A Comparison of General Design and Load Requirements in Building Codes in Canada, Mexico, and the United States

BRUCE R. ELLINGWOOD

ABSTRACT

Safety and serviceability requirements for limit states design (LSD) have been implemented in codes of structural engineering practice in North America during the past 25 years. The load and resistance criteria in LSD take into account inherent variability in strengths and loads and uncertainties due to approximations and lack of information. The framework of structural reliability theory facilitates the treatment and analysis of uncertainties, and yields criteria that are consistent with a performance objective expressed in terms of reliability. Despite the consistency provided by this conceptual framework, the implementation of LSD in structural engineering practice in Canada, Mexico, and the United States has taken somewhat different routes. With recent political developments in North America, the time is opportune to initiate work to remove these differences and to achieve consistency, both in concept and application, of LSD structural codes.

INTRODUCTION

Safety and serviceability provisions in codes and standards used in the design of new buildings and other structures take the form:

$$\text{Design strength } (R_d) > \text{Required strength } (U_d) \quad (1)$$

$$\text{Deflection limit } > \text{Deflection from service loads} \quad (2)$$

The design strength is expressed as factored resistance, while the required strength represents the load effects due to factored loads. Such performance checks are aimed at ensuring that life safety is maintained under extreme loads and occupant comfort and function are maintained under service loads. Until relatively recently, performance checks for steel and other construction materials were accomplished in the context of working or allowable stress design, in which an elastically computed structural response is compared to a maximum allowable stress. During the past 25 years, however, the advantages of limit states design (LSD, or LRFD, as it is termed in the United States) over allowable stress design methods have become apparent (Ellingwood, 1994) and practically all recent advances in the development of standards and specifications have occurred within the context of limit states design.

The design criteria in LSD take inherent variability in strengths and loads and uncertainties due to approximations and lack of information into account. The framework of structural reliability theory, which has advanced as a discipline concurrent to the move toward LSD, facilitates the treatment and analysis of uncertainties, and yields criteria that are consistent with a performance objective expressed in terms of reliability. Many structural loads now are modeled probabilistically, and loads found in design standards are based on specified probabilities.

The knowledge base with regard to scientific theories and data needed to support the development of LSD requirements for steel structures currently is stable. Despite this common knowledge base, the implementation of LSD in structural engineering practice in Canada, Mexico, and the United States has taken somewhat different routes. The recent Eurocode activity in the European Common Market, being coordinated by the Comite Europeen de Normalization CEN/TC 250, Structural Eurocodes, has taken yet another approach. Loads used in design are based on different probabilities of being exceeded; load combination formats are different; and design strengths are specified differently. These differences can create a barrier for structural designers wishing to market their services within the North American economic community or overseas. One of the theses of this paper is that standards for limit states design in North America look more different than they really are. With recent political developments, the time is opportune for initiating work to remove such barriers and to achieve consistency, both in concept and application, of LSD structural codes.

This paper identifies a number of issues that must be addressed and resolved to develop common LSD requirements for steel buildings and other structures in North America. It is customary to organize LSD design standards into a "General Design Requirements and

Bruce R. Ellingwood is a professor in the department of civil engineering, The Johns Hopkins University, Baltimore, MD.

Loads" document coupled with material-specific provisions. The general design provisions are provided by such documents as the National Building Code of Canada (CCBFC, 1995), the Mexico D.F. Code (RCDF, 1996), ASCE Standard 7-95 (ASCE, 1995) and Eurocode No. 1 (CEN, 1994). Such provisions are the focus of this paper. Material-specific provisions are found in the Canadian LSD Specification CAN/CSA S16.1-94 (CSA, 1994), the AISC LRFD Specification (AISC, 1993), and similar documents. A companion paper (Galambos, 1998) deals with strength issues for structural steel design.

STRUCTURAL RELIABILITY—A REVIEW

Uncertainties in structural design arise from variations in loads and material strength properties, dimensions, equations that determine structural actions and resistances, natural and man-made hazards, insufficient knowledge, and human error in design and construction. Modeling these uncertainties probabilistically, the limit state probability becomes the quantitative measure of structural performance; this probability, P_f is expressed as,

$$P_f = \int \cdots \int \mathrm{f}_x(x_1, \ldots x_n)\, \mathrm{d}x_1, \ldots, \mathrm{d}x_\mathrm{n} \tag{3}$$

in which $\mathrm{f}_x() =$ joint density function of resistance and load variables $X = (\mathrm{X}_1, \mathrm{X}_2, \ldots, \mathrm{X}_\mathrm{n})$ and the domain of integration is that region of x corresponding to failure. First-order (FO) reliability analysis provides a tool for performing this numerical integration in approximation (Shinozuka, 1983; Hohenbichler and Rackwitz, 1987; Bjerager, 1990). With a transformation of X to independent standard normal coordinates, U, the limit state can be expressed as $\mathrm{g}(u) = 0$, and a reliability index, β, is defined from the constrained optimization problem:

$$\beta = \min u^{\mathrm{T}} u \tag{4}$$

such that

$$\mathrm{g}(u) = 0 \tag{5}$$

Provided that the limit state is well-behaved, the approximate limit state probability, $P_f = \Phi(-\beta)$, in which $\Phi() =$ standard normal probability integral, often is an excellent approximation for the true limit state probability in Equation 3. If the random variables are lognormal, Equations 4 and 5 yield the expression for β in the commentary to LRFD Section A5.3 (AISC, 1993, p. 6-170).

Statistical data on the mean, standard deviation or coefficient of variation (COV), and probability distribution of each resistance and load variable used in the reliability analysis must be representative of values that would be expected in situ and reflect uncertainties due to inherent variability, modeling and prediction, and measurement. Such data have been described in detail elsewhere (Galambos et al., 1982; Ellingwood et al., 1982; Allen, 1975; Vrouwenvelder et al., 1996), and a portion of these data relevant to steel construction are summarized in Tables 1 and 2. It is interesting to note that the probability models for loads and strengths collected in research programs in North America and Western Europe agree reasonably well when evaluated on a consistent basis (i.e., annual extreme load; 5-percentile yield strength, etc).

It has been customary in developing the first-generation probability-based codes to use β rather than P_f as the quantitative measure of structural performance (e.g., Ellingwood et al., 1982). The load and resistance factors are selected using a code optimization scheme so that the reliability index for any structural member designed by the code is approximately equal to the target value determined by assessment of existing acceptable design practice, or calibration. The reliability analysis of existing standards and specifications in North America and Western Europe indicates that values of β for many members in flexure and/or compression tend to fall in the range of 2.5 to 3.0 (on a 50-year basis) or 3.2–4.0 (on an annual basis), the higher values being observed for gravity load combinations and the lower values for combinations involving wind load. The consistency in overall reliability (and hence on standards of safe performance from a social

Table 1
Statistical Data on Structural Loads

	Annual Extreme			50-Year Maximum		
Load	**Mean/nom[a]**	**COV**	**CDF**	**Mean/nom[a]**	**COV**	**CDF**
Dead	1.05	0.10	Normal	1.05	0.10	Normal
Live	0.42	0.45	Type I	1.00	0.25	Type I
Snow	0.20	0.87	Lognormal	0.85	0.30	Type I
Wind	0.30	0.60	Type I	0.80[b]	0.35	Type I

[a]All mean values are normalized by loads in ASCE 7-95
[b]Wind directionality included

viewpoint), despite the differences in loads, strengths and partial safety factors, is encouraging from the standpoint of developing uniform international design requirements.

