Overview of seismic provisions of the proposed 2005 edition of the National Building Code of Canada[1]

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Abstract: The proposed 2005 edition of the National Building Code of Canada (NBCC) will contain very significant changes in the provisions for seismic loading and design. A brief history of the NBCC seismic provisions is presented followed by a discussion of the reasons for introducing such major changes in the next edition of the code. The major changes to the seismic provisions are summarized; this includes updated hazard in spectral format, change in return period (probability of exceedance), period-dependent site factors, delineation of effects of overstrength and ductility, modified period calculation formulae, explicit recognition of higher mode effects, rational treatment of irregularities, triggers for special provisions incorporated directly in classification of structural systems, and placing dynamic analysis as the normal "default" method of analysis for use in seismic design. The impact of these changes on the seismic level of protection is considered by comparing the 2005 NBCC and 1995 NBCC base shear coefficients for a selection of common structural systems located on a range of site conditions in three urban areas having low to high levels of seismic hazard, i.e., Toronto, Montréal, and Vancouver.

Key words: seismic, design, loading, code, hazard, buildings, structures, foundations, period, analysis.

Résumé : La prochaine édition du Code national du bâtiment du Canada (CNBC), prévue pour 2005, va contenir des changements significatifs pour les clauses concernant les charges et conception sismiques. Une brève histoire des clauses sismiques du CNBC est présentée, suivie par une discussion des raisons de l'introduction de tels changements majeurs dans la prochaine édition du Code. Les changements majeurs aux clauses sismiques sont résumés. Ces changements incluent une mise à jour du risque sous forme de spectre, le changement des périodes de retour (probabilité de dépassement), des facteurs de période dépendants du site, la délinéation des effets de sur-résistance et de ductilité, des formules modifiées des calculs de périodes, la reconnaissance explicite des effets des modes élevés, le traitement rationnel des irrégularités, des amorces pour des clauses spéciales incorporées directement dans la classification des systèmes structuraux, et le placement de l'analyse dynamique comme méthode d'analyse normale « par défaut » pour la conception sismique. L'impact de ces changements sur le niveau de protection sismique est considéré en comparant les coefficients de cisaillement à la base du CNBC 2005 et du CNBC 1995 pour une sélection de systèmes structuraux communs utilisant un éventail de conditions de site dans trois zones urbaines ayant des niveaux de risque sismique de bas à élevé, nommément Toronto, Montréal et Vancouver.

Mots clés : sismique, conception, chargement, code, risque, bâtiments, structures, fondations, période, analyse.

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Introduction

Responsibility for developing seismic provisions

The overall responsibility for developing the seismic provisions of the National Building Code of Canada (NBCC) is that of the Canadian National Committee on Earthquake Engineering (CANCEE), which operates under the direction of the Associate Committee on the National Building Code. In addition to doing the technical and editorial work involved in preparing the seismic provisions, CANCEE has the responsibility of communicating information about these provisions and their impact on building design to professionals in the building industry.

CANCEE is comprised of 20 members who bring together a variety of earthquake-related technical background experience in seismology, geotechnical engineering, and structural engineering. Approximately 40% of its members are engineering practitioners, with the remainder working in univer-

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sities and governmental agencies. In the process of preparing the NBCC seismic provisions, CANCEE obtains input and feedback from design professionals in several ways, e.g., through involving practicing engineers in local working groups, surveying selected designers (Heidebrecht 1999*a*), and soliciting responses to articles in the newsletter of the Canadian Association for Earthquake Engineering (CAEE) (e.g., Heidebrecht 2000).

History of seismic provisions in the NBCC

The first edition of the NBCC in 1941 contained seismic provisions in an appendix, based on concepts presented in the 1937 United States Uniform Building Code (UBC), but specific seismic provisions in the code proper did not appear until the 1953 edition; the history from 1953 is summarized in Table 1, which focuses on the nature of hazard information used to determine seismic design forces. That perspective is of particular interest, since major changes in code provisions have normally been driven by improved knowledge of seismic hazard.

Several significant observations can be drawn from Table 1: (*i*) there has been a movement from general hazard zones that are not at all associated with ground motions to zones that are directly based on peak ground motion values; (*ii*) after the introduction of ground motion parameters, there has been a change in the hazard methodology used to determine those parameters; and (*iii*) there has also been a change in the probability level at which the ground motion parameters have been determined.

Although the historical trend has been to move towards a more explicitly rational use of ground motion parameters in determining seismic design forces, the actual levels of those design forces have remained more or less constant during a period of about 40 years, independent of changes in the ground motion parameter (from peak ground acceleration to peak ground velocity), changes in methodology, and changes in probability level. The 20% reduction in design forces from the 1970 NBCC to the 1975 NBCC was deliberate, reflecting a sense that design forces could be reduced slightly without compromising the level of protection. Actually, that change was also accompanied by a comparable increase in the overturning moment reduction factor for buildings with periods longer than about 0.5 s; the effective level of protection for medium- to long-period buildings sensitive to overturning was therefore about the same as that in the previous code.

When the force expressions were modified to include peak ground motions explicitly, other factors were adjusted to maintain the same design force levels. Such an adjustment also occurred when the annual probability of exceedance was reduced from 0.01 to 0.0021. When the 1990 edition of the NBCC moved to a rational expression for base shear (i.e., one in which the manner of calculating design forces corresponds closely to the dynamics of systems responding to earthquake time histories having a specified peak ground motion) including the use of a force reduction factor that corresponds closely to the estimated realistic ductility factor capacities of building structures, it was then necessary to introduce a calibration factor of 0.6 to maintain the same design force levels.

The determination of seismic hazard for application in specifying seismic design forces has changed very significantly during the past four decades, in parallel with the increasing sophistication of building code seismic provisions. The historical development of hazard mapping in Canada, including the recent full recalculation of seismic hazard by the Geological Survey of Canada, is described and discussed in a companion paper by Adams and Atkinson (2003).

The foregoing historical account illustrates some of the reasons why the NBCC seismic provisions need to be updated from time to time. One of the major reasons is the ongoing improvement in the knowledge of seismic hazard and its geographical distribution throughout the country. As shown in Table 1, this knowledge moved from a general qualitative sense of seismicity based on historical earthquake activity to the expression of hazard using two ground motion parameters (peak ground velocity and acceleration) determined probabilistically. In addition to changes in the way in which seismic hazard is described, earthquake activity in Canada during the recent historical period has been used to produce more reliable estimates of seismic hazard.

Reasons for updating seismic provisions

There are several other major reasons for updating seismic provisions over and above those directly related to seismic hazard. First, studying and learning from the damage due to major earthquakes around the world enables engineers to determine whether or not current Canadian code provisions would be adequate to provide the level of protection required in buildings and other facilities being constructed in Canada. Each major earthquake provides one or more significant lessons that lead to further code improvements. For example, the 1989 Loma Prieta earthquake demonstrated the dramatic amplification of ground motions on soft soil sites; subsequent analysis of measured ground motions during that earthquake was used to improve code provisions for taking those effects into account in the design of structures located on soft soil sites (Borcherdt 1994).

Another major reason for periodic updating of seismic provisions arises directly from the results of broadly based earthquake engineering research being conducted in Canada and around the world. Such research, as reported in the literature and presented at conferences, often demonstrates the need for making changes to improve the code representation of seismic effects on structures. Many of the changes in the NBCC seismic provisions during the past half century have been made based directly on the results of Canadian earthquake engineering research and research done elsewhere in the world.

A very much related reason for changing Canadian provisions is the comparison of our provisions with those appearing in the codes of other countries. When such comparative analysis shows that the Canadian provisions are either inadequate or could be improved, then such provisions are imported and adapted for use in the NBCC provisions.

Why these papers?

The purposes of this paper are as follows: (*i*) provide an overview summary of the major changes in seismic provisions being introduced in the 2005 edition of the NBCC, (*ii*) discuss the significant factors affecting the changes in

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Table 1. History of determination of seismic design forces in the National Building Code of Canada (NBCC).

