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RELIABILITY-CONSISTENT SEISMIC DESIGN LOAD CONTOUR MAPS

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ABSTRACT

Some seismic design provisions given in the 1995 edition of the National Building Code of Canada have been changed or replaced by those given in the 2005 edition of the NBCC. One of the objectives for these changes is aimed to achieving an improved reliability-consistent seismic design. However, information on and detailed quantitative assessment of reliability level and the degree of reliability consistency of the designed structures according to the codes are lacking.

In this study, we indicate that the use of a consistent high return period for specifying the seismic design level in Canada does not ensure the reliability-consistent seismic design since probabilistic characteristics of seismic hazard differ significantly even in a single region. We propose to use a simple approach that can be used for calibrating the required seismic design load levels such that the structures designed according to the NBCC format meet pre-selected target reliability levels. The approach is based on the structural reliability methods and considers both elastic and inelastic structural responses. The obtained design seismic loads are presented in the form of reliability-consistent seismic design load contour maps, and their implication in the codified design is discussed.

Introduction

From time to time, design loads and resistances in structural design codes are changed and modified to incorporate new understandings and to reflect societal needs. Summary of the major changes that are incorporated in the 2005 edition of National Building Code of Canada are described by Heidebrecht (2003). The changes include 1) the use of the uniform hazard spectra (UHS) instead of the scaled standard response spectrum, 2) the adoption of the 2% rather than 10% in 50 years return period for specifying the seismic load parameter, and 3) the use of different design factors and drift limits. One of the objectives for these changes is aimed to achieving a reliability-consistent seismic design for western and eastern Canada. However, information on and detailed quantitative assessment of reliability level and the degree of reliability consistency of the designed structures according to the codes are lacking. Therefore, the notion that the use of the current edition of the NBCC code will result in an improved reliability-consistent seismic design should at least be verified or questioned.

We show in the present study that the use of a consistent and high return period for specifying the seismic design level alone does not ensure the reliability-consistent seismic design for a country, such as Canada, with immense spatial extent and significant different probabilistic seismic hazard characteristics. The differences are evidenced by the results presented by Adams and Halchuk (2003) and Adams and

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Atkinson (2003). To improve the safety consistency of the codified design, we propose to use a simple approach to calibrate the required seismic design load levels or their corresponding return periods such that the structures designed using the calibrated seismic loads and the NBCC format meet pre-selected tolerable probability of failure levels. The formulation of the simple approach is based on the reliability analysis methods and the structure being approximated by a hysteretic bilinear single-degree-of-freedom (SDOF) system. It can also take into account multiple performance levels such as the incipient of damage (i.e., yield) and incipient of collapse (i.e., displacement ductility capacity being exceeded), which are akin to the bi-levels performance-based design and checking advocated by Wen (2001). For given tolerable failure probability levels, the calibrated site-dependent seismic design loads can be represented as seismic design load contour maps, and their implications in the codified design are discussed.

Probabilistic Characterization of Seismic Demand

The seismic hazard assessment is often carried out using Cornell-McGuire method (Cornell 1968; Esteva 1968; McGuire 1976), although other approaches are available in the literature (Hong et al. 2006). The Cornell-McGuire method incorporates the information on seismic source zones, magnitude-recurrence relations, and attenuation relations to estimate the seismic hazard for a selected ground motion parameter such as the peak ground acceleration, and the pseudo-spectral acceleration (PSA) responses. For the estimation, one can use a numerical integration method (McGuire 1976), or directly use the simple simulation technique (Goda and Hong 2006; Hong et al. 2006). The latter could be advantageous because it can easily cope with the Poissonian and non-Poissonian earthquake occurrences. In such a case, the evaluation steps for given probabilistic seismicity information are:

1) Sample the parameters defining the attenuation relations;
2) Sample the seismic catalog for each zone, and error terms of the attenuation relations associated with each event; and
3) Calculate the ground motion parameters at a site of interest.

By adopting the detailed seismicity information given by Adams and Halchuck (2003), and using a computer code implementing the simulation approach (Goda and Hong 2006, Hong et al. 2006), samples of PSA responses are obtained. A post-processing of the samples can be carried out to form the samples of the annual maximum PSA responses, $S_A(T_n,\xi)$, where T_n denotes the natural vibration period of the linear elastic SDOF system, and ξ denotes the damping ratio. The samples of $S_A(T_n,\xi)$ can be used to assess its probabilistic characteristics. It is found that in general $S_A(T_n,\xi)$ is lognormally distributed in the upper tail region. It must be emphasized that throughout this study, the unconditional probability distribution of $S_A(T_n,\xi)$ is used.

