**Ninth Canadian Conference on
Earthquake Engineering
Ottawa, Ontario, Canada
26-29 June 2007**

DEVELOPMENT OF NATIONAL BUILDING CODE OF CANADA (NBCC) SEISMIC PROVISIONS, PAST, PRESENT AND FUTURE

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ABSTRACT

This paper describes the development of the seismic provisions of the National Building Code of Canada (NBCC), beginning with the status of the NBCC as a model code. The major changes of seismic provisions, in terms of design forces, are traced from the 1953 to the 1995 editions of the NBCC. The changes from NBCC 1995 to NBCC 2005 are presented in detail, including numerical comparisons of seismic design forces for typical structures located in regions of high, moderate and low seismicity (Vancouver, Montreal and Toronto respectively). The implications of these changes for seismic design practice are presented in terms of level of protection, hazard format, hazard probability level and the seismic design process itself. This leads to a concluding discussion of issues which should be addressed in future editions of the NBCC, namely prescriptiveness, performance expectations, serviceability and design processes which recognize the differences between low and high seismicity regions..

Introduction – Status of NBCC

The National Building Code of Canada (NBCC) is a "Model Code"; its regulatory use is a provincial responsibility. The first edition in 1941 contained seismic provisions in an appendix, based on concepts presented in the 1937 United States Uniform Building Code (UBC); specific seismic provisions in the code proper did not appear until the 1953 edition. There have been ten editions since 1953, up to and including the edition which is currently in use, i.e. NBCC 2005 (Canadian Commission on Building and Fire Codes 2005). As has been the case with the evolution of most other building codes, there have been major changes in the seismic provisions during that period.

Since 1965, the overall responsibility for developing these seismic provisions has been the responsibility of the Canadian National Committee on Earthquake Engineering (CANCEE), which operates under the direction of the Associate Committee on the National Building Code, National Research Council of Canada (NRCC). In addition to doing the technical and editorial work involved in preparing those 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 composed of twenty members who bring together a variety of earthquake-related technical experience primarily in seismology, geotechnical engineering and structural engineering. Approximately

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40% of its members are engineering practitioners with the remainder working in universities 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 and soliciting responses to articles in the newsletter of the Canadian Association for Earthquake Engineering (CAEE).

History of NBCC Seismic Provisions

Major Changes 1953 to 1995

The history of the determination of seismic design forces from 1953 to 1995 is summarized in Table 1; this table 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.

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

<table>
<tr><th>NBCC edition</th><th>Nature of hazard information</th><th>Manner in which hazard information is used to determine seismic design forces</th></tr>
<tr><td>1953 through 1965</td><td>four zones (0,1,2,3) based on qualitative assessment of historical earthquake activity</td><td>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</td></tr>
<tr><td>1970</td><td rowspan="2">four zones (0,1,2,3) with boundaries based on peak acceleration at 0.01 annual probability of exceedance</td><td>base shear coefficient includes a non-dimensional multiplier (0 for zone 0, 1 for zone 1, 2 for zone 2 and 4 for zone 3)</td></tr>
<tr><td>1975 through 1980</td><td>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 zone3); value of seismic response factor is adjusted so that base shear is approximately 20% below that in NBCC 1970</td></tr>
<tr><td>1985</td><td rowspan="2">seven(0 to 6) acceleration and velocity related zones with boundaries based on 10 % probability of exceedance in 50 years</td><td>base shear coefficient includes zonal velocity "v" which is numerically equal to peak ground velocity in m/s (values are 0, 0.05, 0.10, 0.15, 0.20, 0.30 and 0.40); value of seismic response factor is adjusted by calibration process so that seismic forces are equivalent, in an average way across the country, to those in NBCC 1980 (Heidebrecht et al. 1983)</td></tr>
<tr><td>1990 and 1995</td><td>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); seismic response factor is modified to maintain same design force for highly ductile systems as in NBCC 1985</td></tr>
</table>

Several significant observations can be drawn from this table:

1. There has been a movement from general hazard zones which are not at all associated with ground motions to zones which are directly based on peak ground motion values.

2. After the introduction of ground motion parameters, there has been a change in the hazard methodology used to determine those parameters.

3. There has also been a change in the probability level at which the ground motion parameters have been determined.

While 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 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 NBCC 1970 to 1975 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 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 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 which 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 (GSC), is described and discussed by Adams and Atkinson (2003).

Major Changes 1995 to 2005

Rationale for Changing from 1995 Seismic Provisions

One of the major reasons for revising the NBCC seismic provisions is the ongoing improvement in the knowledge of seismic hazard and its geographical distribution throughout the country (Heidebrecht 1995). 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.

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 which lead to further code improvements. For example, the 1989 Loma Prieta earthquake demonstrated again 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 1994a).

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 as well as on research done elsewhere in the world.

A very much related reason for changing Canadian provisions is the need for our provisions to be responsive to the changes being made in the codes of other countries. We derive benefit from the experience and research which has been used to make changes in other codes; when analysis of such developments shows that the Canadian provisions could be improved, then they are adapted for use in the NBCC provisions.

Table 2 summarizes the major changes in various categories from NBCC 1995 (Associate Committee on the National Building Code 1995) to NBCC 2005. These are discussed below; a more detailed overview is given by Heidebrecht (2003).

Seismic Hazard Format

As noted in Table 2, both the seismic hazard format and probability of exceedance have been changed; seismic hazard has been recomputed using a so-called fourth generation hazard model (Adams and Atkinson 2003) which incorporates new knowledge from recent earthquakes, new strong ground motion relations, measures of uncertainty and a more systematic approach to reference site conditions. Because the spectral acceleration ordinates $S_a(T)$ (calculated on a uniform hazard basis) are being specified directly for each geographical location, design forces for the same structure will vary continuously rather than being the same within a seismic zone or changing abruptly over zonal boundaries, as is currently the case. Similarly, the shape of the spectrum, i.e. its variation with period, varies from location to location. The spectral values tend to fall off more rapidly with increasing period than the equivalent spectrum (i.e. amplified peak ground motions) in NBCC 1995. For example, the ratio $S_a(0.2)/S_a(1.0)$ ranges from approximately 2 to 3.5 in the southwestern plate boundary region and from 3 to 6 in the eastern intraplate region; the corresponding ranges of this ratio for the NBCC 1995 equivalent spectrum are 1.4 to 2 and 2 to 2.8 respectively. This feature has the impact of significantly increasing the design loads of short period structures relative to long period structures; this increase is somewhat ameliorated for structures of limited ductility or better by applying a 2/3 factor to short period loads.

Seismic Hazard Probability Level

The change in probability level was introduced to provide a geographically more uniform margin of safety against collapse; the proposed 2% in 50 year probability level is somewhat nearer to the expected probability of structural collapse or failure of structures designed and constructed in accordance with code provisions. The reason for making this change is that the slopes of the hazard curves (defined as the relationship between spectral acceleration and probability of exceedance) vary considerably between interplate and intraplate regions. For example the ratios of $S_a(1.0)$ at 2% in 50 year to values at 10% in 50 year probabilities are in approximately 2 in Vancouver and 2.8 in Montreal.

Table 2. Summary of Major Changes in Seismic Provisions, NBCC 1995 to 2005.

