PREVIEW OF THE SEISMIC DESIGN PROVISIONS OF THE 2015 NATIONAL BUILDING CODE OF CANADA

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ABSTRACT: The last version of the National Building Code of Canada was issued in 2010. The next version is expected in 2015. In the code cycle between 2010 and 2015, significant changes would take place in the seismic design provisions. The main factor that drives these changes is an improved understanding of the seismic hazard across the country. Changes have also been proposed in the site effect factors. The structural design provisions have been appropriately adjusted to respond to changes in the estimates of hazard and the new site effect factors. There are also new provisions related to design in regions of low hazard, buildings with flexible diaphragms, buildings with inclined columns, passive energy dissipation systems, base isolation, rocking foundations, glazing systems, racks, and elevators. Some of the changes are briefly discussed. The focus of the paper is, however, on the revisions to the structural design provisions.

1. Introduction

Design for seismic forces begins with the specification of the seismic hazard at the site. The steps leading to the determination of the hazard include identifying the source zones, obtaining magnitude-recurrence relations, and developing the ground motion prediction equations. Using a methodology based on implementation of the three steps, the 2010 National Building Code of Canada (NBC 2010) provided the design spectral acceleration values at selected periods and a damping of 5% for a number of localities across Canada. During the code cycle between 2010 and 2015, considerably more data has become available on the ground motions, which has provided a better understanding of the contributors to the determination of hazard. Consequently, major changes have been proposed in the specified design hazard values across whole of Canada.

The availability of significantly more ground motion data has also led to revisions in the site effect factors, such that the two foundation factors F_a and F_v specified in NBC 2010 have been replaced by separate factors for each of the following six periods: 0.2, 0.5, 1.0, 2.0, 5.0 and 10.0 s,, as well as for the peak ground acceleration (PGA) and the peak ground velocity (PGV)..

A number of revisions have also been proposed in the structural design provisions of the code. Some of these provisions reflect the impact of changes in the estimated seismic hazard and site factors. They include specification of the design spectrum, the manner in which the various triggers are specified, short period cap on the design base shear, and higher mode effect factors. A simplified procedure has been introduced for the design of structures in regions of low seismic hazard. Certain new provisions address design issues not previously dealt with, such as, design of buildings with flexible diaphragms, design of buildings having inclined columns, passive energy dissipation, base isolation, design of glazing systems, and design of steel pallet storage racks and elevators. Finally, there are revisions influenced by Canadian and international research, improved understanding of material and structural behaviour, and developments taking place in other international codes. Examples of such changes include: revisions in ductility (R_d) and over-strength (R_o) related modification factors for concrete and masonry, new R_d and R_o

factors for tilt-up construction in reinforced concrete, changes in foundation design provisions, and design provisions for continuous timber construction more than 4 storeys in height.

Brief background is provided to some of the revisions referenced in the preceding paragraph. The focus of the present article is, however, on the equivalent static load method of design including specification of design spectrum and the higher mode effects.

2. Seismic hazard

As in NBC 2010, the seismic hazard in NBC 2015 is specified in terms of 5% damped spectral acceleration, S_a, at selected periods, for a uniform probability of exceedance of 2% in 50 years or a return period of 2475 years. In NBC 2015, spectral accelerations have been specified for periods of 0.2, 0.5, 1.0, 2.0, 5.0, and 10.0 s. In addition, values have been provided for the PGA as well as the PGV. In NBC 2010, the specified values of S_a represented the median, implying that there was 50% chance of the spectral acceleration exceeding the specified value. In NBC 2015, the mean rather than the median is used in the specification of seismic hazard, the mean being closer to the expected value. The NBC 2010 seismic hazard values were based on measured ground motion data collected during earthquakes up to the early 1990s. Since then, considerably more data has been obtained. In addition, new faults, such as the offshore Queen Charlotte fault, and 5 active strike-slip and reverse faults in the Alaska-Yukon region are now explicitly included as the source of hazard. This information along with improved understanding of the seismotectonics has led to improvements in how the source zones are defined and in the ground motion prediction equations (GMPE). In addition, the contributions from different sources are now combined in a probabilistic manner.