An example presentation of the site-dependent mean and coefficient of variation (cov) of $S_A(T_n,\xi)$ for the fitted distributions is shown in Figs. 1 and 2. The figures show that the statistics of $S_A(T_n,\xi)$ for western and eastern Canada are significantly different, especially for the cov values. This has significant implications for assessing seismic hazard. For example, Adams et al. (1999) indicated that the same multiple of the 475-year (i.e., 10% in 50 years) return period values, which is adopted in the 1995 edition of the NBCC (NBCC 1995), does not correspond to the same return period. This observation is also valid if responses at different vibration periods are considered. A plot to illustrate this problem with a multiplication factor of 2 is depicted in Fig. 3 indicating the varied exceedance probability level (i.e., $1/T$) or safety level. Therefore, the significant difference in the cov value of $S_A(T_n,\xi)$ prevents the use of a scaling constant and the 475-year return period values to define the return period values of $S_A(T_n,\xi)$ with the same return period.

To see the implication of the above in the codified seismic design, we note that an overall force modification factor R_A is employed in the 2005 edition of the NBCC, where $R_A = R_d R_o$, in which R_d is the

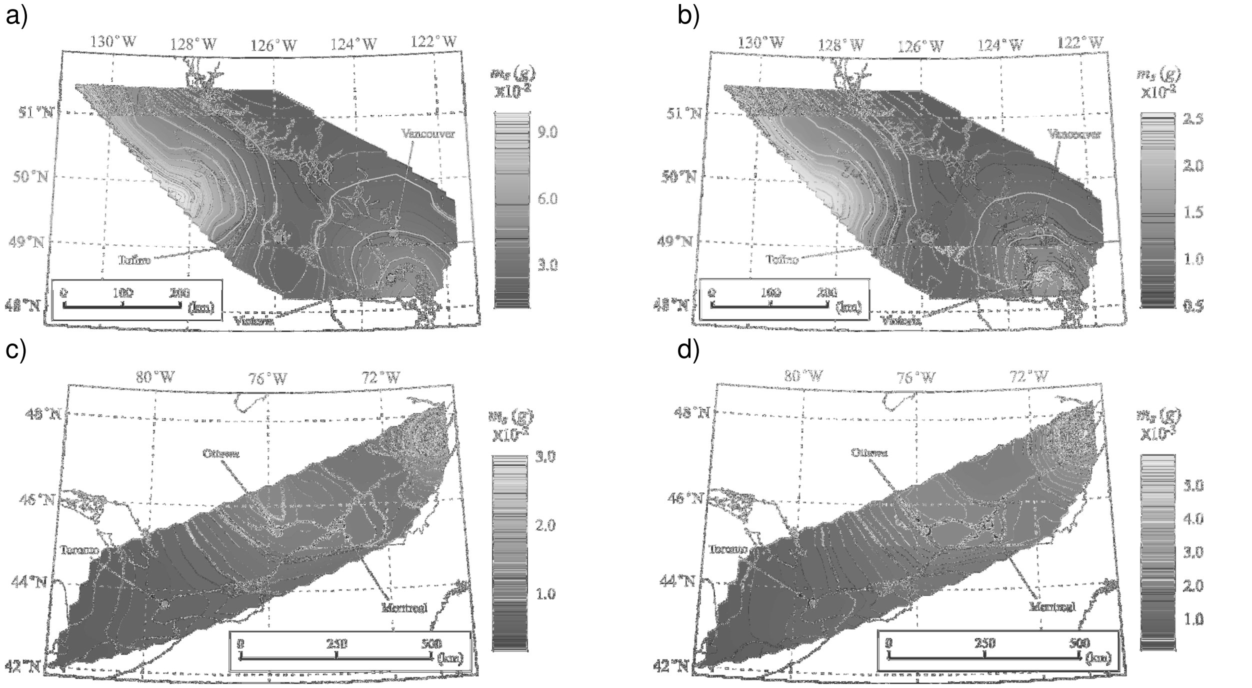

Figure 1. Contour maps of the mean of $S_A(T_n, \xi=0.05)$, m_s, obtained from the fitted probability distributions of the annual maximum $S_A(T_n, \xi=0.05)$: a) T_n = 0.2 s for western Canada, b) T_n = 1.0 s for western Canada, c) T_n = 0.2 s for eastern Canada, d) T_n = 1.0 s for eastern Canada.

