
Earthquake	1.1
Resistance	
Steel	0.1–0.15
Concrete	0.15–0.2
Masonry	0.2–0.3
Wood	0.3

and 1(*b*) accounts for some redundancy already taken into account by the R factor for earthquake. Footnote *e* takes into account the interdependence between the number of people and the live load for assembly occupancies and assumes that the design criteria for wood structures is calibrated to the normal risk category.

A3.3. Uncertainty assumptions

Uncertainty assumptions (V_s and V_r) will be used in [A9] primarily to reduce the NBC load factors. Too high an assumed value of uncertainty is unconservative, therefore care must be taken in its estimation. There are two types of uncertainty to consider: inherent uncertainty which cannot be reduced by better information (e.g., climatic variations) and uncertainty which can be reduced by better information (uncertainties due to scarcity of data, extrapolation, modelling, and systematic deviations such as rate and size effects). Only inherent uncertainties should be included for the application of [A9].

Table A3 recommends uncertainty assumptions for load effects (maximum load in 30 years) and resistances based on available information. The assumption $V_s = 0.3$ applies reasonably well to variable loads (occupancy, wind, and snow). For snow load, $V_s = 0.3$ takes into account variations in maximum ground snow loads (climatic data) as well as variations in other climatic phenomena (wind drifting, temperature-melt runoff) which affect roof snow loads.

The uncertainty assumption for earthquake is more difficult to obtain. A study of over 1000 years of data in Japan (Kanda 1990) indicates an average V_s value (over a period of 50 years) for Japan of 0.7, which includes an uncertainty in peak ground acceleration of 0.5, with an uncertainty in other parameters indicated in [2*b*] of approximately 0.45. Approximately the same assumption applies to other areas of frequent seismic activity. Earthquakes in Canada are rarer, hence the uncertainty in peak rock acceleration is greater. From the data available from the Geological Survey of Canadda, it is estimated that the uncertainty in peak rock motion during a 50-year period is approximately 1.2. Assuming a modelling uncertainty of 0.45, application of [A4] results in a V_s of 1.4. The base shear formula for earthquake is empirical and, although useful for design, it can lead to misleading results if used for a reliability estimation. For example, earthquake damage does not appear to be linearly related to maximum ground acceleration. Therefore, a more conservative (smaller) value for V_s of 1.1 is assumed for inherent uncertainty of earthquake loading.

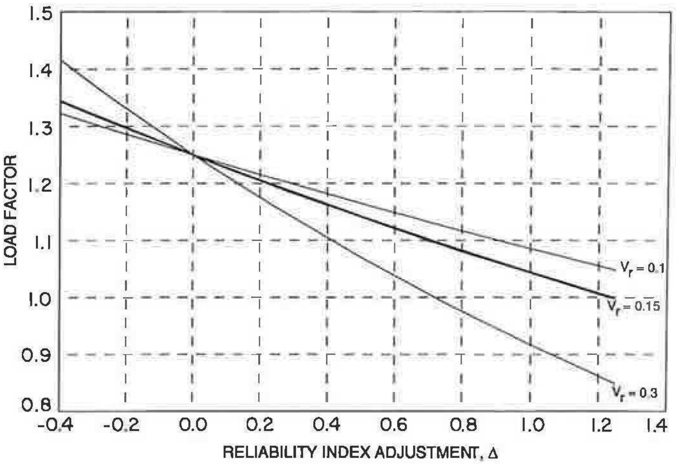

FIG. A3. Dead load factor ($V_s = 0.1$).

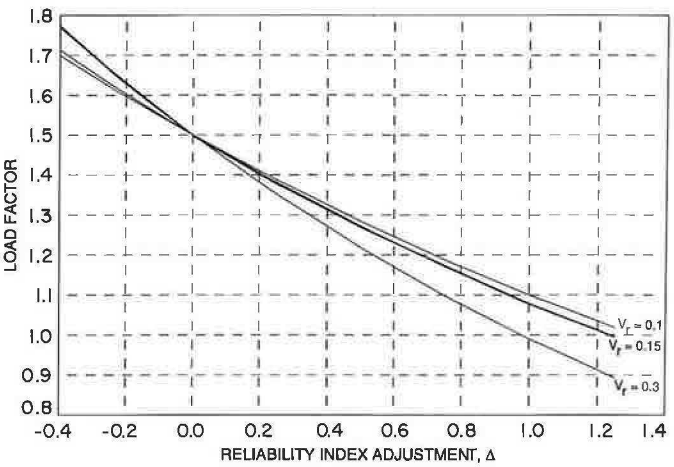

FIG. A4. Variable load factor ($V_s = 0.3$).