LOADS AND GENERAL DESIGN REQUIREMENTS

We begin with a review of the loads and general design provisions referenced by steel design standards in Canada, Mexico, the United States, and Europe. All material standards in Canada refer to the load requirements in Part 4 of the National Building Code of Canada (CCBFC, 1995). The Code issued by the Mexico City Federal District (RCDF-96) is widely used in the rest of Mexico. In the US, the ASCE 7-95 load requirements are the basis for required strength in the LRFD Specification for Steel, the new ASCE LRFD Standard for Engineered Wood Construction, and the new Masonry LSD Standard. The ASCE 7 load requirements also appear in an Appendix to ACI 318-95, along with recalibrated resistance factors. The general design provisions referenced by all Eurocodes are found in Eurocode 1 (CEN, 1994).

The basic provisions in these standards share features in common. All contain general requirements for safety, serviceability, and structural integrity. Specific load combination requirements are provided for strength limit states involving failure by excessive deformation, formation of a mechanism, rupture, loss of equilibrium or stability. The load requirements are based on the same philosophy but are different in their specifics. It often is not appreciated that the loads, load factors and load combinations, and resistance criteria in any code or standard are coupled from a reliability point of view. In other words, if the code performance objective is expressed in terms of a target reliability index, there are many different alternatives for specifying loads, load factors, strengths and resistance factors to achieve the same objective. Harmonization of design codes requires agreement among the stakeholders as to how the goal of essentially uniform reliability can be best achieved for structures designed by the codes.

The NBCC and ASCE 7 load factors and load combinations are tuned to the manner in which the loads are specified. In some cases, such as in the case of certain basic occupancy live loads, the loads have remained unchanged since the beginning of the 20th century, and their values certainly predate any load surveys or rational modeling of loads using probabilistic methods. In other cases, the loads are statistically based. Issues regarding how to specify the individual load must be resolved before common load factors and load combinations can be developed. The following sections illustrate the scope of the problem by providing a selective comparison of some of these differences.

A word on terminology—in the United States and Canada, the unfactored loads appearing in codes and standards are denoted "nominal" loads, "specified" loads, or simply "loads." In the Eurocodes, they are referred to as "characteristic" loads. The intended load intensity in all cases is approximately the same (often corresponding to that value of the annual extreme load with probability 0.02 of being exceeded), and the terms "nominal" and "characteristic" will be used interchangeably in the sequel, depending on the context.

Structural Loads

Live Load

Live load specifications generally contain a set of tabulated basic uniform and concentrated live loads, which depend on occupancy, and a live load reduction (LLR) procedure that permits the basic uniform load to be reduced for larger areas supported in some occupancies. The LLR accounts for area-averaging effects, and can be justified by probabilistic load modeling, but its use in codes

Table 2
Statistical Data on Strength

Component	Mean/nom[a]	COV	CDF
Tension member, yield	1.05	0.11	Lognormal
Compact beam, flexure	1.07	0.13	Lognormal
Continuous beam	1.11	0.13	Lognormal
Plate girder, flexure	1.08	0.12	Lognormal
Column (λ = 0.9)	1.08	0.15	Lognormal
A325 bolts shear	1.50	0.10	Lognormal
E70 Fillet weld, shear	1.47	0.18	Lognormal

[a]All mean values are normalized by strengths in AISC LRFD

predates modern probabilistic load models that were introduced in the early 1970's. The basic live loads are similar in the NBCC, ASCE 7, RCDF, and Eurocode 1, as the comparison of uniform loads specified for common occupancies presented in Table 3 shows. However, the live load reductions permitted are quite different in the three standards. The LLR in ASCE 7 and in the RCDF are based on the concept of an influence area. This concept facilitates specifying the load at the same probability level at all areas, and allows LLR for both beams and columns to be treated by the same procedure. The other standards reviewed have yet to adopt this concept. The comparisons of LLRs in Figure 1 show the variation in the multiplier on basic uniform live load as a function of tributary area, A_t, for a flexural member supporting one floor. None of the standards permit a LLR at small areas, typically less than about 20 m^2 (215 ft^2). For larger areas, the LLR varies among the codes, both in magnitude and in where it is permitted. For example, Figure 1 shows that the NBCC LLR is more conservative than that in ASCE 7 at small areas, but is less conservative at larger areas, since ASCE 7 does not permit more than a 50% reduction for members supporting one floor. The NBCC also permits a LLR to be taken in some heavier assembly or commercial occupancies, mainly those in which the basic live load is 4.8 kPa (100 psf) or greater, whereas ASCE 7 does not. The Eurocode LLR is quite similar to the NBCC and ASCE 7 LLRs for flexural members supporting between 20 m^2 and 100 m^2 (215 ft^2 and 1076 ft^2); however, the Eurocode limits the reduction to 40% for schools, public assembly occupancies, and commercial occupancies. (Interestingly, the Eurocode LLR for columns is dependent on number of floors supported but not supported area, placing it at variance with the LLR in ASCE 7, RCDF, and the NBCC.) The RCDF permits much less live load reduction than the other three standards reviewed. The RCDF live loads reportedly correspond to the 0.9–0.999 fractiles of the 50-yr maximum live load (Ruiz and Sorjano, 1997); in contrast, the live loads in ASCE 7, the NBCC, and Eurocode correspond approximately to the median or mean of the 50-yr maximum live load.

As an example, we consider the live load intensity used to design beams in a floor system in a building housing general and clerical offices with structural bays measuring 9.8 m (32 ft) square. The basic live load is 2.4 kPa (50 psf) in both ASCE 7 and the NBCC and 250 kgf/m^2 (2.45 kPa) in the RCDF. Using the LLR in ASCE 7, the nominal (reduced) live load is 1.4 kPa (29 psf). Using the LLR in the NBCC, the nominal live load is 1.5 kPa (31 psf). Using the LLR in the RCDF, the nominal live load would be 2.2 kPa (46 psf). Using Eurocode 1, the live load would be 1.82 kPa (38 psf).

