Evolution of seismic design provisions in the *National building code of Canada*

Denis Mitchell, Patrick Paultre, René Tinawi, Murat Saatcioglu, Robert Tremblay, Kenneth Elwood, John Adams, and Ronald DeVall

Abstract: The purpose of this paper is to provide a summary of the evolution of seismic design in Canada. This paper presents the significant changes to the approach taken in determining seismic hazards and seismic hazard maps, and describes the evolution of the seismic design provisions of the *National building code of Canada*. The introduction of important parameters in determining the seismic base shear such as the period of vibration of the structure, the influence of type of soil, and the concepts of ductility and energy dissipation capacity of elements and structures are presented. The levels of seismic design base shears, determined from different versions of the National Building Code of Canada, are compared for reinforced concrete frame and wall structures to illustrate the changes.

Key words: National building code of Canada, seismic design, base shear, seismicity maps, natural period, foundation factors, torsional effects, structural systems, ductility.

Résumé : L'objectif de cet article est de présenter un sommaire de l'évolution de la conception parasismique au Canada en se référant aux modifications des efforts sismiques prescrits par le Code National du Bâtiment du Canada. Les changements majeurs concernant l'aléa sismique et les cartes sismiques correspondantes qui ont été créées sont présentés. On présente également les paramètres importants, qui influencent le calcul sismique de l'effort tranchant à la base, tels que la période de la structure, l'influence du type de sols, les concepts de ductilité et de dissipation d'énergie. La variabilité du niveau des charges sismiques, pour les différentes versions du Code National du Bâtiment du Canada, est présentée pour les cadres en béton et les refends, afin d'illustrer ces changements.

Mots-clés : Code National de Bâtiment du Canada, conception parasismique, effort tranchant à la base, cartes sismiques, période naturelle, coefficient de fondation, effet de torsion, systèmes structuraux, ductilité.

[Traduit par la Rédaction]

Introduction

This paper forms part of a major effort by the Canadian Seismic Research Network to develop guidelines for seismic evaluation and retrofit of existing buildings. The evolution of Canadian seismic design codes over the last 70 years is presented, together with numerical comparisons performed for sample structures, to provide engineers with a summary of the key changes to aid in understanding the difference in seismic design force levels of older codes compared to the 2010 *National building code of Canada* (NBCC) (NRCC 2010). Although this paper discusses the evolution of the seismic base shear values, additional papers will present the important aspects of design and detailing in the Canadian Standards Association (CSA) standards. Furthermore, the design philosophy has changed from working stress design to ultimate strength design, with load factors and capacity reduction factors, and then to limit states design, with load factors and material resistance factors. To appreciate the aspects of the original design of an existing building, the engineer should consult the appropriate code and standard along with their commentaries. This paper draws from an overview of the Canadian seismic design provisions up to 1977 authored by Uzumeri, Otani, and Collins (Uzumeri et al. 1978).

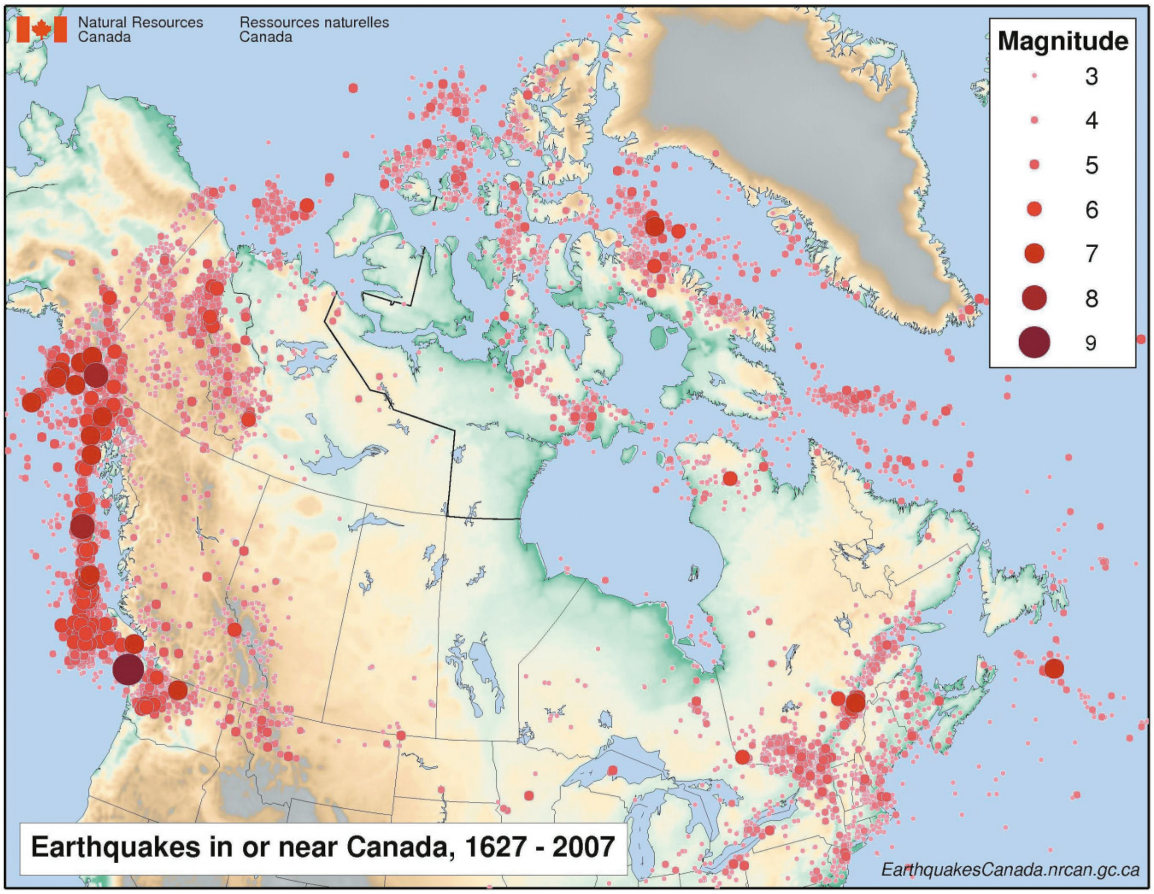
Received 30 September 2009. Revision accepted 4 May 2010. Published on the NRC Research Press Web site at cjce.nrc.ca on 3 August 2010.

D. Mitchell.[1] Department of Civil Engineering, McGill University, 817 Sherbrooke St. West, Montreal, QC H3A 2K6, Canada.
P. Paultre. Department de genie Civil, Université de Sherbrooke, 2000, Avenue Université, Sherbrooke QC J1K 2R1, Canada.
R. Tinawi. Canadian Seismic Research Network, Department of Civil Engineering, McGill University, Montreal QC H3A 2K6, Canada.
M. Saatcioglu. Department of Civil Engineering, University of Ottawa, Ottawa, ON K1N 6N5, Canada.
R. Tremblay. Département de génie civil, École Polytechnique de Montréal, Montréal, QC H3C 3A7, Canada.
K. Elwood. Department of Civil Engineering, University of British Columbia, Vancouver, BC V6T 1Z4, Canada.
J. Adams. Seismologist, Geological Survey of Canada, Natural Resources Canada, Ottawa, ON K1A 0Y3, Canada.
R. DeVall. Senior Consultant, Read Jones Christoffersen Ltd, Vancouver, BC V6H 3X8, Canada.

Written discussion of this article is welcomed and will be received by the Editor until 31 January 2011.

[1]Corresponding author (e-mail: denis.mitchell@mcgill.ca).

Fig. 1. Historical seismicity in Canada (courtesy of Geological Survey of Canada).



Seismic hazard

Figure 1 shows the locations and sizes of earthquakes in Canada from 1627 to 2007 (Adams and Atkinson 2003). There are four main regions of seismic activity: a stable central region with very few earthquakes; an eastern region where about 14% of all earthquakes in Canada have occurred; a western region where about 27% of all earthquakes in Canada have occurred; and a northern region where about 59% of all earthquakes in Canada have occurred. It is noted that there have been a large number of events with magnitudes greater than 6.5.

1941 NBC

In 1941, the first National Building Code (NBC), which contained seismic design provisions in an appendix (NRCC 1941), was published. It was based on the 1935 Uniform Building Code (UBC 1935), where the lateral force, V, located at the center of gravity of the building, is equal to

$$V = CW \quad [1]$$

where C varies between 0.02 and 0.05 depending on the bearing capacity of the soil, and W is the weight of the building.

