Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 1387

PROPOSALS FOR THE SEISMIC DESIGN PROVISIONS OF THE 2010 NATIONAL BUILDING CODE OF CANADA

J. Humar[1], J. Adams[2], R. Tremblay[3], C.A. Rogers[4], S. Halchuk[2]

ABSTRACT

The seismic design provisions of the National Building Code of Canada (NBCC) underwent significant changes between 1995 and 2005. The changes proposed for the seismic requirements of 2010 NBCC are, in comparison, limited in scope as well as in their extent. They include revisions in the seismic hazard values in the eastern region of Canada, changes in some of the site effect factors, a new minimum value for the design spectral acceleration values for concrete shear wall structures, an upper limit on the design forces as determined from a dynamic analysis and some changes related to the design of diaphragms, chords, and collectors. Adjustments have been made in some values of the higher mode effect factors and overturning moment reduction factors. Values of the overstrength and ductility related force modification factors have now been specified for two new lateral force resisting elements and structural types, namely buckling restrained braces and cold-formed structures. The present paper highlights the changes and revisions in relation to the provisions of 2005 NBCC.

Introduction

The National Building Code of Canada is a model code that when adopted by a provincial or territorial authority in its original or modified form becomes a regulation. The code is usually revised at intervals of five years. The last code cycle between 1995 and 2005 was, however, 10 years rather than the usual 5 years. During that cycle the code underwent a major restructuring, resulting in the publication of the code in an objective based format. Division A of the new code contained the objectives of the code provisions and the conditions necessary to achieve compliance. Division B contained acceptable solutions that were deemed to satisfy the objectives listed in Division A. The seismic design provisions of the code are in Division B.

In many ways, the seismic provisions of the 2005 NBCC (NRCC 2005) represented a major departure from the provisions of the 1995 version of the code. The revised provisions were

[1] Dept. of Civil and Environmental Engineering, Carleton University, Ottawa, ON. K1S 5B6
[2] Geological Survey of Canada, 1 Observatory Crescent, Ottawa, ON, K1A 0E4.
[3] Dept. of Civil Geological and Mining Engineering, Ecole Polytechnique, Montreal, PQ, H3T 1J4
[4] Dept. of Civil Engineering and Applied Mechanics, McGill University, Montreal, PQ, H3A 2K6

influenced by new knowledge on seismic hazard, improved understanding of material and structural behavior, Canadian and international research, and developments taking place in other international codes. A series of papers in the April 1993 issue of the Canadian Journal of Civil Engineering provide detailed background to the development of the 2005 provisions (DeVall 2003). Compared to the changes between 1995 and 2005 the proposed revisions are limited both in their scope and in extent. The more important of the revisions are discussed here in the context of the 2005 code.

Seismic Hazard

For NBCC 2005 the Geological Survey of Canada (GSC) produced a new seismic hazard model and from this, a suite of new seismic hazard maps as described in a report by Adams and Halchuk (2003). That report covered the development of (and rationale for) the hazard model, provided the detailed model, and gave results for selected cities and localities across Canada. It specified hazard in terms of spectral acceleration values at selected periods of a 5%-damped single-degree-of-freedom system, and the design spectral accelerations were determined for a uniform probability of exceedence of once in 2475 years. Additional background information on the model, maps and their application in the code appeared in the April 2003 special issue of the Canadian Journal of Civil Engineering.

The 2010 seismic hazard values are based on the same "4th Generation" seismic hazard model as the 2005 values, but the seismic hazard values have been updated by replacing the quadratic fit to the ground motion relations used in NBCC 2005 for earthquakes in eastern, central and north-eastern Canada by an 8-parameter fit. For NBCC 2005, it was recognized that, while the quadratic fit provided a good approximation in the high-hazard zones, it was rather conservative at short periods for the low-hazard zones; however, because the design values are small in the low-hazard zones, the approximation was accepted. The 8-parameter fit gives a good fit across all zones. The changes from 2005 have a complex, but understandable, pattern. In general, PGA and short-period spectral values are reduced in most regions, while long-period values are slightly increased. This is illustrated by the spectral shape changes shown in Figure 1 for four cities.