Snow Load

The snow load on a roof is calculated as the product of a ground snow load (determined from a map) and a ground-to-roof conversion factor in both the NBCC and in ASCE 7. ASCE 7 specifies a ground snow load with a 50-year mean recurrence interval (MRI) based on a lognormal distribution. Loads in the US have been specified this way since 1972. The basic ground-to-roof conversion factor is 0.7. The ground snow load in the NBCC has been specified at a MRI of 30 years based on a Type I distribution of largest values for at least two decades, and the basic ground-to-roof conversion factor is 0.8. A rain-on-snow surcharge of 0.3 kPa is added to the roof snow load. For an ordinary building with a flat roof at International Falls, MN/Fort Frances, ON (a border location, where loads from both standards should be comparable), ASCE 7-95 would require the roof to be designed for a snow load of 1.7 kPa (35 psf), while the NBCC would require the roof to be designed for 2.0 kPa (41 psf). At Detroit, MI/Windsor, ON, the flat-roof snow load is 0.96 kPa (20 psf) by both standards. The RCDF has no snow load requirements, but has an allowance for concentration of hail in roof valleys (RCDF, 1996).

Table 3
Basic Uniform Live Loads in Modern Codes, kPa (psf)

Occupancy	NBCC	RCDF	ASCE 7	Eurocode 1
Residential	1.9(40)	1.7(35)	1.9(40)	2.0(42)
Offices	2.4(50)	2.5(51)	2.4(50)	3.0(63)
Classrooms	2.4(50)	3.4(72)	1.9(40)	3.0(63)
Assembly*	4.8(100)	4.5(94)	4.8(100)	5.0(104)
Commercial	4.8(100)	3.5(73)	3.6(75)	5.0(104)
Storage	4.8(100)	3.5(73)**	6.0(125)	6.0(125)

*without fixed seating
**minimum value

Wind Load

Load standards determine wind pressure as the product of stagnation pressure (dependent on the square of windspeed), gust response factor, pressure coefficient, and exposure factor. The wind loads in ASCE 7-95 are calculated from a 50-yr MRI 3-second gust windspeed, which is provided in a map. The NBCC provisions are based on a 30-yr MRI mean hourly windspeed pressure, presented in a table. Essential facilities are designed for a 100-yr MRI rather than the basic 50-yr MRI pressure in both. Both standards model the windspeed with a Type 1 distribution of largest values. The mean hourly windspeed averages about 66% of the 3-sec gust for the same MRI. Moreover, a 30-yr MRI windspeed averages about 95% of a 50-yr MRI wind speed. Thus, the 30-yr mean hourly windspeed as specified in the NBCC would be on the order of 63% of the 50-yr 3-sec gust in ASCE 7, and the stagnation pressures would differ by the square of the windspeed. On the other hand, the gust factors in the NBCC are much higher than those in ASCE 7. For a building in International Falls, MN/Fort Frances, ON, of height 60 m (197 ft), situated in a suburban exposure, the total wind pressure on windward and leeward surfaces at a point 30 m (100 ft) above grade, used in the design of a main wind force-resisting frame, would be 1.11 kPa (23 psf) using ASCE 7-95 and 0.91 kPa (19 psf) using the NBCC. The wind pressure in Eurocode 1 is based on a 50-yr MRI 10-minute average windspeed; for these same conditions, the characteristic pressure would be 1.53 kPa (32 psf). In Mexico D.F., ordinary buildings are designed using a 3-sec gust windspeed with a MRI of 50 years; essential facilities require a 200-yr MRI windspeed.

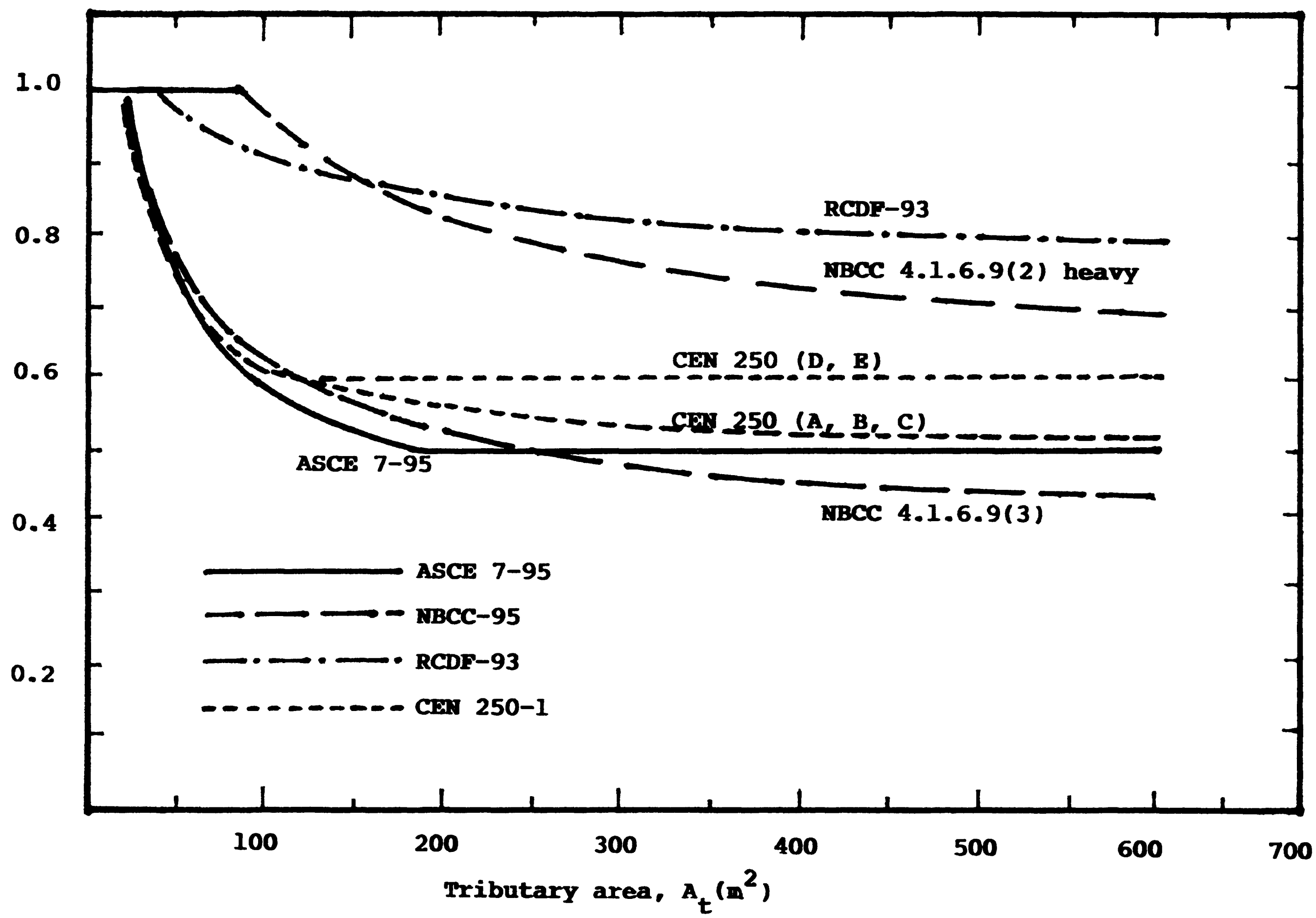

Fig. 1. Comparison of live load reduction vs. tributary area for structural members supporting one floor.