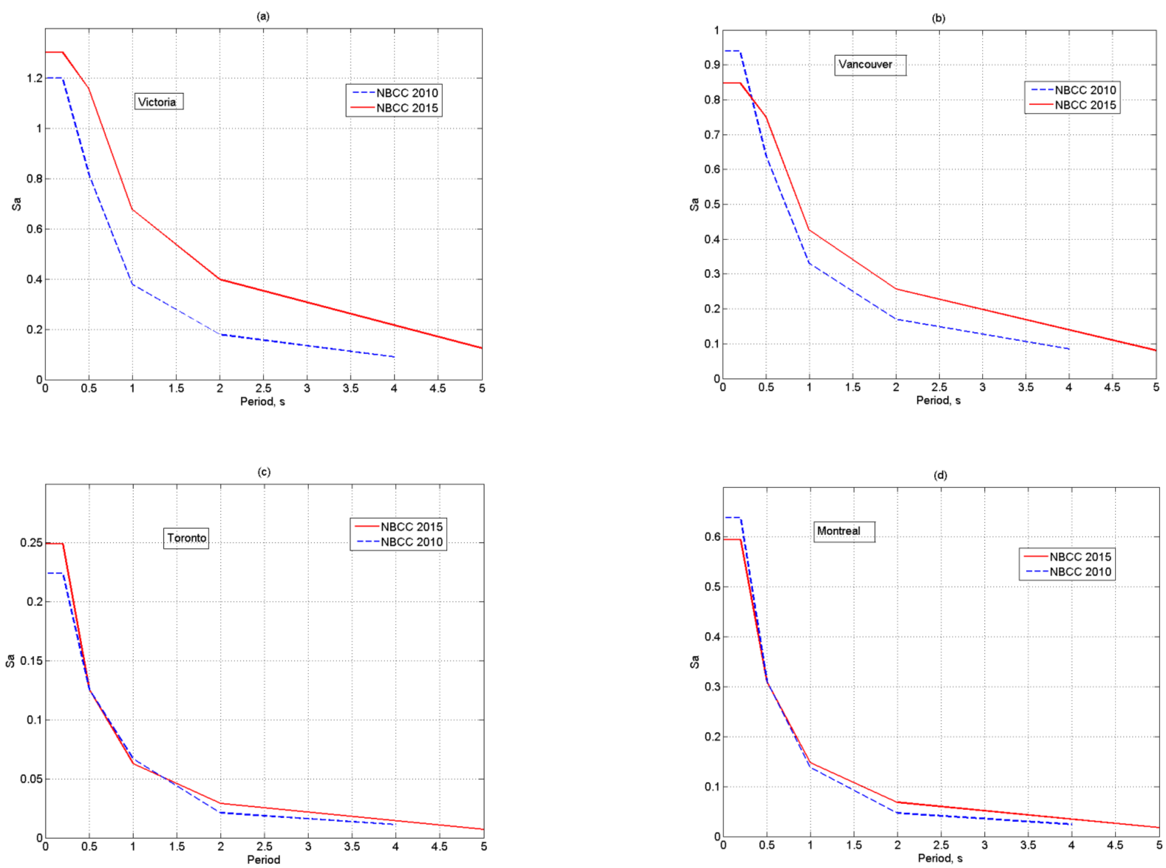


Figure 1: Comparison of UHS for NBC 2010 and NBC 2015: (a) Victoria, (b) Vancouver, (c) Toronto, (d) Montreal

2.1. Impact on seismic hazard

The various factors that contribute to the differences in seismic hazard between NBC 2010 and NBC 2015 have been outlined in the previous paragraph. The most important among these factors is the influence of the GMPEs used in the calculations. For the western regions the probabilistic treatment of Cascadia subduction zone has strongly affected the hazard estimates in regions influenced by that zone. Overall, in the eastern regions estimates of long-period hazard have increased while the estimates of short-period hazards have decreased. In the west the long period hazard has increased significantly for regions affected by Cascadia subduction zone. In addition, regions in the vicinity of recognized faults have experienced large increases in hazard.

Figure 1 compares the 2010 and 2015 uniform hazard spectra (UHS) for four cities: Victoria, Vancouver, Toronto, and Montreal. All spectra have been plotted for Class C site, which is the reference site used for the specification of hazard in the code. The 2010 spectra are plotted only up to a period of 4 s. Figure 1 shows that the hazard in Victoria has increased for the entire range of periods. Vancouver sees a decrease in hazard for short periods and marked increase for long periods. The long-period hazard in both cities has increased because of the probabilistic combination of the contribution from Cascadia subduction zone. For Toronto and Montreal, the hazard has not changed much, although there is an increase for long periods, chiefly due to the GMPEs used. However, the absolute value of the long period hazard for these cities is comparatively low.

3. Site Effect Factors

It is well known that the characteristics of the underlying soil have a profound effect on the amplitude of seismic waves arriving at the surface. The NBC 2010 specified five different soil categories 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. The NBC 2010 provisions on the classification of soils are adopted for NBC 2015.

