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Implications of New Canadian Uniform Hazard Spectra for Seismic Design and the Seismic Level of Protection of Building Structures

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ABSTRACT

This paper presents the new seismic hazard results for Vancouver and Montreal and considers the implications of these results with reference to uncertainty, probability level and seismic design forces relative to those determined using the seismic provisions of the National Building Code of Canada. The results of recent research concerning the drift performance of low rise reinforced concrete and steel moment resisting frame structures are evaluated in light of the new seismic hazard information.

CANADIAN SEISMIC HAZARD

During the past three decades, seismic hazard for use in determining design forces in the seismic provisions of the National Building Code of Canada (NBCC) has been described in terms of peak ground motions. The NBCC 1970 zoning maps were expressed in terms of peak ground acceleration at an annual probability of exceedance of 0.01; these were calculated using extreme value methodology. The 1985 and subsequent editions of NBCC use zoning maps of peak ground velocity and peak ground acceleration at a 10% probability of exceedance in 50 years; these were calculated using the Cornell-McGuire seismic hazard methodology. The Geological Survey of Canada has now calculated seismic hazard in terms of Uniform Hazard Spectra (UHS) rather than peak ground motions; these calculations were based on the most recent seismological, geological and tectonic information using so-called fourth generation methodology involving inclusion of both aleatory and epistemic uncertainties. UHS values calculated at probability levels of 2% in 50 years and 10% in 50 years, as well as a description of the methodology, are given by Adams et al (1999a).

Figure 1 shows median and 84%ile UHS at both probability levels for Vancouver and Montreal. The difference between the median and 84%ile values represents the extent of epistemic uncertainty, i.e. that due to uncertainty in the seismic hazard model, primarily in the strong ground motion relations. The extent of epistemic uncertainty is substantial with ratios of 84%ile to median values ranging as high as 2 to 3. Median values are preferred since they are closer to "expected values" but engineers need to be aware that, at probability of exceedance, there is a significant likelihood that values can be up to 3 times larger.

PROBABILITY LEVEL FOR DESIGN

While it is common for design seismic hazard information to be supplied at a 10% in 50 year probability of exceedance, the contribution of various sources of conservatism (e.g. overstrength) in the design process using hazard information at this probability level results in a much lower probability that structural failure or collapse will occur due to strong seismic ground motion. A study by SEAOC (Vision 2000 Committee 1995) recommends that normal structures be designed for "near collapse" performance under "very rare" earthquake ground motion conditions, in which "very rare" is defined in terms of a probability of exceedance of 10% in 100 years (return period of 970 years). The most recent NEHRP provisions (Building Seismic Safety Council 1997) have adopted the principle that ground motions used in design should result in a uniform margin against collapse; for this purpose they have specified use of "maximum considered earthquake ground motion" which is defined as that having a 2% in 50 year probability of exceedance (return period of about 2500 years). It is therefore useful to examine the relationship between 10% in 50 year and 2% in 50 year hazard. Figure 2 shows the variation of spectral values at a period of 1.0s with return period for both Vancouver and Montreal, which are typical for locations in western and eastern Canada respectively. The values in this figure have been normalized to hazard at 10% in 50 years (return period of 475 years).

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Since the seismic provisions of NBCC and other design codes are based on experience in regions of moderate to high seismicity (e.g. Vancouver), it can be assumed from this figure that conservatism in the design process results in an effective increase in seismic resistance from the nominal design value by a factor of 2 (i.e. the ratio of 2% in 50 year to 10% in 50 year hazard in Vancouver). However, applying the same factor in Montreal corresponds to a return period of 1400 years, i.e. approximately 3.5% probability of exceedance in 50 years; Adams et al (1999b) show the corresponding return period to be 1600 years for spectral values with a period of 0.2s. The implication is that the use of 10% in 50 year hazard as the basis for design results in significantly dissimilar risks of structural failure in different regions of the country. One solution to this dilemma is to use the 2% in 50 year hazard values for design and then apply a reduction factor of ½ to recognize conservatism in the design process. Applying this procedure to the information in Figure 2 would result in no change of design forces in Vancouver but a 40% increase in Montreal, an increase which would be justified in order to ensure the same risk of failure (due to seismic hazard) as in Vancouver. While this figure is only applicable for structures having a period of 1.0s, the effect would be similar for structures with other periods and in other locations in eastern Canada.