Zones of low hazard cover a large part of the country, so the climatic information for about 550 of the 650 localities listed in the code has changed (often in a minor way); only some western localities are unaffected. The 2010 changes have the following engineering implications: geotechnical design levels (based on PGA values) are generally reduced, the design forces for short-period buildings are reduced in most places, and the design forces for tall buildings are increased in all but the highest-hazard regions. The tilting of the spectra (shorter periods lower, longer periods slightly higher) requires some minor adjustments to the higher mode effect factor described below in a subsequent paragraph.

Site Effect Factors

It is well known that the characteristics of the underlying soil have a profound effect on the amplitude of the seismic waves reaching the foundation. The 2005 NBCC specified five different categories of soil from Class A, hard rock, to Class E, soft soil. Another soil class F was used to cover liquefiable soils, and sensitive, organic and highly plastic clays. Classification of the soil into categories A to E was based on the average shear wave velocity in the top 30 meters of soil below the spread footing, mat foundation, or pile cap. Where applicable, energy corrected average standard penetration resistance or undrained shear strength could be used in lieu of the shear wave velocity.

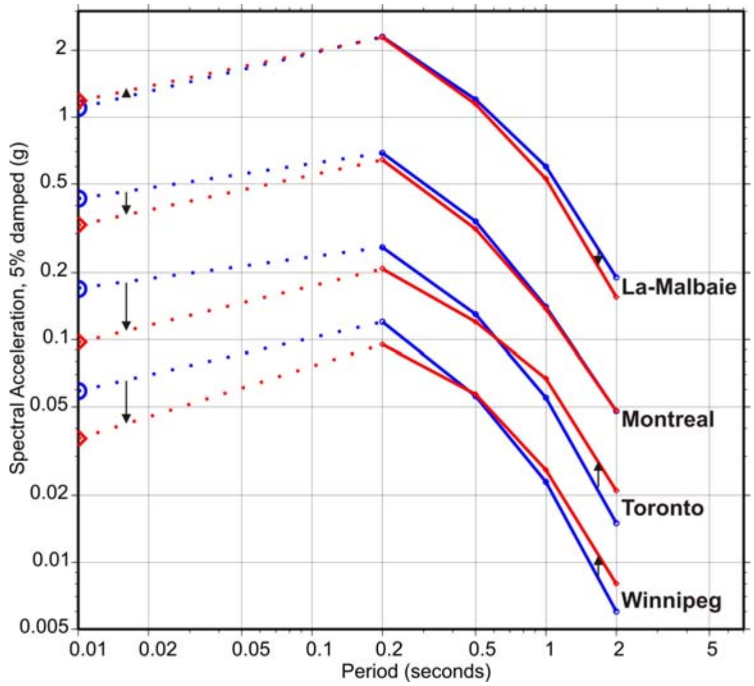

Figure 1. Uniform Hazard Spectra changes from 2005 (blue lines) to 2010 (red lines) for four localities which are representative of the range in hazard values for eastern Canada. Peak ground acceleration is plotted at a period of 0.01 seconds.

To account for the influence of soil characteristics the 2005 NBCC specified two different site effect factors, one for the short period range and another for the long period range. The two factors continue in 2010. It is now explicitly stated that the Class A or B classification must not be used if there is a more than a 3-m thick intervening layer of softer soil in the top 30 meters, and that in such a case the classification should be based on the average shear wave velocity in just the softer layer. This provision recognizes that the intervening softer layer may cause significant amplification of the earthquake waves as they propagate to the foundation level.

The 2010 NBCC provisions also specify that for structures with a fundamental period of vibration equal to or less than 0.5 s, the values of F_a and F_v may be determined using the site class definitions and criteria assuming that liquefaction does not occur. This exception is provided because ground motion data obtained in liquefied soil areas during earthquakes indicate that short-period ground motions are generally attenuated due to liquefaction whereas long-period ground motions may be amplified. This exception is only for the purposes of defining the site class and obtaining site coefficients. It is still required to assess liquefaction potential and its effects on structures as a ground failure hazard.