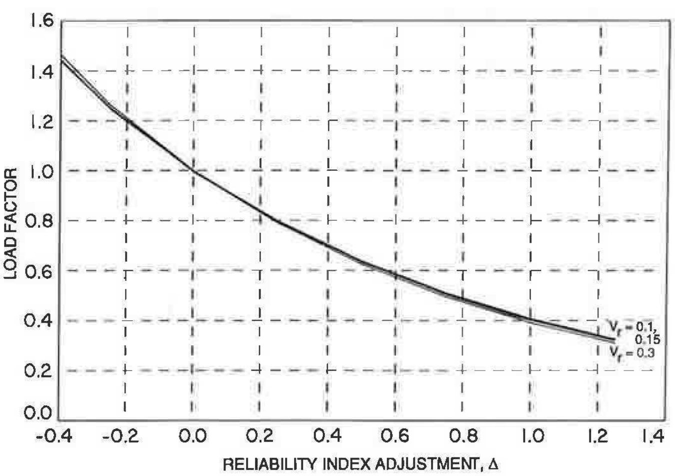

FIG. A5. Earthquake load factor ($V_s = 1.1$).

A3.4. Load factors

Figures A3–A5 show plots of load factors determined by [A9] as a function of the reliability index adjustment, Δ, where Δ is determined from [A8] and Table A2. These plots show

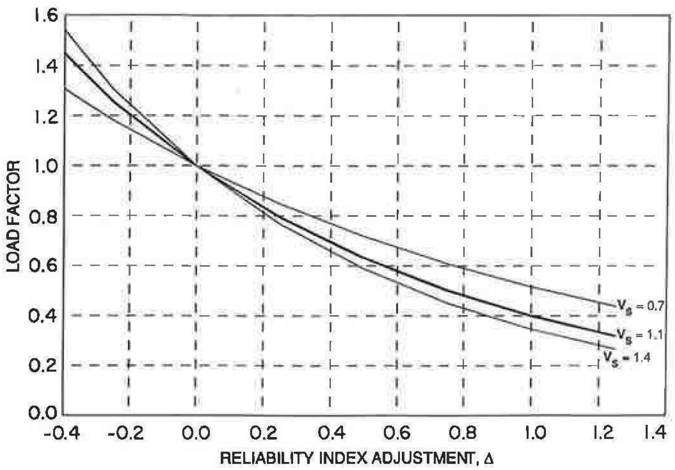

FIG. A6. Earthquake load factor (V_s varies, V_r = 0.2).

that the influence of V_r on load factor is greatest for dead load, less for variable loads, and negligible for earthquake. On the basis of Figs. A3 and A4, conservative values of the dead load factor and the variable load factor for occupancy, snow, and wind are recommended in Table 1. Figure A6 shows the effect of V_s assumptions on the earthquake load factor. The assumption V_s = 0.7 gives a range of load factors from approximately 0.45 to 1.0, similar to those recommended for areas of frequent earthquakes (NZNSEE 1985). Earthquake load factors for Canada based on the assumption V_s = 1.1 are presented in Table 1.

TABLE A4. Reduction in dead load factor due to a decreased dead load uncertainty (V_s = 0.05)

Resistance uncertainty V_r	β		
	2	2.5	3
0.1	0.946	0.932	0.919
0.2	0.969	0.961	0.952
0.3	0.979	0.973	0.967

As the load factors decrease, in Table 1, the probability of simultaneous occurrence of factored loads increases more rapidly than the probability of the factored loads individually. This means that the load combination factor, 0.70 for design, must be adjusted upwards as is indicated in Table 1.

The NBC design load factor can also be decreased if there is a decrease in its uncertainty due to better information. The uncertainty of dead load, for example, can be reduced by measurements. A reduction in V_s from 0.1 to 0.05, where 0.05 is a minimum value for structural analysis, results, through application of [A3], in a reduction in the dead load factor in Table A4 of between 0.92 and 0.98. A reduction in the dead load factor of 0.96 corresponds to a Δ of 0.25 in Table 1, and is therefore introduced as part of Δ_1 in Table A2, although such a decrease does not actually alter β. It would be difficult to justify a similar reduction in uncertainty for most other loads.