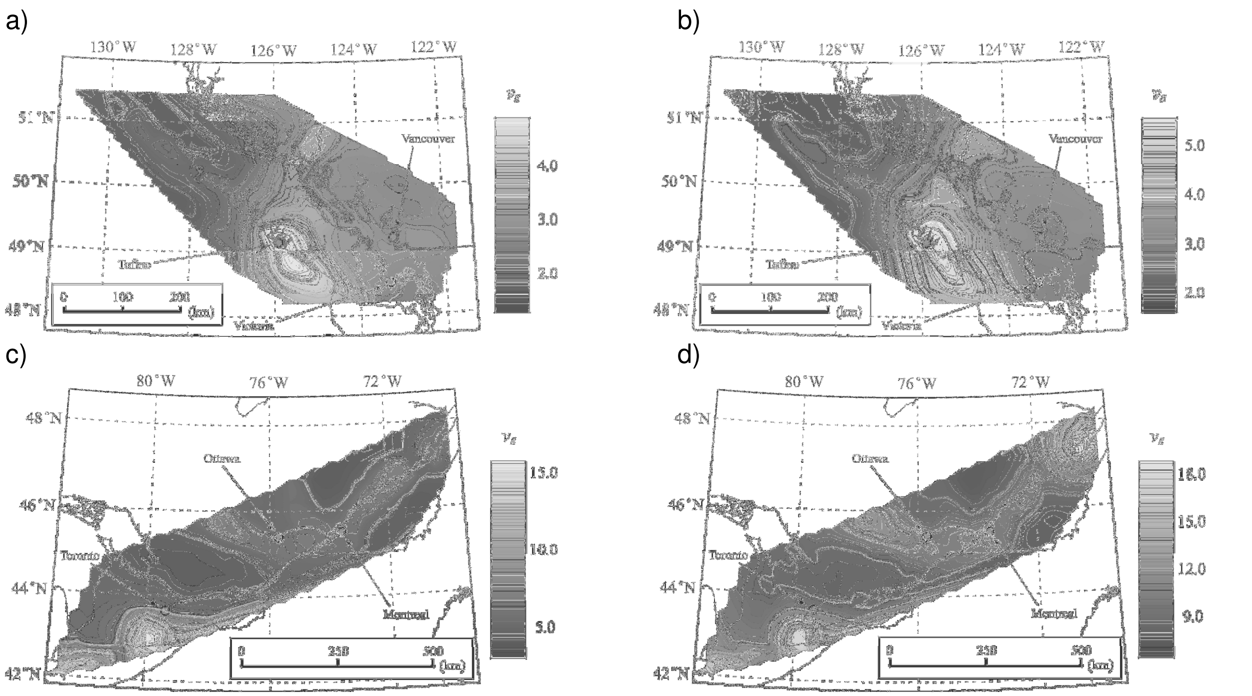

Figure 2. Contour maps of the cov of $S_A(T_n, \xi=0.05)$,v_s, obtained from the fitted probability distributions of the annual maximum $S_A(T_n, \xi=0.05)$: a) T_n = 0.2 s for western Canada, b) T_n = 1.0 s for western Canada, c) T_n = 0.2 s for eastern Canada, d) T_n = 1.0 s for eastern Canada.

a) b)

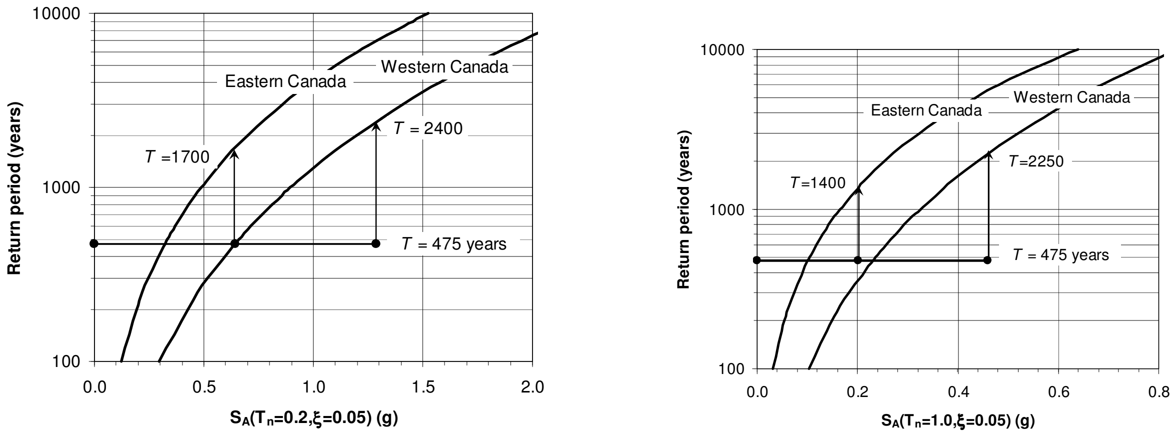

Figure 3. Illustration of the twice of the 475-year return period values that do not correspond to the same return period.