Figure 2 compares the variation of nominal (characteristic) wind drag pressure as a function of elevation above grade for a simple building (rectangular plan, no setbacks over height) on a suburban site characterized by a 3-sec windspeed of 90 mph (40 m/s). Comparing on the basis of design wind pressure allows differences in windspeed averaging time, mean recurrence interval, exposure coefficient, and gust response factor to be considered together. The wind pressures in ASCE 7, NBCC, and Eurocode 1 differ significantly, as will be shown subsequently.

Earthquake Load

In ASCE 7, the structural action due to earthquake is based on a ground motion with an estimated probability of 0.1 in 50 years (FEMA, 1994) or 0.0021/yr (equivalent to a 475-yr MRI). For a building situated on rock in Vancouver, BC, designed with an ordinary moment-resisting frame and a fundamental period of vibration of 1 sec, the required base shear according to ASCE 7 would be $V = 0.053W$, in which W = gravity weight of the building. Using the provisions in the NBCC, which also are based on a 475-yr MRI, $V = 0.06\ W$. A comparable building in Mexico D.F. would be designed for a base shear of 0.08 W, corresponding to a spectral acceleration with a 100-yr MRI.

Load Combinations

All structural loads, with the exception of dead load, vary in time. This variation in time must be taken into account properly in developing load combinations for design purposes, and the failure of ASD to do it rationally is one of its notable drawbacks. Load combinations specified for design in all modern structural codes utilize what is referred to as a "principal action–companion action" load combination scheme. This scheme arises from the observation, confirmed by theoretical load combination analysis and Monte Carlo simulation, that the maximum combined structural action during some suitable period of reference nearly always occurs when (only) one of the time-varying loads (termed the "principal action") reaches its maximum value while the concurrent time-varying loads are at near-average values. The probability that two (or more) time-varying loads reach their maxima simultaneously is negligible unless the loads are highly correlated. Accordingly, accounting for this small probability by multiplying the sum of all structural actions, including dead load, by a common multiplier (such as the traditional value 0.75) is inconsistent with the way that load combinations occur. However, since one generally does not know which principal action will govern, one needs to consider

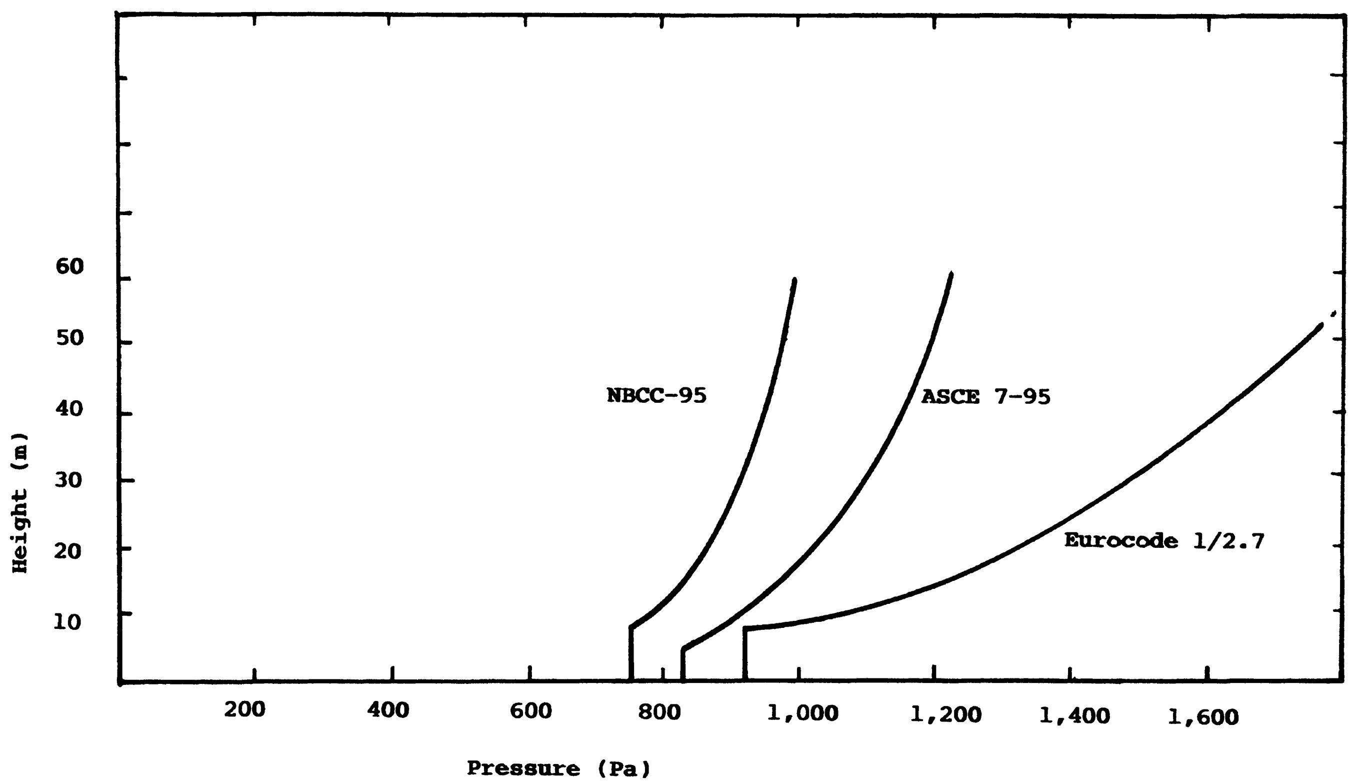

Fig. 2. Design wind pressures in suburban exposure (windward plus leeward wall surfaces) for main wind force-resisting systems.

several load combinations, taking each time-varying load to be the principal action, in turn, until the maximum combined structural action is obtained. As an illustration, for combinations involving dead, live, and wind load, the designer would consider the maximum of the following:

$$D + \max L + W \tag{6a}$$

$$D + L + \max W \tag{6b}$$

in which max L and max W are the principal actions, and W and L are the companion actions, or values assumed to act concurrent to the principal actions. It usually is obvious to an experienced designer which of a set of such load combinations is likely to govern in a particular situation.

In all formats described below, the principal action can be identified as the load with the load factor greater than 1.0. The companion actions often have load factors less than 1.0, since the average loads usually are less than the nominal (or characteristic) loads, which have a small probability of being exceeded. For simplicity, only dead, live, wind, snow, and earthquake loads are considered.

Strength Limit States

Structural analysis using the following load combinations defines the required strength in Equation 1. Although the combinations are presented as a sum of load effects, it should be realized that when the structural behavior is nonlinear, the structural actions may not sum in a linear fashion.