The site effect factors for Class A soils were based on ground motion measurement on rocks with shear wave velocity of ~1500 m/s, the lower level of Class A. In recognizing that rock with shear wave velocity higher than 1500 m/s will cause lower amplification in ground motion, 2010 NBCC allows the use of a smaller amplification factor obtained by multiplying the standard code values by the ratio $(1500/V_s)^{1/2}$.

Design Response Spectrum

In the 2005 NBCC, the seismic hazard was specified through uniform hazard spectra, which provided the spectral acceleration ordinates at selected periods, from 0.2 s to 2.0 s, calculated at a uniform probability of exceedance of 2% in 50 years. Spectral acceleration values were provided for a number of geographical locations throughout the country and for a foundation soil belonging to the reference site class C. For soils other than Class C the design spectral accelerations were to be obtained by applying the site effect factors Fa and Fv.

For want of sufficient data spectral accelerations values for periods longer than 2 s could not be determined with certainty. Based on available information on the drop-off gradient of spaectral acceleration with period the design acceleration, S(T), for $T \geq 4.0$ s was conservatively specified to be one-half of that at $T = 2.0$ s. Given the uncertainty in the acceleration values for $T > 2$ s and the vulnerability of tall structure to the concentration of inelastic deformations in certain storeys, the 2005 NBCC required that the equivalent static design base shear for structures with fundamental period greater than 2 s should be taken as being no less than that corresponding to a period of 2 s. In the 2010 NBCC this lower limit has been relaxed to the base shear value at $T = 4$ s for structures in which the lateral resistance is provided by shear walls, coupled walls or wall frame systems. In structures of this type the inelasticity is, in general, more uniformly distributed across the height.

The damage caused to a structure due to earthquake is deformation related rather than strength related. As a consequence it is the deformations and displacements, rather than the induced forces, that control the performance. Deformations in short period structures are small because the spectral displacements at short periods are quite small. Also, the effective spectral acceleration experienced by short period structures is smaller than the specified values because of factors such as finite foundation size and energy dissipation at the foundation-structure interface, e.g. due to sliding. The short period structures also tend to have greater overstrength and stiffness than could be readily estimated. For the reasons cited above, short-period structures, particularly those with some ductlity, have performed better than others in past earthquakes. In view of this the 2005 NBCC specified an upper limit on the equivalent static design shear that was 2/3 of that for a period of 0.2 s. This practice continues in the 2010 NBCC except that the upper limit is not applicable to structures sited on Class F soil. For such soils the design spectrum may have a peak in the spectral acceleration value at a fairly long period and the application of the 2/3 factor would then affect buildings with an even longer period. Since the upper cap is intended for short period structures it should not be applied to structures on Class F site where it may affect even those structures that are in the long period range.

Higher Mode Effect

The equivalent static base shear determined from the uniform hazard spectrum corresponds to the fundamental perid of the structure. The 2005 NBCC specified a higher mode effect factor M_v to account for the effect of higher modes on the design base shear. The M_v factor depends on the spectral shape, the type of structure (frame, wall or brace), and the value of the fundamental period. All of these were taken into account in calculating M_v, values for which were given for periods up to 2.0 s. This factor continues to be specified in 2010 NBCC, except that new values are given for periods between 2.0 and 4.0 s for wall, coupled wall and wall frame systems. In addition some minor adjustments have been made in the M_v values for structures in the eastern region to account for the change in spectral shape.

In the 2005 and 2010 NBCC the equivalent static base shear is distributed across the height primarily in accordance with an assumed shape that corresponds to the first mode. The effect of higher modes on the base overturning moment is taken into account through a base overturning moment reduction factor J. The J values in 2010 NBCC are the same as those in 2005 NBCC, except that new values of J are specified wherever new values of M_v are given.

Analysis Method

The 2005 NBCC recognized the dynamic analysis method as the preferred method for determining the seismic forces. The equivalent static load method was still permitted for buildings located in zones of low seismicity, or when the building was regular and of medium height, or when it was irregular but of low height. When a dynamic analysis method was employed, the calculated base shear was to be scaled if necessary, so that for a regular building it was no less than 80% of the equivalent static base shear and for an irregular building no less than 100% of the equivalent static base shear. In calculating the base shear, the fundamental period determined from methods of engineering mechanics could be used as long as it was no greater than a given factor times the period detrermined from empirical equations given in the code. The spectrum to be used in the dynamic analysis was the same as the uniform hazard spectrum without any modification. All of these provisions continue in the 2010 NBCC except that a short period cap similar to that used in the equivalent static method is also applied in the dynamic anlysis method.