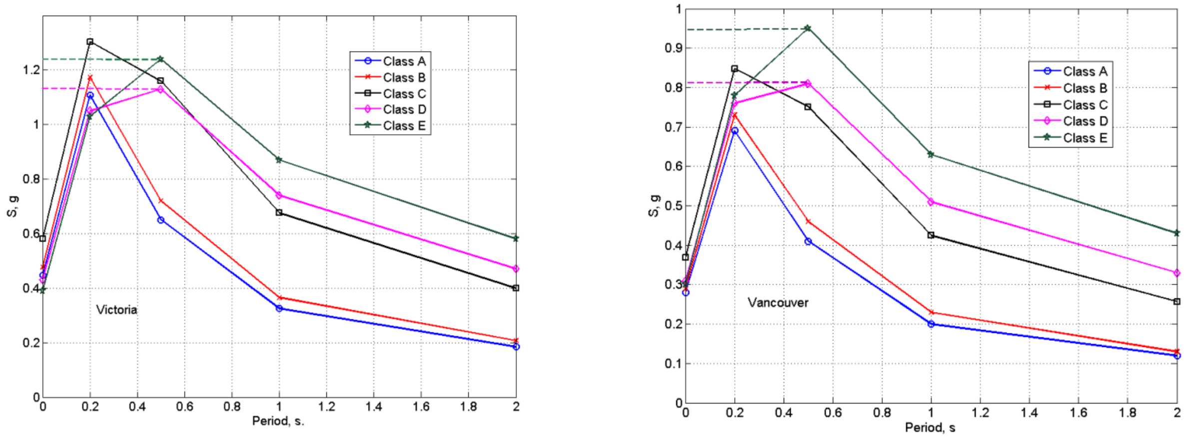
In NBC 2015, as in NBC 2010, Soil Class C is treated as the reference soil condition. For such soils the S_a values specified in the code could be used in design without any modification. For soils of the other classes, the code values must be multiplied by a site effect factor before being used in design. The site effect factor depends on the nature of soil, and the period and intensity of earthquake shaking.

The site factors in NBC 2015 are specified for six different periods: 0.2, 0.5, 1.0, 2.0, 5.0 and 10.0 s. The V_{S30} values that define the boundaries between the various classes of soil are: between A and B 1500 m/s, between B and C 760 m/s, between C and D 360 m/s, and between D and E 180 m/s. Factors $F(T)$ depend on a reference peak ground acceleration PGA_{ref} that serves as a measure of the earthquake intensity. For locations in the east, and for a given sustained shaking, ground motions have higher amplitudes for PGA than in the west. However, these amplitudes sometimes arise from small earthquakes with short duration of shaking and hence have low damage potential, as their ground displacements are very small. In the context of soil amplification and deamplification, the direct use of PGA would give $F(T)$ values with larger non-linear deamplification effects in the east than appropriate for their sustained level of shaking. Accordingly, in the computation of $F(T)$, PGA_{ref} is used instead of PGA, where PGA_{ref} = 0.8*PGA when $S_a(0.2)$/PGA is less than 2.0, but is equal to PGA otherwise. Some localities in the west will also be affected by this provision. The design spectral acceleration is denoted by S and is given by $S(T) = F(T)S_a(T)$.

4. Specification of Seismic Hazard

As in NBCC 2010, the seismic hazard is represented in NBCC 2015 by a uniform hazard spectrum (UHS) for the given site. Figure 1 shows the uniform hazard spectra for some selected sites. As stated earlier, NBCC 2015 provides spectral acceleration values for periods of 0.2, 0.5, 1.0, 2.0, 5.0, and 10.0. Spectral values at periods of 0.05, 0.1 and 0.3 s have also been determined and will be available from the Geological Survey of Canada.

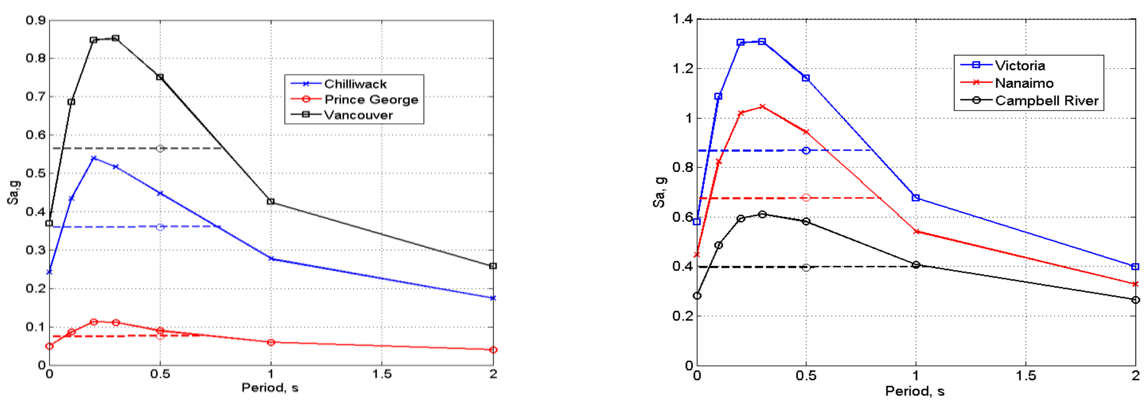
At most sites, the spectral acceleration decreases continuously from its value at 0.2 s to the peak ground acceleration at a period equal to zero. However, in the UHS an upper plateau is defined at T = 0.2 s, so that for T < 0.2 s the spectral acceleration is equal to $S_a(0.2)$. This is because it is not considered good practice to design on the basis of a spectrum in which *S* increases with the period. With yielding of the structure, expected during an earthquake, its period becomes longer and if that causes the structure to migrate into a region of the spectrum in which the spectral value is greater, the structure will attract greater force than it was designed for.