United States. Design load requirements were developed for all construction materials using probabilistic methods and a code optimization procedure (Ellingwood et al., 1982). A partial list of these load requirements, which now appear in the ASCE 7 Standard on Minimum Design loads (ASCE, 1995), is:

$$1.2D + 1.6L + 0.5S \tag{7a}$$

$$1.2D + 1.6S + (0.5L \text{ or } 0.8W) \tag{7b}$$

$$(0.9 \text{ or } 1.2)D + 1.3W + 0.5L \tag{7c}$$

$$(0.9 \text{ or } 1.2)D + 1.0E + 0.5L + 0.2S \tag{7d}$$

in which D, L, S, W, and E = nominal actions due to dead, live, snow, and wind loads and earthquake ground motions. The factor 0.9 on dead load applies when the dead load has a stabilizing effect on the structure. The load factor on E is set equal to 1.0 because it is based on an MRI of 475 years rather than 50 years.

Canada. The load combinations can be expressed in a more concise form than in the U.S. requirements because of the decision to apply the same load factor to all principal loads acting in the combination.

$$\gamma[(0.85 \text{ or } 1.25)D + \Psi(1.5L + 1.5W)] \tag{8a}$$

in which W is wind load, γ = importance factor [0.8 for storage buildings with low human occupancy; 1.0 for other buildings]. Load combination factor Ψ equals to 1.0 if either L or W is acting, and 0.7 if both L and W are acting. Earthquake effects are treated through the combination,

$$1.0D + 1.0E + 0.5L \tag{8b}$$

Here, the dead load factor is not reduced to 0.85 when dead and lateral forces counteract one another.

Mexico. The load combinations in the RCDF are:

$$1.4(D + L) \tag{9a}$$

$$1.1(D + E + L_{red}) \tag{9b}$$

$$1.1(D + W + L_{red}) \tag{9c}$$

For important buildings, the load factor in Equation 9a is increased to 1.5. In contrast to the load combinations in the ASCE 7 and NBCC, one overall load factor is applied in each combination. However, a two-level specification of live load is employed, which is akin to the "arbitrary point-in-time" live load that appeared in some early LRFD proposals back in the 1970's. For example, for general and clerical offices, L_{red} = 180 kgf/m^2 (1.8 kPa), or 72% of the basic live load; no LLR ever is applied to L_{red}.

Eurocode 1. The load combination scheme has features in common with both the US and Canadian approaches. For ordinary ultimate limit states, the required strength is determined from:

$$(0.9 \text{ or } 1.35)D + 1.5(Q_1 + \sum \Psi_{0i} Q_i) \tag{10a}$$

in which Q = characteristic load and Ψ_{0i} = companion action factor defining the "combination" value of Q_i. Eurocode 1 provides a table of ψ_{0i}-values. For example, for combinations of dead, live, and wind load, the engineer would check:

$$1.35D + 1.5(L + 0.6W) \tag{10b}$$

$$1.35D + 1.5(W + 0.7L) \tag{10c}$$

$$1.50W - 0.9D \tag{10d}$$

The Eurocode 1 treats earthquake effects as a "special" limit state, as described in the following section.

Conditional or Accidental Limit States

Load effects arising from rare accidental events, such as fires, explosions of natural gas, detonation of bombs, or vehicular impact, may initiate a catastrophic partial or total collapse of a structure. Here, the primary concern is life safety rather than damage prevention, and the design philosophy is to create a robust structural system in which damage is contained rather than prevented entirely. As a consequence, while the conditional limit states are ultimate or strength-related, in the general sense described above, sources of load-carrying capacity not ordinarily permitted might be mobilized in the design analysis.

United States. ASCE 7/ANSI A58 first introduced a requirement for general structural integrity against unforeseen events in 1972, shortly after the 1968 Ronan Point collapse. Section 1.4 of ASCE 7-95 contains a performance requirement that a building be designed to sustain local damage, with the structural system as a whole remaining stable. Its Commentary Section C1.4 discusses general design approaches to general structural integrity. Section 2.5 requires, in some instances, a check of strength and stability of structural systems. Its Commentary Section C2.5 recommends the following load combination for checking the ability of a damaged structure to maintain its overall stability for a short time following an accidental load event:

$$(0.9 \text{ or } 1.2)D + 0.5L + 0.2W \tag{11a}$$

This check requires the notional removal of load-bearing elements. If certain key elements in the structure must be designed to withstand the effects of the accident (perhaps to allow the development of alternate load paths), they should be designed using the combination,

$$(0.9 \text{ or } 1.2)D + A_k + (0.5L \text{ or } 0.2S \text{ or } 0.2W) \tag{11b}$$

in which A_k is the postulated action due to the abnormal load. Normally, only the main load-bearing structure would need to be checked using these equations.

Building code officials in the US are not enthusiastic about provisions related to general structural integrity because they are difficult to cast in prescriptive code language. Most building codes in the United States do not contain such provisions.

Canada. Section 4.1.1.3(1) of the NBCC requires structures to be designed for sufficient structural integrity to withstand all effects that may reasonably be expected to occur during the service life. Commentary C on Part 4 defines structural integrity as "the ability of the structure to absorb local failure without widespread collapse." Designers are advised to consider and take measures against severe accidents with probabilities of occurrence of approximately 10^{-4}/yr or more. Several general approaches—local resistance, minimum tie forces, provision of alternate paths of support—are suggested and a list of references is provided. The general structural integrity provisions date back to the 1970 edition of the NBCC, issued not long after the Ronan Point collapse. Through the 1977 edition, the commentary guidelines were quite detailed; however, in the 1980 and later editions, the guidelines were stated in a more general way, and specific prescriptive measures currently are not presented.

Eurocode 1. In contrast to North American standards, Eurocode 1 treats earthquake-resistant design in this category rather than as an ordinary load combination. Structural effects of fires also are included. The design combination is,

$$D + A_k + \Psi_1 Q_1 + \sum \Psi_{2i} Q_i \tag{12}$$

in which Ψ_1 and Ψ_{2i} are companion action factors for "frequent" and "quasi-permanent" values of load, presented in a table in Eurocode 1 along with Ψ_0.

Serviceability Limit States

Serviceability limit states are becoming increasingly important as the use of higher strength materials and better integrated structural systems leads to flexible and lightly damped structures (Ad Hoc Committee on Serviceability, 1986). These limit states relate to expectations on the part of the occupants regarding general quality of building performance in service rather than life safety. Building occupants object to cracks, deflections, and movement in a building that are far less than would represent structural damage, let alone failure. Guidelines that are not material-dependent should be covered in a document dealing with general design and load requirements. While the motion-related limit states require special consideration of the dynamic properties and response of the building (Ad Hoc Committee on Serviceability, 1986), the deformation-related limit states can be checked using load combinations similar to those above.