NBCC edition	Nature of hazard information	Manner in which hazard information is used to determine seismic design forces
1953–1965	Four zones (0, 1, 2, and 3) based on qualitative assessment of historical earthquake activity	Base shear coefficients are prescribed for design of buildings in zone 1; these are doubled for zone 2 and multiplied by 4 for zone 3
1970	Four zones (0, 1, 2, and 3) with boundaries based on peak acceleration at 0.01 annual probability of exceedance	Base shear coefficient includes a nondimensional multiplier (0 for zone 0, 1 for zone 1, 2 for zone 2, and 4 for zone 3)
1975–1980	Four zones (0, 1, 2, and 3) with boundaries based on peak acceleration at 0.01 annual probability of exceedance	Base shear coefficient includes factor A, which is numerically equal to the zonal peak acceleration (0 for zone 0, 0.02 for zone 1, 0.04 for zone 2, and 0.08 for zone 3); the value of the seismic response factor is adjusted so that base shear is approximately 20% below that in the 1970 NBCC
1985	Seven (0–6) acceleration- and velocity-related zones with boundaries based on a 10% in 50 year probability of exceedance	Base shear coefficient includes zonal velocity v, which is numerically equal to peak ground velocity in metres per second (values are 0, 0.05, 0.10, 0.15, 0.20, 0.30, and 0.40); the value of the seismic response factor is adjusted by calibration process so that seismic forces are equivalent, in an average way across the country, to those in the 1980 NBCC (see Heidebrecht et al. 1983)
1990 and 1995	Seven (0–6) acceleration- and velocity-related zones with boundaries based on a 10% in 50 year probability of exceedance	Elastic force coefficient includes zonal velocity v (as above) with total seismic force V calculated as elastic force divided by force reduction factor and then multiplied by a calibration factor of 0.6 (see eq. [1]); the seismic response factor is modified to maintain the same design force for highly ductile systems as that in the 1985 NBCC

seismic design forces, and (*iii*) present summary information on the impact of those changes on the level of protection against strong seismic ground motions. The companion papers in this volume provide detailed information on specific technical aspects of these provisions, e.g., hazard, analysis, and design.

Summary of major changes in provisions of the 2005 edition of the NBCC

The major changes made from the 1995 NBCC to the proposed 2005 edition of the NBCC are summarized in the following sections. To facilitate discussion of these changes and the comparison of base shear forces later in this paper, Table 2 shows the formulae for base shear in both codes, including an identification of the comparable parameters in each code.

Updated hazard in spectral format

As indicated in Table 1, seismic hazard in the 1985, 1990, and 1995 editions of the NBCC was described in terms of peak ground velocity and acceleration determined at a probability of 10% in 50 years. These ground motion parameters are then amplified (i.e., using the seismic response factor S in the 1995 NBCC) to obtain the period-dependent variation of seismic forces. The Geological Survey of Canada is now calculating hazard in the form of uniform hazard spectra (UHS) at specific geographical locations (Adams and Atkinson 2003), which provides a much better period-dependent representation of earthquake effects on structures. Because the spectral ordinates are determined directly at each geographical location, the differences in spectral shape across the country are reflected directly in the determination of design forces rather than being approximated by amplifying peak ground velocity or acceleration.

It should be noted that the UHS is defined by spectral ordinates at different periods calculated at the same probability of exceedance which distinguishes it from the classical response spectrum (RS). The ordinates of the UHS at different periods are affected by the ground motions from earthquakes of different magnitudes and distances from the site; the RS ordinates at different periods represent the response of different single degree of freedom systems to one specific ground motion. For the purpose of determining seismic loads on building structures, these differences have little engineering significance.

Change in return period (probability of exceedance)

During the past several decades it has been common, in the NBCC and in seismic codes in many other countries (e.g., United States and New Zealand), to specify seismic hazard information at a 10% in 50 year probability of exceedance, i.e., corresponding to a return period of 475 years. At the same time it has been recognized that the contribution of various sources of conservatism (e.g., overstrength) in the design and construction process leads to a much lower probability that structural failure or collapse will occur due to strong seismic ground motion. This would not cause any concern if the ground motions used in design provided an approximately uniform margin against collapse in different parts of the country.

Seismic hazard calculations at different probabilities of exceedance, however, have demonstrated that the slopes of the hazard curve vary considerably in different parts of the country. The hazard curve is defined as the relationship between the level of ground shaking, e.g., a spectral ordinate at

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Table 2. Base shear formulae in the 1995 and proposed 2005 editions of the NBCC.

Parameter	1995 NBCC	2005 NBCC
Static lateral earthquake force	$V = U(V_e/R)$; $V_e = vSIFW$	$V = S(T)M_vI_EW/(R_dR_o)$; $S(T) = F_aS_a(T)$ or $F_vS_a(T)$
Level of protection experience factor	$U = 0.6$	None
Importance factor	$I = 1.0$, 1.3, or 1.5	$I_E = 1.0$, 1.3, or 1.5
Foundation or site factor	$F = 1.0$, 1.3, 1.5, or 2.0	F_a and F_v are functions of site class and intensity of ground motion: $0.7 \le F_a \le 2.1$ and $0.5 \le F_v \le 2.1$
Seismic hazard parameter	v = zonal velocity ratio, determined at 10% in 50 year probability of exceedance	$S_a(T)$ = 5% damped spectral response acceleration, determined at 2% in 50 year probability of exceedance
Site response factor	S is a function of period T and zonal acceleration ratio a	Equivalent to $M_vS_a(T)$
Factor to take into account higher modes	Included in long-period shape of site response factor $S \propto 1/T^{0.5}$	M_v is a function of T, type of system, and shape of spectral response acceleration curve: $0.4 \le M_v \le 2.5$
Force modification factor	$1.0 \le R \le 4.0$	R_dR_o: $1.0 \le R_d \le 5.0$ and $1.0 \le R_o \le 1.7$

Note: The descriptions of some parameters in this table have been simplified to facilitate comparison. *W* refers to dead load.

a specific period or peak ground acceleration or velocity, and the probability that the particular level of ground shaking will be exceeded. The main variation in hazard curve shape occurs between regions that are near plate boundaries (e.g., Vancouver and Victoria) and intraplate regions (e.g., eastern Canada). To provide a more uniform margin of collapse it is necessary to specify seismic hazard at a lower probability of exceedance, i.e., one that is much nearer to the probability of failure or collapse.

The 1997 National Earthquake Hazard Reduction Program (NEHRP) provisions (Building Seismic Safety Council 1997), based on the approach of providing a more uniform margin against collapse, specified the use of "maximum considered earthquake ground motion," which is defined as that having a 2% in 50 year probability of exceedance (return period of approximately 2500 years). Adams and Halchuk (2003) provides a rationale for using this probability level in Canada, including a numerical comparison of hazard curves between Vancouver and Montréal. On this basis, the seismic code provisions in the 2005 NBCC are based on using 2% in 50 year seismic hazard values.

Period-dependent site factors

It has long been recognized that amplification of seismic motions from rock to soil sites can be significant, especially at sites with soft soil conditions. As noted in Table 2, the 1995 NBCC includes a foundation factor *F*, which ranges from 1.0 to 2.0 but does not vary with period or with the intensity of the underlying rock motion; the type and depth of rock and soil in each of four categories are defined only in a qualitative manner.

As noted earlier, research by Borcherdt (1994) and others has enabled quantification of important seismic response effects for code purposes, including categorization of soil profiles using quantitative properties (shear wave velocity, standard penetration resistance, or undrained shear strength), period dependence, and effects of the intensity of underlying rock motion. This work was incorporated into the seismic provisions of the NEHRP 1997 (Building Seismic Safety Council 1997). Those provisions are used as the basis for the site factors in the 2005 NBCC, as discussed by Finn and Wightman (2003). An important feature of the revised site factors is the deamplification of seismic motions at sites located on rock or hard rock, i.e., sites at which the shear wave velocity is greater than that of the reference site condition, which is described as "very dense soil and soft rock."

Delineation of effects of overstrength and ductility

For many years the NBCC seismic provisions have recognized, either implicitly or explicitly, that seismic forces are reduced when structural response goes into the inelastic range. This is an important feature in enabling structures to resist strong earthquake shaking, provided of course that the structure has the capacity to deform inelastically through several load reversals without a significant loss of strength. The 1995 NBCC incorporates this recognition by including a force modification factor *R* in the denominator of the expression used to calculate the lateral seismic force *V*; as noted in Table 2, the value of *R* ranges from 1.0 (for nonductile structural systems) to 4.0 (for ductile structural systems).