1953 NBCC

The first seismic zoning map was introduced in the 1953 NBCC (NRCC 1953) and is shown in Fig. 2. This zoning map, developed and described by Hodgson (1956), delineated four zones with relative seismic intensity, based on the locations of large historical earthquakes, with the highest intensity values in the western part of British Columbia and in the St. Lawrence and the Ottawa River valleys. It is noted that this is a qualitative map with no probability level specified and has abrupt changes in zones (e.g., upper Ottawa valley). After the 1940 Imperial Valley earthquake, the 1943 Los Angeles Building Code made the seismic force coefficient, C, in eq. [1] a function of the stiffness of the structure based on the number of storeys, N, (Hawkins and Mitchell 1977). Based on these developments the lateral seismic design force in the 1953 NBCC was given as

$$F_i = C_i W_i \quad [2]$$

where F_i is the applied lateral seismic design force at the ith level, W_i is the total weight (taken as dead load plus 25% of the design snow load) tributary to the ith level, and C_i is the seismic force coefficient for minimum earthquake loads of $0.15/(N+4.5)$ for zone 1 and N is the number of storeys above the ith level. The seismic force coefficient for minimum earthquake loads, C_i is multiplied by 2 for zone 2 and multiplied by 4 for zone 3.

1960 NBCC

The seismic design provisions of the 1960 NBCC (NRCC 1960) were essentially the same as the 1953 NBCC. This

Fig. 2. Seismic zoning map from the 1953 *National building code of Canada* (NRCC 1953).

was the first Canadian code to refer to the need to consider torsional effects; however, no specific guidance was given.

1965 NBCC

The 1965 NBCC (NRCC 1965) used the same seismic zoning map as the 1953 NBCC, shown in Fig. 2. The seismic design provisions of this code departed from the US codes of the day with the introduction of an importance factor; a foundation factor and consideration of torsion. The minimum seismic base shear, V, was given as

$$[3] \qquad V = RCIFSW$$

where R is the seismic regionalization factor with values of 0, 1, 2, and 4 for seismic intensity zones 0, 1, 2, and 3, respectively; C is the type of construction factor with values of 0.75 for moment resisting frames and reinforced concrete shear walls that are adequately reinforced for ductile behaviour, and 1.25 for other types of buildings; I is the importance factor with values of 1.0 and 1.3 (buildings with large assemblies of people, hospitals, and power stations); F is the foundation factor with values of 1.5 for highly compressible soils and 1.0 for other soil conditions; S is the structural flexibility factor of $0.25/(N + 9)$, where N is the number of storeys; W is the total weight (dead load plus 25% snow plus live load for storage areas). The total lateral seismic force was assumed to be linearly distributed proportional to the height and weight of the floors, similar to the 1959 Structural Engineers Association of California (SEAOC) provisions (SEAOC 1959). This edition contained torsional design provisions based on the 1966 Mexican code (DDF 1966; Ward 1966). The code required a torsional eccentricity equal to

$$[4] \qquad e_x = 1.5e \pm 0.05D$$

where e is the distance between the centre of mass and the centre of rigidity, and D is the plan dimension in the direction of the computed eccentricity. The factor 1.5 applied to e accounts for the dynamic amplification of torsional moments resulting in larger design forces on the more "flexible side" of the structure and the term $0.05D$ represents the accidental torsional eccentricity (Bustamante and Rosenblueth 1960). In the 1965 NBCC, if e_x exceeded $D/4$, either dynamic analysis was required or the computed torsional moment was doubled.

It is noted that, in general, working stress design was used, however, ultimate strength design was permitted for concrete structures in 1965, as an alternative method, based on the American Concrete Institute (ACI) Code approach (ACI 1963) with load factors and capacity reduction factors. The 1965 NBCC ultimate load, U, for earthquake design is given by

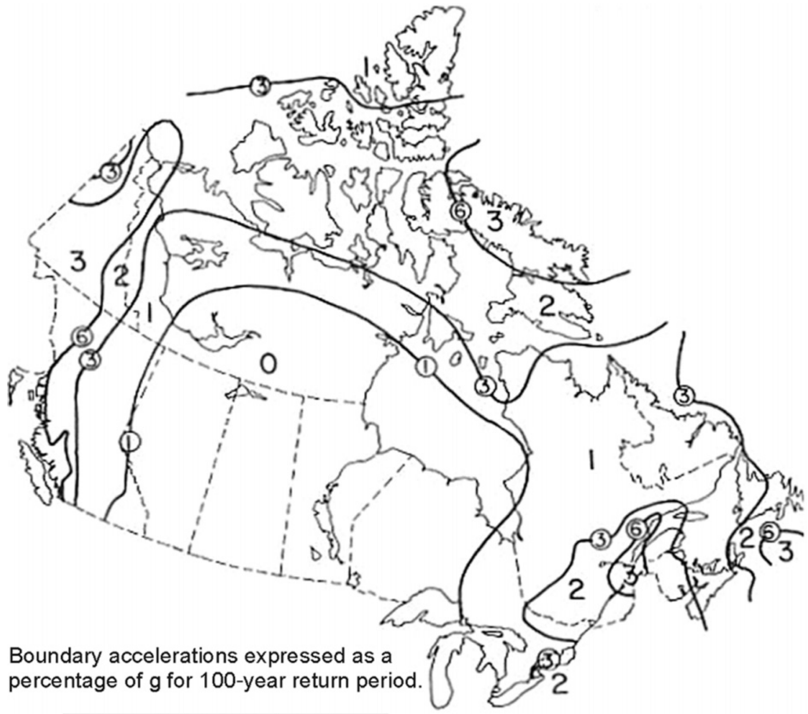
$$[5] \qquad U = 1.35(D + L + E)$$

where D, L, and E are the effects from dead, live, and earthquake loads, respectively.

1970 NBCC

Milne and Davenport (1969) developed the first truly probabilistic seismic zoning map (Fig. 3), using extreme-value statistics that were applied to known Canadian earthquakes. This map was introduced in the 1970 NBCC (NRCC 1970) and was based on expected accelerations, A_{100}, having a probability of exceedance of 0.01 (100-year return period). There were four zones with numbers on the zonal boundary lines indicating accelerations as a percentage

Fig. 3. Seismic zoning map in 1970 NBCC (NRCC 1970).



Zone	3	2	1	0
R	4	2	1	0

of g. These acceleration values are not used directly to determine the seismic lateral force but rather seismic zones were introduced. It is noted that Montreal and Ottawa changed from zone 3 to zone 2 in this edition of the code.

In the 1970 NBCC, the structural flexibility factor, C, depended on the period of vibration of the structure and higher mode effects were accounted for through the application of a portion of the lateral force, V, as a concentrated force, F_t, at the top of the structure and a reduction of the overturning moment. The minimum lateral seismic force (base shear), V, was given as

$$[6] \qquad V = \frac{1}{4}R(KCIFW)$$

where R is the seismic regionalization factor (Fig. 3), K is the type of construction factor (see Table 1), C is the structural flexibility factor, where,

for one and two storey buildings,

$$[7] \qquad C = 0.1$$

for other buildings,

$$[8] \qquad C = 0.05/T^{1/3} \quad < 0.10$$

for moment resisting frames,

$$[9] \qquad T = 0.1N$$

for other cases,

$$[10] \qquad T = 0.05h_n/D^{1/2}$$

where T is the fundamental period of structure; h_n is the height of structure, in feet (1 ft = 0.3048 m); D is the dimension of the building parallel to the seismic force, in feet; N is the number of storeys. This was the first Canadian code where the structural response factor was made a function of the period of the structure.

While the 1970 NBCC referred to ductile moment resisting frames, design and detailing provisions for ductile frame members and ductile flexural walls were not provided until the *Special provisions for seismic design* were introduced in the 1973 CSA A23.3 Standard (CSA 1973). For the alternative, the ultimate strength design approach for concrete structures, the ultimate load, U, for earthquake design was given by

$$[11] \qquad \begin{aligned} U &= 1.15D + 1.35(L + E) \\ U &= 1.5D + 1.8E \\ U &= 0.9D + 1.35E \end{aligned}$$

1975 NBCC

The seismic zoning map developed for the 1970 NBCC was used for the 1975 NBCC (NRCC 1975; Fig. 3). The minimum seismic base shear, V, was given as

$$[12] \qquad V = ASKIFW$$

Table 1. Summary of K factors representing type of construction, damping, ductility, and energy absorption.