United States. Section 1.3.2 of ASCE 7-95 is a mandatory performance requirement for sufficient stiffness to limit deflections, lateral drift, vibrations, or other deformations that might adversely impact structural performance. This provision first appeared in 1982. Appendix B, which is nonmandatory, contains further guidelines and specific information, including load combinations for deformation-related limit states. For serviceability checks involving short-duration static effects, that value of the total load effect that has a 5-percent probability of being exceeded in any given year is,

$$D + L \tag{13a}$$

$$D + 0.5L + 0.7W \tag{13b}$$

For long-term static effects, such as creep, the corresponding load combination is,

$$D + 0.5L \tag{13c}$$

Some general guidelines for addressing objectionable floor or building vibrations are provided as well. Appendix B represents the first attempt in a U.S. standard to implement quantitative load requirements for serviceability limit states.

There appears to be a belief in the US that serviceability issues are best relegated to nonmandatory guidelines rather than to codes and standards. The AISC LRFD Specification (AISC, 1993) has no specific serviceability criteria or guidelines, but makes reference to the technical literature; sentiment in the profession seems to be against quantitative serviceability requirements.

Canada. The NBCC contains mandatory provisions for deflection (4.1.1.5) and vibration (4.1.1.6) that date back to 1970. There is one load combination in section 4.1.3.3 (2) for checking deflections:

$$D + \Psi\,(L + W + T) \tag{14}$$

with load combination factor Ψ defined as in Equation 8a. Wind pressures are based on a 10-yr MRI windspeed. There is a requirement in clause 4.1.10.6 (1) to investigate by dynamic analysis floor structures supporting assembly occupancies if the fundamental frequency is less than 6 Hz. Commentary A of the National Building Code of Canada (1995) contains guidelines for excessive structural deflections and floor vibrations. CAN/CSA-S16.1-94 also has three Appendices—G, H, and I—with specific serviceability limits. These CSA and NBCC guidelines are consistent and are far more comprehensive than what is found in U.S. codes and standards. Appendix G—Guide for Floor Vibrations - was first introduced in 1974; it currently indicates thresholds of annoyance, distinguishes between steady-state and transient vibrations, and provides some guidance on dynamic modeling, damping, and corrective action for troublesome floors. Engineers also are warned to avoid coincidence of excitation frequencies (or harmonics) with the fundamental floor frequency. Appendix H—Wind Sway Vibrations—notes the annoyance problem, and refers the designer to Commentary B, Part 4 of the NBCC for calculating alongwind, acrosswind, and torsional accelerations. Appendix I—Recommended Maximum Values for Deflections—recommends specific vertical and lateral deflection limits for specified design loads; these range from 1/240 span for live loads on roofs to 1/800 span for crane runway girders.

Mexico. The RCDF contains general indications on service limit states, and provides limits on vertical and lateral displacements.

Eurocode 1. The Eurocode considers three different categories of serviceability limit state: rare, frequent, and quasi-permanent. For "rare" combinations (connection slip, permanent inelastic deformation, cracking),

$$D + Q_1 + \sum \Psi_{0i} Q_i \tag{15a}$$

For "frequent" combinations (excessive temporary elastic deformations),

$$D + \Psi_{1i} Q_1 + \sum \Psi_{2i} Q_i \tag{15b}$$

For "quasi-permanent" combinations (creep, differential settlement),

$$D + \sum \Psi_{2i} Q_i \tag{15c}$$

The companion action factors Ψ_{0i}, Ψ_{1i}, and Ψ_{2i} in these equations, which all are less than 1.0, are provided in a table in Eurocode 1. Limit states related to dynamic response are not addressed explicitly.

DESIGN STRENGTH

In CSA S16.1-94, the RCDF-96, and the AISC LRFD specification, the design strength in Equation 1 is represented by $R_d = \phi R_n$, in which R_n = nominal (or code-specified) strength for the limit state of interest computed using specified nominal strengths, F_{ni}, and dimensions and ϕ = a resistance or performance factor. The resistance factor depends on the variability in strength and nature of the limit state. In AISC LRFD, one ϕ is specified for each limit state. This practice is generally followed in CSA S16.1 as well; however, for composite construction, different ϕ's are assigned to steel, concrete and shear connector strengths. In contrast, in Eurocodes 2–5 for different construction materials, R_d is determined by first dividing the individual nominal material strengths, F_n, by partial material factors greater than unity, i.e., $F_{di} = F_{ni}/\gamma_{mi}$, and then calculating R_d directly using these F_{di}. Both code formats are philosophically similar and lead to approximately the same structural designs for similar materials and load envelopes. For design of many steel structures, there is no compelling advantage to either format. However, for composite construction, applying a separate material factor to steel yield strength and concrete compressive strength results in more uniform reliability when both materials contribute to strength (Ellingwood, 1982). Typical values of γ_{mi} would be 1.1–1.15 on F_y and 1.5 on f'_c.

There are a number of differences in particular strength provisions for steel structures. Included here would be plate slenderness ratios; the use of single vs multiple column curves; treatment of second-order effects in frames and K-factors; and treatment of lateral-torsional buckling of beams. A companion paper (Galambos, 1998) addresses those issues.

DESIGN COMPARISONS

To show how the design requirements and reliabilities compare for structural components designed using the four sets of specifications reviewed above, we consider three design cases: (1) a simply supported beam supporting an interior floor in a general office building with 9.8 m (32 ft) square bays; (2) a similar beam supporting a flat roof;

and (3) wind bracing in a vertical truss. In all cases, the load requirements define the required strength; for simplicity, the required member is selected from the AISC LRFD Manual in each case to illustrate the differences in required section and weight. It is understood in the following illustrations that D, L, and W correspond to structural actions due to dead, live, and wind load. The structural analysis is elastic, which is permitted unconditionally by all standards reviewed; failure corresponds to formation of the first "hinge" in the structural system. The resistance factor for steel in flexure or yielding in tension is 0.9 in CSA S16.1-94, AISC LRFD, and RCDF-96; for Eurocode 1, the partial material factor on yield is 1.1.

Subsequent to obtaining a specific beam or brace design, a first-order reliability analysis is performed using the procedure described previously. To provide a common basis for comparison, the reliability index is obtained for a 50-year service life using the load and resistance statistics summarized in Tables 1 and 2. To the extent that fundamental differences in loads within North America may be reflected similarly in both nominal and mean loads (e.g., there is some evidence that live loads in Mexico D.F. tend to be higher than in Canada and the US), the differences in β would diminish.

Simply supported beam. The dead load for the floor is 3.6 kPa (75 psf). The live loads are the appropriate values identified in each standard in the previous discussion of live loads in office buildings; thus, the values L in the following design equations are not the same numerically. The beam is assumed to have full lateral support, and any permitted live load reduction is taken. The following design equations, structural members, and reliability indices are obtained:

Standard	Criteria	Section	β
CSA S16.1-94/NBCC	$0.9F_yZ_x = 1.25D + 1.5L$	W24×84	2.9
RCDF-96	$0.9F_yZ_x = 1.4D + 1.4L$	W27×94	4.1
AISC/ASCE 7	$0.9F_yZ_x = 1.2D + 1.6L$	W24×84	2.9
Eurocode 1/3	$(F_y/1.1)Z_x = 1.35D + 1.5L$	W27×84	3.4

The beam designed by the RCDF-96 is heavier because very little live load reduction is permitted. The beam designed by the Eurocode is somewhat heavier because the basic live load is higher.