On the other hand, there has been considerable mystique about the quality that has become referred to as "overstrength." Various features of structural systems and their design (e.g., material factors used in design, minimum design requirements, load combinations, and the redistribution of forces arising from redundancy) often lead to a lateral strength that is considerably larger than that used as the basis for design. This has been implicitly recognized by using design ground motions at probabilities well above the expected probability of failure or collapse and by calibrating code seismic design forces to those used in previous editions of the code (e.g., calibration of the 1985 NBCC forces to those in the 1980 NBCC, as described in Heidebrecht et al. 1983). The 1990 NBCC introduced a calibration factor $U = 0.6$ in the calculation of the lateral seismic force *V*, which has sometimes been interpreted as an explicit representation of overstrength (Tso 1992).

The calculation of the lateral seismic force *V* in the 2005 NBCC introduces an explicit system overstrength factor R_o; this factor is intended to represent the minimum level of overstrength that can be counted on for each particular seismic force resisting system (SFRS). It ranges from 1.0 to 1.7 and is applied as a reduction factor in the denominator of the expression used to calculate *V*, as shown in Table 2. A force modification factor is also used in the denominator, and is

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now labelled R_d to denote a more explicit linkage to the ductility capacity of each SFRS. The rationale for the use of these two factors is given by Mitchell et al. (2003), who also detail the reasons for particular values or ranges of values for various structural systems.

Period calculations

The calculation of the fundamental period T_a is significant because this value determines the spectral response acceleration $S(T_a)$. On the one hand, the determination of T_a needs to be relatively simple, whereas on the other hand its value should not be overestimated; values of T_a that are larger than can be realistically expected result in an underestimate of the seismic design force V.

Although the formulae for calculating the periods of moment resisting frames have not been changed, the formula for other structures (e.g., walls and braced frames) is simplified so that it is no longer dependent on the length D_s of those elements. This change is made because of the considerable confusion as to the appropriate value of D_s, which is often ill-defined. The rationale for the new formula is given by Saatcioglu and Humar (2003).

Although the revised provisions continue the practice of allowing period calculations by other established methods of mechanics, the upper limit on calculated periods is now expressed as $1.5T_a$ rather than by placing a lower limit on the seismic force. The justification for an upper limit arises because of the concern that structural models frequently overestimate the flexibility of a structural system (e.g., by neglecting nonstructural stiffening elements), giving rise to an overestimate of the natural period.

Higher mode effects

The static equivalent lateral seismic force calculated in the NBCC provisions, as with that in other codes, is based on the assumption that the main features of the dynamic response of the structure can be represented by a single mode response at the fundamental period T_a. Since many structures, particularly those with longer periods, have significant higher mode effects, these are taken into account by modifications in both the value of the seismic design force and the distribution of the shears and moments along the height of the structure. In the 1995 NBCC, higher mode effects are simulated by an additional top force F_t and by an overturning moment reduction factor J. These are also used in the 2005 NBCC provisions, but a higher mode factor M_v is also applied directly in the determination of the lateral seismic force V, as shown in Table 2. The rationale for this factor and for the values prescribed in the provisions is given by Humar and Mahgoub (2003).

The simulation of higher mode effects in an equivalent static procedure is not valid for structures with long periods. Consequently, the 2005 NBCC requires that a dynamic analysis procedure be used for regular structures with periods of more than 2 s or with heights greater than 60 m located in regions of moderate to high seismicity.

Treatment of irregularities

The only specific treatment of irregularities in the 1995 NBCC concerns the analysis required for torsional effects. General statements are made concerning discontinuous vertical resisting elements and the need to take into account the possible effects of setbacks, but the provisions contain no specific requirements. The 2005 NBCC includes definitions of eight types of irregularities and specifications concerning analysis and design for each of those types. The kinds of specifications applicable to the different types of irregularities include limitations on the use of the static analysis procedure, restrictions on irregularities permitted in relation to the extent of the seismic hazard, restrictions applicable to post-disaster buildings, increases in seismic design forces, and specific design requirements (e.g., related to diaphragms, openings, and discontinuities). There continue to be specific requirements for taking into account torsional effects; a torsional sensitivity parameter B is used to determine whether or not dynamic analysis is required. Detailed descriptions of the rationale for the irregularity provisions and for the revisions to the torsional requirements are given by DeVall (2003) and Humar et al. (2003), respectively.

Dynamic analysis requirements

Dynamic analysis plays a very small role in the 1995 NBCC provisions; designers are given the option (sentence 4.1.9.1.(13)(b)) of determining the distribution of seismic forces within the structure, but these must be scaled so that the lateral seismic force V is the same as that determined using the normal static method. As indicated in Commentary J of the 1995 NBCC, the dynamic option is primarily applicable for buildings with significant irregularities and buildings with setbacks or major discontinuities in stiffness or mass. The main reason for using dynamic analysis in those situations would be to obtain a better distribution of forces within the building; this would also apply to tall buildings in which the dynamic analysis would include higher mode effects, which cannot be well represented by static equivalent loads. Designers are also allowed to use dynamic analysis for the determination of torsional moments but with the proviso that the effects of accidental torsion must be determined statically and added to the effects of a three-dimensional dynamic analysis. To enable the designer to use dynamic analysis, Commentary J includes a normalized design distribution spectrum and a very brief procedure for conducting an elastic dynamic modal analysis.

Dynamic analysis plays a very prominent role in the 2005 NBCC seismic provisions. It is stated as the required method of analysis with the exception that the equivalent static force procedure may be used for structures in any of the following situations: (*i*) relatively low seismic hazard, as defined by the short-period (0.2 s) design spectral acceleration; (*ii*) regular structures less than 60 m in height and with fundamental lateral period less than 2 s; or (*iii*) certain irregular structures less than 20 m in height and with fundamental lateral period less than 0.5s. The general rationale for this radical change is that linear dynamic analysis (particularly modal analysis) is now a straightforward procedure that simulates the effects of earthquakes on a structure much better than the equivalent static force procedure. The exceptions recognize, however, that (*i*) there is not likely to be any significant negative consequence in allowing the static procedure in areas of low seismic hazard; (*ii*) the equivalent static loads can simulate dynamic effects for medium-height regular structures, provided that the fundamental period is not

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too long; and (*iii*) both overall force and distributional effects are determined quite well by the static force method for relatively squat, short-period, irregular structures, except those which are torsionally sensitive.

Conducting dynamic analysis in accordance with the 2005 NBCC provisions is also facilitated by the fact that seismic hazard is now specified in terms of spectral acceleration. The provisions specify design spectral acceleration values for all fundamental periods; these are determined directly from 5% damped spectral response acceleration values $S_a(T)$ multiplied by site amplification factors for periods T of 0.2, 0.5, 1.0, and 2.0 s. This means that the input to a dynamic analysis is based directly on the best available estimates of ground motion, at the specified probability of exceedance. The 2005 NBCC requires that the spectral acceleration values used in the modal response spectrum method be the design spectral acceleration values (which are also used as the basis for determining the minimum lateral seismic force V in the equivalent static force procedure) and that, if a numerical integration linear time history method of dynamic analysis is used, the ground motion histories shall be compatible with a response spectrum constructed from the design spectral acceleration values.

Although the 2005 NBCC makes dynamic analysis the normal procedure, there is still a concern that the resulting seismic forces may be too low because the parameters used in the analysis (e.g., structural stiffness) are totally at the designer's discretion rather than being specified by the code. For example, although there are limitations on the maximum value of the fundamental lateral period T_a which can be used in the equivalent static force procedure, there are no such limitations in the specifications for dynamic analysis. To provide some protection against inappropriate choices of such parameters, the 2005 NBCC provisions require that the dynamically determined base shear shall not be less than 80% of that determined using the static method and that, in the case of irregular structures in which dynamic analysis is required (rather than being optional), the minimum dynamic base shear is 100% of the statically determined value.

Triggers for special provisions

The 1995 NBCC, primarily in section 4.1.9.3, contains a number of special provisions in which certain restrictions are "triggered" when the velocity- or acceleration-related seismic zone is at a certain level or higher. These special provisions include limiting the kind of structural system that can be used, restricting the height of buildings with structural systems having limited ductility capacity, ensuring that reinforcement is provided in certain kinds of masonry elements, and requiring specific foundation design requirements.

The restrictions on structural systems in the 2005 NBCC are included in the same table (Table 4.1.8.9), which defines the force modification factor R_d and the system overstrength factor R_o for each SFRS. The restrictions are now triggered by the design spectral acceleration values (including the earthquake importance factor I_E) determined at periods of 0.2 and 1.0 s. Inclusion of these restrictions in this table simplifies the design process in that the designer can immediately see the consequences of choosing a particular SFRS, both in terms of the factors R_d and R_o and in terms of any restrictions that may be applicable to the particular system.