Category	NBCC 1995	NBCC 2005
Seismic hazard format	Zonal peak ground velocity and acceleration	Location-specific uniform hazard spectral acceleration values at 0.2, 0.5, 1.0 and 2.0s, with linear interpolation
Seismic hazard probability	10% in 50 years	2% in 50 years
Site effects	Single foundation factor F ranging from 1.0 to 2.0 for four foundation categories; short period force cap equiv to F of 1.0 for all sites	Site factors F_a and F_v with values dependant upon spectral accelerations at 0.2s and 1.0s respectively (direct adaptation of approach used by NEHRP*)
Vertical irregularities	No specific requirements	Six types defined with restrictions on method of analysis and design for each type
Torsion	Static torsional moments include amplified natural eccentricity and accidental eccentricity 0.1 x plan dimension; same accidental eccentricity added to 3D dynamic analysis	Torsional sensitivity defined on basis of ratio of max edge displ to ave displ; dynamic analysis required for torsionally sensitive structures; static method may be used for non-sensitive structures, with no amplification of natural eccentricity
Structural system force modification factors	Single factor R; values range from 1.0 (e.g. unreinforced masonry) to 4.0 (e.g. steel or RC moment-resisting frame)	Ductility related factor R_d (range 1.0 to 5.0) and system overstrength factor R_o (range 1.0 to 1.7); product R_dR_o for actual systems ranges from 1.0 to 7.5
Analysis	Equivalent static load prescribed; dynamic analysis permitted	Dynamic analysis prescribed (normally linear modal response or numerical integration); equivalent static load permitted as exception (e.g. low seismicity, most regular structures and short period irregular structures)
Level of Design Load (Calibration)	Level of protection factor U = 0.6 applied in determination of seismic load	No calibration but maximum seismic load limited to 2/3 of short period maximum for structures with R_d of 1.5 or higher, i.e. limited ductility or better

*National Earthquake Hazards Reduction Program (Building Seismic Safety Council 2001)

Site Effects

It has long been recognized that the amplification of seismic motions from rock to soil sites can be significant, especially for sites with soft soil conditions. The site factor approach adopted in NBCC 2005 is an adaptation of that used in NEHRP 2001 (Building Seismic Safety Council 2001) which is based largely on research done by Borcherdt (1994b). The substantive impacts of this change are to include: a) short period amplification, b) non-linearity of site amplification, i.e. amplification decreasing with increasing levels of rock motion, and c) de-amplification of seismic motions at rock or hard rock sites, i.e. those having shear wave velocities higher than that of the reference site condition, which is described as “very dense soil and soft rock”. Short period amplification occurs primarily on soft soils in regions of low seismicity and can increase ground motions by as much as a factor of 2; NBCC 1995 has no short-period amplification on soft soils because of a cap on short period force levels. Non-linearity has the effect of eliminating short period amplification on soft soil sites in regions of high seismicity and reducing medium to long period amplification by 20 to 40%. De-amplification for hard rock sites (shear wave velocities of 1500 m/s or more) can range from 20 to 50% depending upon period and the intensity of rock motion.

Irregularities

As noted in Table 2, NBCC 1995 has no specific requirements for vertical irregularities, although it does require that building design take into account the effect of setbacks; the commentary provides a few paragraphs describing setbacks and their effects. The significant effect of such irregularities on the performance of structures during earthquakes is recognized in NBCC 2005 by defining six types of irregularity (stiffness, mass, geometric, discontinuities (in-plane and out-of-plane) and weak storey) and specifying restrictions applicable to the different types. The kinds of restrictions include: analysis (i.e. requiring dynamic rather than static analysis), design (e.g. specific requirements associated with diaphragms, openings and discontinuities) and use (e.g. restrictions in use related to type and level of seismicity). One of the major use restrictions is the prohibition of weak storeys in regions of moderate to high seismicity.

The consideration of torsional effects for all structures continues to be a requirement but NBCC 2005 requires dynamic analysis for structures which are torsionally flexible, based on studies (e.g. Humar et al 2003) which show that a static approach cannot consistently represent torsional effects for such structures. Rather than requiring designers to compute the ratio of torsional to lateral period, a torsional sensitivity parameter B is introduced. This parameter is defined as the maximum value, in both orthogonal directions, of the ratio of edge displacement to average displacement in each storey when the static seismic load is applied at distances of ±0.1 x plan dimension from the centres of mass at each floor. A structure is deemed to be torsionally sensitive when $B > 1.7$ in which case dynamic analysis is required; otherwise torsional effects can be determined statically by applying torsional moments based on the natural eccentricity plus an accidental eccentricity of 0.1 x plan dimension. Accidental eccentricity must also be included when dynamic analysis is used.

Structural Systems

NBCC 1995 specifies a force modification factor R, which is equivalent to the maximum system ductility capacity, for a number of types of lateral-force-resisting systems for which the design and detailing requirements are specified in the steel, reinforced concrete, timber and masonry materials standards published by the Canadian Standards Association (CSA). The linkage to updated editions of these materials standards continues to be important in the NBCC 2005 requirements because of the significance of these design and detailing requirements in assuring that these systems have the properties associated with the specified force modification factors. As noted in Table 2, NBCC 2005 specifies both a ductility related factor R_d and an overstrength related factor R_o for each structural system; the product R_dR_o appears as a composite reduction factor in the denominator of the expression for calculating the seismic design force V. The introduction of R_o is intended to recognize the dependable portion of the reserve strength in the various structural systems; this is consistent with the use of seismic hazard at a lower probability of exceedance, as discussed previously. Mitchell et al. (2003) describe the components used to determine R_o and show the detailed calculations for the various values assigned to the different structural systems.

NBCC 2005 also includes height limits for structural systems having limited ductility when these are to be built in regions of high seismicity. The most common limit is 60 m although limits as low as 15 m are specified for so-called “conventional construction” in steel and concrete, i.e. buildings designed with no specific attention to ductility capacity. Also, NBCC 2005 prohibits very brittle structures such as unreinforced masonry in regions of moderate and high seismicity.

Analysis

As indicated in Table 2, NBCC 2005 specifies dynamic analysis as the “default” method of analysis, with static analysis permitted as an exception. Linear methods of dynamic analysis (either modal response or numerical integration time history) are specified although nonlinear dynamic analysis is permitted provided that a special study is performed. The input for the linear methods must conform to the site specific spectral acceleration values, i.e. either using these as response spectrum ordinates or using accelerograms having spectra which are compatible with such a spectrum. The dynamically determined base shear must be at least 80% of that determined using the static method for regular structures and 100% of the static value for irregular structures; these restrictions are intended to provide a safeguard against the use of structural models which are inadvertently much more flexible than actual structures. Of course, it is intended that the designer use the actual dynamic base shear if it is larger than the static value, which is likely to be the case for structures in which the higher modes dominate the dynamic response, e.g. tall long period structures.

The NBCC 2005 static method uses the spectral acceleration at the fundamental period of the structure to compute the elastic base shear coefficient. However, since spectral acceleration represents the maximum force in a single-degree-of-freedom system, a higher mode factor M_v is applied for structures with fundamental periods in excess of 1.0 s. That factor varies with period and with the type of lateral load resisting system; it can be as high as 2.5 for long period wall and wall-frame systems (Humar and Mahgoub 2003).

Level of Design Load (Calibration)

In this context, the value of the seismic design load is considered a proxy for the level of protection although there are several other factors which contribute significantly to the actual level of protection, e.g. maximum interstorey drift. NBCC 1995 deliberately calibrated the “average” seismic design load to that in the previous code by incorporating the multiplier U = 0.6 in the expression for determining that load.