Ductility and overstrength related factors

The practice of reducing the design forces to benefit from the ductility and overstrength present in a structure continued in the 2005 NBCC with the specification of two different factors R_d and R_o along with restrictions on the height of the structure, for a range of structural materials and structure types. In the 2010 NBCC, R_d and R_o factors have been specified for buckling restrained braces as well as for cold-formed steel structures, not previously covered. Other adjustments have been made to coordinate the structure types and height limitations with the related material standards. In particular, for building occupancies other than assembly occupancies, height limits have been extended for steel seismic force resisting systems of conventional construction.

Buckling restrained braced frame (BRBF) seismic force resisting systems

R factors and restrictions were specified in the 2005 edition of NBCC for steel force resisting systems made of moment-resisting frames, concentrically braced frames, eccentrically braced frames and plate walls. In 2010, provisions related to the buckling restrained braced frame (BRBF) system have been introduced. BRBF is a special form of concentrically braced frame where the braces are specially designed and detailed such that they yield in both tension and compression, without buckling. This bracing system was introduced in Japan in the late 1980’s to act as a hysteretic damper designed to help in controlling drifts in steel moment-resisting frames (Watanabe et al. 1988). A typical BRB member is illustrated in Fig. 2a. It consists of a steel core plate with a reduced cross-section inserted in a steel tube. The volume between the tube and the core is filled with mortar or concrete. At both ends of the brace, the core plate extends outside of the tube and is connected to the framework. The core is covered with an un-bonding material prior to filling the tube, so that the core can freely deform in the axial direction. As a result, the core and resists most of the axial load. The unsupported projections of the core at the two ends of the brace are generally made larger in size and/or stiffened to avoid buckling in compression. Under seismic loading, yielding is expected to take place both in compression and tension in the reduced segment of the core, L_c, leading to a nearly symmetrical hysteretic response as shown in Fig. 2b. A small gap must be left between the concrete fill and the core to allow for transverse expansion of the core upon compression yielding.

In North America, the BRB system was introduced in the late 1990’s (Tremblay 1999), essentially as a substitute to conventional bracing members in concentrically braced steel frames, not as a damping device. The first application of the system in Canada was for the TELUQ (Télé-Université) building, of the Université du Québec network, in Québec City, in 1999.

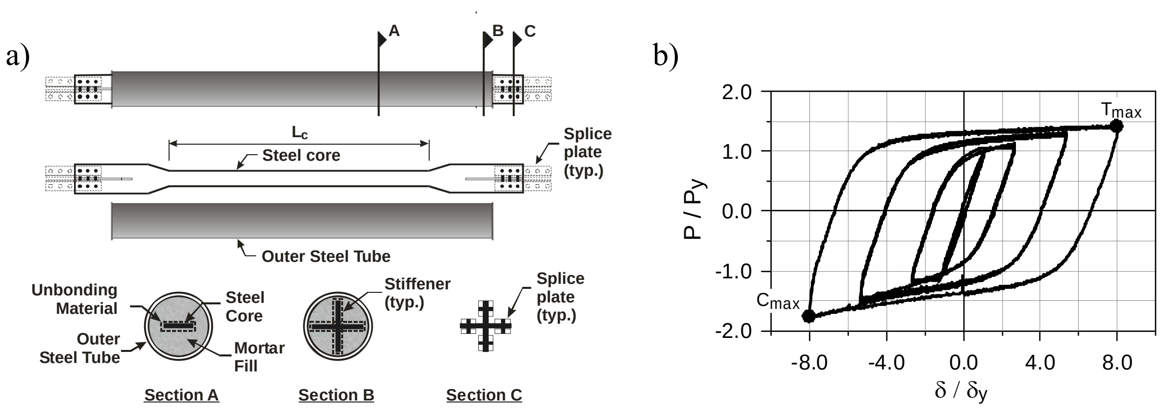

Figure 2 Buckling restrained bracing member: a) Typical design, b) Hysteretic response.