The 2005 NBCC requirements also include triggered restrictions on designs having structural irregularities and on foundation design requirements. The rationale for triggering specific structural and foundation design restrictions are discussed in more detail in the companion papers by DeVall (2003) and Finn and Wightman (2003).

Deflections and drift limits

The drift limits in the 1995 NBCC are specified as $0.01h_s$ for post-disaster buildings and $0.02h_s$ for all other buildings, in which h_s is the interstorey height. In the 2005 NBCC, the limit for post-disaster buildings remains the same, the value of $0.02h_s$ is specified for schools, and the value for all other buildings is increased to $0.025h_s$. Although these appear to be the same as or more liberal than those in the 1995 NBCC, they are actually more restrictive because displacements are now determined using loads based on a 2% in 50 year hazard rather than a 10% in 50 year hazard. The rationale for these changes and their impact are discussed by DeVall (2003).

Impact of changes on level of protection

Common structural systems

In 1999, under the auspices of CANCEE, the author conducted a survey of designers in Vancouver, Montréal, and Québec City to determine the most common structural systems in use at that time; the purpose of the survey was to assist in determining the impact of changes in the code on the seismic level of protection in the most populous regions of the country having moderate to high levels of seismic hazard. The results of the survey were presented to CANCEE (Heidebrecht 1999*b*) and are summarized here.

In Vancouver, respondents to the survey indicated that two reinforced concrete wall systems were the most common types of structural systems being used in building construction. These were ductile wall systems (case 12 in the 1995 NBCC) for buildings of 14 storeys and higher, and wall systems having nominal ductility (case 14) for four- to seven-storey buildings; between them, these two situations made up 44% of all structures in Vancouver. In Montréal and Québec City, respondents indicated that over 50% of the buildings being constructed were steel-braced frames with nominal ductility; almost all of these were in the one- to three-storey height range.

Between the two locations, making up over 75% of all systems, six structural systems were identified as being the most common. These are shown in Table 3, including the case numbers, the description of each system, and the force modification factor R for each system. Table 3 also provides a description of the equivalent SFRS in the 2005 NBCC and the corresponding factors R_d and R_o. There are slight differences in nomenclature between the 1995 NBCC and the 2005 NBCC descriptions, due primarily to the fact that the 2005 NBCC descriptions are selected to be the same as those used in the Canadian Standards Association materials codes for steel, reinforced-concrete, timber, and masonry structures.

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Table 3. Common structural systems and equivalence between the 1995 and 2005 NBCC.

Types of lateral force resisting system in the 1995 NBCC			Types of seismic force resisting systems (SFRS) in the 2005 NBCC		
Case	Description	R	Description	R_d	R_o
7	Steel: braced frame with nominal ductility	2	Steel: limited ductility concentric braced frame	2	1.3
9	Steel: other lateral force resisting systems not defined in cases 1–8	2	Steel: conventional construction of moment frames, braced frames, or shearwalls	1.5	1.3
11	Reinforced concrete: ductile coupled wall	4	Reinforced concrete: ductile coupled wall	4	1.7
12	Reinforced concrete: other ductile wall systems	4	Reinforced concrete: ductile shearwall	3.5	1.6
14	Reinforced concrete: wall with nominal ductility	2	Reinforced concrete: moderately ductile shearwall	2	1.4
16	Timber: nailed shear panel with plywood, waferboard, or oriented strand board (OSB)	3	Timber: nailed shearwalls, wood-based panel	3	1.7

Table 4. Relationship between 1995 and 2005 NBCC site classifications.

1995 NBCC category (Table 4.1.9.1.C)	1	2	3	4
1995 NBCC foundation factor F	1	1.3	1.5	2
2005 NBCC site classification (Table 4.1.8.4.A)	A, B, and C	Midway between C and D	D	E

It should also be noted that R_d for each system is identical to R, which is to be expected because the force modification factor in the 1995 NBCC is in fact primarily due to ductility. This is true for almost all of the types of SFRS listed in Table 4.1.8.9 in the 2005 NBCC; there are a very small number of exceptions due to a reassessment of the ductility capacity of several structural systems. For example, $R_d = 5.0$ for ductile moment resisting steel frames, whereas $R = 4.0$ for such frames in the 1995 NBCC.

Foundation equivalence

As indicated earlier in this paper, there are significant changes in the foundation factors used to amplify ground motions for the determination of seismic loads. These are accompanied by changes in the descriptions of the site classifications. To make comparisons among seismic forces determined for different foundation conditions, it is necessary to define the relationships between the site classification definitions in the two editions of the code, which are shown in Table 3.

The equivalences of Table 4 have been determined by comparing descriptions in the two codes. It is important to note that the 1995 NBCC category 1, which includes rock and stiff soil, is now broken into three different site classifications: A (hard rock), B (rock), and C (very dense soil and soft rock). These distinctions are important because the acceleration-based site coefficient F_a and the velocity-based site coefficient F_v, which are used to amplify ground motions in the determination of design spectra response accelerations, can vary among these three classifications, ranging between 0.5 and 1.0, depending on the intensity of the ground motion at the site. As noted earlier in the paper, values less than 1.0 represent deamplification of ground motions from the reference site classification (C).

Base shear comparisons

For the purpose of considering the impact of changes in seismic provisions from the 1995 NBCC to the 2005 NBCC, base shear forces were calculated using both codes for the six common structural system types shown in Table 3 in the three most populous cities in Canada (Vancouver, Toronto, and Montréal). Not only are these the largest concentrations of population, but also they represent low (Toronto), moderate (Montréal), and high (Vancouver) seismic hazard. Forces were calculated for structures located on site classifications A, C, and E, i.e., including the two extreme classifications and the reference classification C.

Figures 1–6 show the results of these calculations, in the form of base shear coefficients (V/W, where W is the dead load) as a function of fundamental structural period. All figures present results for periods ranging from 0.1 to 4.0 s, although it is clear that some structural systems (e.g., timber shearwalls or steel-braced frames with limited ductility) would never be used for long-period (i.e., very high) building structures. Also, these figures do not incorporate the restrictions included with the SFRS description in Table 4.1.8.9 of the 2005 NBCC; for example, these restrictions would limit the use of conventional steel moment frames to a maximum of 15 m in Vancouver.

Each of Figs. 1–6 gives the results for one type of structural system, organized in the same order as the description in Table 3, reading from the top down. Each figure contains three graphs, one for each of the three site classifications (A, C, and E, in that order), and each graph contains six lines showing the base shear coefficients for the three cities using the 1995 and 2005 NBCC.

The following sections of the paper use Figs. 1–6 in discussing the particular factors that have the largest impact on seismic design forces and the resulting total impact on such forces. For the purpose of this paper, seismic design forces are considered to represent the seismic level of protection. It should be recognized, however, that there are other significant aspects, e.g., detailing and quality of construction, which contribute significantly to that level of protection and are not included in this discussion.

Seismic hazard

Geographical distribution

Adams and Atkinson (2003) give detailed comparisons of changes in geographical distribution of seismic hazard, but the discussion here will be restricted to the impact of distri-