As the provisions for NBCC 2005 were being developed, the consensus among the members of CANCEE was that it would be preferable, if possible, for the calculation of the seismic design load to be done rationally without resorting to a calibration factor. However, as studies were done to compare seismic design forces using the proposed code provisions with those determined in accordance with the NBCC 1995 provisions it became clear that the resulting increases in design forces for short period structures would be unacceptably large; in many situations such forces would be nearly doubled. There are several major reasons for such changes: a) the spectral shape, i.e. higher ratios of short to long period values and b) short period site amplification in regions of low to moderate seismicity, both of which have been discussed previously in this paper.

While dramatic increases in short period design forces could be explained, these go counter to experience during earthquakes which shows that it is very unusual for well designed short period structures to collapse, especially if they have even a limited amount of ductility capacity. As a consequence, NBCC 2005 limits the design force to 2/3 of the short period maximum value for systems having $R_d \geq 1.5$, i.e. in all but the least ductile structures.

Other codes also reduce seismic loads on the basis of experience. NEHRP 2001 (Building Seismic Safety Council 2001), which also uses hazard computed at a 2% in 50 year probability of exceedance, applies a factor of 2/3 for all structures at all periods on the basis of an experience-based estimated lower bound margin against collapse of approximately 1.5 inherent in structures designed in accordance with those provisions. The 1992 New Zealand Code of Practice (Standards New Zealand. 1992) includes a structural performance factor S_p = 0.67 in the static base shear expression; one of the arguments for this factor is that experience in past earthquakes indicates that, on average, buildings sustain less damage than would be predicted from simplified calculations.

Comparison of NBCC 1995 and 2005

Static Force Formulation

As noted earlier in this paper, NBCC 2005 seismic provisions call for dynamic analysis as the default method of analysis. However, static analysis can still be used in a large number of design situations, i.e. sites with relatively small ground motions, for regular structures less than 60 m in height and having fundamental periods less than 2 seconds and low rise short period irregular structures. Given these circumstances and the fact that static analysis was the norm in NBCC 1995, a comparison of NBCC 1995 and 2005 static force formulations can provide some significant insights into resulting changes in seismic design forces. Since it would be inappropriate to go into code level detail on these formulations, a simplified set of seismic force parameters for both codes is given in Table 3.

Static Lateral Force Equation

While the NBCC 2005 equation shown in Table 3 is more complicated than the corresponding NBCC 1995 equation, the parameters in the equation are more rationale and have a direct equivalence to dynamic analysis parameters. The product S(T) W represents the force exerted on a single-degree-of-freedom (SDOF) system by seismic motions on the specified site conditions. The other parameters modify that force to take into account importance, multi-degree-of-freedom effects, and response modification due to inelastic behaviour and overstrength.

Level of Protection Experience Factor

NBCC 1995 includes an experience factor U = 0.6 which is essentially a calibration factor to maintain design loads at approximately the same level as previous editions of NBCC. There is no equivalent calibration factor in NBCC 2005, i.e. design forces correspond to what would be determined by rationale analysis for seismic hazard at the 2% in 50 year level. However, the application of the NBCC 2005 static force equation without a calibration factor results in very high values of V at short periods, particularly when compared with the forces determined using the NBCC 1995 equation. These high values of V arise for three reasons:

a) hazard being computed at a lower probability level, i.e. 2% in 50 years rather than 10% in 50 years,
b) use of hazard spectra determined directly from seismic hazard computations; NBCC 1995 used zonal values of peak ground motions and the short period amplification of those were too low, and
c) NBCC 1995 put a cap on the short period site amplifications on soft soils equivalent to F = 1, whereas actual short period amplifications, as observed in real earthquakes, are expected to much higher.

While the increased short period design forces are rational, as outlined above, experience has demonstrated that short period structures, with even limited ductility, are rarely damaged. Short period structures are inherently stiff and, for those with even limited ductility, are not likely to reach deformation levels which would cause significant damage. Consequently, based on the foregoing, NBCC 2005 includes a short period factor of 2/3 for all structures except those which are brittle, i.e. no reduction for $R_d < 1.5$.

Importance Factor

Importance factors in NBCC 1995 and 2005 are essentially the same with the exception that NBCC 2005 includes a factor of 0.8 for structures of low importance, e.g. minor storage buildings or low human-occupancy buildings for which collapse is not likely to cause injury or other serious consequences.

Table 3. NBCC 1995 and 2005 static force parameters.

Parameter	NBCC 1995	NBCC 2005
Static lateral earthquake force	$V = U\ (V_e / R)$ $V_e = v\ S\ I\ F\ W$	$V = S(T)\ M_v\ I_e\ W / (R_d\ R_o)$ $S(T) = F_a\ S_a(T)$ or $F_v\ S_a(T)$ T = structural fundamental period
Level of protection experience factor	U = 0.6	None; however, design loads for non-brittle ($R_d \leq 1.5$)short period structures reduced by 1/3
Importance factor	I = 1.0, 1.3 or 1.5	I_e = 0.8, 1.0, 1.3 or 1.5
Foundation or site factor	F = 1.0, 1.3 or 1.5	F_a and F_v based on site class and ground motion intensity $0.7 \leq F_a \leq 2.1$ $0.5 \leq F_v \leq 2.1$
Seismic hazard parameter	Zonal velocity “v” at 10% in 50 year probability of exceedance	$S_a(T)$ = 5% damped spectral acceleration at 2% in 50 year probability of exceedance
Site response factor	S is a function of period T and zonal acceleration “a”	Equivalent to $M_v\ S_a(T)$
Higher mode factor	Included in long period shape of $S \propto 1 / \sqrt{T}$	M_v is a function of T, type of structural system and shape of $S_a(T)$ $0.4 \leq M_v \leq 2.5$
Force modification factor	$1.0 \leq R \leq 4.0$	Factor is product of $R_d\ R_o$ $1.0 \leq R_d \leq 5.0$ $1.0 \leq R_o \leq 1.7$

Foundation or Site Factor

NBCC 1995 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.

The non-linear period-dependent site effects described earlier are represented in NBCC 2005 by the acceleration-based site coefficient F_a and the velocity-based site coefficient F_v. The theoretical basis for this approach and the methodology used to obtain the specific values are described in Commentary J of NBCC 2005. The site classes which are used to determine the values of these coefficients require a quantitative evaluation of site conditions, normally the determination of the average shear wave velocity in the top 30 m.

Both F_a and F_v have values below 1.0 for sites which are harder than the reference site condition (very dense soil and soft rock); the lowest values are for low levels of seismic hazard. As a consequence, force levels for buildings built on hard rock are as low as 50% of the value for comparable buildings on the reference site condition.

The maximum values of F_a and F_v are both 2.1 for buildings located on soft soils and subjected to low seismic ground motions. However, the values of these coefficients are reduced for high levels of seismic hazard.

Seismic Hazard Parameter

Replacing the zonal acceleration and velocity values by period-dependent spectral acceleration $S_a(T)$ is the most significant change from NBCC 1995 to 2005. Because the spectral accelerations in NBCC 2005 are determined from a new seismic hazard model, as discussed previously, changes in design forces due to changes in seismic hazard are complex and not amenable to simple comparisons. However, the following aspects are particularly pertinent for the calculation of seismic loads:

a) the geographical distribution of seismic hazard is now continuous rather than based on zonal values; as a consequence, there are no longer step changes of seismic design forces across zonal boundaries, which were particularly problematic when those boundaries passed through urban areas,

b) period-dependent spectral acceleration ordinates calculated at the same probability of exceedance, i.e. uniform hazard spectra, provide a much better representation of earthquake effects on structures with different periods than the multiplication of a peak ground velocity by a seismic response factor.