(a) (b)

Figure 2: Uniform hazard spectra: (a) Victoria for Classes A to E soil; (b) Vancouver for Classes A to E soil

In several cases *S*(0.5) is larger than *S*(0.2). Such is the case for Class D and E spectra for Victoria, as shown in Figure 2a, and Vancouver, as shown in Figure 2b. There exist other similar cases, particularly for the softer soils. As stated earlier, it is not considered a good practice to design on the basis of a spectrum in which the *S* value increases with period. It is recommended that in such cases the upper plateau should be moved to the highest peak in the spectrum. Such adjustment to the spectrum is shown by dashed lines in Figures 2a and b for Class D and E spectra for Victoria and Vancouver. The adjustment to the spectra is achieved by specifying that for a period of 0.2 s the design spectral acceleration $S(T)$ be taken as the larger of $F(0.2)S_a(0.2)$ and $F(0.5)S_a(0.5)$.



(a) (b)

Figure 3: Uniform hazard spectra for Class C soil and 2/3 cut-off shown by dashed lines; (a) Chilliwack, Prince George, Vancouver; (b) Victoria, Nanaimo, Campbell River

5. Short Period Cut-off

Recognizing that short period structures usually have more reserve strength than accounted for, undergo small displacement, and have performed relatively well during the past earthquakes, NBC 2015 allows such structures to be designed for 2/3 the calculated base shear provided the structure has been detailed to have at least a limited amount of ductility. This is achieved by specifying a short period cap at 2×S(0.2)/3 on the UHS. However, in some cases the spectral shape is so flat that the cap extends to periods considerably longer than 0.5 s, the period range for which such cap was not intended. To avoid such a situation NBC 2015 provides that the cap be 2×S(0.2)/3 or S(0.5), whichever is greater. This can be appreciated by referring to Figure 3a for Chilliwack, Prince George and Vancouver, and Figure 3b for Victoria, Nanaimo, and Campbell River.

6. Higher Mode Effect

The code specifies a method of determining the equivalent static design base shear that is based on the assumption that the structure responds in its first mode. The base shear is thus obtained from the estimated fundamental period T_a and the site-adjusted UHS for the location of the structure. The shear so obtained is multiplied by a factor M_V to account for the effect of higher modes to obtain a better estimate of the elastic base shear. The elastic shear is divided by a ductility related modification factor R_d and an overstrength related modification factor R_o to obtain the design base shear. The rationale for the use of these two factors and their values for various structural systems are given by Mitchell et al. (2003). For certain important buildings, the adjusted shear is multiplied by an importance factor I_E. Thus, the design base shear is given by

$$V = \frac{S(T_a) M_V I_E W}{R_d R_O} \tag{1}$$

where $S(T_a) = F(T_a)S_a(T_a)$ is the design spectral acceleration in units of gravity, $S_a(T_a)$ is obtained from the UHS for the site, and W is the seismic weight, equal to the dead load plus a portion of the live load.

The higher mode adjustment factor depends on the period of the building, shape of the design spectrum, and the characteristics of the structure. The last of these include the spread between the values of the fundamental and higher modes, and the modal participation factors. A steeper spectral shape attracts greater participation from the higher modes. Shear walls attract greatest higher mode participation because they show the largest spread between period values and the largest participation factors for the upper modes. Shear wall structures are followed by braced frame structures and moment frame structures, in that order.

The base shear is distributed along the height of the buildings by first applying a lateral force F_t at the top of the building to account for the effect of higher modes. The remaining portion of the base shear is distributed according to a shape that is representative of the first mode, giving the storey level forces F_x. Empirical equations to determine the forces F_t and F_x are identical in NBC 2010 and NBC 2015. The overturning moments computed from these equations overestimate the true overturning moments. This is because, while the shear V, and hence F_x, account for the participation of higher modes, the overturning moments produced by such modes is comparatively small. The code therefore specifies base overturning moment reduction factor J and similar factor J_x at each level x. The expressions to determine J and J_x are identical in NBC 2010 and NBC 2015.

The methodology used to determine the M_v and J factors has been described by Humar and Mahgoub (2003). Factor M_v is given by

$$M_V = \frac{\sqrt{\sum [S(T_i) W_i]^2}}{S(T_a) W} \tag{2}$$

where $S(T_i)$ is the site-adjusted spectral acceleration corresponding to the ith modal period, W_i is the corresponding modal weight, and W is the total weight of the building. The summation in Equation 2 is carried out over all modes included in the computation. The numerator on the right hand side of that equation is the base shear obtained from a modal response spectrum analysis, while the denominator is the base shear assuming that the entire weight participates in the first mode.

The lateral forces F_x are used to calculate the base overturning moment M_{bc} and the storey level moments M_{xc}. The corresponding moments obtained from a response spectral analysis are M_{be} and M_{xe}. The overturning moment reduction factors are then given by

$$J = \frac{M_{be}}{M_{bc}} \qquad J_x = \frac{M_{xe}}{M_{xc}}$$

The higher mode adjustment factors are determined here separately for the three major categories of structures, moment frame structures, braced frame structures, and shear wall structures. Representative ten-storey structures are used for this purpose; they have been described in the paper by Humar and Mahgoub (2003).