In Canada, buckling restrained bracing members have typically been produced by local steel fabricators using simple designs such as that proposed by Tremblay et al. (1999, 2006). Design and detailing requirements for BRBFs have now been introduced in the 2009 edition of CSA-S16 (CSA 2009). In view of the stable hysteretic response exhibited by bracing members (Fig. 2b), a ductility-related factor R_d = 4.0 is specified for the system, which is the same as for ductile eccentrically braced steel frames. Typically, the core plate of BRB members is cut to achieve exactly the required factored axial resistance for the brace, taking into account the measured steel properties as obtained from mill test certificates or coupon tension tests. Furthermore, BRB members are generally used in frames with simple shear beam-to-column connections exhibiting no or very limited bending moment capacity. Hence, the dependable system overstrength, as defined by Mitchell et al. (2003), results only from the difference between nominal and factored brace resistances and strain hardening of the core steel when subjected to large inelastic deformations. Consequently a low overtrength-related force modification factor $R_o = 1.2$ is specified in the 2010 NBCC for BRBFs. Buckling restrained braced frames with non moment-resisting beam-to-column joints have limited capacity to redistribute the inelastic demand over the building height and may be prone to soft-storey response under strong ground shaking (Tremblay and Poncet 2007). Height limits have therefore been introduced for BRBF applications in moderate and high seismic regions to minimize the potential for such undesirable behavior. Qualifying tests are prescribed in CSA S16-09 to ensure that the braces supplied will perform as intended in design. The *R* values and restrictions for the BRBF system included in the 2010 NBCC are shown in Table 1.

Table 1: R-values and restrictions for BRBF and CFS seismic force resisting systems

Type of seismic force resisting system	R_d	R_o	Building height (m) limitations[1]				
			$I_EF_aS_a(0.2)$				$I_EF_vS_a(1.0)$
			<0.2	≥0.2 to <0.35	≥0.35 to ≤0.75	> 0.75	>0.3
Steel Structures Designed and Detailed According to CSA S16							
Ductile buckling-restrained braced frames	4.0	1.2	NL	NL	40	40	40
Conventional construction of moment-resisting frames, braced frames or plate walls							
Assembly occupancies	1.5	1.3	NL	NL	15	15	15
Other occupancies	1.5	1.3	NL	NL	40	60	40
Cold-Formed Steel Structures Designed and Detailed According to CSA S136							
Shear walls							
Screw connected shear walls – wood based panels	2.5	1.7	20	20	20	20	20
Screw connected shear walls – wood based and gypsum panels in combination	1.5	1.7	20	20	20	20	20
Diagonal strap concentrically braced walls							
Limited ductility (LD)	1.9	1.3	20	20	20	20	20
Conventional construction (CC)	1.2	1.3	15	15	NP	NP	NP
Other cold-formed SFRS(s) not defined above	1.0	1.0	15	15	NP	NP	NP

[1] NL = No Limit; NP = Not permitted

Cold formed steel (CFS) lateral framing systems

Prior to 2010, the National Building Code of Canada did not contain seismic design provisions for cold-formed steel (CFS) lateral framing systems. Table 4.1.8.9 of the Building

Code has been updated (Table 1), and now lists seismic force modification factors and height limits for structures framed with CFS. Shear walls sheathed with wood structural panels (plywood or OSB) and with a combination of wood and gypsum panels are listed, as well as steel strap braced walls. Two categories of braced wall have been established; limited ductility (LD) and conventional construction (CC). The NBCC refers to the CSA Standard S136 (2007) for the requisite design and detailing of CFS walls. The CSA S136 Standard addresses cross-section and member properties of cold-formed steel in general. It does not provide information on specific seismic design or detailing. CSA S136, for this reason, refers to the American Iron and Steel Institute's (AISI) North American Standard for cold-formed steel framing – lateral design (AISI S213) (2007). This newly published standard contains Canadian specific design and detailing provisions for CFS framed shear and strap braced walls. These provisions have been written such that they provide the ductility and overstrength corresponding to the new R-values for CFS lateral systems. The basis of these design provisions and R-values can be found in the work of Boudreault et al. (2006), Branston et al. (2007) and Comeau et al. (2010), among others. This research at McGill University dates back to 2000; over 300 single storey CFS wall assemblies of various configurations have been tested using monotonic and reversed cyclic loading protocols. Design and detailing provisions were first developed through analyses of the resulting test data. This was followed by extensive non-linear time history dynamic analyses of multi-storey CFS framed structures using the approach which is now known as FEMA P695 (2009); the quantification of building seismic performance factors. Synthetic and recorded earthquake records were scaled to the site-specific uniform hazard spectra for Vancouver, Calgary, Quebec and Halifax. Failure probabilities were then shown to be within acceptable levels for the strap braced and sheathed shear wall systems listed in the 2010 NBCC.