Site Response Factor

The site response factor S in NBCC 1995 was used to estimate structural response, i.e. the product “Sv W” represented the maximum dynamic force in the structure for a specified value of “v”. While S varies with period T, including several low-period branches to recognize differences in the “a/v” ratio, it did not recognize the significant differences which can occur in responses for different kinds of structures.

In NBCC 2005, the spectral acceleration $S_a(T)$ multiplied by W represents the maximum dynamic force in a SDOF system, so that no modification would be required if the actual building is a SDOF system. However, since most building structures are actually multi-degree-of-freedom (MDOF) systems, NBCC 2005 includes a higher mode factor M_v to include those effects. In practice, while M_v is a function of the type of structural system, the period T. and the period-dependency of $S_a(T)$, its value is 1.0 for all structures with periods of 1 s or lower and is only slightly above 1.0 for many other circumstances. It only has a significant effect for long period wall or wall-frame structures, for which its value can be as high as 2.5 in Eastern Canada.

Force Modification Factor

NBCC 1995 included a single force modification factor R to reduce seismic forces to take into account the beneficial effects of structures being able to undergo inelastic deformations without significant loss of capacity. The value of R varied from 4.0 for the most ductile structural systems to 1.0 for structural systems with little or no ductility.

As discussed previously, NBCC 2005 uses two force reduction factors (R_d and R_o) whose product ranges from 1.0 to 7.5. As a consequence, the total effective force reduction for structural systems with the most ductility and the largest amount of dependable overstrength (e.g. ductile moment-resisting steel frames) relative to the least ductile systems having little or no dependable overstrength is almost twice that in NBCC 1995.

Design Force Comparisons

The effects of changes in seismic provisions on design forces can be visualized most clearly by comparing design forces for a few specific situations. Figures 1 to 4 show the period dependency of base shear coefficients for several common types of structural configuration in locations with varying levels of seismicity, i.e. Vancouver (high seismicity), Montreal (moderate seismicity) and Toronto (low seismicity). A range of site conditions is also included in these comparisons. In all of these figures, the thin lines show the NBCC 1995 values and the thick lines the NBCC 2005 values.

Ductile RC Walls – Dense Soil and Soft Rock Sites – Toronto, Montreal and Vancouver

Figure 1 shows the changes in base shear coefficient for ductile RC wall structures located on the reference site condition (dense soil and soft rock) in three cities with different levels of seismicity. Base shear coefficients are comparable in the short period (i.e. T less than 0.5s), primarily because NBCC 2005 includes the 2/3 short period “cap” which was discussed previously; without that adjustment, the NBCC 2005 values would have been 50% larger.

In the intermediate period range (i.e. T between 0.5 and 1.0s), NBCC 2005 values are significantly larger than NBCC 1995 values in Vancouver but as one moves to a low seismicity location (Toronto), the values become similar throughout this period range. In the long period range (i..e. T greater than1.0s), the NBCC 2005 values are smaller than the NBCC 1995 values and become quite low for very long periods. This change in period-dependency arises because the NBCC 2005 values are based on actual seismic hazard calculations whereas the NBCC 1995 period dependency in the long period range is simply proportional to $1/\sqrt{T}$.

Conventional Steel Frames – Dense Soil and Soft Rock Sites – Toronto, Montreal and Vancouver

Figure 2 is similar to Figure 1 except that this figure is for steel frames of conventional construction; the major difference is that the ductility-related force modification factor R_d is now 1.5, compared with 3.5 for ductile reinforced concrete walls. Except for the longer periods, NBCC 2005 force levels are significantly larger than NBCC 1995 levels in the moderate and high seismicity locations of Montreal and Vancouver. Values are comparable in Toronto in the short and intermediate period ranges, but the NBCC 2005 values are significantly lower in the long periods, primarily for the reason outlined in the discussion of Figure 1.

Ductile RC Walls – Range of Site Conditions – Vancouver

Figure 3 shows changes in force levels for ductile reinforced concrete wall structures located on three different site conditions in Vancouver, i.e. hard rock, soft rock/dense soil, and soft soil. In NBCC 1995, there is no distinction between soft rock/dense soil and hard rock; the reason for including this distinction in NBCC 2005 is that hard rock actually deamplifies ground motion (in comparison with motions on the reference site condition, soft rock/dense soil). That deamplification ranges from 25% to 50%, depending upon the seismic hazard level and the period range. While the comparisons for soft rock/dense soil are as previously discussed in Figure 1, the effect of deamplification means that NBCC 2005 design forces on hard rock sites are lower than the NBCC 1995 values at almost all periods, and are substantially lower in the low and high period ranges.

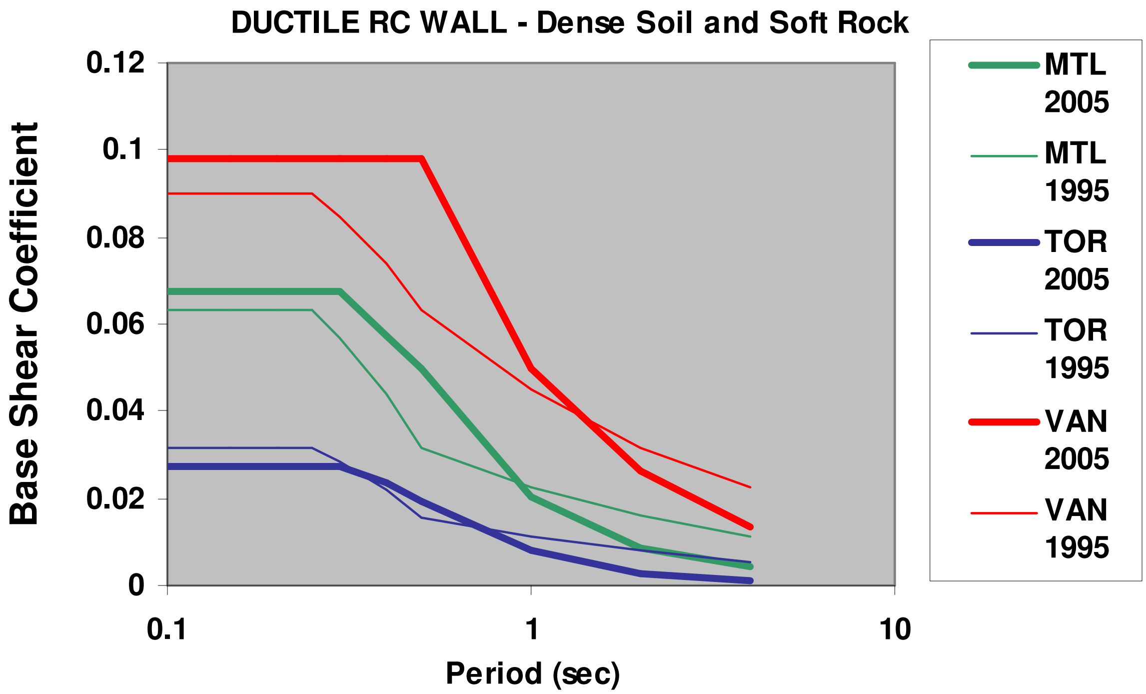

Figure 1. Base Shear Coefficient Comparisons for Ductile Reinforced Concrete Wall Structures Located on Dense Soil and Soft Rock Sites.