One obtains very similar results when considering beams in roofs. We consider a simply supported beam spanning 7.6 m (25 ft) that is part of a flat roof over a heated building with a normal exposure in the International Falls, MN/Ft. Frances, ON region. Two cases are considered: a normal dead load of 1.9 kPa (40 psf) and a light dead load of 0.72 kPa (15 psf). The design snow loads were presented earlier. The design criteria are the same as in the live load example above, replacing L with S. The following designs and reliabilities were obtained:

Standard	Dead Load	Section	β
CSA S16.1-94/NBCC	1.90 kPa	W18×35	3.1
	0.72 kPa	W14×30	2.9
AISC LRFD/ASCE 7	1.90 kPa	W18×35	3.1
	0.72 kPa	W16×26	2.6

As with the live load, the CSA/NBCC and AISC/ASCE 7 designs are comparable, with small differences that arise from the different snow loads (1.7 vs 2.0 kPa).

Wind bracing member. In the truss considered, the structural actions due to gravity loads in the diagonal bracing were found to be negligible, and the tensile and compressive forces in the brace are entirely due to wind. The diagonal bracing is assumed to be provided by structural WT sections. The following design equations, structural members, and reliability indices were obtained:

Standard	Criteria	Section	β
CSA S16.1-94/NBCC	$0.9F_yA_g = 1.5W$	WT5×15	2.3
AISC/ASCE 7	$0.9F_yA_g = 1.3W$	WT5×15	2.3
Eurocode 1/3	$(F_y/1.1)A_g = 1.5W$	WT4×24	4.4

The required section and reliability are much larger for the brace designed by Eurocodes 1 and 3 than for any of the North American standards. Part of the reason is the higher characteristic wind load (Figure 2); e.g., the characteristic wind force in Eurocode 1 is about 1.5 times that in ASCE 7. Coupled with the higher load factor of 1.5, the required area of the tension member is higher by more than 60 percent. The variation in wind load reliability among the standards is much higher than the variation in gravity loads. Clearly, the issue of wind load reliability requires further investigation in all standards examined in this study.

The use of conservative loads and small load factors or small loads and high load factors is a matter of custom and preference. One can obtain essentially the same design (and thus the same reliability) in any one of several ways. However, there must be agreement among the stakeholders on how the loads are to be specified before common load combinations and load factors can be devised. Structural engineers attempting to market their services and to develop software for routine design find such differences an irritation and a potential source for design error.

CURRENT ISSUES

The above review has indicated that it should be possible to develop a common set of load requirements for the design of steel (and other) structures. The following issues may be barriers to implementation and acceptance and should be addressed by international committees.

Loads and Load Combinations

The brief comparison of structural load requirements indicates that the engineer ends up with approximately the same required strength in many cases. A common set of load requirements might be developed and implemented that would adopt the best features of each format. The difficulties will lie more in the specification of the loads than in their combinations, since all specifications examined treat the load combination issue in a similar way.

It would be desirable to specify a common return period for wind and snow loads. Eurocode 1 and ISO have recommended that occupancy and environmental loads be based on the 50-yr MRI. It seems reasonable to adopt this recommendation, as it will provide a consistent basis for the treatment and analysis of environmental load data worldwide. A common averaging time for windspeed also must be selected, since the gust response factor and pressure coefficient are keyed to this averaging time. ASCE 7 abandoned the "fastest mile" specification for 1995 because of changes in meteorological instrumentation. It would be desirable in the design load analysis to utilize the meteorological information essentially as provided by the weather service. Local weather information occasionally must be used to develop design loads, particularly in regions with severe local terrain features, and most structural designers do not have the expertise to perform the sort of data reduction that underlies the load provisions in the NBCC or ASCE 7.

It should be recognized that earthquake-resistant design is based on a different philosophy than design for dead, live, snow, and wind loads. The concepts of "load" or "strength" have little meaning in this context. Rather, the current safety checks are simply part of a design process aimed at providing a structural system with the ability to dissipate energy through cyclic inelastic deformations.

Reliability Measures

The benchmark reliability measures on which LSD is based have been determined through a probability-based calibration of the new load and resistance requirements to acceptable (frequently archaic ASD) practice in the US. While essential uniformity in reliability has been advanced as one goal of LSD, there may be social, economic, political, and technical reasons, why this might not be desirable in all cases. These reasons stem from the notions of risk perception and tolerance on the part of the profession, building owner, and the public (Ellingwood, 1994). Public expectations are that buildings and other structures will perform without jeopardizing occupant or user safety. The engineering profession expects that abiding by minimum code requirements will protect the building owner, occupant, and the engineer of record. Since most structures designed by recent codes have indeed fulfilled these societal and professional expectations, calibration to acceptable performance usually is a reasonable strategy for setting reliability benchmarks, especially since quantifiable structural failure data are rare. However, expectations may differ from country to country, depending on sociopolitical factors. Occasionally, these expectations are not met, particularly during and following a natural disaster resulting in extreme loads and structural challenges. At such a time, public officials subsequently may increase minimum structural safety requirements for certain classes of buildings to a level well above that otherwise justified by either cost-benefit analysis of building performance or calibration to historical design procedures. Such increases may be necessary to restore the confidence of the public in the regulatory process governing the built environment.

As an example, there is evidence that design for wind load in the US should be more conservative than currently recognized in ASCE 7. The current factor of 1.3 on wind in ASCE 7 arose from the calibrations performed nearly 20 years ago on simple structural members, such as compact laterally supported beams and tension members in trusses, that were designed by ASD. Those calibrations indicated that existing criteria for gravity loads were not consistent with those for wind load, and that reliabilities apparently were less for designs governed by wind load than for those governed by gravity load. This difficulty could not be resolved at that time, and rather than to allow the entire movement toward LSD to founder on this one issue, the inconsistencies were allowed to remain. The intervening years and additional data have indicated that code performance levels for wind indeed may not be equivalent to those for gravity loads for certain types of buildings. Anecdotal evidence reveals that many structural engineers design structural systems for wind load at higher levels of conservatism than required by the code. Moreover, the trend in modern steel building construction is toward the use of higher strength steels, leading to lighter and more flexible building systems that are more susceptible to wind effects. The time has come to resolve this issue.