IMPLICATIONS OF NEW HAZARD RESULTS FOR SEISMIC DESIGN FORCES

While the above discussion provides a rationale for changing the probability level used in determining seismic design forces, the implications of the new hazard results can be best evaluated by comparisons using the probability level used for the determination of design forces in the current (1995) edition of NBCC, i.e. 10% in 50 year probability of exceedance. In NBCC 1995, the minimum lateral seismic force V is given by

$$V = U(v\ S\ I\ F\ W/R) \tag{1}$$

in which U = 0.6 is a calibration factor, R = a force modification factor (values range from 1 to 4), v = zonal velocity ratio (corresponding to peak ground velocity in m/s), S = seismic response factor (a function of period T), I = importance factor (1 for buildings of normal importance), F = foundation factor (1 for buildings on rock or stiff soil), and W = dead load. For the purpose of comparison, let F = I = R = 1 (i.e. elastic force); values of v for Vancouver and Montreal are 0.20 and 0.10 respectively. The NBCC 1995 base shear coefficient V/W is then given by

$$[V/W]_{NBCC} = 0.6\ v\ S \tag{2}$$

For the same conditions and assuming that no calibration factor is applied, the base shear coefficient using new seismic hazard results in UHS format is given by

$$[V/W]_{UHS} = S_a / g \tag{3}$$

in which S_a is the median value of the UHS at the particular structural period T.

Figure 3 shows a comparison of NBCC and 10% in 50 year median UHS base shear coefficients for Vancouver and Montreal. Intermediate period values (T from 0.3s to 0.5s) in Montreal are similar but the UHS coefficients are slightly higher in the short period region. The UHS coefficients in Vancouver are much higher than the NBCC values in both the short and intermediate period regions. UHS values fall below the NBCC coefficients in the long period region, but this is to be expected since the NBCC S factor includes recognition of higher mode effects while the UHS coefficient uses only spectral values. An important observation is that, at the 10% in 50 year probability level, the design forces in Vancouver would increase relative to those in Montreal. If a 0.5 multiplier were applied to 2% in 50 year hazard values, as described in the previous section, then the relative force values between these two locations would be quite similar to the relative NBCC 1995 values.

PERFORMANCE OF STRUCTURES DESIGNED USING NBCC 1995 SEISMIC PROVISIONS

Heidebrecht and Naumoski (1999) present the results of an extensive investigation into the seismic performance of a six storey ductile moment-resisting reinforced concrete frame building (height of 25.2 m) designed for Vancouver seismic hazard. The seismic resistance in the transverse direction (plan dimension of 63 m) is taken entirely by two end frames for which the design is dominated by seismic loads; all frames (9m spacing) in the longitudinal direction are designed to have seismic resistance, but the design of these frames is dominated by gravity loads. Inelastic models of both the transverse and longitudinal frames in this building were subject to an ensemble of 15 time-history excitations having spectral shapes similar to those of design level seismic ground motions expected in Vancouver. The performance of the frames was evaluated in terms of maximum interstorey drift. A SEAOC study (Vision 2000 Committee 1995) indicates that maximum drifts should be below 0.5% for operational performance, below 1.5% for life safe performance and below 2.5% for near collapse performance.

Figure 4 presents the mean plus one standard deviation (M+SD) drift for the frames in two directions in relation to the spectral acceleration at the fundamental period of each frame. The excitation of the transverse frames (in terms of fundamental period spectral acceleration) should not exceed 0.36g and 0.52g to have a high level of confidence of life safe and near collapse performance respectively. The corresponding excitations for the longitudinal frame would be 0.5g and 0.7g (extrapolated) respectively. Since the period of both frames is quite near to 1.0s, the results in this figure can be evaluated with reference to the 1.0s UHS values in Figure 1, median and 84%ile spectral values of 0.17g and 0.35g respectively. Since the excitation time-histories were each scaled to the fundamental period spectral value in the analyses used to obtain the results shown in Figure 4, it is appropriate to make comparisons in relation to the median spectral value of 0.17g. The transverse frame could sustain excitations of more than twice the design level with a high level of confidence that the life safe performance drift limit would not be exceeded and not reach near collapse for excitations up to three times the design level.

A similar study on six storey frame building was conducted by Biddah (1998). The building is three-bay by four-bay office building with a floor area of 770 m^2 and a total height of 22.5m; the frames considered in this study are three-bay ductile moment-resisting perimeter frames. Seismic design was based on Vancouver seismic hazard and was done using several different design philosophies. The results reported here concern frames designed using the strong column weak beam (SCWB) and weak column strong beam (WCSB) design philosophies.