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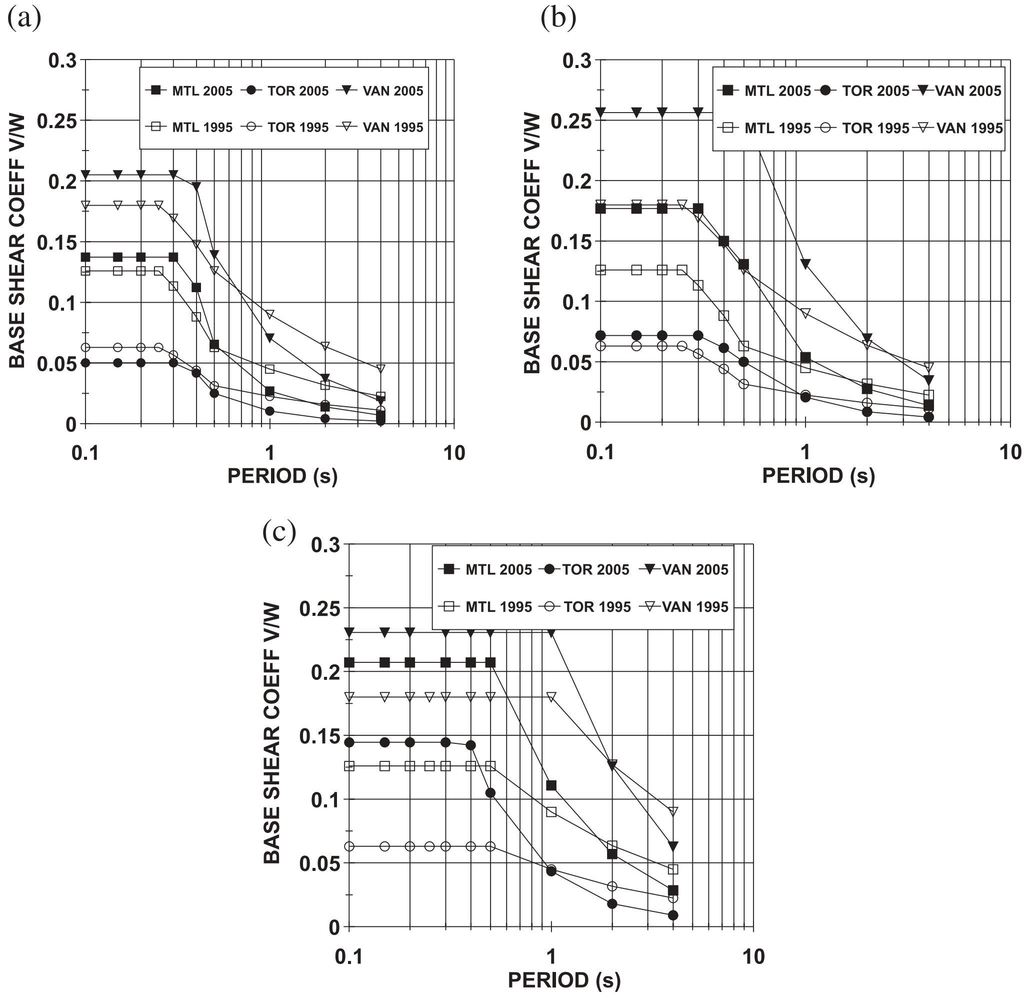
Fig. 1. Base shear coefficients (*V*/*W*) for limited ductility steel-braced frame (1995 NBCC case 7): (*a*) hard rock (site A); (*b*) very dense soil and soft rock (site C); and (*c*) soft soil (site E). MTL, Montréal; TOR, Toronto; VAN, Vancouver.



bution changes on seismic forces in Vancouver, Montréal, and Toronto. Figure 1*b* shows the base shear coefficients for limited ductility steel-braced frames situated on the reference site condition (C). At a fundamental period of 1.0 s, approximate changes in base shear are –10% in Toronto, +20% in Montréal, and +50% in Vancouver. The relative changes at that period would be the same for other types of structural systems. The implication of the changes shown in Fig. 1*b* is that the long-period hazard in Vancouver increases significantly relative to that in Montréal and Toronto, with a modest relative increase from Toronto to Montréal.

Making the same kind of comparison at a fundamental period of 0.1 s shows approximate changes in base shear of +15% in Toronto and +40% in both Montréal and Vancouver. Relative short-period hazard increases only modestly from Toronto to both Montréal and Vancouver, and there is no relative increase between Montréal and Vancouver.

A word of caution is needed here: these observations are valid only for the three cities included in this comparison. Although relative changes of similar magnitudes can be seen at other locations, there is no particular pattern, i.e., it cannot be concluded that locations in high-hazard regions always have a relative increase in hazard compared with locations in low-hazard regions. The apparently random nature of changes in geographical distribution of hazard is due primarily to the changes in the modelling of seismicity which have been used in the revised calculations of seismic hazard; these are discussed in some detail by Adams et al. (1999).

Spectral shape

To discuss the impact of changes in spectral shape on the calculation of seismic base shear forces, it is necessary to review the variations of spectral shape which are implicit in the provisions of the 1995 NBCC. Variations in spectral shape are incorporated by different formulations for the seismic response factor S, depending on the ratio of acceleration-related to velocity-related seismic zones, Z_a and Z_v. Table 5 shows Z_a, Z_v, and the ratio $S(0.2\ s)/S(1.0\ s)$ for Montréal, Toronto, Vancouver, and Prince Rupert, which has been included to cover the full range of Z_a/Z_v ratios. The variation of spectral shape is such that $S(0.2\ s)/S(1.0\ s)$ ranges from 1.4 to 2.8.

Table 5 includes the spectral acceleration values (at 0.2 s and 1.0 s) computed for use in the 2005 NBCC for the same four cities (Adams and Halchuk 2003) and the corresponding ratio $S_a(0.2\ s)/S_a(1.0\ s)$. This ratio ranges from 2.10 to 5.09, i.e., approximately 50–80% greater than the ratio

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Fig. 2. Base shear coefficients for conventional steel moment frame (1995 NBCC case 9): (*a*) hard rock (site A); (*b*) very dense soil and soft rock (site C); and (*c*) soft soil (site E).

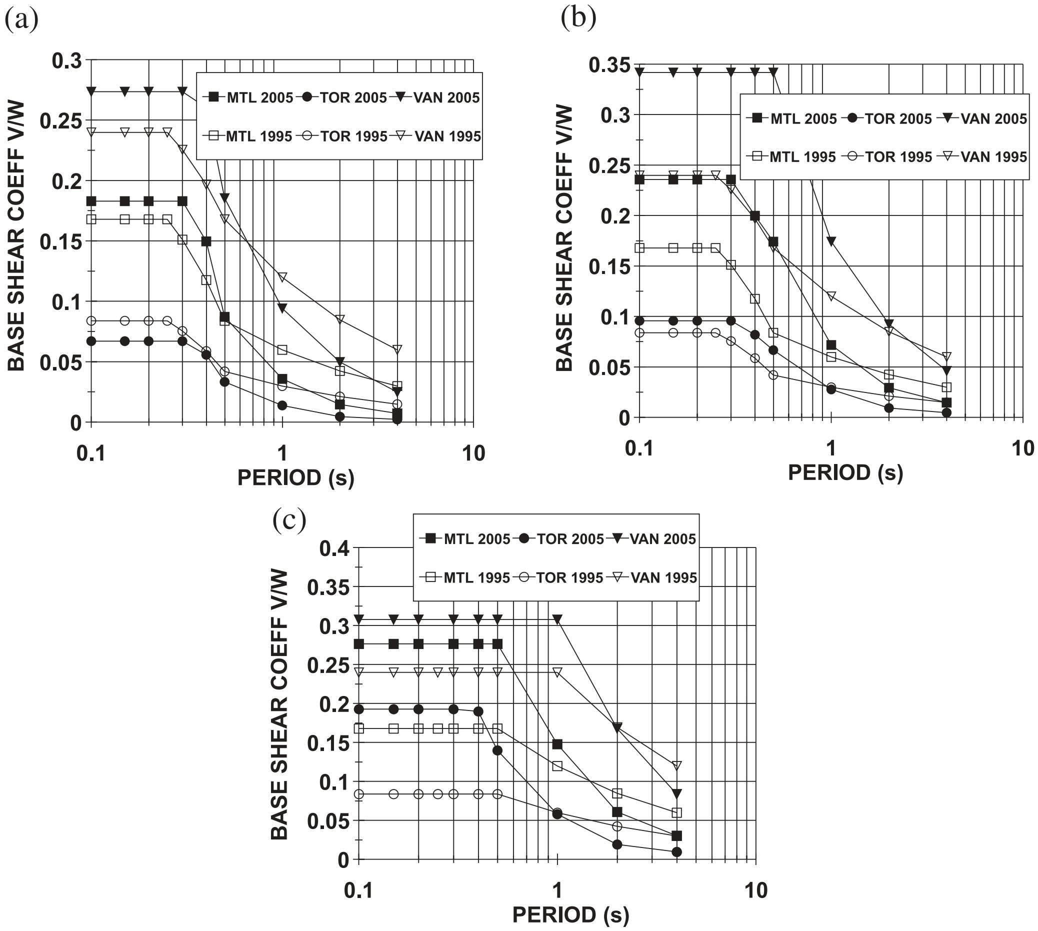

Table 5. Seismic hazard in terms of spectral shape and probability level information.