The spectral shape, or how steep is the spectrum, can be determined by considering the spectral ratio *S*(0.2)/*S*(5.0). For the locations included in the climatic data table of the code, this ratio for Class C soil ranges from 3.8 to 51.3. Factors M_V and J are derived here for specific values of the spectral ratio, namely 5, 20, 40 and 65. This covers almost the entire expected range of the spectral ratios. The database of UHS used for the derivation comprises Class C uniform hazard spectra for 27 cities, selected from the table of climatic data, and Class A to E spectra for Victoria, Vancouver, Calgary, Montreal, Toronto, and Fredericton, giving a total of 57 spectra.

The results for M_V and J values are presented in Figure 4 as scatter diagrams. Also shown are piece-wise linear representations of these values. Separate curves are provided for T = 0.5, 1.0, 2.0, and 5.0 s, except that when the M_V value is less than 1 for the entire range of the ratio *S*(0.2)/*S*(5.0), the corresponding curve is not shown. It is apparent from Figure 4 that for a given structural type the spectral ratio and the fundamental period are the two parameters that most influence the response.

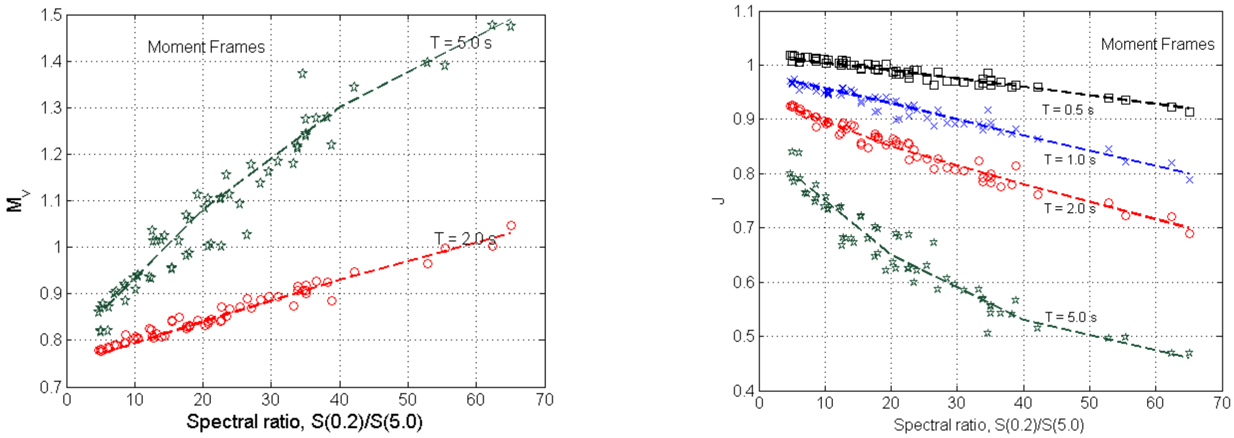
The code provisions on M_V and J factors are derived from the straight-line representations in Figure 4 and are shown in Table 1 for different spectral ratios and different values of the fundamental period T_a. For intermediate values of the spectral ratio, M_V and J are obtained by linear interpolation. The values so obtained for M_v and J are then used to obtain the product $S(T_a)\bullet M_V$ *and* J, respectively, for intermediate values of the fundamental period.

The code specifies that for moment resisting frames and braced frames the design base shear be taken as not less than that for a period of 2.0 s. Accordingly M_V and J values are not specified for such structures for periods greater than 2.0 s. For shear wall structures the minimum design base shear specified in the code is that corresponding to a period of 4.0 s. The product M_VS at 4 s may be obtained by interpolation between its values for 2 and 5 seconds. The coupled walls respond to earthquake motion in a manner that is somewhere between those of moment frames and shear walls. In most practical cases the behaviour is closer to that of a moment frames, except that the likelihood of inelastic demand being concentrated in a storey is smaller. Thus, the M_v and J factors for coupled walls are similar to those for moment frame, but the minimum design shear is that corresponding to a period of 4.0 s as in the case of shear wall structures.

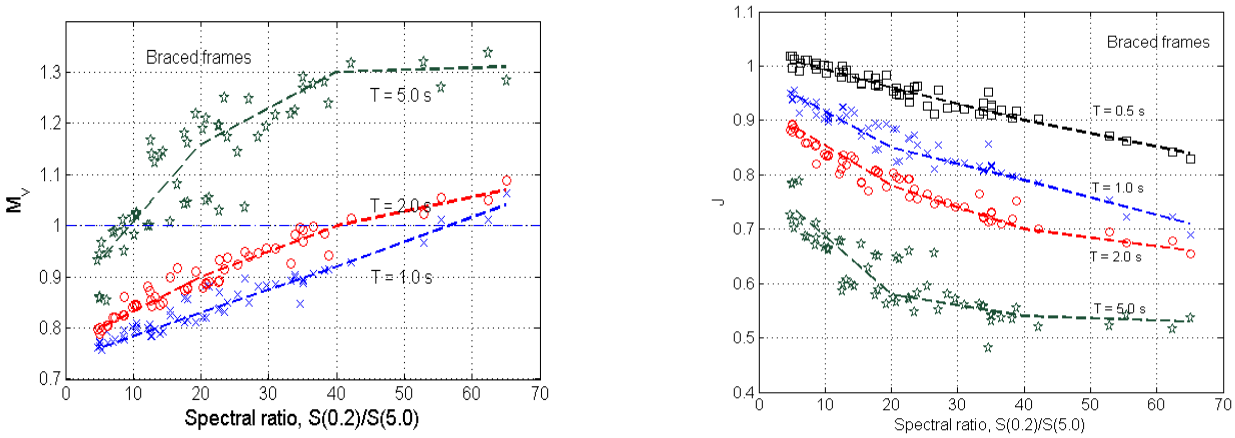
In the code provisions, the M_V factor is specified as being no less than 1. This is in keeping with similar provision in NBC 2010, is on the conservative side, and is justified by the fact that as the number of storeys approaches 1, M_V value will approach 1. Similarly, J is taken as no larger than 1.