a) b)

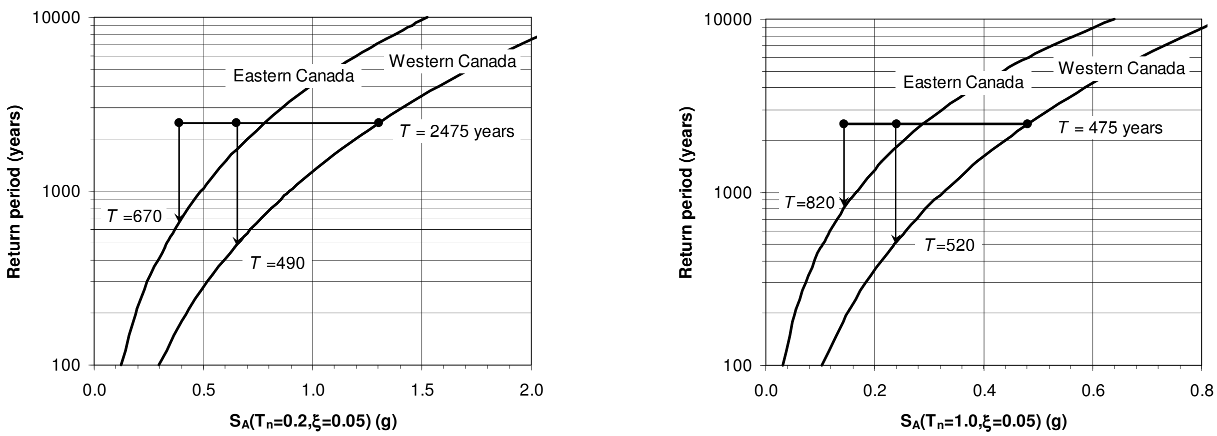

Figure 4. Illustration of 50% of the 2475-year return period values that do not correspond to the same return period.

ductility-related force modification factor and R_o is the overstrength-related force modification factor. We also note that the 2% in 50 years (2475-year) return period value of $S_A(T_n, \xi)$, denoted by $S_{AT}(T_n, \xi)$ with T=2475 (years), is recommended in the 2005 edition of the NBCC, and that the designed structure is actually overstrengthed by a factor of R_n. If this overstrengthening and the overall modification factor lead to the structural yield capacity equal to 50% of $S_{AT}(T_n, \xi)$, these yield capacities do not correspond to the same return period level or safety level as illustrated in Fig. 4. Therefore, again the significant differences in the cov values of $S_A(T_n,\xi)$ prevent the use of the overall force modification factor and $S_{AT}(T_n, \xi)$ to achieve a consistent level of safety for yielding. The implication of this observation as well as the level of safety for collapse will be discussed in the following sections.

Note that $S_A(T_n,\xi)$ represents the seismic demand for a linear elastic SDOF system. If the inelastic demand for a hysteretic bilinear SDOF system is of interest, Hong and Hong (2006) suggested that the displacement ductility demand $\mu(\mathbf{A})$ for a given normalized yield strength level ζ can be modeled as a Frechet variate with typical cov of less than about 1.0, and mean of $\mu(\mathbf{A})$, m_μ, predicted using the following empirical equation,

$$m_\mu = \exp\left(\left(-\alpha_1 \ln \zeta\right)^{\beta_1}\right) \tag{1}$$

where $\mathbf{A}$=[ϕ,γ,T_n,ξ], γ is the ratio of the post-yield stiffness to initial stiffness, and the parameters α_1 and β_1 are given by empirical equation in their study. In particular, for γ=0.05 and ξ=0.05, α_1 =1.4885 and β_1 =1.28 for T_n = 0.2 s, and α_1 =0.9937 and β_1 =1.11 for T_n =1 s.