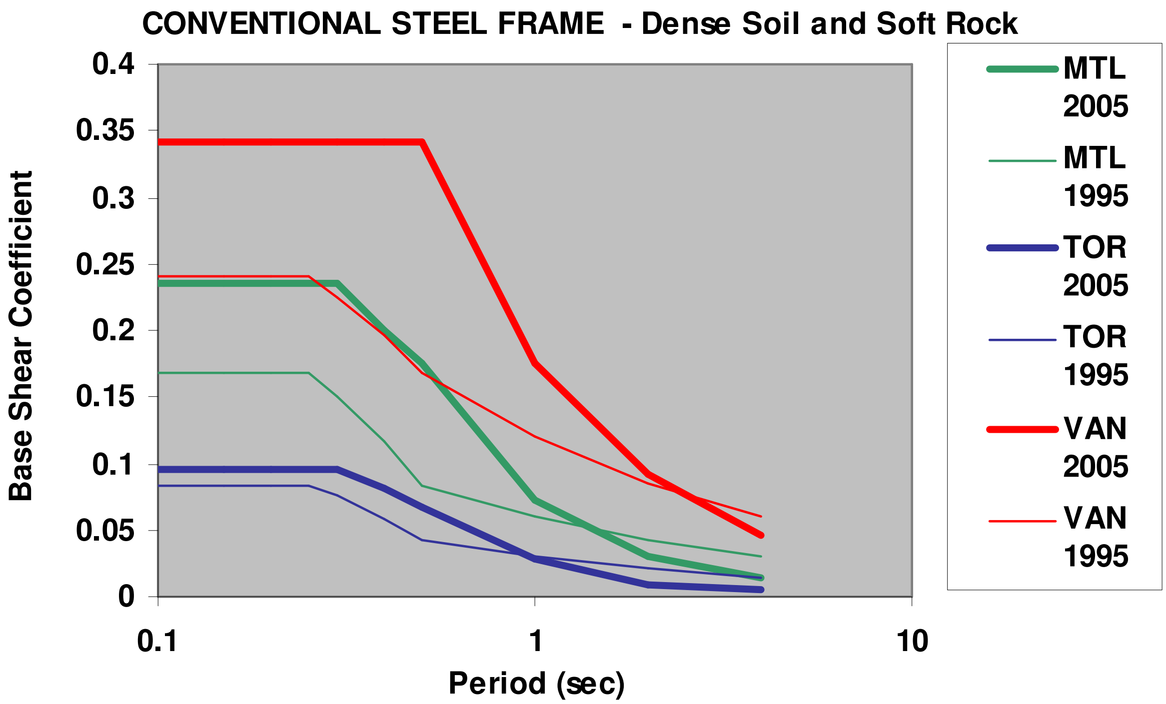

Figure 2. Base Shear Coefficient Comparisons for Conventional Steel Frame Structures Located on Dense Soil and Soft Rock Sites.

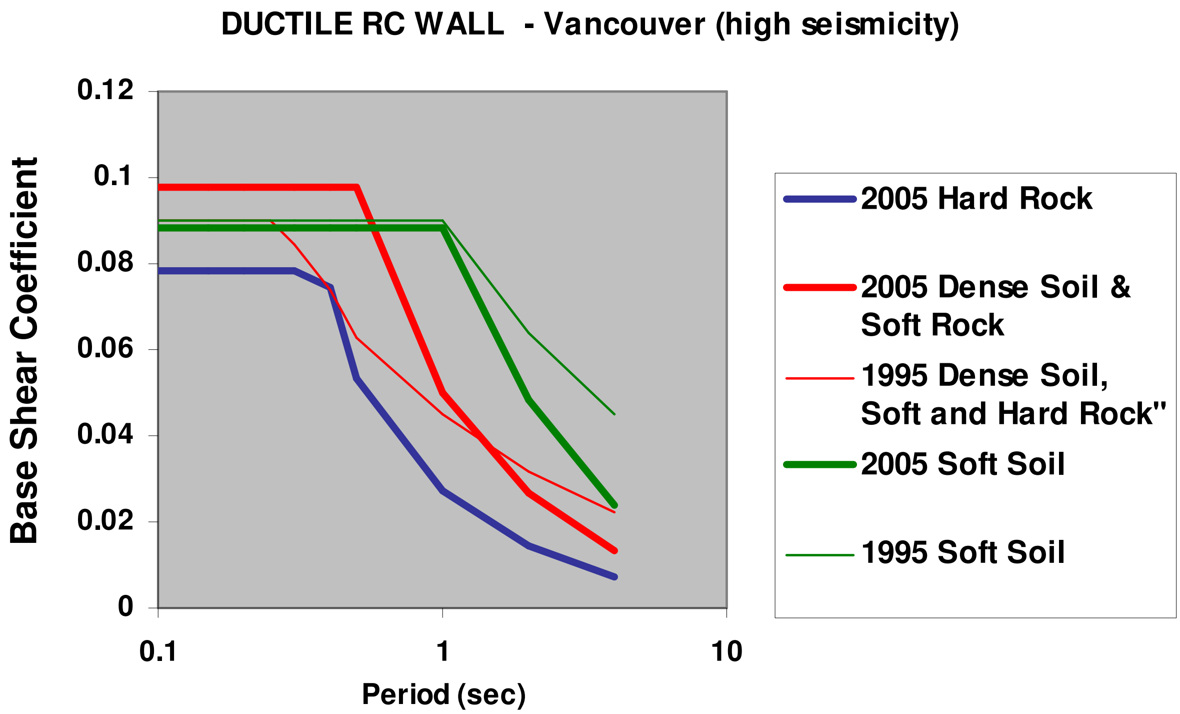

Figure 3. Base Shear Coefficient Comparisons for Ductile Reinforced Concrete Wall Structures in Vancouver Located on a Range of Site Conditions.