Table 1 shows summary performance evaluation results for both the reinforced concrete and steel frame studies outlined above. In comparing the performance of the reinforced concrete and steel frames, it is clear that the more flexible steel frames have much higher maximum drifts than the reinforced concrete frames, even though both are designed to the same NBCC drift limit of 2%. All frames are below the design drift limit when excitation is at the design level. However both reinforced concrete frames are at the SEAOC operational performance limit while both steel frames are at or above the SEAOC life safe performance limit. Both concrete frames remain at or below the near collapse drift limit for excitations at three times the design level while both the steel frames reach or exceed that limit at twice the design excitation. The performance of the WCSB steel frame is poorer than that of the frame designed using the SCWB philosophy; the difference in performance is accentuated at higher excitations.

Consider now the evaluation of performance when excitation is expressed in terms of 10% in 50 year median UHS spectral acceleration at the fundamental period of each structure. Performance at the median UHS and twice the median UHS levels is in general slightly better than when excitation is expressed in terms of peak ground velocity relative to design velocity. Both steel frames have drifts below the SEAOC near collapse performance limit when excitations are at twice the median UHS level. Both reinforced concrete frames have drifts below the SEAOC life safe performance limit at this excitation level. For excitations at the median UHS level, the steel frames are below the life safe drift limit while the reinforced concrete frames are at or below the operational drift limit.

The sections of Table 1 which concern probabilities of exceedance are based on the Vancouver curve in Figure 2. The results show that the probabilities of exceedance for reaching the SEAOC life safe performance limit drift for the two steel frames are of the same order of magnitude as the 10% in 50 year level used to express the median UHS. However the probabilities of exceedance for reaching the SEAOC near collapse performance limit for the two steel frames are in the order of 1% in 50 years. The performance advantage of the SCWB frame relative to the WCSB frame is quite marked, nearly a factor of two in terms of exceedance percentage in a 50 year period. The reinforced concrete frames both have probabilities of exceedance which are well below those of the steel frames.

DISCUSSION AND CONCLUSIONS

The new seismic hazard results show that modelling uncertainty can have a significant effect on the seismic hazard level; these uncertainties need to be recognized both in seismic design and in the evaluation of performance, particularly recognizing the high likelihood that ground motions may be much larger than the values used in design. Comparisons of design forces in Vancouver and Montreal determined using the new results in relation to those determined from the current NBCC seismic provisions indicate significant relative differences; even larger differences can be expected for a number of other geographical locations in the country.

Results of recent research on structures with fundamental periods in the range of 1 to 2 seconds interpreted in light of the new hazard results indicate that the probabilities of reaching near collapse drift levels are in the order of 1% in 50 years, which is well below the design hazard probability level. However, for the more flexible structures, life safe drift limits are quite likely to be reached at probabilities at the same order of magnitude as the design hazard probability. This may not be acceptable and suggests that more flexible structures may need to be designed using a lower drift limit.

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Table 1 Summary of Drift Performance Evaluations for Six Storey Steel and Reinforced Concrete Frames

	Reinforced Concrete Frames		Steel Frames	
	Longitudinal	Transverse	SCWB	WCSB
Fundamental Period (sec)	1.04	1.16	2.24	2.17
	Mean plus one standard deviation maximum interstorey drift (%)			
Excitation at design level (0.2 m/s)	0.5	0.5	1.5	1.8
Excitation at 2 x design level (0.4 m/s)	1.2	1.4	2.5	2.8
Excitation at 3x design level (0.6 m/s)	2.0	2.5	3.0	4.2
Excitation at 10%/50 yr median UHS $S_a(T_1)$	0.35	0.5	1.2	1.4
Excitation at twice median UHS $S_a(T_1)$	1.0	1.3	2.0	2.3
	Exceedance percentage in 50 year period associated with specified interstorey drift			
Mean plus one standard deviation drift = 1.5%	0.6	1.6	5.5	8
Mean plus one standard deviation drift = 2.5%	<0.5	0.6	0.8	1.4