Resisting elements	K (1970)	K (1975 to 1985)
Ductile moment-resisting space frame resisting 100% of required force	0.67	0.7
Dual system of ductile moment-resisting space frame and ductile flexural walls (frame must be designed to resist at least 25% of total base shear)	0.8	0.7
Dual system of ductile moment-resisting space frame and shear walls or steel bracing (frame must be designed to resist at least 25% of total base shear and walls or bracing must be designed to resist 100% of base shear)		0.8
Other framing systems not defined above	1.0	
Ductile flexural walls and ductile framing systems not defined above		1.0
Systems without space frames (box systems)	1.33	
Dual system with ductile space frame with masonry infill (infilled wall system must be designed to resist 100% of base shear and frame; without infill, must be designed to resist at least 25% of total base shear)		1.3
Systems not defined above with continuous reinforced concrete, structural steel, or reinforced masonry shear walls		1.3
Other structural systems not defined above	2.0	2.0
Unreinforced masonry		2.0

where A is the horizontal design ground acceleration with values of 0, 0.02, 0.04, and 0.08 for seismic zones 0, 1, 2, and 3, respectively; and S is the seismic response factor defined in eq. [13]. The K factors for different types of construction are given in Table 1. In addition, an intermediate foundation factor F = 1.3 was introduced to account for soft soils or for compact coarse-grained or stiff fine-grained soils with a depth greater than 50 ft (= 15.24 m).

[13] $$S = 0.5/T^{1/3} \leq 1.0$$

The expression for the period, T, was the same as in the 1970 NBCC. As indicated by Uzumeri et al. (1978), the term AS in the 1975 NBCC was calibrated to be 20% less than the term $RC/4$ in the 1970 NBCC.

The torsional design eccentricity, e_x, was given as

[14] $$e_x = 1.5e + 0.05D$$
$$e_x = 0.5e - 0.05D$$

The introduction of the 0.5 factor on e was aimed at increasing the design force levels of the "stiff side" of the structure. If e_x exceeds $D/4$, then dynamic analysis is required; otherwise the computed torsional moment is doubled.

The 1975 NBCC permitted the use of dynamic analysis as an alternative procedure to determine the seismic design forces. However, for irregular structures the Commentary to the code recommended the use of the dynamic analysis procedure. A response spectrum compatible with that proposed by Newmark et al. (1973) with 5% damping was adopted for the dynamic analysis that was scaled to the design ground acceleration, A, equal to 0, 0.02g, 0.04g, and 0.08g for zones 0, 1, 2, and 3, respectively. Furthermore, the response spectrum was divided by $\sqrt{2\mu - 1}$ for shorter periods ("equal energy concept") and by μ for longer periods ("equal displacement" concept), where μ is the structural ductility factor (Blume et al. 1961; Table 2). Commentary K to the 1975 NBCC recommended the use of the square root of the sum of the squares modal combination method for calculating design forces.

Table 2. Structural ductility factor μ for dynamic analysis (Commentary K, 1975 NBCC (NRCC 1975)).

Building type	μ
Ductile moment resisting space frame	4
Combined system of 25% ductile moment resisting space frame and ductile flexural walls	3
Ductile reinforced concrete flexural walls	3
Regular reinforced concrete structures, cross-braced frame structures and reinforced masonry	2
Structures having no ductility, plain masonry	1

It is noted that the reciprocal of the K factor should be approximately equal to the μ factor, where the "equal displacement" concept is applicable, because both factors account for structural ductility.

The relatively low ground accelerations (100-year return period) used in the static base shear equation were not representative of ground motions for dynamic analyses. In addition, the long periods for buildings obtained using computer models of "bare structures" (without considering nonstructural components) together with high values of μ (relative to $1/K$) resulted in dynamic base shear values considerably less than the equivalent static approach.

For concrete structures, the CSA Standard A23.3 (CSA 1973) specified an ultimate strength approach with load factors specified in the standard and load combination factors, as given in the 1970 NBCC. The resulting required factored strength, U, was given as

[15] $$U = 0.75(1.4D + 1.7L + 1.8E)$$
$$U = 1.4D + 1.8E$$
$$U = 0.9D + 1.4E$$

In the 1975 NBCC (NRCC 1975), limit states design was introduced as an alternative design approach to working stress design with load factors and material resistance factors or capacity reduction factors (concrete standard). The factored load combinations including seismic effects were

$$[16] \quad U = 1.25D + 0.7(1.5L + 1.5E)$$
$$U = 1.25D + 1.5E$$
$$U = 0.85D + 1.5E$$

These load combinations were used in the CSA S16.1 standard (CSA 1974) starting in 1974; however, concrete structures were designed using ultimate strength design load factors until the introduction of limit states design in the 1984 CSA A23.3 standard (CSA 1984).

1977 NBCC

In the 1977 NBCC, seismic zoning maps and seismic provisions remained essentially the same as in the 1970 NBCC (NRCC 1977). A key change in the dynamic analysis design procedure was the introduction of a minimum base shear equal to 90% of the base shear determined from the static analysis procedure, to limit the difference between the base shears determined from static and dynamic analyses. It was recognized that this limit for dynamic analysis was necessary because the probability of exceedance of the A_{100} acceleration values provided inadequate protection (i.e., 40% probability of exceedance in 50 years) compared to the probability of exceedance of other structural loads. The static load approach, however, remained the same as in the 1975 NBCC.

1980 NBCC

The 1980 NBCC introduced SI units, with an introduction to SI units given in a supplement to the 1977 NBCC. The 1980 NBCC (NRCC 1980) used the same seismic zoning map as the 1970 NBCC. The minimum design lateral seismic force equation did not change from the 1975 NBCC, except that the seismic response factor, S, was changed to

$$[17] \quad S = 0.5/T^{1/2} \leq 1.0$$

This resulted in a longer plateau, with $S = 1.0$ up to $T = 0.25$ s compared to $T = 0.125$ s in the 1975 and 1977 NBCC (Heidebrecht et al. 1983). This change increased the seismic design forces for the period range from 0.125 to 1.0 s, affecting a large portion of low- and mid-rise buildings, but resulted in smaller earthquake design forces for structures having periods greater than 1.0 s. A procedure was proposed for the determination of the structural eccentricity, e, for each floor level in a structure.

The limit state load factors and combination factors remained the same as in the 1977 NBCC; however, for concrete structures, the ultimate strength procedure from the 1977 CSA A23.3 Standard (CSA 1977) was still used.

1985 NBCC

New seismic zoning maps, based on the point source model developed by Cornell (1968), were introduced in 1985 (NRCC 1985). The seismic zoning map was based on a probability of exceedance of 10% in 50 years or 0.0021 per annum (return period of 475 years), which was judged to be closer to the probability of exceedance of other design loads. These maps provided accelerations and velocities for each zone (see Fig. 4) and the number of seismic zones was increased from four to seven. These maps followed the ATC-3 Guidelines (ATC 1978) and their development is described by Basham et al. (1985). Peak horizontal ground accelerations (units of g) with corresponding values of the peak zonal acceleration ratios, a, were given for each seismic zone, Z_a. Peak horizontal ground velocities (in m/s) with values of the velocity zonal ratios, v, were given for each seismic zone Z_v.

This 1985 code introduced the influence of the acceleration–velocity ratio (a/v), with ground motions with high a/v ratios having high frequency content and high spectral amplification for short period structures (e.g., eastern Canada). On the other hand, low a/v ratios indicate the dominance of long period motion and hence reduced spectral response for short period structures. This change recognized that the spectral shape was not the same as that for California, and indeed varies geographically in response to the number and sizes of local earthquakes and the different characteristics of earthquakes in the east and west.

The seismic base shear, V, was given as

$$[18] \quad V = vSKIFW$$

where v is the velocity zonal ratio and S is the seismic response factor. For periods greater than or equal to 0.5 s, $S = 0.22/T^{1/2}$. For $T \leq 0.25$, $S = 0.62$, 0.44, 0.31 for Z_a/Z_v greater than 1, equal to 1, or less than 1, respectively. Linear interpolation was used for S values between 0.25 and 0.5 s. The term vS can be interpreted as the spectral acceleration. The S value of 0.44 for the case of $Z_a/Z_v = 1$ was chosen to calibrate the base shear values to the previous code.