Height limits for conventional construction in steel

A height limit of 15 m was specified in NBCC 2005 for steel SFRSs of the conventional construction category, i.e., SFRSs not specifically detailed to achieve a ductile seismic response, in regions of moderate and high seismicity. This limit was intended to retain the traditional three-storey height limit stipulated in previous NBCC editions for these systems. This height limit was not intended for single-storey, steel industrial structures. In particular, structures such as steel mills and aircraft hangars could exceed 15 m in height and were permitted to be built using conventional construction. On the other hand, other tall steel structures such as stadia, exhibition halls, arenas, and convention centers were to satisfy the 15 m height restriction in view of the higher hazard to human life associated to these occupancies. In the 2010 NBCC, the situation was clarified by specifying two different sets of height limits, one set for assembly building occupancies and another set for other occupancies. The first set is the same as in the 2005 NBCC; in the second set extended limits are now specified for building occupancies other than assembly occupancies (see Table 1). These extended height limits do not come without restrictions, because several new requirements have been introduced in CSA S16-09 for conventional construction in buildings that exceed 15 m in height. In particular, the seismic design loads must be increased linearly by 2% per meter of height above 15 m, without exceeding forces corresponding to $R_dR_o = 1.0$, and the seismic forces and deformations must be determined using the Dynamic Analysis Procedure described in the NBCC. The steel and weld metal used in these structures must meet minimum requirements for ductility and all members of the seismic force resisting system must be Class 1 or Class 2 sections to delay the occurrence of local buckling. Columns and connections must be designed for amplified earthquake loads to protect further their integrity. In addition, the connections must be detailed such that the governing failure mode is ductile. Also, minimum factored seismic forces are prescribed for diaphragms to control inelastic diaphragm response and members of the seismic-force-resisting system that intersect at an unbraced location are required to have out-of-plane transverse resistance.

Force cut-off and diaphragm design

Capacity design principle has been adopted for seismic design in Canada: the seismic force resisting system (SFRS) includes components that are specifically designed and detailed to withstand cyclic inelastic deformations while the remaining components are intended to remain essentially elastic. The yielding components are sized for the effects of gravity loads combined with earthquake loads reduced by the R_o and R_d factors. The elastic components are designed to carry the gravity loads together with seismic effects corresponding to the forces generated upon yielding of the ductile components. In cases of oversized yielding components (for instance, when drift limits govern the design), the seismic force effects for the non-yielding components may become very large; the 2005 NBCC therefore prescribed an upper limit on these effects equal to the elastic force level. In recognition that non-yielding components may possess some overstrength, the 2010 NBCC has reduced this limit to earthquake forces determined with R_oR_d = 1.3, when permitted in the applicable CSA material standards. This relaxation only applies, however, to components that exhibit a ductile failure mode, the reason being to prevent brittle failures that can potentially lead to structural collapse in case the seismic demand exceeds the code design level.