Parameter	Montréal	Toronto	Vancouver	Prince Rupert
1995 NBCC				
Z_a	4	1	4	3
Z_v	2	0	4	5
$S(0.2\ s)/S(1.0\ s)$	2.8	2.8	2	1.4
2005 NBCC				
Values at a probability of 2% in 50 year hazard[a]				
$S_a(0.2\ s)$	0.69	0.28	0.97	0.35
$S_a(1.0\ s)$	0.14	0.055	0.34	0.17
$S_a(0.2\ s)/S_a(1.0\ s)$	4.96	5.09	2.82	2.1
$V(0.2\ s)/V(1.0\ s)$[b]	3.31	3.39	1.88	1.4
Values at a probability of 10% in 50 year hazard[a]				
$S_a(0.2\ s)$	0.29	0.11	0.51	0.19
$S_a(1.0\ s)$	0.052	0.022	0.18	0.091
$S_a(0.2\ s)/S_a(1.0\ s)$	5.58	5	2.85	2.05
Ratio of 2% in 50 year hazard to 10% in 50 year hazard				
$T = 0.2$ s	2.39	2.55	1.90	2.03
$T = 1.0$ s	2.67	2.5	1.92	1.85

[a]Spectral acceleration values expressed as a ratio to gravitational acceleration g.
[b]The ratio $V(0.2\ s)/V(1.0\ s)$ is valid for an SFRS with $R_d \geq 1.5$; for smaller values of R_d this ratio is the same as the ratio $S_a(0.2\ s)/S_a(1.0\ s)$.

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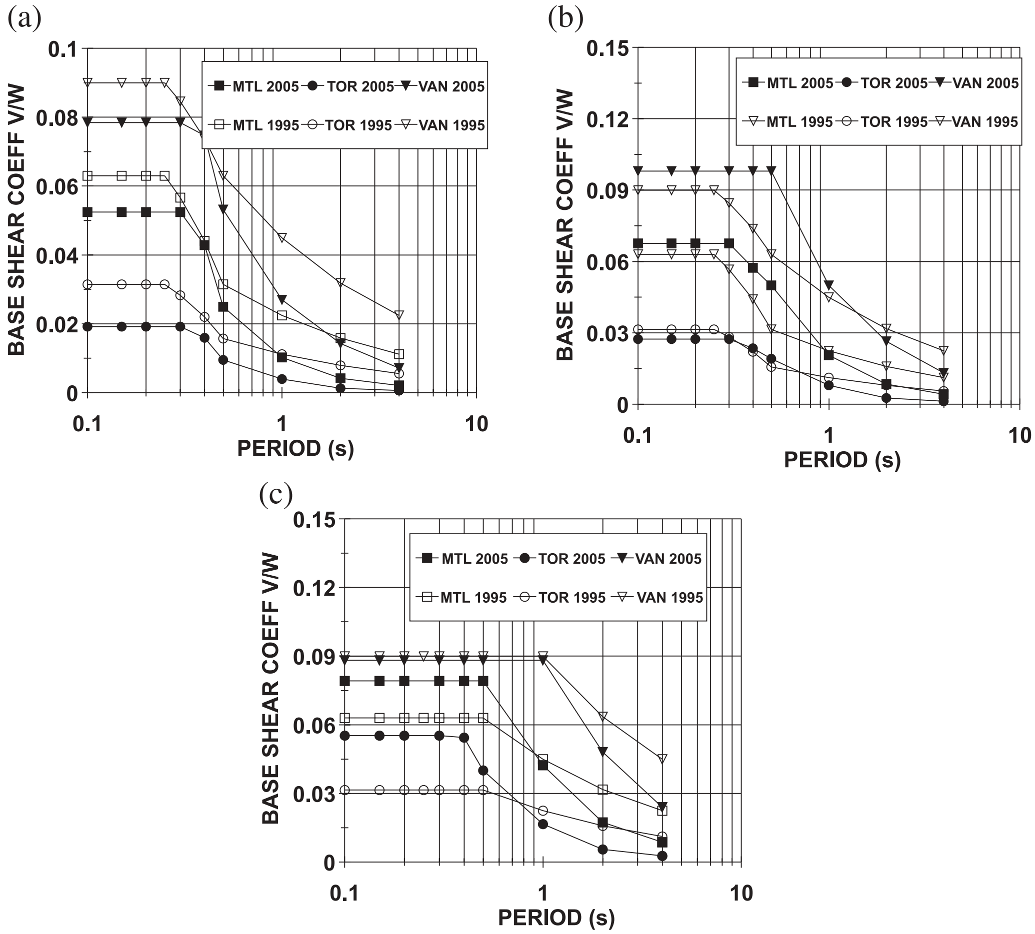
Fig. 3. Base shear coefficients for reinforced concrete ductile coupled wall (1995 NBCC case 11): (*a*) hard rock (site A); (*b*) very dense soil and soft rock (site C); and (*c*) soft soil (site E).



S(0.2 s)/S(1.0 s) for the same cities. This means that spectral shapes are now steeper, i.e., falling off much more rapidly from the short-period peaks to the long-period values. The implication is that, based on the actual calculated spectral accelerations, the design forces in short-period structures relative to those in long-period structures would be much larger than is the case for structures designed in accordance with the 1995 NBCC provisions.

When CANCEE reviewed this information, considerable concern was expressed as to the impact of these changes in spectral shape on the design of short-period structures; since such structures have traditionally not suffered much damage during earthquakes, it did not seem reasonable to require such dramatic increases in the design forces for them. It was therefore decided that the static equivalent base shear V for structures with some ductility (R_d of 1.5 or greater) need not be more than two thirds of the value that would ordinarily be required. In reaching this decision, it was also noted that the spectral shape is extremely steep in the short-period region (i.e., falling off by as much as 50% from 0.2 s to 0.5 s), which means that introducing this "cutoff" would result in less pressure on designers of such structures to calculate unrealistically high periods to reduce design forces.

The impact of the two-thirds cutoff factor is shown in Table 5 in the line for the parameter V(0.2 s)/V(1.0 s). These values now range from 1.40 to 3.39 and correspond more closely to the 1995 NBCC ratios of the spectral response factor.

The full impact of spectral shape changes for the three major cities is shown in Figs. 1–6. Consider in particular Fig. 3*b*, which shows base shear for ductile coupled reinforced concrete walls on the reference site condition C. The variation of base shear with period in Toronto using the 2005 NBCC provisions is very similar to that obtained using the 1995 NBCC provisions for periods of 1.0 s and shorter. In Montréal and Vancouver, however, the 2005 NBCC forces in this period range increase somewhat. The 2005 NBCC plateau in the short-period region is a direct consequence of the two-thirds cutoff factor; this plateau extends to 0.5 s in Vancouver and results in the largest force increases in the neighbourhood of that transition period. Long-period forces in the 2005 NBCC are generally lower than those in the 1995 NBCC because the fall off of spectral values with period is proportional to $1/T^k$; the exponent $k \geq 1$ in the 2005 NBCC, whereas $k = 0.5$ in the 1995 NBCC.

As discussed in more detail by Humar and Mahgoub (2003), a higher mode factor M_v is included in the calculation of the equivalent static force; this factor is 1.0 for periods of 1.0 s and shorter but has values greater than 1.0 for periods of 2.0 s and longer. It is a function of both the type of structural system (frame, braced frame, or wall) and the spectral shape. It has only a minimal effect on frames or