The period of the structure is determined from eq. [19] with the exception that $T = 0.1N$ for moment-resisting space frames resisting 100% of the lateral forces:

$$[19] \quad T = 0.09h_n/D_s^{1/2}$$

where h_n is the height of the building above the base in metres; D_s is the dimension of the lateral force resisting system in a direction parallel to the applied forces in metres, rather than using the building dimension, D; thus resulting in longer periods and reduced seismic design forces for most structures. The 1985 NBCC also allowed, for the first time, the use of the period obtained from modal analysis, without exceeding 1.2 times the value given by eq. [19], which could result in further reductions in seismic design loads. If dynamic analysis were selected by the designer, the results had to be scaled such that the base shear corresponds to 100% of the static earthquake force, not 90%, as was permitted in the previous editions of NBCC.

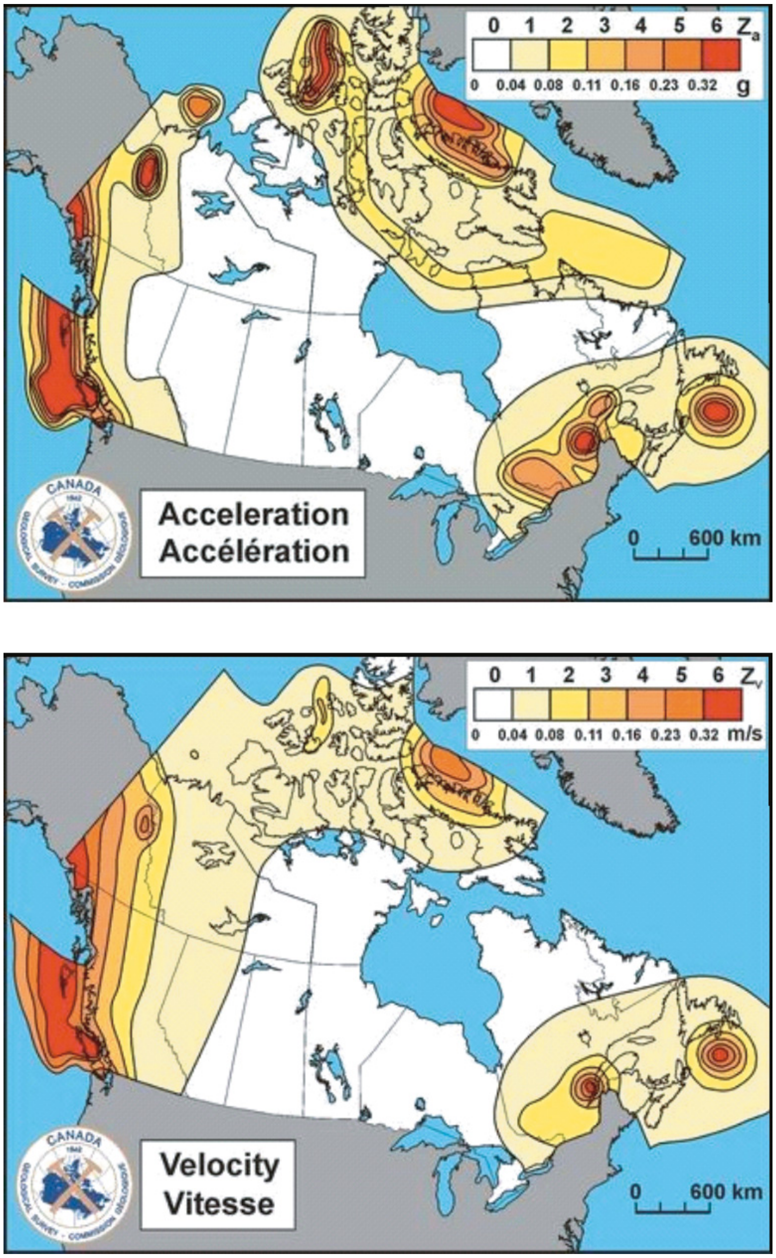
The torsional design eccentricity, e_x, was given as

$$[20] \quad e_x = 1.5e + 0.10D$$
$$e_x = 0.5e - 0.10D$$

The accidental torsional eccentricity was increased from $0.05D$ to $0.10D$; however, doubling of torsional moments was no longer required for cases with large e values. Dynamic analysis was required if the centroids of mass and the centres of stiffness of different floors did not lie on approximately vertical lines.

The load factors and combination factors were the same, as given by eq. [16], for buildings with various material types.

Fig. 4. Contours of peak horizontal acceleration and velocity having a probability of exceedance of 10% in 50 years (NRCC 1985), courtesy of the Geological Survey of Canada.



1990 NBCC

The 1990 NBCC used the same seismic zoning maps as the 1985 NBCC. Significant changes that were introduced included the replacement of the K factor by the force modification factor, R, and the use of a load factor of 1.0 on the seismic forces to reflect the onset of yielding in the structure. The base shear was determined from

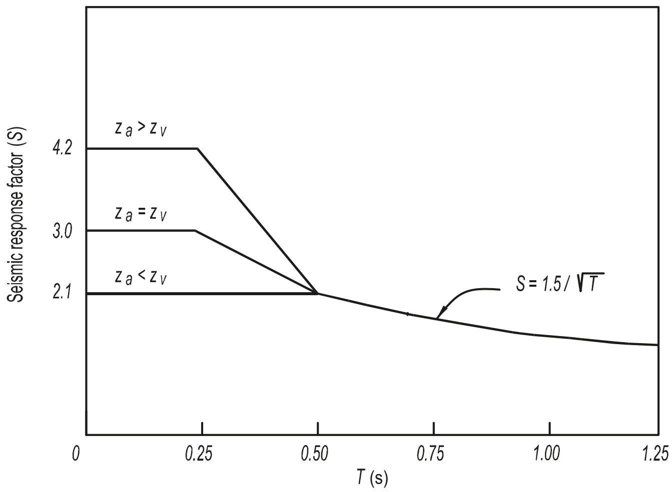
[21] $$V = U(vSIFW)/R$$

where U is a calibration factor (U = 0.6) to "maintain the design base shears at the same level of protection for buildings with good to excellent capability of resisting seismic loads consistent with the R factors used" (Commentary to 1990 NBCC NRCC 1990). The base shear was therefore calibrated to previous code values.

For $T \le 0.25$, S = 4.2, 3.0, 2.1 for Z_a/Z_v greater than, equal to, or less than 1, respectively; $S = 1.5/(T)^{1/2}$ for $T > 0.5$ (Fig. 5); I = 1.0, 1.3 or 1.5; and F = 1, 1.3, 1.5 or 2. Following the damage to structures in the soft soil region of Mexico City in the 1985 earthquake (Finn and Nichols 1988), the 1990 NBCC introduced the fourth category, F equal to 2.0, for "very soft and soft grained soils with depths greater than 15 m."

It is noted that the K factor from the 1985 NBCC was replaced by a force modification factor, R. The R factor "reflects the capability of a structure to dissipate energy through inelastic behaviour." The R factor varies from 1.0

Fig. 5. Seismic response factor, *S*, in the 1990 NBCC (NRCC 1990).



for unreinforced masonry to 4.0 for ductile moment-resisting space frames. Intermediate values of R were introduced with R = 2.0 for nominally ductile walls, concrete frames, and braced steel frames. The 1990 NBCC required that the design and detailing be in accordance with the provisions in the CSA standards for concrete, steel, timber, and masonry, consistent with the R factor chosen.

The earthquake load factor was reduced to 1.0 to reflect the extreme character of the loading considered in design, resulting in the following load combinations for earthquake design:

$$[22] \quad \begin{aligned} U &= 1.25D + 0.7(1.5L + 1.0E) \\ U &= 1.25D + 1.0E \\ U &= 0.85D + 1.0E \end{aligned}$$

The interstorey deflections, determined from analysis and multiplied by R to account for inelastic effects, were limited to $0.01h_s$ for post-disaster buildings and $0.02h_s$ for all other buildings, where h_s is the interstorey height. Tso (1992) provided a detailed comparison of the seismic design provisions in the 1985 NBCC and the 1990 NBCC.