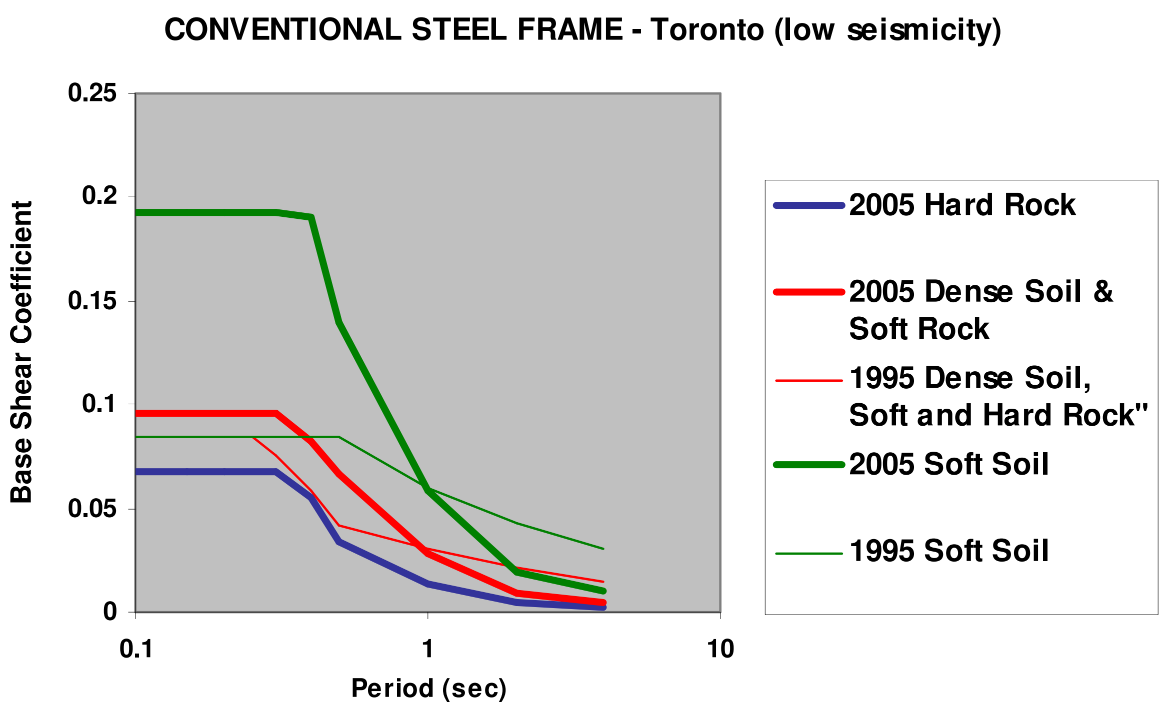

Figure 4. Base Shear Coefficient Comparisons for Conventional Steel Frame Structures in Toronto Located on a Range of Site Conditions.

For soft soil sites, the NBCC 2005 and NBCC 1995 values are essentially the same in the low and intermediate period ranges, but the NBCC 2005 values are much lower for longer periods. This effect occurs because soft soil motions for strong ground motions (i.e. at high seismicity locations) are lower than those on the reference ground condition; the reasons for this are described in the NBCC 2005 Commentary.

Conventional Steel Frames – Range of Site Conditions – Toronto

Figure 4 shows changes in force levels for conventional steel frames located on the same three different site conditions in Toronto. Comparing force levels in Toronto for this type of structure is important because limited ductility capacity structures are more likely to be used in a low seismicity location.

The effect of the including the hard rock site category in NBCC 2005 has the same effect as discussed above, i.e. reducing forces to levels which are below those in NBCC 1995 for all periods.

However, in this case, the low and intermediate period design forces on soft soil sites in NBCC 2005 are much larger than those in NBCC 1995. This increase occurs because there is significant amplification on soft soil sites for all periods whereas NBCC 1995 did not include any amplification in the short period range. This amplification is particularly significant for low levels of ground motion, i.e. at locations with low seismic hazard.

Implications of NBCC 2005 Provisions for Seismic Design

Seismic Level of Protection

While several other factors have a significant impact on level of protection (e.g. quality of construction), in this context the level of design base shear is taken as a proxy for level of protection. The changes listed below are deemed to have the most significant effect on level of protection. Other changes also have some impact, but none on their own are likely to match those listed here.

a) The use of more rational site factors, recognizing short period amplifications and non-linearity, improves the level of protection; also the use of site categories which are defined quantitatively should improve the consistency in classifying sites.

b) The specification of the different types of irregularity, including torsional sensitivity and the corresponding restrictions on analysis, design and use, reduces the vulnerability of irregular structures.

c) The explicit delineation between structural types on the basis of minimum overstrength results in a relative range of force reduction of 7.5 in NBCC 2005 compared with a range of 4.0 in NBCC 1995; the relative increase of design loads for the less ductile structural systems reduces the vulnerability of such structures.

d) The impact of changes in seismic hazard is probably the largest single factor in improving the level of protection, the effects of format and probability level are described below.

Seismic Hazard Format and Probability Level

NBCC 2005 makes explicit use of computed spectral acceleration ordinates rather than pseudo-spectra created by amplifying peak ground motions as was done in NBCC 1995. The major impact of this change is that the spectral shape, i.e. the variation of the spectral acceleration with period, has changed significantly. This can be seen in Figure 5 which shows NBCC 1995 and 2005 spectra normalized to the 1.0s value for the reference site condition (soft rock/dense soil) in moderate and high seismicity locations, i.e. Montreal and Vancouver respectively. The NBCC 2005 spectra are much steeper in terms of the ratio of short period (0.2s) to intermediate (1.0s) values. For Vancouver, the normalized short period value in NBCC 2005 is 1.5 times the NBCC 1995 value. That ratio is nearly 1.8 for Montreal. However, the 2/3 factor applied to short period design forces for structures with some ductility largely sets off those changes so the net effect is to increase short period design loads for non-ductile systems.

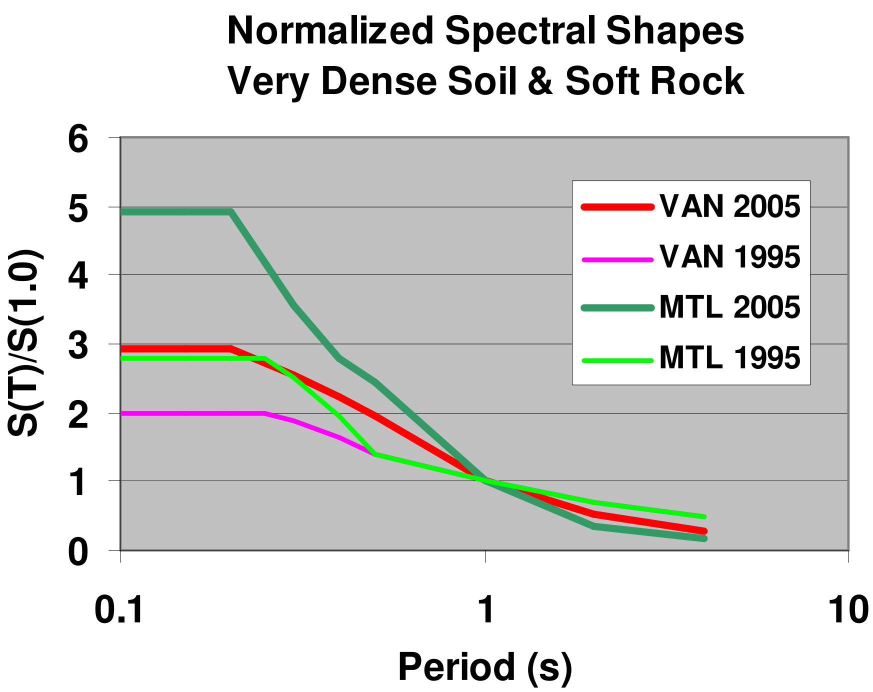

Figure 5. NBCC 1995 and 2005 Normalized Spectral Shapes for Vancouver and Montreal.

Figure 6 shows the hazard curves at T = 1s for Vancouver, Victoria, Montreal and Vancouver; these are normalized to the value at an annual exceedance probability of 0.0021, i.e. corresponding to a 10% in 50 year probability of exceedance; this was the probability level used in the NBCC 1995 seismic hazard parameters. For the reason described earlier in this paper, the seismic hazard parameters in NBCC 2005 have been determined at the 2% in 50 year probability level, i.e. an annual probability of approx 0.0004.

As shown in this figure, this change has the major effect of improving the level of protection in eastern intraplate locations because of the distinct differences in hazard curve shape. The ratio of 2% in 50 year values to 10% in 50 year values is about 2 for Vancouver and Victoria but increases to around 2.5 for intraplate locations such Montreal and Toronto.