7. Low Hazard Zones

A large portion of Canada's land mass consists of tectonically stable region with low seismic activity. The central part of Canada including the prairies falls in this region. However, in spite of the low seismic activity, large earthquakes can occur in the region. Experiences in stable tectonic regions around the world confirm this. For example, the 2012 Christchurch (New Zealand) Earthquake, the 1995 Kobe (Japan) Earthquake, and the 1989 Newcastle (Australia) earthquake all occurred in areas of moderate to low seismic activity. Although moderate in intensity, these earthquakes were located close to an urban centre and caused considerable damage.



(a)



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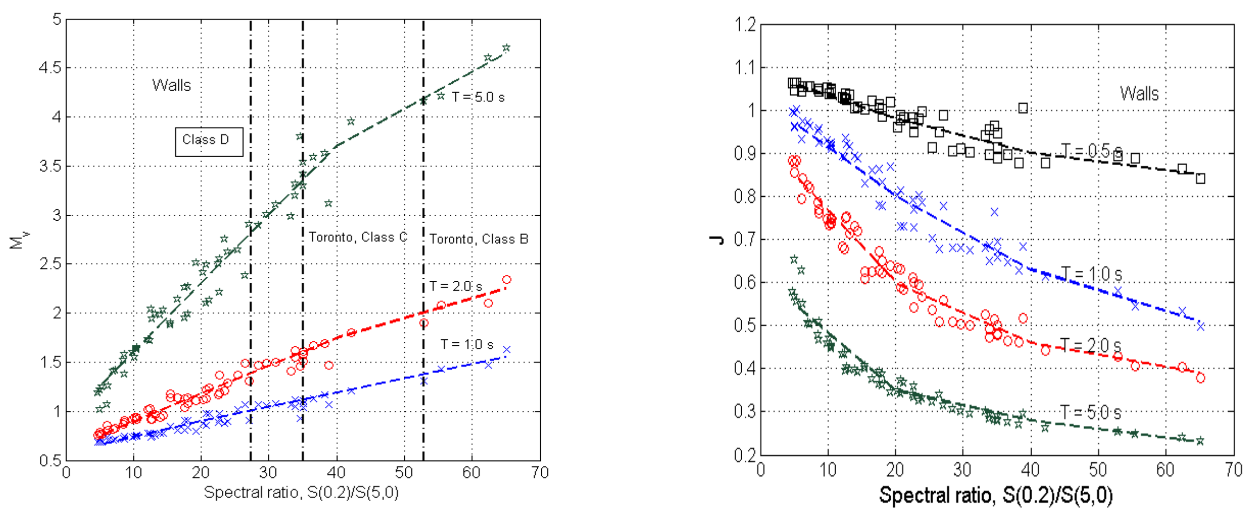

(c)

Figure 4: M_v and J factors: (a) Moment frames, (b) Braced frames, (c) Shear walls

Table 1: Higher Mode Factor M_V and Base Overturning Reduction Factor J

S(0.2)/ S(5.0)	M_v for $T_a \le 0.5$	M_v for $T_a = 1.0$	M_v for $T_a = 2$	M_v for $T_a \ge 5.0$	J for $T_a \le 0.5$	J for $T_a = 1.0$	J for $T_a = 2.0$	J for $T_a \ge 5.0$
				Moment-resisting frames				
5	1	1	1	-	1	0.97	0.92	-
20	1	1	1	-	1	0.93	0.85	-
40	1	1	1	-	1	0.87	0.78	-
65	1	1	1.03	-	1	0.80	0.70	-
				Coupled walls				
5	1	1	1	1	1	0.97	0.92	0.80
20	1	1	1	1.08	1	0.93	0.85	0.65
40	1	1	1	1.30	1	0.87	0.78	0.53
65	1	1	1.03	1.49	1	0.80	0.70	0.46
				Braced frames				
5	1	1	1	-	1	0.95	0.89	-
20	1	1	1	-	1	0.85	0.78	-
40	1	1	1	-	1	0.79	0.70	-
65	1	1.04	1.07	-	1	0.71	0.66	-
				Walls, wall frame systems				
5	1	1	1	1.25	1	0.97	0.85	0.55
20	1	1	1.18	2.30	1	0.80	0.60	0.35
40	1	1.19	1.75	3.70	1	0.63	0.46	0.28
65	1	1.55	2.25	4.65	1	0.51	0.39	0.23
				Other systems				
5	1	1	1	-	1	0.97	0.85	-
20	1	1	1.18	-	1	0.80	0.60	-
40	1	1.19	1.75	-	1	0.63	0.46	-
65	1	1.55	2.25	-	1	0.51	0.39	-