1995 NBCC

The three major changes to the 1995 NBCC (NRCC 1995) were the additional R factors, new expressions for building periods, and new torsional eccentricity expressions. Additional lateral-load resisting systems introduced include nominally ductile and ordinary steel plate shear walls (R = 3 and 2, respectively), ductile coupled walls (R = 4), and reinforced masonry walls with nominal ductility (R = 2).

In the 1995 NBCC, the fundamental period, T, for moment-resisting frames was determined as $0.1N$ or, alternatively, as $0.085(h_n)^{3/4}$ for steel moment resisting frames and $0.075(h_n)^{3/4}$ for concrete moment resisting frames when the frame resists 100% of the lateral forces, where h_n is the total height in metres of the building above the base. As in previous editions, the period from dynamic analysis could be used in design, however, NBCC required that the resulting base shear should be not less than 80% of the static base shear.

The torsional moment, T_x, at a floor level x, was given as

$$[23] \quad \begin{aligned} T_x &= F_x(1.5e_x + 0.1D_{nx}) \\ T_x &= F_x(1.5e_x - 0.1D_{nx}) \\ T_x &= F_x(0.5e_x + 0.1D_{nx}) \\ T_x &= F_x(0.5e_x - 0.1D_{nx}) \end{aligned}$$

where e_x is the distance measured perpendicular to the direction of seismic loading between the centre of mass and centre of rigidity, D_{nx} is the plan dimension of the building at level x perpendicular to the direction of seismic loading, and F_x is the lateral force applied to level x. When a three-dimensional dynamic analysis is used, the effect of the accidental static torsion $\pm F_x \cdot 0.1D_{nx}$ needs to be added to the results of the dynamic analysis.

A companion load format was adopted for the load combinations involving earthquake loads to reflect the "probable" dead and live loads expected to be acting when the earthquake load occurs:

$$[24] \quad \begin{aligned} &U = 1.0D + 1.0E \\ &\text{for storage occupancies} \\ &U = 1.0D + 1.0E + 1.0L \\ &\text{for other occupancies} \\ &U = 1.0D + 1.0E + 0.5L \end{aligned}$$

2005 NBCC

Several major changes were incorporated in the 2005 NBCC (NRCC 2005) that are described in detail by DeVall (2003) and Heidebrecht (2003). The uniform hazard spectrum (UHS) approach (NEHRP 1997) was adopted essentially giving site-specific response spectral accelerations for numerous locations in Canada (Adams and Atkinson 2003). These spectral accelerations have a probability of exceedance of 2% in 50 years (2475-year return period). This lower probability provided a more uniform margin of collapse, one that is much nearer to the probability of structural failure (Heidebrecht 2003).

It is noted that the dynamic analysis approach became the preferred method of analysis and must be used for structures with certain irregularities.

The minimum lateral earthquake design force, V, at the base of the structure (equivalent static force procedure), is

$$[25] \quad V = S(T_a)M_v I_E W / R_d R_o$$

Except that V shall not be taken as less than $S(2.0)M_v I_E W/R_d R_o$.

where $S(T_a)$ is the design-spectral-response acceleration at the fundamental period of vibration, M_v is a factor to account for higher mode effects on the base shear (Humar and Mahgoub 2003; see Table 3). The 2005 NBCC introduced two separate force modification factors, the ductility-related factor R_d and the overstrength-related factor R_o (Mitchell et al. 2003), as defined in Article 4.1.8.9. The ductility-related force modification factor, R_d, reflects the capability of a structure to dissipate energy through inelastic behaviour while the overstrength-related force modification factor, R_o, accounts for the dependable portion of reserve strength in a structure designed according to the 2005 NBCC and the corresponding CSA standards. Table 4 gives some typical val-

Table 3. Higher mode factor M_v in 2005 NBCC.

$S_a(0.2)/S_a(2.0)$	Type of lateral resisting systems	M_v for $T_a < 1.0$	M_v for $T_a > 2.0$
<8.0	Moment resisting frames or "coupled walls"	1.0	1.0
	Braced frames	1.0	1.0
	Walls, wall-frame systems, and other systems	1.0	1.2
≥8.0	Moment resisting frames or "coupled walls"	1.0	1.2
	Braced frames	1.0	1.5
	Walls, wall-frame systems, and other systems	1.0	2.5

Note: Linear interpolation should be used for intermediate values.

ues for R_d and R_o for different seismic-force resisting systems (SFRS).

The earthquake importance factor, I_E, is taken as 1.0 for normal structures, 1.3 for "high iportance" structures (e.g., schools and community centres), and 1.5 for "post-disaster" structures (e.g., hospitals and emergency response facilities).

For SFRS with $R_d \geq 1.5$, V need not be taken greater than $\frac{2}{3}[S(0.2)I_E W/R_d R_o]$.

The design spectral acceleration values, $S(T)$ are given as

$$[26] \quad \begin{aligned} &S(T)=F_a S_a(0.2) \quad \text{for } T\leq 0.2\text{s} \\ &S(T)=F_v S_a(0.5) \quad \text{or} \quad F_a S_a(0.2) \\ &\text{whichever is smaller for } T=0.5\text{s} \\ &S(T)=F_v S_a(1.0) \quad \text{for } T=1.0\text{s} \\ &S(T)=F_v S_a(2.0) \quad \text{for } T=2.0\text{s} \\ &S(T)=F_v S_a(2.0)/2 \quad \text{for } T\geq 4.0\text{s} \end{aligned}$$

where $S_a(T)$ is the 5% damped spectral response acceleration, expressed as a ratio to gravitational acceleration, at a period of T; T_a is the fundamental lateral period of vibration of the building or structure (in seconds) in the direction under consideration; and F_a and F_v are the acceleration and velocity-based site coefficients, respectively, (Finn and Wightman 2003) depending on the site class (Table 5).

Figure 6 shows the values of spectral response acceleration [$S_a(T)$] for Vancouver, Montreal, and Toronto. These values also corresponding to the $S(T)$ values for site class C ($F_a = F_v = 1$). This UHS approach, through the use of $S_a(T)$ together with the site coefficients, F_a and F_v, results in site-specific spectra.

The fundamental lateral period of vibration of the building, T_a, (in seconds) can be evaluated empirically (Saatcioglu and Humar 2003) as

$$[27] \quad T_a = \lambda(h_n)^{3/4}$$

where λ is 0.085 for steel moment frames and 0.075 for concrete moment frames, while for other frames $T_a = 0.1N$, where N is the number of storeys. For braced frames

$$[28] \quad T_a = 0.025h_n$$

where h_n is the total height (in metres) of the building above the base. For shear walls and other structures, $\lambda = 0.05$ in eq. [27]. If a dynamic analysis is used, the resulting T_a values shall not be taken greater than 1.5 times that calculated using the empirical formula for moment resisting frames, and shall not exceed two times that calculated using the empirical formula for braced frames and shear wall structures. These limitations are placed on T_a to ensure that the period is in general agreement with typical measured periods on existing structures. This can represent a significant increase in period and hence a reduction in base shear compared to previous code editions.

Torsional effects (Humar et al. 2003) are considered by applying torsional moments, about a vertical axis at each level, derived separately for each of the following load cases considered:

$$[29] \quad \begin{aligned} T_x &= F_x(e_x + 0.1D_{nx}) \\ T_x &= F_x(e_x - 0.1D_{nx}) \end{aligned}$$

where F_x is the lateral force at each level and D_{nx} is the plan dimension of the building at level x perpendicular to the direction of seismic loading being considered.

The 2005 NBCC has greatly simplified the determination of torsional effects by eliminating the factor on the eccentricity e_x. This enables the designer to account for torsion directly, including accidental torsion, by performing 3-D analyses and shifting the mass at floor level by $\pm 0.1D_{nx}$. This approach no longer requires the very complex determination of e_x. Alternatively, the accidental torsion may be accounted for separately by adding the static effects of torsional moments due to $\pm 0.1D_{nx}F_x$ at each floor level.