Seismic Design Process

As stated earlier in this paper, the membership of CANCEE includes engineering practitioners as well as individuals in academia and government. In addition, groups of engineering designers in the more seismically active cities in the country, e.g. Vancouver and Montreal, regularly contribute to the code development process either by functioning as local subcommittees or by acting as more informal sounding boards for proposed code changes. The following comments are based on input obtained at several distinct stages during the development of the NBCC 2005 provisions as well as discussions with several of the engineering practitioner members of CANCEE concerning the implications of the seismic code changes on the seismic design process. The context of these comments is that, as a rule, designers do not like to see code changes, especially if they do not perceive them to be essential.

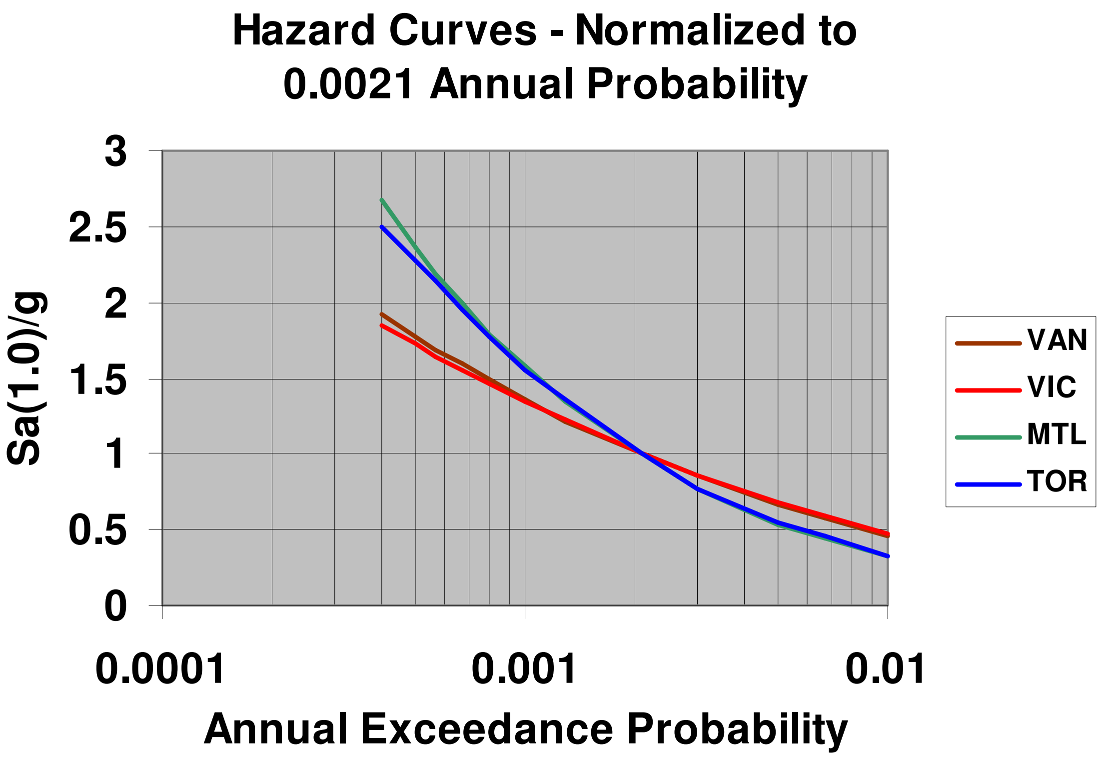

Figure 6. Normalized Hazard Curves for Vancouver, Victoria, Montreal and Toronto.

The NBCC 2005 provisions make the seismic design process more complex in several respects, even when the static analysis option is allowed. The location-specific nature of seismic hazard means that a designer will need to determine the spectral acceleration values for each project rather than determining the zones which are applicable to the particular geographical area, as was the case for NBCC 1995. The determination of the site factors F_a and F_v requires testing and analysis to establish the site class and interpolation to determine the factors associated with the location-specific spectral acceleration values. The other complicating factor in the static seismic load expression is the determination of the higher mode factor, which is a function of the type of structural system, the period of the structure and the spectral shape (i.e. the ratio $S_a(0.2)/S_a(2.0)$)

There are of course both pros and cons to this additional complexity. Designers who develop an appreciation for the nature of seismic response of structures appreciate the reasons for using a spectral shape which is location-specific and understand the need for intensity-dependent site factors. Such knowledgeable designers also appreciate the flexibility which goes with this additional complexity, e.g. the rewards and penalties associated with the choice of systems with different ductility capacities and the benefits of being able to compute fundamental structural periods using alternate methods. Such designers can also be expected to understand the dynamic nature of seismic response and appreciate the need for dynamic analysis in many circumstances; in many cases they will be as familiar with the "normal "dynamic analysis, i.e. the linear modal response spectrum method, as with static analysis.

On the other hand, many designers perceive seismic design requirements as a design "hurdle", either because such requirements do not often govern actual design or because they simply do not recognize the need for seismic protection. Such designers may well find the additional complexity of the NBCC 2005 provisions to be unduly onerous. However, even knowledgeable designers in high seismicity regions have concerns about the growing complexity of code provisions. The following is a brief list of concerns arising from complexity, not necessarily in order of priority:

a) The use of uniform hazard spectral ordinates does not provide the same “feel” for seismic hazard as was the case when seismic zones were used.

b) The non-linear variation of site factors with spectral ordinates, including de-amplification on hard-rock sites, is puzzling in comparison with a foundation factor which is uniform for each site category and whose value increases as soil sites become softer.

c) Complexity makes it difficult for a designer to detect errors since the final product, e.g. seismic design load, is dependent upon many more interrelated factors; designers may be prone to seeing the process as “number-crunching” rather than as one based on rational engineering.

d) Dynamic analysis, which is required for many more design situations, is not familiar to many designers; they may simply rely on the output of dynamic analysis options in structural analysis computer software without having the experience or judgment to recognize errors.

In general, the additional complexities are likely to be justifiable for seismic design in regions of high seismicity but are questionable for design in other parts of the country. Designers also have “credibility” concerns; these tend to be more common in regions of low seismicity:

a) The use of the 2/3 short period factor appears to be arbitrary and not based on any recognizable rationale, even in comparison with the “calibration” factor U = 0.6 in NBCC 1995.

b) Even with the above 2/3 factor in place, short period forces in regions of low seismicity are often significantly larger than in NBCC 1995; this result is often not seen as credible in areas which have never suffered a damaging earthquake, at least in the recorded history of less than 400 years.

c) While the move from seismic zones to location-specific values will remove the sharp changes of design load across zonal boundaries, designers tend to be skeptical of the validity of rather steep gradients of spectral acceleration values over short distances in some regions of the country.

d) The severe restrictions on post-disaster buildings, e.g. prohibiting weak storey irregularity throughout the country, is perceived as “overkill” in regions of low seismicity; such a restriction prohibits the construction of hospitals with large bottom storey openings in cities such as Winnipeg, which is located in the stable aseismic central region of Canada.

While the NBCC 2005 commentary provides quite detailed explanations and reasons for the changes from NBCC 1995, it is doubtful that these are sufficient to overcome all of the complexity and credibility concerns. Consequently, in order to ensure proper use of the new seismic provisions and for the improvements in the level of protection to be achieved, it is important that there be ongoing educational opportunities for designers, including dialogue between designers and educators/researchers who contribute to code development.

Issues for Future Development of NBCC Seismic Provisions

The author’s participation in the development of the NBCC seismic provisions since the late 1960s has stimulated thinking about the role of seismic codes and their current and future development. This part of this paper raises some of the issues which have arisen without attempting to suggest particular solutions or directions; these issues are applicable to seismic code development in other parts of the world as well.