Reliability Assessment and Design Consideration

The reliability analyses of structures under seismic excitations have been reported in the literature including those by Bazzurro and Cornell (1994), Cornell (1996), Han and Wen (1997), Shome and Cornell (1999), Vamvatsikos and Cornell (2002, 2005), and Hong and Hong (2006). The studies of Cornell and associates for assessing the structural reliability are formulated by considering that the seismic demand for a structure can be characterized based on the results of the nonlinear dynamic analyses, such as the incremental dynamic analysis for a few records. This is then combined with probabilistic characterization of a seismic intensity measure for estimating the probability of failure. Therefore, it effectively separates the structural analysis and seismic hazard assessment which is efficient, although it requires some detailed structural characteristics and a set of selected ground motion records for the structural analysis. The approach advanced by Han and Wen (1997) combines the seismic hazard assessment together with the nonlinear dynamic analysis of the structure to estimate the probability of failure. It also requires some detailed structural information, and provides accurate reliability estimates. This could slightly more CPU-intensive if one is interested in carried out analysis for many sites. The approach proposed by Hong and Hong (2006) is based on the consideration that the response of a structure can be approximated by a hysteretic bilinear SDOF system. Therefore, it sacrifices some accuracy and it is “generic”. This approach, which is explained in the following, is employed in this study for its simplicity. Note that the adequacy of the use a hysteretic bilinear system to approximate the structural behaviour is at least supported by the results of nonlinear static pushover analysis and the incremental dynamic analysis of six steel frames designed according to the 1995 edition and 2005 edition of the NBCC (Wang 2006).

To evaluate the probability of incipient of yield, which is referred to as the probability of incipient of damage P_D, and the probability of the displacement ductility capacity being exceeded, which is referred to as the probability of incipient of collapse P_C, we first relate the structural capacity to the code recommended seismic design load. We note that the minimum strength requirement dictates that the yield capacity of the designed structure should be equal to or greater than the base shear force V given by,

$$V = S(T_n) M_V I_E W / R_A \tag{2}$$

where $S(T_n)$ denotes the design spectral acceleration, M_V is the higher mode factor, I_E is the importance factor, W represents the building dead load weight plus 25% of snow load and R_A= $R_d R_o$. V should be greater than or equal to $S(2.0)W/(R_d R_o)$, and for seismic force resisting systems with an $R_d \geq 1.5$, V should be less than or equal to two thirds of $S(0.2)M_V I_E W/(R_d R_o)$.

Note that $S(T_n)$ is related to the “median estimate” rather than the “mean estimate” of the 2475-year return period value of $S_A(T_n,\xi)$. The debate on whether the former or the latter is a more adequate measure is ongoing in the literature, and is directly rooted in the philosophical view of probability, which is outside of the scope of the present study. For simplicity and for the purpose of illustrating the proposed approach in this study, we shall follow the Bayesian view and consider that $S(T_n)$ can be related to the “mean estimate” of the T-year return period value. In such a case, it can be shown that by considering that the design is governed by the minimum design base shear requirement given in the 2005 edition of NBCC the ratio of the yield capacity of a designed structure to the elastic seismic demand, ζ, can be expressed as,

$$\zeta = \frac{R_n L_m}{R_A} \frac{1}{L}, \tag{3}$$

where R_n denotes the ratio of the yield capacity of the constructed structure to the design yield capacity,

L_m represents the ratio of the total mass M to the generalized mass Λ considering the first vibration mode because M (i.e., W) rather than Λ is used in Eq. 2 estimating V, and $L=S_A(T_n,\xi)/S_{AT}(T_n,\xi)$ denotes the normalized seismic hazard. Note that in writing the above equation it is considered that the values of M_V, I_E, the velocity-based site coefficient F_v, and the acceleration-based site coefficient F_a correspond to those suggested in the code. Consequently, these variables need not be considered in Eq. (3).

Since the cov of the material strength is relatively small as compared with that of $S_A(T_n,\xi)$, the uncertainty in R_n can be ignored without introducing any noticeable error. The values of L_m for a few designed steel moment resisting frames are about 1.25 (Wang 2006). Since, as indicated in the previous section, $S_A(T_n,\xi)$ is lognormally distributed with cov of v_s, L is lognormally distributed with the cov of v_s and mean m_L given by,

$$m_L = \sqrt{1+v_s^2}\,\exp\left(-\beta_T\sqrt{\ln\left(1+v_s^2\right)}\right), \tag{4}$$

where $\beta_T=\Phi^{-1}(1-1/T)$ and $\Phi^{-1}(\bullet)$ is the inverse standard normal distribution function. Based on this and above, we can show that $\ln(\zeta)$ is a normal variate with the mean $m_{\ln(\zeta}$ given by,

$$m_{\ln(\zeta)} = \ln\left(R_n L_m / R_A\right) + \beta_T\sqrt{\ln\left(1+v_S^2\right)}\,, \tag{5}$$

and the standard deviation $\sigma_{\ln(\zeta)}$ equals $(\ln(1+v_s^2))^{1/2}$. Therefore, P_D, is,

$$P_D = \Phi\left(-m_{\ln(\zeta)}/\sigma_{\ln(\zeta)}\right) = \Phi\left(-\left(\ln\left(R_n L_m / R_A\right) + \beta_T\sqrt{\ln\left(1+v_S^2\right)}\right)/\sqrt{\ln\left(1+v_S^2\right)}\right). \tag{6}$$