Buildings with high torsional eccentricity are vulnerable to severe damage due to large displacements imposed on the "soft side" of the structure. Torsional sensitivity is determined by calculating the maximum value, B, from the calculated ratios B_x for each level x, where $B_x = \delta_{max}/\delta_{ave}$. The maximum storey displacement, δ_{max}, determined at the extreme points of the structure at level x is induced by the equivalent static forces acting at distances $\pm 0.1D_{nx}$ from the centres of mass at each floor and , δ_{ave} is the average of the displacements, at the extreme points of the structure at level x produced by the above forces. When B exceeds 1.7 and $I_E F_a S_a(0.2) > 0.35$, then a 3-D dynamic analysis is required.

Table 6 summarizes the different types of structural irregularities that were introduced in the 2005 NBCC. Such irregularities have resulted in significant damage by earthquakes, and therefore should ideally be avoided by designers. The presence of one or more of these irregularities may trigger the need to perform a dynamic analysis. There are also severe limits on irregularities in post-disaster buildings, so as to better ensure continued operations after a significant seismic event. The presence of irregularities in existing buildings should be considered an indication that the building is vulnerable to damage during strong ground shaking and further assessment of the seismic performance

Table 4. Seismic-force resisting systems (SFRS) ductility-related force modification factors (R_d), overstrength-related force modification factors (R_o) in 2005 NBCC.

Type of seismic-force resisting systems (SFRS)	R_d	R_o
Steel structures designed and detailed according to CSA S16		
Ductile moment resisting frames	5.0	1.5
Moderately ductile moment resisting frames	3.5	1.5
Limited ductility moment resisting frames	2.0	1.3
Moderately ductile concentrically braced frames	3.0	1.3
Limited ductility concentrically braced frames	2.0	1.3
Ductile eccentrically braced frames	4.0	1.5
Ductile frame plate shear walls	5.0	1.6
Moderately ductile plate shear walls	2.0	1.5
Conventional construction of moment frames, braced frames, or shear walls	1.5	1.3
Concrete structures designed and detailed according to CSA A23.3		
Ductile moment resisting frames	4.0	1.7
Moderately ductile moment resisting frames	2.5	1.4
Ductile coupled walls	4.0	1.7
Ductile partially coupled walls	3.5	1.7
Ductile shear walls	3.5	1.6
Moderately ductile shear walls	2.0	1.4
Conventional construction (Moment resisting frames and shear walls)	1.5	1.3
Timber structures designed and detailed according to CSA 086		
Shear walls		
• Nailed shear walls — wood based panels	3.0	1.7
• Shear walls — wood based and gypsum panels in combination	2.0	1.7
Braced or moment resisting frame with ductile connections		
• Moderately ductile	2.0	1.5
• Limited ductility	1.5	1.5
Other wood or gypsum based SFRS(s) — Not listed above	1.0	1.0
Masonry structures designed and detailed according to CSA S304.1		
Moderately ductile shear walls	2.0	1.5
Limited ductility shear walls	1.5	1.5
Conventional construction (Shear walls and moment resisting frames)	1.5	1.5
Unreinforced masonry	1.0	1.0

Table 5. Acceleration and velocity-based site coefficient as a function of site class in the 2005 NBCC.

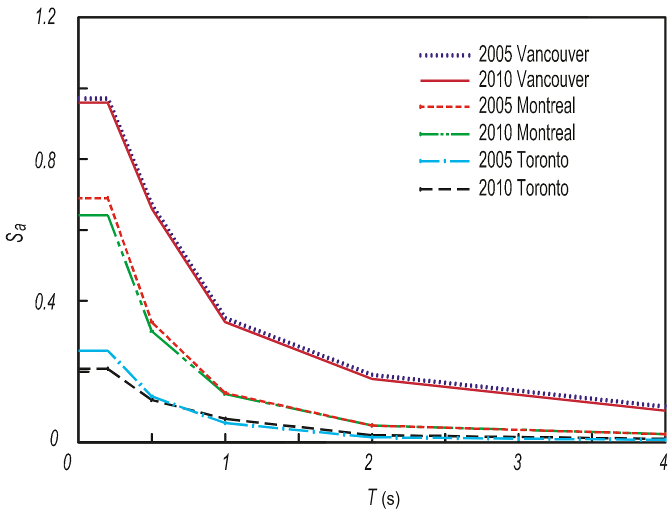
Site class		Shear wave	Values of F_a for site classes					Values of F_v for site classes				
			S_a (0.2)					S_a (1.0)				
		$\bar{V}_s$ (m/s)	≤0.25	= 0.50	= 0.75	= 1.0	≥1.25	≤0.10	= 0.20	= 0.30	= 0.40	≥0.50
A	Hard rock	>1500	0.7	0.7	0.8	0.8	0.8	0.5	0.5	0.5	0.6	0.6
B	Rock	760–1500	0.8	0.8	0.9	1.0	1.0	0.6	0.7	0.7	0.8	0.8
C	Very dense soil or soft rock	360–760	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
D	Stiff soil	180–360	1.3	1.2	1.1	1.1	1.0	1.4	1.3	1.2	1.1	1.1
E	Soft soil	<180	2.1	1.4	1.1	0.9	0.9	2.1	2.0	1.9	1.7	1.7
F			Site specific evaluation required					Site specific evaluation required				

of the building should be conducted. It is noted that there are also height restrictions for different structural systems, depending on the value of $I_E F_a S_a(0.2)$.

Dynamic analysis is the preferred method of analysis in the 2005 NBCC. However, if the base shear from dynamic analysis is lower than the earthquake design force V from eq. [25], the results must be amplified such that the base shear corresponds to V. For regular structures, V can be replaced by $0.8V$ in this adjustment process.

The calculated elastic maximum interstorey deflection at any level, including accidental torsional moments, shall be multiplied by $R_d R_o / I_E$ to get an estimate of the maximum interstorey deflections due to nonlinear response. These deflections are limited to $0.01h_s$ for post-disaster buildings,

Fig. 6. Values of spectral response acceleration, S_a, for Vancouver, Montreal, and Toronto according to 2005 NBCC (NRCC 2005) and 2010 NBCC (NRCC 2010).



$0.02h_s$ for schools, and $0.025h_s$ for all other buildings, where h_s is the interstorey height.

It is noted that the load factor for earthquake effects is taken as 1.0 because of the low probability of exceedance used in the UHS approach and the loading cases for earthquake effects are

[30] $U = 1.0D + 1.0E$
for storage occupancies
$U = 1.0D + 1.0E + 1.0L + 0.25S$
for other occupancies
$U = 1.0D + 1.0E + 0.5L + 0.25S$

2010 NBCC

The values for the UHS for all but western localities were recalculated using an improved fit to the ground motion relations used in 2005. In general short-period hazard in low seismic zones were slightly reduced though long-period hazard increased slightly (e.g., Toronto). The resulting changes to the UHS are illustrated in Fig. 6 for Montreal and Toronto, also shown is the UHS for Vancouver.

A change was made to the minimum lateral earthquake force, V, for walls, coupled walls and wall-frame systems such that

[31] $V \geq S(4.0)M_v I_E W/R_d R_o$

for moment-resisting frames, braced frames and other systems, V is defined as

[32] $V \geq S(2.0)M_v I_E W/R_d R_o$

As a consequence of this change, the values of M_v given in Table 3 were revised and the case of "M_v for $T_a > 2.0$" was changed to "M_v for $T_a = 2.0$" and an additional case for "M_v for $T_a > 4.0$" was added.

Additional force modification factors were added for cold-formed steel structures and for steel structures with ductile buckling-restrained braced frames. Some of the types of structural systems for steel structures were renamed.

An additional restriction was added for post-disaster buildings to disallow vertical stiffness irregularities.

Comparisons of seismic design force levels for concrete frame structures and concrete wall structures

Figure 7 gives a comparison of static design base shears from different versions of the *National building code of Canada* and CSA A23.3 standards. For this study, two-storey and 10-storey concrete moment resisting frame structures in Montreal and Vancouver were chosen. It was assumed that the storey heights were 3.5 m, the importance factor was 1.0, and the foundations were on very stiff soil (with F equal to 1.0 for earlier codes and Site Class C for 2005 and 2010 codes). For concrete structures, the "factored" values of V/W for the codes from 1965 to 1980 were based on factored loads (ultimate strength design), while from the values from 1985 to 2005 were based on limit states design in accordance with the NBCC. It is noted that prior to 1965 working stress design was used. Because the reinforcing steel stress was typically limited to 50% of the yield stress for concrete structures, an implied load factor of two was assumed to determine the "equivalent factored" values of V/W. While the load factors have been accounted for in these comparisons, the capacity reduction factors (ϕ) and the material resistance factors (ϕ_c, ϕ_s) have not been accounted for. The more recent codes have different design base shears depending on the "ductility" level and the corresponding detailing requirements in the CSA Standards. For convenience, structural systems have been categorized as "conventional", "nominal" (includes moderate ductility in the 2005 and 2010 NBCC), and "ductile". The design force levels are similar for 10-storey ductile frame structures after 1975. However, it must be recognized that the design and detailing requirements have become more stringent and hence the ductility and performance of older "ductile" structures would typically be less than more recently designed structures. The nominally ductile structures were introduced in the 1990 NBCC and the 1995 NBCC. It should be emphasized that the values of V/W have increased significantly in recent years for "conventional" construction, highlighting the greater risk for older frame structures. In addition, many older frame buildings have irregularities (Table 6) that makes such structures more vulnerable.