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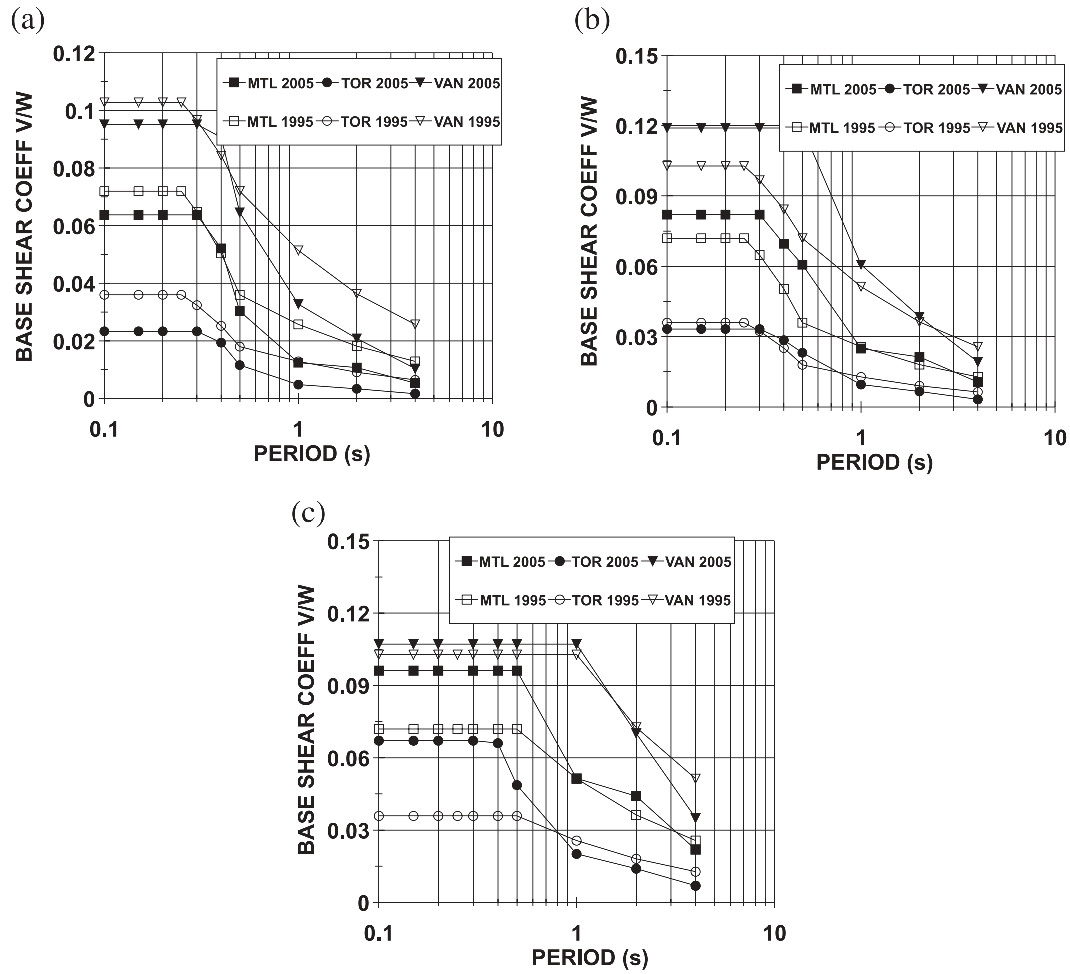
Fig. 4. Base shear coefficients for reinforced concrete ductile shearwall (1995 NBCC case 12): (*a*) hard rock (site A); (*b*) very dense soil and soft rock (site C); and (*c*) soft soil (site E).



coupled walls but can have a major effect on long-period forces in shearwall structures. Consider Figs. 4*b* and 5*b*, which show the base shear coefficients for ductile and moderately ductile reinforced concrete shearwalls on the reference site condition C. Both show a clear “knee” kink at a period 1.0 s for Montréal and Toronto; this kink is not present in Vancouver because the higher mode effect has very little effect when the fall off of spectral values (with increasing period) is lower, i.e., for smaller ratios of S_a(0.2 s)/S_a(1.0 s).

Probability level

As indicated previously, seismic hazard values used in the 2005 NBCC are calculated at a 2% in 50 year probability of exceedance, compared with a 10% in 50 year probability of exceedance in the 1995 NBCC. It is not feasible to evaluate the effect of changing the probability level by examining the design forces because of simultaneous changes in other parameters, including elimination of the level of protection experience factor U = 0.6. The impact of this change can best be observed by comparing 2% in 50 year and 10% in 50 year spectral acceleration values, both of which have been calculated and tabulated by Adams et al. (1999).

The lower part of Table 5 includes the 10% in 50 year spectral acceleration values at 0.2 s and 1.0 s for the same cities (Toronto, Montréal, Vancouver, and Prince Rupert) and the ratios of the 2% in 50 year to 10% in 50 year values at those same periods. Differences in those ratios represent the effect of probability change, i.e., if the ratios were more or less constant, then there would be no effect except possibly for an overall scale effect. There are significant differences, however, between the two western locations (ratios ranging from 1.85 to 2.03) and the two eastern locations (ratios ranging from 2.39 to 2.67). Although only these four locations are tabulated here, the same pattern of relative differences between eastern and western locations can be seen in the data for other locations as tabulated by Adams et al. (1999). These differences arise primarily due to differences in the seismotectonic environment, i.e., western locations being primarily near plate boundaries and eastern locations being intraplate, as noted earlier in the paper. The impact of probability level change on seismic design is that base shear in eastern locations increases (from the 1995 NBCC to the 2005 NBCC) in the neighbourhood of 25% due to this factor alone.

Site response effects

As noted in Table 4, the 1995 NBCC foundation factor F varies from 1.0 to 2.0; F is 1.0, 1.0, and 2.0 for the 2005

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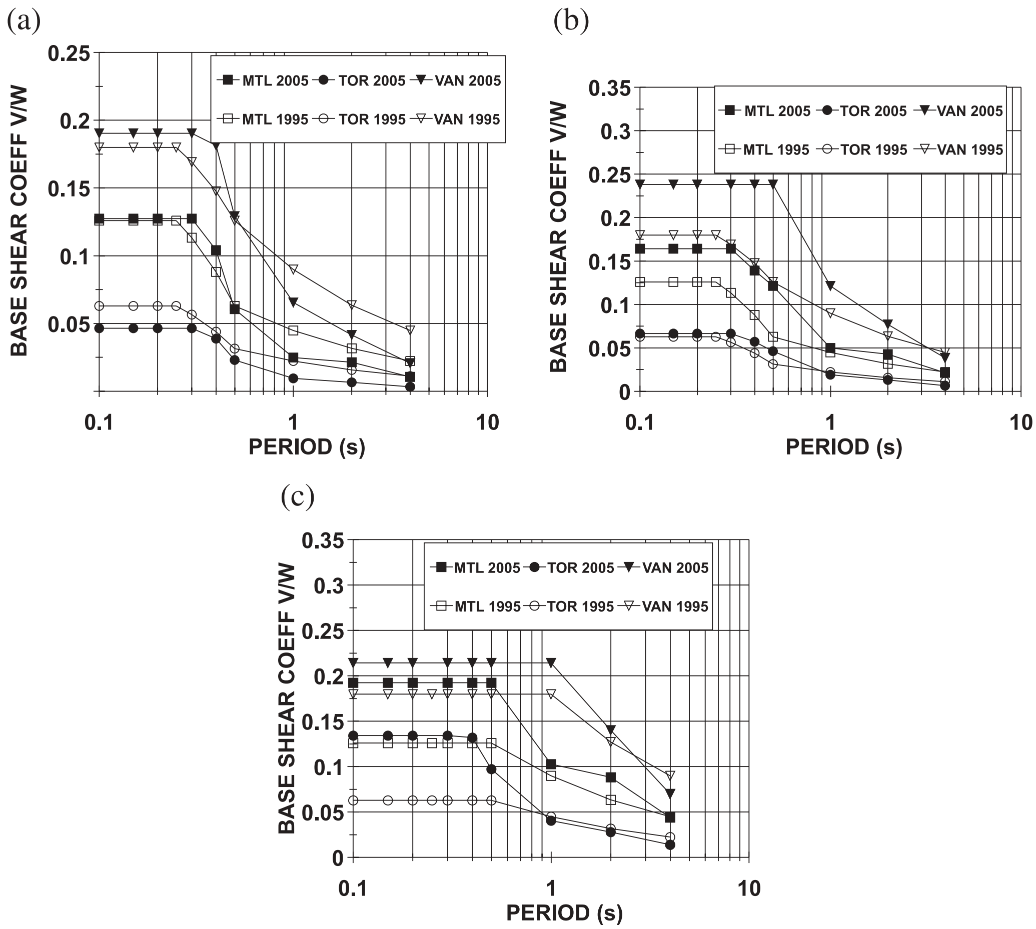
Fig. 5. Base shear coefficients for reinforced concrete moderately ductile shearwall (1995 NBCC case 14): (*a*) hard rock (site A); (*b*) very dense soil and soft rock (site C); and (*c*) soft soil (site E).



NBCC site categories A, C, and E, respectively. The 2005 NBCC site coefficients F_a and F_v for each site category are functions of the spectral accelerations $S_a(0.2\ s)$ and $S_a(1.0\ s)$, respectively. Values of F_a and F_v for category C are 1.0 for all spectral accelerations, since category C is the reference site condition. The impact of site response on changes in base shear coefficients can be seen by examining the three diagrams in any of Figs. 1–6. For the purposes of this discussion, consider Fig. 6, which shows the base shear coefficients for timber nailed shearwalls with wood-based panels.