To evaluate the probability of incipient of collapse P_C, we note that for a structure with the displacement ductility capacity, μ_R, the limit state defining the incipient of collapse, g_c, can be expressed as,

$$g_c = \mu_R / \mu(\mathbf{A}) - 1\,. \tag{7}$$

Therefore, the probability of collapse P_C, can be evaluated using,

$$P_C = \int_{\Omega_\zeta}\left(\int_{g_c<0} f_{\mathbf{X}}(\mathbf{x}|\zeta)d\mathbf{x}\right)d\Phi\left(\frac{\ln(\zeta)-m_{\ln(\zeta)}}{\sigma_{\ln(\zeta)}}\right), \tag{8}$$

where Ω_ζ denotes the domain of ζ, $\mathbf{X}$ denote the set of random variables μ_R and $\mu(\mathbf{A})$ and $f_{\mathbf{X}}(\mathbf{x}|\zeta)$ denote the joint probability density function of $\mathbf{X}$ conditioned on ζ. Note that μ_R could be considered as a lognormal variate (Diaz-Lopez and Esteva 1991) with the cov of μ_R, denoted by $v_{\mu r}$. Since by definition both μ_R and $\mu(\mathbf{A})$ are defined for values greater than 1.0, it is considered that μ_R can be modeled as truncated lognormal variate and $\mu(\mathbf{A})$ can be modeled as truncated Frechet variate.

Eq. 8 can be evaluated using the nested reliability methods Wen and Chen 1987) or the combination of the point estimate method and the first-order reliability method (Hong 1996). Alternatively, one could use a simple simulation to evaluate Eq. 8 which is employed in the following section for numerical analyses.

Sensitivity of probability of incipient of damage and incipient of collapse

From Eq. 6, it can be observed that the probability of incipient of damage P_D is completely controlled by the quantity $R_n L_m/(R_d R_o)$, the return period T, and the cov of $S_A(T_n, \xi)$, v_s. These quantities, which define the distribution parameters of ζ (i.e., $\ln(\zeta)$), also control the probability of incipient of collapse P_C if the probabilistic model of μ_R and $\mu(\mathbf{A})$ are given. To assess the sensitivity of P_D and P_C to v_s and T, first it is considered that R_n/R_o equals 1, and L_m equals 1.25; the mean and cov of μ_R before the truncation, denoted by $m_{\mu R}$ and $v_{\mu R}$, are equal to R_d and 0.5; and the mean and cov of $\mu(\mathbf{A})$ before the truncation equal to m_μ given in Eq. (1) and 0.8. Note that the consideration of R_n/R_o=1 and R_d/m_μ=1 implies that the ductility-related force modification factor and the over-strength related force modification factor reflect adequately the actual overstrength and ductility capacity without bias. Based on these considerations, the obtained values of P_D and P_C are shown in Figure 5. The results shown in the figure suggest that:

1) For a given seismic design level (i.e., T), the differences between P_D values are more significant than that between P_C values. As v_s increases, the rates of decrease in P_D and in P_C are most significant for lower values of v_s. The differences between the log of P_D values and between log of P_C values are insensitive to the considered T.
2) The rate of decrease in P_C depends somewhat on the considered ductility level.
3) P_D for the considered case is independent of the T_n which can be see from Eq. 6.
4) To maintain the same tolerable level of P_D or of P_C, the required T value differ significantly for different values of v_s. This is because the lines shown in the figure are very flat and a slight change in the failure probability (P_D or P_D) leads to a significant change in T.