Figure 8 gives a comparison of design base shears from different versions of Canadian codes for 10-storey concrete wall structures in Montreal and Vancouver. It was assumed that the storey heights were 3.5 m, that the dimension of the building in the direction of lateral loading, D, was 30 m and that there were a series of parallel walls, with a length, D_s, in the direction of lateral loading of 5 m. From 1970 to 1980, the period was a function of D, resulting in a value of T varying from 0.55 to 0.58 s. From 1985 to 1995, the period was a function of D_s, resulting in a T of 1.41 s; and in the 2005 NBCC (NRCC 2005) and the 2010 NBCC (NRCC 2010) the period is a function of height and structural system, resulting in a value of 0.72 s. This significant difference in the period results in lower values of "factored"

Table 6. Structural irregularities in 2005 NBCC.

Type	Definition of irregularity
Vertical stiffness	If lateral stiffness of SFRS in a storey is <70% of adjacent storey, or <80% of average of three stories above or below
Weight	If weight of any storey is >150% of adjacent storey (excluding roofs)
Vertical geometry	If horizontal dimension in a storey of the SFRS is >130% of adjacent storey
In-plane discontinuity in SFRS	If there is an in-plane offset of the SFRS or a reduction in lateral stiffness of resisting element in the storey below
Out-of-plane offsets	Discontinuities in the lateral force path, such as out-of-plane offsets of SFRS elements
Weak stories	Storey shear strength less than that of storey above
Torsional sensitivity	If $B = \delta_{max}/\delta_{ave}$>1.7
Non-orthogonal systems	If the SFRS is not oriented along a set of orthogonal axes

Fig. 7. Comparisons of "factored" design base shears for concrete moment resisting frame structures in Montreal and Vancouver. Note that values of V/W before 1965 were based on working stress design and hence were multiplied by 2 for comparison (*a*) two-storey frame (Montreal), (*b*) ten-storey frame (Montreal), (*c*) two-storey frame (Vancouver), and (*d*) ten-storey frame (Vancouver).

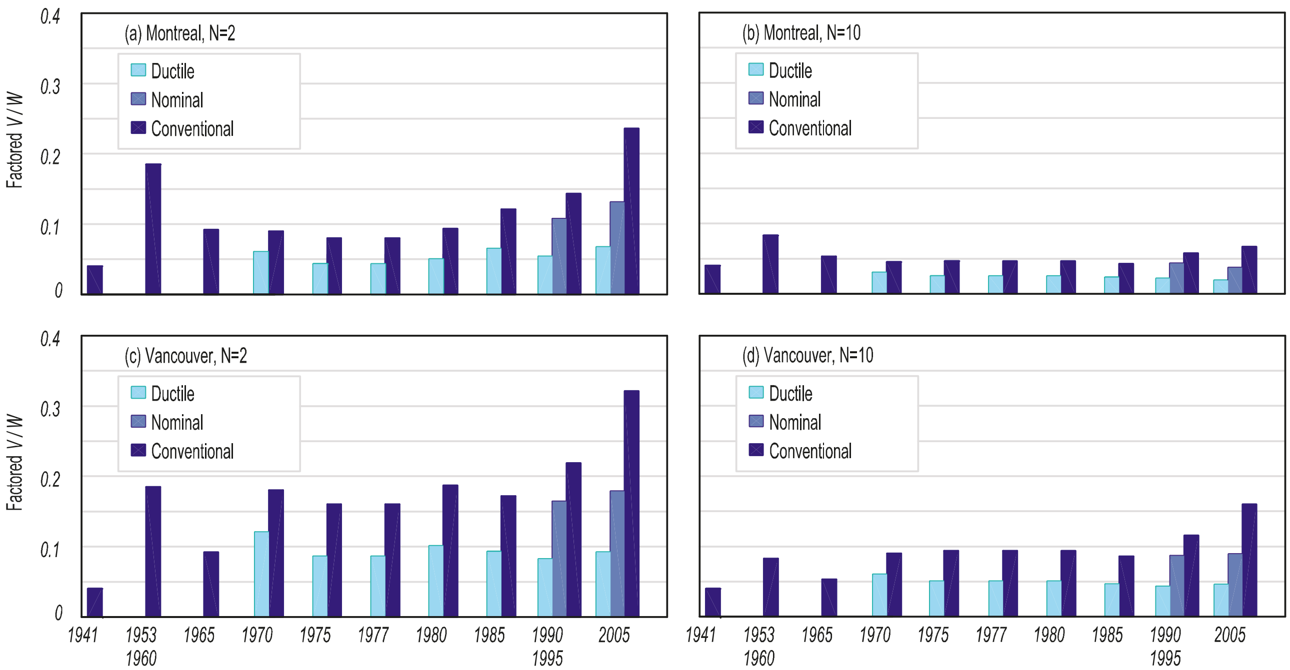
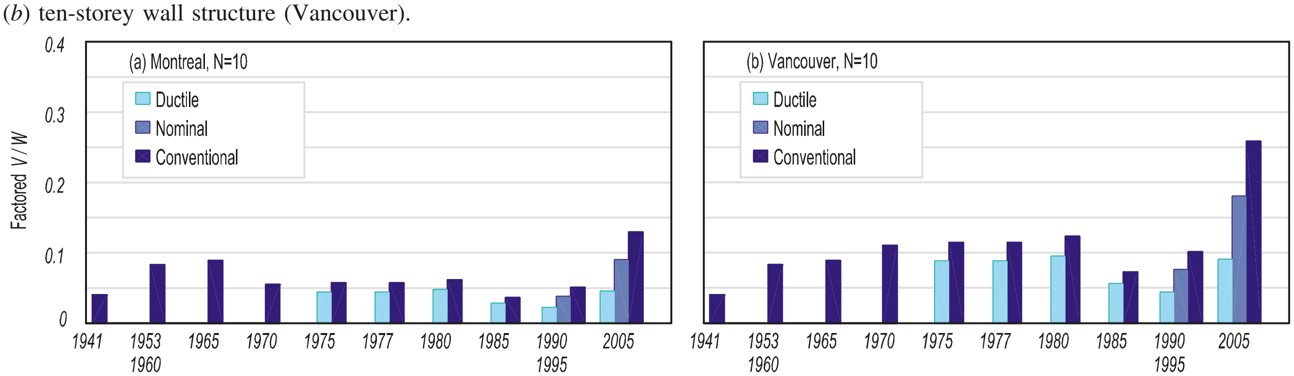


Fig. 8. Comparisons of "factored" design base shears for concrete wall structures in Montreal and Vancouver. Note that values of *V*/*W* before 1965 were based on working stress design and hence were multiplied by two for comparison (*a*) ten-storey wall structure (Montreal) and (*b*) ten-storey wall structure (Vancouver).



base shear from 1985 to 1995, as shown in Fig. 8. In 1990 and 1995, the *R* factors replaced the *K* factors for the different structural systems, a calibration factor (*U* of 0.6) was introduced and the load factor was reduced to 1.0. The significant increase for the "factored" base shear in 2005 is due mainly to the introduction of the UHS based on a probability of exceedance of 2% in 50 years, which is partially compensated for by the force modification factors, by the steep drop of the spectral values in the low period range, by the fact that the period was a function of the building height rather than on D_s, and by the fact that the design base shear is no longer scaled to previous code values. Ghorbanirenani et al. (2009) studied the nonlinear performance of walls designed and detailed in accordance with requirements of the NBCC between 1975 and 2005. They concluded that the walls designed in accordance with the older codes (1975 to 1995) are likely to lack sufficient shear capacity over their height as well as flexural strength above the plastic hinge region.