In view of the experience in other stable areas of the world and the fact that the eastern region of Canada has seen considerable seismic activity, NBC 2015 requires that seismic design be carried out in all regions of Canada. However, the code provides a much simplified design procedure for areas of low hazard. The low hazard areas are defined as those where $I_E F_s S_a(0.2)$ is less than 0.16 and $I_E F_s S_a(2.0)$ is less than 0.03. The foundation factor F_s, which substitutes for F_a, does not require the measurement of shear wave velocity and is given by

F_s = 1.0 for rock sites or when $N_{60} > 50$ or S_u is > 100 kPa,
= 1.6 when $15 \le N_{60} \le 50$ or 50 kPa ≤ S_u ≤ 100 kPa,
= 2.8 for all other cases,

where N_{60} is the average energy-corrected standard penetration resistance of the soil and S_u is the average undrained shear strength for the top 30 m below the footing, pile cap, or mat foundation. Values of F_s are selected to be the most conservative estimates for periods between 0.2 and 1.0 s. The trigger value of $I_E F_s S_a(0.2)$ beyond which detailed design would be required has been raised from 0.12 in NBC 2010 to 0.16, since even the structures below the trigger will be designed for earthquake forces, although using a simplified procedure. On the other hand, unlike in NBC 2010, the trigger now includes the importance factor to ensure that important structures perform better.

The design earthquake base shear V_s is calculated from

$$V_s = F_s S_a(T_a) I_E W_t / R_s \tag{4}$$

The code specifies that the design shear V_s should be no less than $F_s S_a(1.0) I_E W_t / R_s$. This limit is conservative in comparison to that prescribed in the detailed design procedure. Since for most cases M_V value is 1 for periods less than or equal to 1 as shown in Table 1, M_V factor is not included in Equation 4.

There is also an upper limit on the design shear equal to $F_s S_a(0.5) I_E W_t / R_s$ provided R_s is equal to or greater than 1.5.

8. Buildings with Flexible Diaphragms

Single storey buildings with large footprints, such as those used for commercial, educational, or institutional purposes, often have a flexible steel deck or wood panel diaphragm. The response of such buildings to seismic loads is strongly affected by the flexibility of the roof diaphragm. Diaphragm flexibility alters the manner in which the inertia forces, shears, and bending moments are distributed along the length of the diaphragm. In addition, it causes a significant increase in the ductility demand on the lateral load resisting system that is expected to be strained into the inelastic range under the design earthquake. NBC 2015 includes design specifications that account for the effect of diaphragm flexibility on the period of the building and the increased ductility demand on the lateral force resisting system These specifications are based on several research studies related to the seismic response of buildings with flexible diaphragms (Humar and Popovski, 2013, Tremblay and Stiemer 1996, Trudel-Languedoc et al 2012).

Flexibility of the diaphragms elongates the period of a building, so the empirical formulas for determining the period should account for it. Based on modal analysis of a large number of prototype buildings with flexible diaphragm the following empirical formulas for the fundamental period have been developed for NBC 2015.

$$\begin{aligned} T_a &= 0.05 h_n^{3/4} + 0.004L \quad \text{for shear walls} \\ T_a &= 0.035 h_n + 0.004L \quad \text{for steel moment frames and steel braced frames} \end{aligned} \tag{5}$$

where L is the span of the diaphragm in meters, between adjacent vertical elements of the SFRS. In case of multiple spans, the shortest span is to be used. The term containing L accounts for the flexibility of the diaphragm. As an alternative to the empirical formulas, the period may be determined by the method of mechanics, but the T_a so determined should not exceed 1.5 times the empirical period. As in the case of other structures, the upper limit on the analytical period for buildings with flexible diaphragm is meant to account for the possibility that the model used to calculate the period may not have considered all of the structural and non-structural elements that could contribute to the stiffness.

Studies have shown that in single storey buildings with flexible diaphragm the flexibility of the diaphragm generally causes an increase in the ductility demand on the SFRS. Consequently, to keep the ductility demand unchanged the ductility related force modification factor must be assigned a value that is smaller than the ductility capacity R_d. The revised force modification factor is denoted here by $\tilde{R}_d$.

To estimate the increase in ductility demand Humar and Popovski (2013) carried out time history analyses on 65 single-storey buildings with flexible steel deck diaphragm designed by Tremblay and Stiemer (1996) for their response to El Centro 1941 ground motion as well as to 5 other spectrum compatible ground motions. Based on the results of their study, Humar and Popovski suggested an equation relating the force reduction factor to the ductility capacity of the lateral load resisting system and the drift ratio r. The drift ratio is defined as the ratio of the maximum horizontal deformation of the diaphragm along its length (Δ_D) to the average inter-story drift of the lateral load resisting elements supporting the diaphragm (Δ_B) produced by the action of uniform static lateral load acting along the length of the diaphragm.