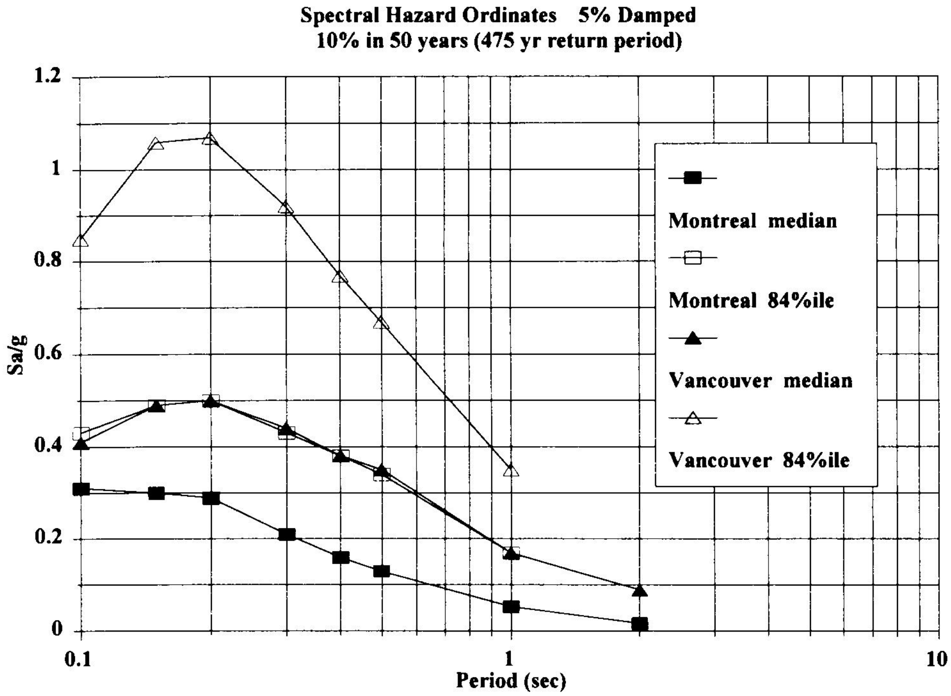

Figure 1 New Hazard Spectra for Montreal and Vancouver

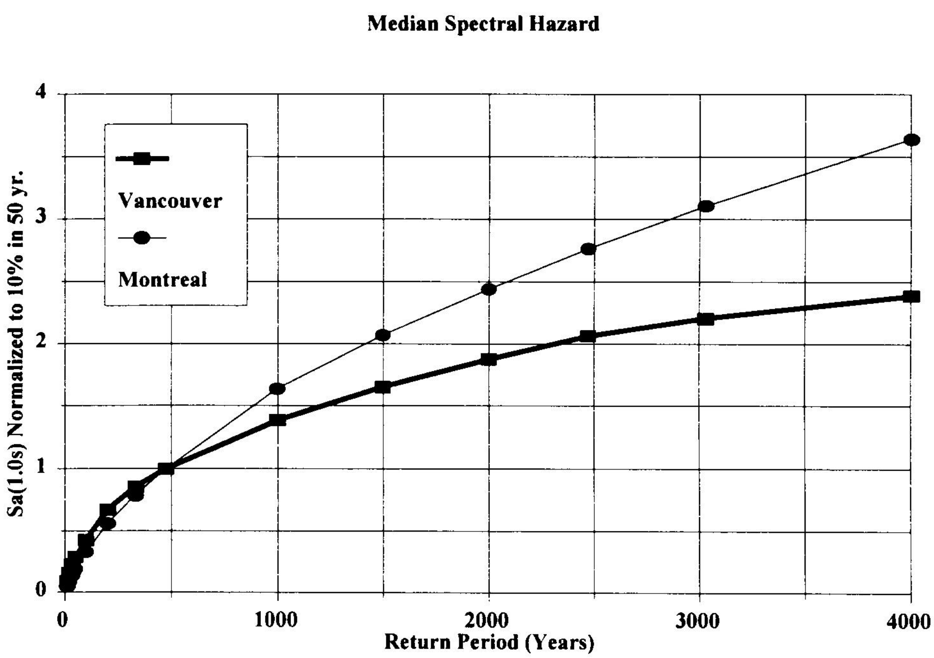

Figure 2 Effect of Return Period on Hazard at T = 1.0 sec