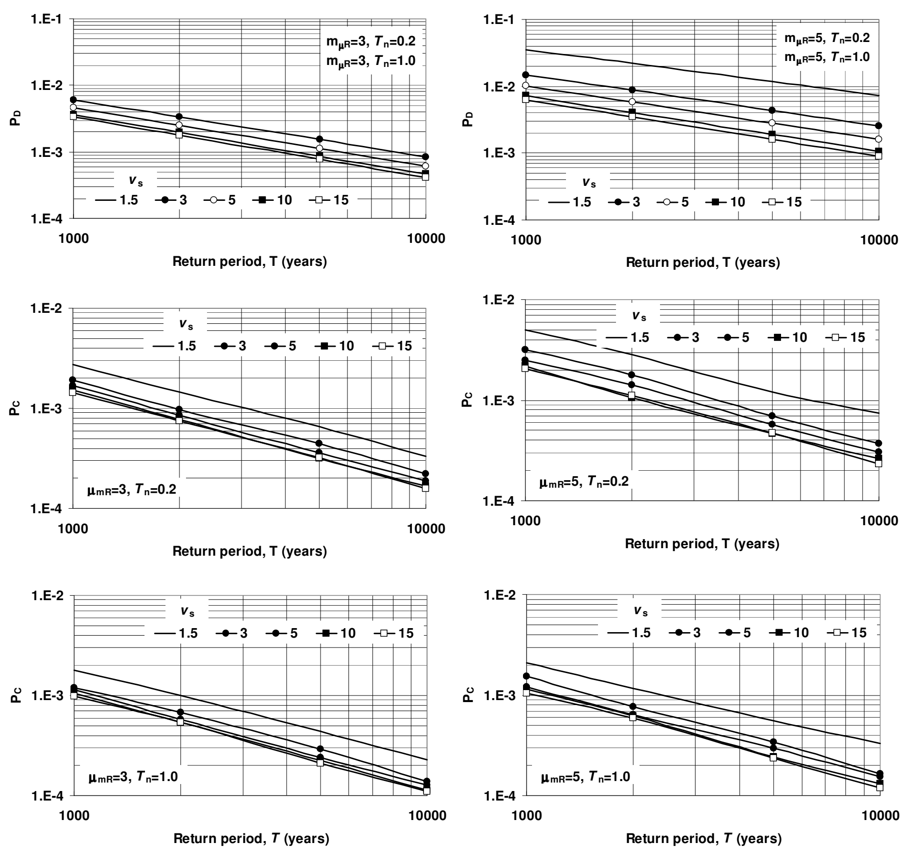

Figure 5. Sensitivity of P_D and P_C to T, v_s, and $m_{\mu R}$.

The above observations suggest that one must vary the return period T used to specify the seismic design load to maintain the same consistent tolerable probability of incipient of damage P_D for western and eastern Canada. This is the case even if one is only interested in a single region since the cov of $S_A(T_n,\xi)$ for a region alone varies significantly as shown in Fig. 2. Similar conclusion is obtained if a consistent P_C rather than P_D is considered.

Observation 2) indicates that if a single load contour map for different ductility capacity levels is desirable, besides of the current ductility-related force modification factor, one must introduce an adjustment factor which depends on the ductility capacity level. Observation 4) suggests that a significantly different T value must be employed to achieve the same desirable reliability level for structures with different displacement ductility capacity.

Seismic Load contour maps

To illustrate the concept of reliability-consistent seismic design load contour maps, only a single class of structures is considered for western Canada. The parameters for the class of structures are T_n=1.0, ξ=0.05, γ=0.05, R_n/R_o=2, L_m=1.25, $m_{\mu R}$=5, $v_{\mu R}$=0.5, R_d=5, m_μ as defined Eq. 1, and cov of $\mu(\mathbf{A})$ equal to 0.8. It should be emphasized that the statistics of μ_R and $\mu(\mathbf{A})$ are those before the truncation of the probability distribution functions. The arbitrarily selected tolerable (annual) P_D and P_C are considered to be equal to 6.21×10^{-3} and 5×10^{-4}, (i.e., reliability index of 2.5 and 3.29 per year), respectively.

For the given value of P_D, T can be evaluated by rewriting Eq. 6 as,

$$T = 1/\left(1 - \Phi\left(-\Phi^{-1}(P_D) - \ln(R_n L_m / R_A)/\sqrt{\ln(1+v_S^2)}\right)\right) \tag{9}$$

The obtained values of T are presented in Fig. 6a. The figure shows that for the considered class of structures, the required return period T that meets the tolerable P_D level varies from 940 to 4050 years.

a) b)

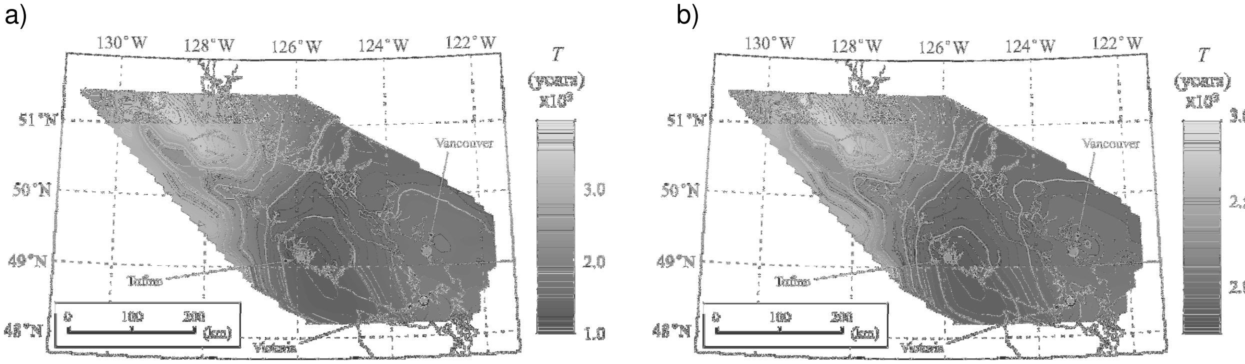

Figure 6. Contour maps of the required return periods: a) To meet the selected P_D, b) To meet the selected P_C.