Conclusions

This paper provides a comparative study of the seismic design codes in Canada from the first code published in 1941 to the present. The comparison of factored base shears for design of concrete structures provides a guide for designers faced with the difficult task of seismic evaluation of existing structures. The key parameters that influence these factored base shears include seismicity, load factors, foundation conditions, determination of fundamental period, the seismic response factor, structural systems, and the corresponding design and detailing requirements. This comparison was made possible by assuming a load factor of 2.0 for structures that were designed using the working stress design (before 1965). This study illustrates the vulnerability of low period structures designed with older codes. In evaluating an older building, the engineer must be aware that major changes have taken place, not only with the design base shears but also for the classifications of structural systems that depend on the design and detailing requirements. To appreciate the evolution of seismic design codes in Canada, it must be recognized that there have been significant improvements to the design and detailing requirements, in the CSA materials standards, that are consistently linked to the ductility-based and the overstrength-based force modification factors.

Acknowledgements

The authors gratefully acknowledge the financial support provided by the Natural Sciences and Engineering Research Council of Canada (NSERC) for the Canadian Seismic Research Network (CSRN).

References

ACI. 1963. ACI standard building code requirements for reinforced concrete – ACI 318–63. American Concrete Institute, Detroit, Michigan, 144 p.

Adams, J., and Atkinson, G. 2003. Development of seismic hazard maps for the proposed 2005 edition of the National Building Code of Canada. Canadian Journal of Civil Engineering, **30**(2): 255–271. doi:10.1139/l02-070.

ATC. 1978. Tentative provisions for the development of seismic regulations for buildings. ATC 3–06, Applied Technology Council, National Bureau of Standards, Special Publication 510, U.S. Government Printing Office, Washington, D.C., 514 p.

Basham, P.W., Weichert, D.H., Anglin, F.M., and Berry, M.J. 1985. New probabilistic strong seismic ground motion maps of Canada. Bulletin of the Seismological Society of America, **75**: 563–595.

Blume, J.A., Newmark, N.M., and Corning, L.H. 1961. Design of multistory reinforced concrete buildings for earthquake motions. Portland Cement Association, Skokie, IL, 318 p.

Bustamante, J.I., and Rosenblueth, E. 1960. Building code provisions on torsional oscillations. *In* Proceedings 2nd World Conference on Earthquake Engineering, Japan, 879–894.

Cornell, C.A. 1968. Engineering seismic risk analysis. Bulletin of the Seismological Society of America, **58**: 1583–1606.

CSA. 1974. Steel structures for buildings – Limit states design – CSA S16.1–1974. Canadian Standards Association, Rexdale Ontario.

CSA. 1965. CSA Standard A23.3, Code for the design of concrete structures for buildings. Canadian Standards Association, Mississauga, ON.

CSA. 1973. CSA Standard A23.3, Code for the design of concrete structures for buildings. Canadian Standards Association, Mississauga, ON.

CSA. 1984. CSA Standard A23.3, Code for the design of concrete structures for buildings. Canadian Standards Association, Mississauga, ON.

DDF. 1966. Reglamento de Construcciones para el Distrito Federal, Departamento del Distrito Federal, Mexico City. [In Spanish.]

DeVall, R. 2003. Background information for some of the proposed earthquake design provisions for the 2005 edition of the National Building Code of Canada. Canadian Journal of Civil Engineering, **30**(2): 279–286. doi:10.1139/l02-048.

Finn, W.D.L., and Nichols, A.M. 1988. Seismic response of long period sites, lessons learned from the September 19, 1985 Mexican earthquake. Canadian Geotechnical Journal, **25**: 128–137.

Finn, W.D.L., and Wightman, A. 2003. Ground motion amplification factors for the proposed 2005 edition of the National Building Code of Canada. Canadian Journal of Civil Engineering, **30**(2): 272–278. doi:10.1139/l02-081.

Ghorbanirenani, I., Rallu, A., Tremblay, R., and Léger, P. 2009. Distribution of inelastic demand in slender R/C shear walls subjected to eastern North America ground motions. *In* Proceedings of the ATC-SEI Conference on Improving the Seismic Performance of Existing Buildings and Other Structures, San Francisco, Calif. pp. 1–13.

Hawkins, N.M., and Mitchell, D. 1977. Historical perspective, American Concrete Institute Special Publication, SP-53, Reinforced Concrete Structures in Seismic Zones, American Concrete Institute, pp. 1–23.

Heidebrecht, A.C. 2003. Overview of seismic provisions of the proposed 2005 edition of the National Building Code of Canada. Canadian Journal of Civil Engineering, **30**(2): 241–254. doi:10.1139/l02-068.

Heidebrecht, A.C., Basham, P.W., Rainer, J.H., and Berry, M.J. 1983. Engineering application of new probabilistic seismic ground motions of Canada. Canadian Journal of Civil Engineering, **10**(4): 670–680. doi:10.1139/l83-096.

Hodgson, J.H. 1956. A seismic probability map for Canada. Research Paper No. 22, Division of Building Research, National Research Council of Canada, Ottawa, Ont.

Humar, J., and Mahgoub, M.A. 2003. Determination of seismic design forces by equivalent static load method. Canadian Journal of Civil Engineering, **30**(2): 287–307. doi:10.1139/l02-067.

Humar, J., Yavari, S., and Saatcioglu, M. 2003. Design for forces induced by seismic torsion. Canadian Journal of Civil Engineering, **30**(2): 328–337. doi:10.1139/l02-029.

Milne, W.G., and Davenport, A.G. 1969. Distribution of earthquake risk in Canada. Bulletin of the Seismological Society of America, **59**: 729–754.

Mitchell, D., Tremblay, R., Karacebeyli, E., Paultre, P., Saatcioglu, M., and Anderson, D.L. 2003. Seismic force modification factors for the proposed 2005 edition of the National Building Code of Canada. Canadian Journal of Civil Engineering, **30**(2): 308–327. doi:10.1139/l02-111.

NEHRP. 1997. Recommended provisions for seismic regulations for new buildings and other structures. National Earthquake Hazard Reduction Program, Building Seismic Safety Council, Washington, D.C.

Newmark, N.M., Blume, J.A., and Kapur, K.K. 1973. Seismic design spectra for nuclear power plants. Journal of the Power Division, **99**: 287–303.

NRCC. 1941. National Building Code of Canada, Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, ON.

NRCC. 1953. National Building Code of Canada, Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, ON.

NRCC. 1960. National Building Code of Canada, Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, ON.

NRCC. 1965. National Building Code of Canada, Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, ON.

NRCC. 1970. National Building Code of Canada, Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, ON.

NRCC. 1975. National Building Code of Canada, Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, ON.

NRCC. 1977. National Building Code of Canada, Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, ON.

NRCC. 1980. National Building Code of Canada, Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, ON.

NRCC. 1985. National Building Code of Canada, Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, ON.

NRCC. 1990. National Building Code of Canada, Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, ON.

NRCC. 1995. National Building Code of Canada, Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, ON.

NRCC. 2005.National Building Code of Canada, Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, ON.

NRCC. 2010. National Building Code of Canada, Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, ON.

Saatcioglu, M., and Humar, J. 2003. Dynamic analysis of buildings for earthquake-resistant design. Canadian Journal of Civil Engineering, **30**(2): 338–359. doi:10.1139/l02-108.

SEAOC. 1959. Recommended lateral force requirements, SEAOC Code. Structural Engineers Association of California, Seismological committee, Sacramento, Calif.

Tso, W.K. 1992. Overview of seismic provision changes in National Building Code of Canada 1990. Canadian Journal of Civil Engineering, **19**(3): 383–388. doi:10.1139/l92-046.

UBC. 1935. Uniform Building Code 1935. International Conference of Building Officials (ICBO), Long Beach, California.

Uzumeri, S.M., Otani, S., and Collins, M.P. 1978. An overview of Canadian code requirements for earthquake-resistant concrete buildings. Canadian Journal of Civil Engineering, **5**(3): 427–441. doi:10.1139/l78-047.

Ward, H.S. 1966. Earthquake load provisions of the National building code of Canada. Technical Paper No. 211, Division of Building Research, National Research Council of Canada, Ottawa, ON.