This modification to the NBCC is likely to affect the design of floor and roof diaphragms. Traditionally, yielding of diaphragms has been considered an undesirable method of energy dissipation (DeVall 2003) and it was specifically required in the 2005 NBCC that diaphragms be designed so that they do not yield. This requirement also ensured that diaphragms could maintain their structural integrity and, thereby, properly distribute the horizontal forces induced at each level to the vertical elements of the SFRS (walls, braced frames, etc.) under strong ground motions. In many buildings, the diaphragm design was controlled by the upper force limit corresponding to R_oR_d = 1.0 due to the overstrength present in the vertical elements of the SFRS. For such diaphragms, this change to NBCC will reduce the design forces while maintaining the original objective of elastic diaphragm response. Recent studies showed, however, that properly designed and detailed metal roof deck diaphragms and wood diaphragms could exhibit ductile behavior. This new data motivated a second change to the 2005 NBCC by which reduced seismic design loads are now specified for these diaphragms so that they may yield under strong earthquakes. For metal roof diaphragms in buildings of less than four stories and for wood diaphragms, these reduced loads are determined using R_oR_d = 2.0. In case of wood diaphragms acting in combination with vertical wood shear walls, the diaphragm design loads can be reduced to the values used for the walls, that is, R_oR_d = 5.1 for nailed shear walls with wood based panels or R_oR_d = 3.4 for shear walls made of wood based and gypsum panels in combination. For these yielding diaphragms, the struts must be designed so as not to yield whereas the collectors, chords and the connections between the diaphragms and the vertical elements of the SFRS must be designed for forces corresponding to the actual capacity of the diaphragms.

Conclusions

The National Building Code of Canada underwent extensive revisions between its 1995 and 2005 versions. The changes in the 2010 version are comparatively quite limited. They include revisions to hazard calculations for the eastern and central regions, some new provisions related to site effect factors for poor soils, minor changes to the analytical procedures for determining the design forces, introduction of overstrength and ductility related factors for buckling restrained braces and cold formed structures, and changes to provisions related to height restrictions for conventional steel construction and diaphragm design.

References

Adams, J., and Halchuk, S., 2003. Fourth generation seismic hazard maps of Canada: Values for over 650 Canadian localities intended for the 2005 National Building Code of Canada, Geological Survey of Canada Open File 4459, 155 p.

AISI, 2007. North American standard for cold-formed steel framing – lateral design, AISI S213, American Iron and Steel Institute, Washington, DC.

Boudreault, F.A., Blais, C., Rogers, C.A., 2007. Seismic force modification factors for light-gauge steel-frame – wood structural panel shear walls, *Canadian Journal of Civil Engineering*, 34(1), 56-65.

Branston, A.E., Boudreault, F.A., Chen, C.Y., Rogers, C.A., 2006. Light-Gauge Steel-Frame – Wood Structural Panel Shear Wall Design Method, *Canadian Journal of Civil Engineering*, 33(7), 872-889.

Comeau, G., Velchev, K., Rogers, C.A., 2010. Development of seismic force modification factors for cold-formed steel strap braced walls, *Canadian Journal of Civil Engineering* (In press).

CSA, 2009. Design of Steel Structures, CSA-S16-09, Canadian Standards Association, Toronto, ON.

CSA, 2007. North American specification for the design of cold-formed steel structural members. Standard CSA-S136, Canadian Standards Association (CSA), Mississauga, ON.

DeVall, R., 2003. Background information for some of the proposed earthquake design provisions for the next edition of the National Building Code of Canada, *Canadian Journal of Civil engineering*, 30, 279-286.

FEMA, 2009. Quantification of building seismic performance factors, FEMA P695, Federal Emergency Management Agency, Washington DC.

Mitchell, D., Tremblay, R., Karacabeyli, E., Paultre, P., Saatcioglu, M., and Anderson, D.L., 2003. Seismic force modification factors for the proposed 2005 NBCC, *Canadian Journal of Civil Engineering*, 30, 308-327.

NRCC, 2005. National Building Code of Canada, National Research Council of Canada, Ottawa, ON.

Tremblay, R. and Poncet, L., 2007. Improving the Seismic Stability of Concentrically Braced Steel Frames, *Engineering Journal, AISC*, 44, 103-116.

Tremblay, R., Bolduc. P., Neville, R., and DeVall, R., 2006. Seismic Testing and Performance of Buckling Restrained Bracing Systems, *Canadian Journal of Civil Engineering*, 33, 183-198.

Tremblay, R., Degrange, G., and Blouin, J., 1999. Seismic Rehabilitation of a Four-Storey Building with a Stiffened Bracing System, *Proceedings 8th Canadian Conference on Earthquake Engineering*, Vancouver, BC, 549-554.