Cross-Referencing

All design standards and specifications cross-reference documents prepared by other organizations. In the LRFD Specification, ASTM, AWS, AISI, and ASCE are referenced extensively. In the CSA S16-94 Standard on LSD, documents produced by CSA, ASTM/ANSI, CISC, and CGSB are referenced. Some of the documents cross-referenced are consistent; others are not, and a common set of references would have to be developed early in any unification process. The cross-referencing also caused difficulty in the European community when the Eurocodes were being developed.

Organization

CSA S16.1, RCDF-96, and the LRFD Specification all begin with general design requirements, followed by technical provisions, and end up with fabrication, erection, and quality assurance criteria; a number of specialized appendices and a commentary also are provided. In between, however, they are organized differently. LRFD is organized along the lines of structural actions (tension, compression, etc.). CSA S16.1-94 has this information in one section on Member and Connection Resistance, but organizes other sections along the type of structural element: Beams and Girders, Open-web Steel Joists (which LRFD does not cover), and so on. Agreement on organization would have to be reached early in the process.

Fire-Resistant Structural Design

Fire is one of the primary causes of loss of life and property in buildings. In the US and other countries, acceptable performance during a fire has been determined for many years on the basis of a standard fire test rather than by structural analysis (e.g., ASTM, 1990). In such a test, structural and nonstructural components are qualified or assigned a "fire rating," in hours, based on their performance during a test with a standard fire exposure. While this design-by-test procedure generally results in conservative fire ratings, recent research has shown that the requirements may be excessively conservative in some cases and unrealistic in many others (Ellingwood and Corotis, 1991). The ASTM E119 test requirements appear to have been developed without rational consideration of structural performance, do not provide for modern structural analysis capabilities, and may place an undue burden on certain types of steel construction. Recent research has made possible improvements in fire-resistant design. Load surveys have provided data on typical fire and live loads for various building occupancies. Advances in computational structural analysis now make it possible to determine structural response to realistic fire exposure as part of the design process without difficulty. General design requirements for buildings should take full advantage of these advances.

Evaluation of Existing Buildings

Any move toward common LSD requirements in North America would provide a splendid opportunity to develop requirements for evaluating safety of existing steel buildings and other structures. A significant amount of structural engineering practice involves existing buildings scheduled for rehabilitation or retrofit as a result of modernization, changes in occupancy, or damage from extreme events. The economic impact of this area of structural engineering practice is substantial, especially in urban areas where costs of new or replacement construction are high. The concerns—safety, serviceability, and durability—are the same as for a new structure. However, the relative cost of making required changes to existing buildings to meet updated safety requirements may be very large (Allen, 1991). Currently this is a significant issue in seismic rehabilitation (FEMA, 1997).

Current standards merely touch upon this issue, and do not address factors that distinguish existing from new construction. There are a few guidelines for evaluating existing structures (e.g., ACI, 1982). None has a reliability basis. None of the steel standards reviewed in this study contain quantitative safety and/or serviceability checks of existing steel buildings. Numerous questions must be addressed to evaluate such criteria. For example, what action is taken if the code loads have gone up and the building cannot meet the new code? What are the legal implications of using assessment criteria for older buildings that are different in terms of risk from those used to design new buildings? The entire calibration process requires rethinking for existing construction.

Although at first glance serviceability may not seem to be an issue for existing construction, appropriate criteria present an issue for existing construction, particularly for old or historical buildings constructed with archaic materials. Structural components in old buildings may be weakened due to weathering or other deterioration, foundation movement, or previous extreme loads. Such behavior would reduce strength and stiffness, but may increase damping. Accordingly, serviceability checking procedures for such structures may be different from what would be suitable for new construction. Indeed, deflection or vibration limits might be only a fraction of what they would be for new construction. Additional research is required to identify and implement acceptable procedures.

Content and Scope

The review of the Eurocode requirements has identified some desirable features that might be considered in the long term in developing common design requirements for steel structures in North America.

The Eurocodes include criteria for safety, serviceability, and durability. Issues related to durability in North American building practice are left up to the building owner, architect, and engineer. However durability issues can affect both safety and serviceability limit states. Most durability-related research is conducted by material scientists who often have only a vague idea of the structural engineering implications of corrosion and other common modes of deterioration. Moreover, most durability problems in service are caused by improper selection of materials or design detailing, both of which fall in the purview of structural engineering. Any move toward common LSD requirements in North America should come to grips with the role of durability in structural standards.

Design requirements for fire are being developed as part of the Eurocode activity for each material. US structural design standards do not have mandatory structural design requirements for fire, relying instead on ASTM E119 to provide the required fire performance.

Safety of structures under construction is included in Eurocodes. North American standards generally contain requirements only for completed building structures, leaving safety during construction to the general contractor.

The scope of the Eurocodes generally is broader than that of North American standards and specifications used in structural design. This will facilitate widespread adoption. Constraints have been placed on the standard development process in the U.S. by the influence of building codes and code officials, and the desire to maintain simple prescriptive and easily enforceable provisions. While such provisions are more easily interpreted, they also stifle creativity and innovation, particularly when the structural systems or materials are novel.

RECOMMENDED LOAD COMBINATIONS

A common technical database on structural loads already exists, and accordingly, it is feasible to develop a common set of loading and general design requirements that would meet the needs of structural engineers in North America. The following set of load combinations are recommended for the ultimate limit states involving dead, live, wind, snow, and earthquake loads:

$$1.2D + 1.6L \tag{15a}$$

$$1.2D + 1.6S + 0.7L \tag{15b}$$

$$(0.9 \text{ or } 1.2)D + W + 0.7S + 0.7L \tag{15c}$$

$$(0.9 \text{ or } 1.2)D + E + 0.7L + 0.2S \tag{15d}$$

The load factors for W and E are set equal to 1.0, based on the notion that one specifies a low-probability (say, 500-yr MRI) wind or seismic event as a design basis, and details to provide adequate strength and ductility, as appropriate, to withstand that event. It can be shown that mapping the design-basis environmental event rather than a 50 to 100 yr MRI event achieves more uniformity in reliability over wide geographic areas. The alternative of a two-level specification for wind or earthquake—one at a moderate level at which damage should not occur and a second at an extreme level at which life safety must be preserved—is worthy of further consideration as well.

CONCLUSIONS

The move toward limit states design has taken different paths in Canada, Mexico, and the United States during the past twenty years. As a result, existing standards and specifications appear more different than they really are, which may be a barrier to their use by engineers. The time is right to strive for some uniformity in design requirements for steel structures in North America. The necessary knowledge base exists. The technical barriers do not appear to be insurmountable. Basic organizational issues will have to be agreed upon at the beginning, including formatting, cross-referencing of other specifications, and standardization of structural loads. Resolving these issues will involve numerous people outside the steel community and is likely to involve national pride as much as technical details. General load combinations would follow quickly. Specific requirements for steel structures could be devised more or less concurrent to the resolution of load issues.

Common load and design requirements for steel structures in North America is a worthy goal for the new millennium. The availability of such design requirements would be of enormous benefit to the steel industry in particular and to structural engineers in general.

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