The proposed relationship between $\tilde{R}_d$ and R_d is as follows:

$$\begin{aligned} \tilde{R}_d &= \kappa R_d \\ \kappa &= -0.5r + 1.5 \end{aligned} \tag{6}$$

$$1 \geq \kappa \geq 0.64 \quad \text{for } \mu = 2, \quad 1 \geq \kappa \geq 0.5 \quad \text{for } \mu = 3, \quad 1 \geq \kappa \geq 0.4 \quad \text{for } \mu = 4$$

Figure 5 shows the response data obtained from the time history analyses of the 65 buildings for ground motions compatible with the UHS for Vancouver. Also shown, are the straight-line relationships given by Equation 6.

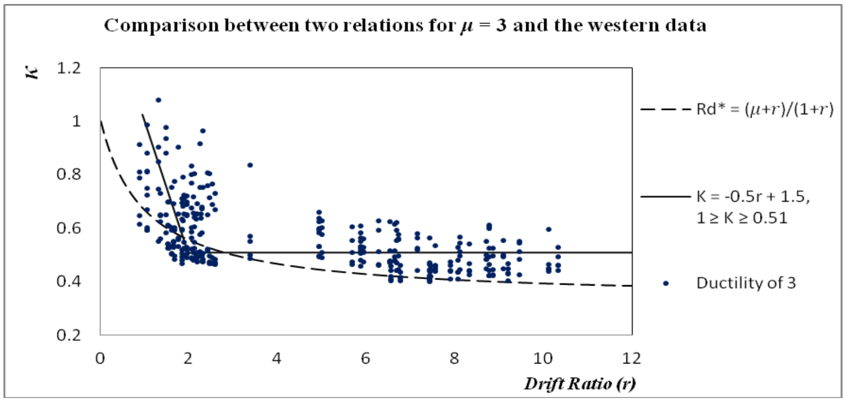

Figure 5: Relationship between κ and drift ratio *r*

Tremblay has developed a simple relationship between $\tilde{R}_d$ and the drift ratio *r* by analysing a system that consists of two springs in series with a mass attached to the end of the second spring. The first spring represents the lateral force resisting system while the second represents the diaphragm. The relationship obtained by Tremblay is

$$\tilde{R}_d = \frac{R_d \Delta_B + \Delta_D}{\Delta_B + \Delta_D} = \frac{R_d + r}{1 + r}, \quad \kappa = \frac{R_d + r}{R_d (1 + r)} \tag{7}$$

Equation 7 has also been plotted in Figure 5. It provides almost a lower bound to the dynamic analyses data. The NBC 2015 provisions on the design of buildings with flexible diaphragm are based on Equation 7. NBC 2015 also provides that as an alternative to increasing the design force by using $\tilde{R}_d$ instead of R_d according to Equation 7, the SFRS may be designed to accommodate the increased displacement.

9. Acknowledgement

A significant portion of the background material presented in this paper is based on the work of author's colleagues on the Standing Committee on Earthquake Design; their contributions are gratefully acknowledged. Where available, references have been provided. However, some of the background material is available only in the internal documents of SCED. Special thanks are extended to John Adams, Gail Atkinson, Adrian Wightman, Liam Finn, Ron De Vall, Robert Tremblay, Ken Elwood, and John Tsertobitoff for their contributions.

10. References

ATKINSON, G. M. and ADAMS, J., Ground motion prediction equations for application to the 2015 Canadian national seismic hazard maps, *Canadian Journal of Civil Engineering,* Vol. **40,** 2013, pp 988-998.

HUMAR, J. and MAHGOUB, M.A., Determination of seismic design forces by the equivalent static load method, *Canadian Journal of Civil Engineering,* Vol. **30**, 2003, pp 287-307.

HUMAR, J. and POPOVSKI, M., Seismic response of single-storey buildings with flexible diaphragms. *Canadian Journal of Civil Engineering*, Vol. **40**, 2013, pp 875-886.

MITCHELL, D., TREMBLAY, R., KARACABYLI, E., PAULTRE, P., SAATCIOGLU, M., and ANDERSON, D.L. 2003. Seismic force modification factors for the proposed 2005 edition of the National Building Code of Canada. *Canadian Journal of Civil Engineering*, Vol.**30**, 2003, pp 308-327.

TREMBLAY, R. and STIMER, S.F., Seismic Behavior of Single-storey Steel Structures with a Flexible Roof Diaphragm, *Canadian Journal of Civil Engineering*, Vol. **23,** 1996**,** pp 49-62.

TRUDEL-LANGUEDOC, S., TREMBLAY, R., SHRESHTHA, K., and ROGERS, C.A., Seismic force and ductility demand on the braced bents of single-storey buildings with flexible roof deck diaphragms *15th World Conference on Earthquake Engineering*, Lisbon, Portugal, 2012.