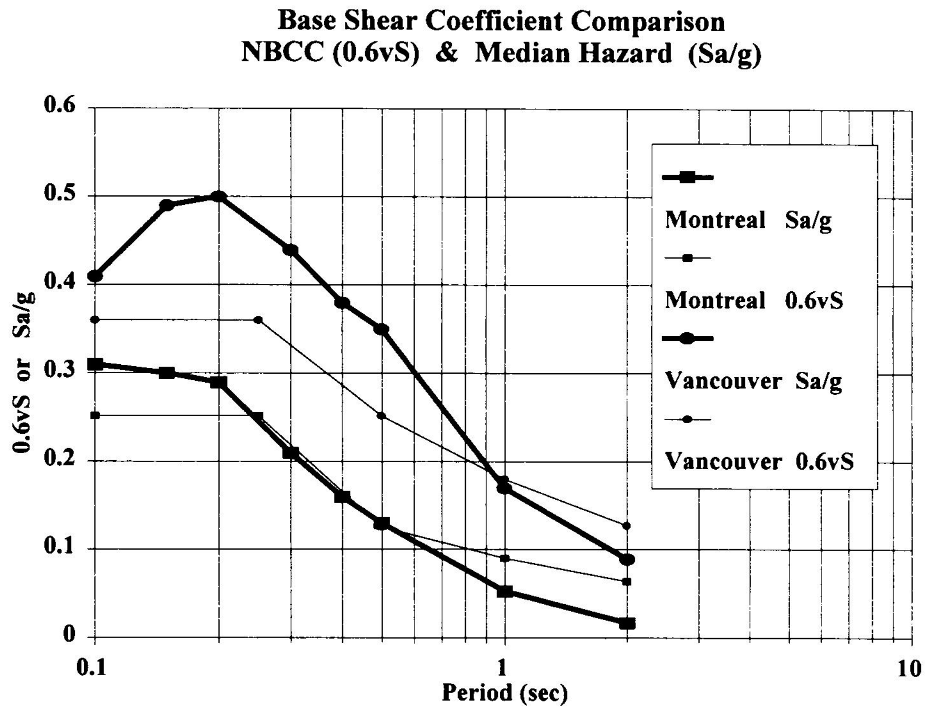

Figure 3 Comparison of Base Shear Coefficients, NBCC and Median 10% in 50 Year Hazard

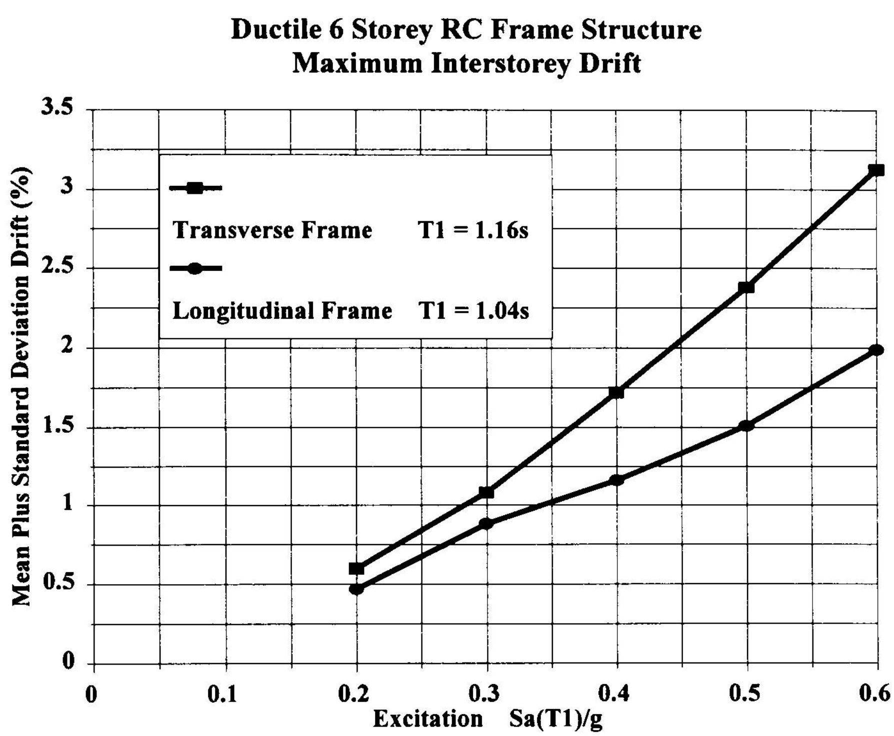

Figure 4 Maximum Dynamic Drifts in Six Storey RC Frame Structure