To evaluate the required return period to meet the tolerable P_C, one could first develop results similar to those shown in Fig. 5 for the considered case and then interpolate or extrapolate to find the required T. The results obtained in this way are shown in Fig. 6b and compared with those dictated by the required by P_D. In this case, the required return period T that meets the tolerable P_C level varies from 1720 to 3010 years. The contour maps for the seismic design loads, $S_A(T_n,\xi)$, corresponding to the return periods shown in Fig. 6 are presented in Fig. 7.

Comparison of the results between Fig. 6a and Fig. 6b, and between Fig. 7a and Fig 7b suggests that the required return period (value) for the selected P_C is often greater than that for the selected P_D if v_s is relatively small (i.e., less than about 2).

It must be emphasized that the results presented in this section only serves as an illustration of the concept of the reliability-consistent seismic design load contour maps since the obtained return period are highly dependent on the selected tolerable probability values P_D and P_C. To provide definite recommendations on the return periods for selecting the seismic design, one must assess first what are the acceptable target reliability levels or tolerable probability of failure levels. This assessment should take into account experience, engineering judgment, socio-economic considerations and well-being of societal members. Note that since the calculated P_C and P_D depend somewhat on the displacement ductility capacity factor and vibration period, the overall strength modification factor needs to be adjusted for different values of these parameters if only two maps are to be implemented.

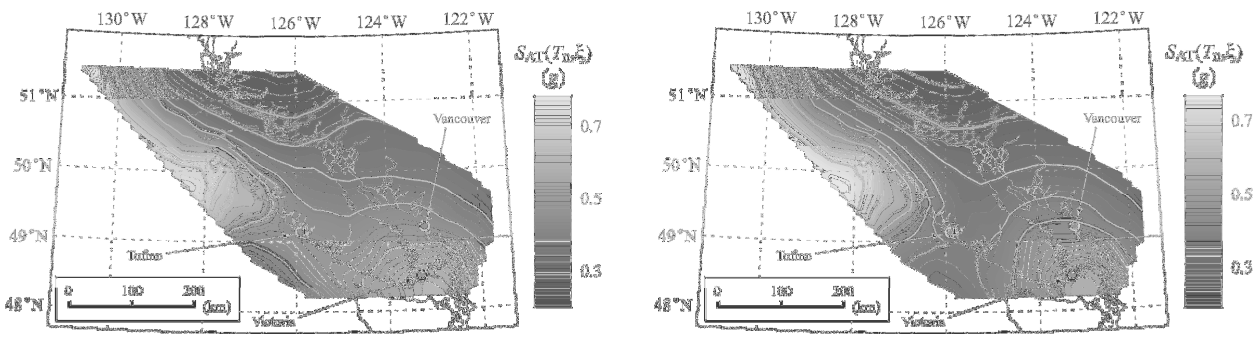

Figure 7. Contour maps of the required seismic design load to achieve reliability-consistent design: a) To meet the selected P_D, b) To meet the selected P_C.

Conclusions

It is shown in this study that the use of a consistent high return period for specifying the seismic design load, as was done for the 2005 edition of the National Building Code of Canada, may not ensure the reliability-consistent seismic design. Results of reliability analysis suggest that the probability of incipient damage and the probability of incipient of collapse vary very slowly with the return period employed to specify the seismic design load, and the required return period to achieve consistent reliability levels differs significantly for different values of the coefficient of variation of the seismic ground motion parameter. Therefore, to achieve a more reliability-consistent seismic design in Canada, it is suggested that the site-dependent return period is to be used. Further, since the required safety levels for incipient of damage and incipient of collapse are different, it is suggested that two reliability-consistent seismic design load contour maps are to be developed, one for a given tolerable probability of incipient of damage level and the other for a given tolerable probability of incipient collapse level. An illustrative numerical example on how to develop such contour maps is presented. Left for the future studies and discussions are the selection of the target reliability levels and on how to combine the contours maps considering structures with different displacement ductility factor and natural vibration periods.

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