It is important to note that NBCC 2005 continues to be a model code but the format has being changed to that of an “objective-based” code. The fundamental objectives of the code (e.g. structural safety) and a number of specific functional requirements (stated in qualitative terms) are stated in Division A Compliance, Objectives and Functional Statements. The NBCC 2005 seismic provisions discussed in this paper are included in Division B Acceptable Solutions, which corresponds to the full content NBCC 1995 (and previous editions of NBCC). Division A of NBCC 2005 states that compliance can be achieved by using the acceptable solutions in Division B or by using alternate solutions that will achieve at least the minimum level of performance required by Division B.

How Prescriptive?

The seismic provisions of NBCC 1995 and previous editions tended to be quite prescriptive with regard to aspects such as: calculation of the static seismic load, the distribution of that load with height and calculation of torsional moments. A few other aspects were handled more generally, e.g. stating that building design shall take into account the possible effects of setbacks without specifying how that should be done. The NBCC 2005 provisions, while somewhat more complex, continue that trend, including some additional prescriptive requirements, e.g. concerning the input for dynamic analysis. In NBCC 1995, the commentary included a recommended approach for certain matters on which the code is non-prescriptive, e.g. for determining P-Delta effects. The NBCC 2005 commentary continues that approach as well as providing considerably more explanatory material, e.g. with regard to seismic hazard and site response effects. The commentary, including particularly the additional explanatory material, is likely to be used by designers who take the “alternate solutions” approach to compliance as the basis for justifying that the alternate solution achieves at least the minimum level of performance of the NBCC 2005 seismic provisions.

It is not clear that the movement to an objective-based code in itself will have a particular steering direction on the prescriptiveness of future seismic provisions. The steering is likely to come from regulatory bodies (e.g. those who approve building plans on behalf of a municipality are likely to prefer more prescriptive provisions because it is easier to check whether a design meets code requirements) and from design engineers (e.g. knowledgeable designers are likely to prefer less prescriptive provisions so that they have more flexibility in choosing how to meet stated performance requirements). In theory, the objective-based code approach should satisfy both ends of the spectrum by providing a more prescriptive alternative for those who prefer that route and by allowing alternate solutions which meet the stated objectives and functional requirements. If that is the case, then it may well be that the seismic provisions, as the “acceptable solution” will continue to be quite prescriptive.

Performance Expectations

Traditionally, performance expectations associated with use of NBCC seismic provisions have not been included in the code and are only stated in very general terms in the commentary. The expected performance of structural systems with different levels of ductility capacity when subjected to the design ground motions are implicit for those involved in developing the code and for a few very knowledgeable designers. One can understand the reluctance of codes to state performance expectations explicitly, given the possibility or even likelihood that these could in the future be the basis for litigation. Performance-based engineering approaches, e.g. such as developed by the Structural Engineers Association of California (Vision 2000 Committee 1995) have gained prominence in seismic design and it is likely that codes will need to reflect that trend, which includes more explicit performance objectives than is the case in the NBCC 2005 and previous editions of NBCC. The NBCC 2005 specification of seismic hazard at the 2% in 50 year probability level and the inclusion of an overstrength-related force reduction factor provides a fairly clear, albeit still somewhat implicit, understanding that these seismic provisions are associated with near-collapse performance. The NBCC 2005 commentary contains a more explicit set of statements concerning seismic design objectives and expected performance than previous commentaries.

Serviceability

The primary objective of the NBCC seismic provisions is safety, i.e. the reduction of the loss of life through prevention of serious damage or collapse of building structures. While the serviceability is included in the broad objectives of building design (i.e. in the objectives concerned with fire and structural protection of buildings), earthquakes are treated as rare events so the code provisions do not include serviceability limit state requirements for seismic loading. This is in marked contrast to other codes, e.g. the 1992 New Zealand Code (Standards New Zealand 1992), which include specific serviceability limit state design requirements.

While serviceability is not an explicit objective in the NBCC seismic provisions, it should be noted that it is recognized implicitly in several respects. First, the use of interstorey drift limits, while applied to deflections computed at the design load level, have a significant role in ensuring serviceability at lower loads, i.e. associated with ground motions at higher probabilities of exceedance. Second, for post-disaster buildings, the significantly lower drift limit (i.e. 1% of interstorey height) is intended to permit such buildings to remain functional during and after design level ground motions. Nevertheless, the need and desirability for more explicit serviceability requirements for all buildings will need to be considered in future NBCC seismic provisions.

Design Processes for Low and Moderate Seismicity (LMS) Regions

The NBCC seismic provisions, in the 2005 edition and in previous editions, are based predominantly on the needs and concerns of high seismicity regions. Various code provisons, e.g. large force reductions for structures with high ductility capacity, explicitly or implicitly arise from codes, practice and experience in regions of high seismicity. In particular, most lessons learned from damage during earthquakes have been the result of investigations of moderate to large earthquakes occurring in regions of high seismicity.

However, while some level of seismic protection is necessary in low to moderate seismicity (LMS) regions, the design processes which would be best for those situations are not necessarily the same as for high seismicity regions. This is particularly important for the NBCC, which is a model code for the entire country but in which a large proportion of the population resides in LMS regions. Consequently, it is this author's view that the development of an alternate design process for these regions should be included in the development of future NBCC seismic provisions.

The following are few relevant aspects which need to be considered in developing LMS design procedures:

a. Seismic design is not likely to be the main lateral loading concern, although in may govern in particular circumstances.

b. There may not be a good database for quantification of seismic hazard, i.e. calculated seismic hazard is likely to be more uncertain than in high seismicity regions.

c. Designers may not be familiar with dynamic behaviour of structures nor are they likely to want to spend a lot of time familiarizing themselves with dynamic response analysis methods.

d. Seismic design should be a relatively simple procedure; checking for errors should be relatively straightforward; the amount of time spent should be commensurate with its significance in the overall design of the particular building or facility.

e. Qualitative aspects of design (e.g. type of structural system) are likely to be as important or more important than quantitative aspects.

f. Design objectives need to be clarified, e.g. protection against damage may be more important than life safety as a primary design objective.

Conclusions

The seismic provisions of the NBCC have been updated at frequent intervals over a period of approximately 50 years. Updating on a frequent basis is needed to recognize:

a. Lessons learned from damage which has occurred during major earthquakes around the world.

b. Results of earthquake engineering research conducted in Canada and elsewhere.

c. Changes made in seismic codes in other countries.

Of the NBCC seismic provision updates, the changes incorporated in the NBCC 2005 seismic provisions are probably the most substantial. The NBCC 2005 provisions should, if used effectively, provide for an improved and more consistent seismic level of protection. However, these changes will make seismic design significantly more complex, which could have a negative effect because the less knowledgeable designers may not use its provisions appropriately; designers in regions of low to moderate seismicity are likely to view this additional complexity as unwarranted. The development of the NBCC 2005 provisions has raised certain issues, e.g. complexity, prescriptiveness, and performance objectives, which are of ongoing concern in the development of seismic codes, whether in Canada or elsewhere in the world.

Acknowledgements and Thanks

The author would like to acknowledge and thank the members of CANCEE, past and present, for their contributions to development of seismic design in Canada and, in particular, for the stimulation of the discussions at CANCEE meetings which has led to the views and ideas expressed in this paper. Thanks are also due to the organizing committee of the Ninth Canadian Conference on Earthquake Engineering for the invitation to make a presentation of this paper at that conference.

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