In the 1995 NBCC, the short-period base shear coefficient is identical in each location for all three site conditions because of a cap on forces in short-period structures, due to the assumption that site amplification does not occur in the short-period region. That assumption is quite reasonable when the ground motions are strong but is incorrect for relatively weak ground motions. For example, the high ground motion in Vancouver results in F_a values (which apply to the short-period region) of 0.8 and 0.9 for site categories A and E, respectively (i.e., a slight deamplification both for hard rock and soft soil sites). The low ground motion in Toronto, however, results in F_a values of 0.7 and 2.0 for those two site categories; there is a significant deamplification on hard rock sites and near-maximum amplification on soft soil sites. Figure 6 confirms that the short base shear coefficients in Vancouver vary only to a minor extent between the three site conditions, but there is substantial variation of those coefficients in Toronto. As a result, although there is little change (from the 1995 NBCC to the 2005 NBCC) in short-period force levels on site category C in Toronto, those force levels nearly double for Toronto structures on site category E and are reduced significantly for structures on hard rock.

The changes in forces in long-period structures on soft soil sites (category E) are not very significant because F_v is 2.0 for weak ground motions and only decreases to 1.7 for the strongest ground motions; the factor F in the 1995 NBCC for this site category is 2.0. Figure 6*c* shows relatively modest changes between the 1995 NBCC and the 2005 NBCC force levels for structures located on soft soil sites. There is significant deamplification of long-period ground motions on hard rock sites, however, regardless of the strength of the ground motion, i.e., values of F_v for site category A range from 0.5 to 0.6. Consequently, as shown in Fig. 6*a*, long-period base shear coefficients in all three locations are substantially lower in the 2005 NBCC.

Type of lateral load resisting system

Leaving aside changes in restrictions because of triggering and irregularities, the changes arising due to the type of lateral load resisting system are entirely captured in the factors R_d and R_o as specified for each SFRS in Table 4.1.8.9 of the 2005 NBCC. As stated previously, the force modification

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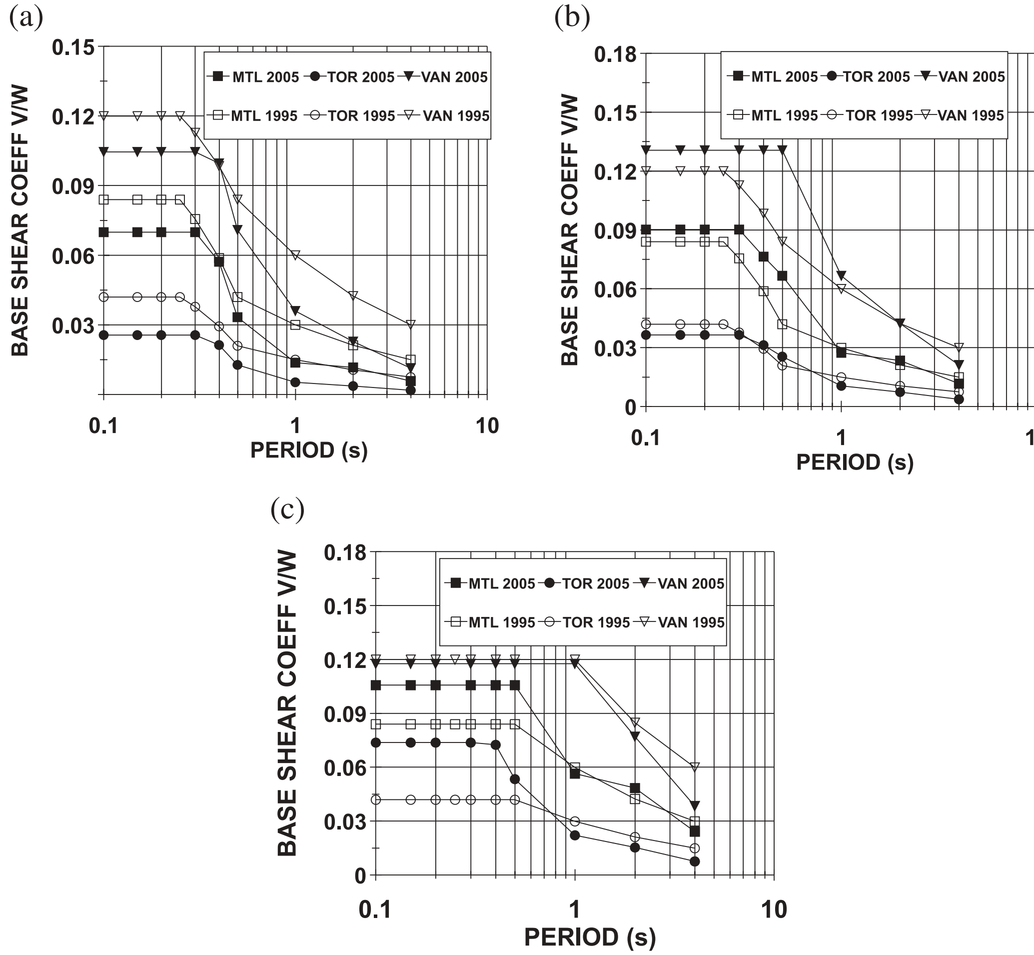
Fig. 6. Base shear coefficients for timber nailed shearwalls with wood-based panels (1995 NBCC case 16): (*a*) hard rock (site A); (*b*) very dense soil and soft rock (site C); and (*c*) soft soil (site E).



factor R_d for most systems is identical to the corresponding factor R in the 1995 NBCC. Consequently, changes in force levels for the different systems are primarily due to differences in the system overstrength factor R_o. The overstrength factor ranges from 1.5 to 1.7 for the most ductile systems, has a value of 1.0 for the most nonductile systems, and has values of 1.3 or 1.4 for conventional construction. Consequently, because of these differences, forces in ductile systems decrease (from the 1995 NBCC to the 2005 NBCC) by 30–40% relative to those in nonductile systems and by 10–30% relative to those in conventional systems.

Changes as noted previously can be seen by comparing Figs. 3*b* and 5*b*, i.e., reinforced concrete ductile coupled walls and shearwalls with nominal ductility, both on the reference site condition (C). The short-period forces in ductile coupled walls located in Vancouver (Fig. 3*b*) are about 10% higher in the 2005 NBCC than in the 1995 NBCC. However, Fig. 5*b* shows that the increase is about 35% for shearwalls with nominal ductility.

Conclusions

In this paper, the author has attempted to describe briefly the substantive changes being introduced in the seismic provisions of the proposed 2005 edition of the National Building Code of Canada (NBCC) and to present the major impact of these changes on the seismic level of protection provided to structures designed according to these provisions. In this context, seismic base shear is considered to be a simplified measure of the seismic level of protection, noting that many other aspects of the design and construction process have a significant influence on the actual level of protection.

The changes being introduced in the 2005 NBCC seismic provisions are very significant, both in concept and in their effects on the level of protection. Revisions in seismic hazard are the single major source of change; these arise from overall recalculation of hazard and from a change in format (from peak ground motion to spectral accelerations) and a change in the probability level at which the hazard is determined. Other significant changes arise because of inclusion of period-dependent site factors, a delineation of ductility-based reduction and overstrength factors for structural systems, defining irregularities in a systematic manner, and placing dynamic analysis as the normative method of analysis. The static equivalent force method can still be used in many circumstances; it is now based directly on the same design spectral values as those used in dynamic analysis and includes higher mode effects and some changes in period calculations. As detailed in the companion papers, the changes being introduced are based both on research and on

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experience, i.e., post-earthquake damage investigations in other countries.

The impact of the major changes on seismic base shear coefficients of the most common types of structural systems located in three cities having low to high levels of seismic hazard (Toronto, Montréal, and Vancouver) shows that, although there are some systematic influences, the reasons for the changes in design force for any particular structural system are complex. There are both significant increases and significant decreases in force levels, depending on the combination of circumstances. Although there are some deviations, the general trend is for forces to increase in the short-period region and decrease in the long-period region.

The complexity of the overall changes being introduced and the resulting apparent randomness of changes in force levels make it difficult to draw simple conclusions as to the change in the seismic level of protection. Because of the strong scientific and experiential basis for the individual changes, however, the overall effect is to provide for more consistency and uniformity of seismic level of protection throughout the country. When force levels increase significantly, there are good reasons for such increases to be required; similarly, decreases are also justifiable. Also, the delineation of situations for which static equivalent analysis can be used, with dynamic analysis being required for all other circumstances, means that design engineers are made aware of situations in which a simplified approach is not warranted and need to take additional care in seismic design, analysis, detailing, and construction.

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