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The Impact of Low Probability Ground Motions on Canadian Geotechnical Engineering Practice

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ABSTRACT

The adoption of design motions with a 2% rate of exceedance in 50 years in the National Building Code of Canada (NBCC 2005) has had a major impact on geotechnical engineering practice in Canada. The peak ground accelerations were doubled compared with the previous motions which had an exceedance rate of 10% in 50 years. The increase in accelerations has had a huge effect on assessments of liquefaction potential and slope stability, because the methods of assessment in common use depend on peak ground acceleration. This paper describes typical problems encountered in Canadian practice with use of the low probability motions and describes some measures for alleviating the impact on design, while maintaining the code objective of life safety.

INTRODUCTION

In 2005 the National Building Code of Canada (NBCC 2005) adopted design ground motions with a 2% chance of being exceeded in 50 years. This change resulted in about a doubling of peak ground accelerations (PGA) compared to the PGA associated with the design ground motions in the previous code (NBCC 1995) as shown in Table 1.

Table 1. PGA hazard in Canadian cities, NBCC 1995 and 2005

Median frequency of exceedance	Vancouver	Toronto	Montreal
10% in 50 yrs NBCC 1995	0.24	0.08	0.20
2% in 50 yrs NBCC 2005	0.46	0.20	0.43

The impact of the increases in ground motions on geotechnical engineering practice depends on the type of design. Conventional procedures for assessing liquefaction potential and slope stability have been based traditionally on peak ground acceleration. Designs based on these procedures have been strongly and directly affected by the increased peak ground accelerations. Sites and structures which would have been safe under the old code may now be considered unsafe for the new hazard levels. Geotechnical engineers and their clients have been expressing concerns about the great impact of the changes in ground motions on projects.

The impact on seismic slope stability makes an interesting case history. Following the adoption of the NBCC 2005 design motions by the province of British Columbia (BC) in the BC building code in 2006, sites on slopes slated for residential development failed to be approved for the use intended that would have been considered safe under the previous code. Developers and municipalities were understandably upset by this abrupt turn of events and appealed to the BC government for relief. The government responded by issuing provincial regulation M268 in December 2006 restoring the 10% in 50 years motions for slope stability assessment as a temporary measure and setting up a task force on seismic slope stability (TFSSS) under the direction of the Association of Professional Engineers and Geologists of British Columbia (APEGBC) to study the issues and make recommendations for future action. The writer is a member of the task force. The TFSSS approach to assessment of slope stability is described herein. A later extension by the writer is described in which procedures recommended by the TFSSS are coupled with reliability analysis to allow uncertainties in soil properties and seismic input to be taken into account.

The problems associated with liquefaction were studied by a Task Force of the Vancouver Geotechnical Society. A state of the art approach for dealing with liquefaction, especially under extreme motions is described in a comprehensive report (Anderson et al. 2007). The report advocates attention to the consequences of liquefaction, primarily expressed in settlements and lateral displacements as the key to rational and cost–effective engineering. The report describes a state of the art level of practice.

In more conventional practice, the Seed and Idriss (1971) simplified method for assessing liquefaction, as updated in Youd et al. (2001), is widely used, especially in Eastern Canada. The method is based on a measure of soil resistance to liquefaction, earthquake magnitude as a surrogate for duration of shaking and the peak ground acceleration. The probabilistic ground motions in NBCC 2005 are the combined contribution to hazard of all earthquakes in the seismic sources contributing to hazard. Which magnitude should be associated with the code PGA for implementation of the Seed-Idriss method? Finn and Wightman (2006, 2007) suggested two approaches for dealing in a logical way with probabilistic motions. One method is based on the concept of a hazard analysis based on earthquake magnitudes weighted according to the relative contributions they make to liquefaction potential. This concept was first proposed by Idriss (1985). The second approach is based on de-aggregating the hazard and summing up the contributions of the individual magnitudes to liquefaction potential. These methods are described later.

SEISMIC SLOPE STABILITY: REVIEW OF CURRENT PRACTICE IN BC

In BC, the most common method currently used to carry out seismic slope stability analysis is the pseudo-static limit equilibrium method. In this method, earthquake loading is represented by a constant horizontal force, kW, applied to the centre of gravity of the potential sliding mass, as shown in Fig. 1. W is the weight of the sliding mass and the coefficient, k, is called the seismic coefficient.

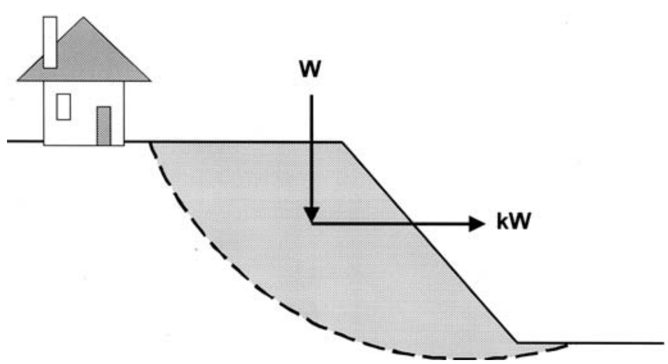

Fig.1. Pseudo-static method of seismic slope stability analysis

There is, however, no generally accepted method in BC practice for selecting seismic coefficients for slopes. From a limited survey of BC practice, the TFSSS found seismic coefficients in the range 0.5(PGA) ≤ k ≤ 1.0(PGA), where PGA is the peak ground acceleration.

The choice of k = 1.0(PGA) may be very conservative as shown by the acceleration time history in Fig. 2. The PGA occurs only for an instant and most of the record indicates accelerations much less than the maximum. The PGA has no significant impact on the response of the slope to shaking by the time history. Therefore the TFSSS recommends the use of k = PGA only as a preliminary screening tool. If FS ≥ 1.0, when k = PGA is used in a pseudo-static limit equilibrium slope stability analysis, no further stability analyses are required.

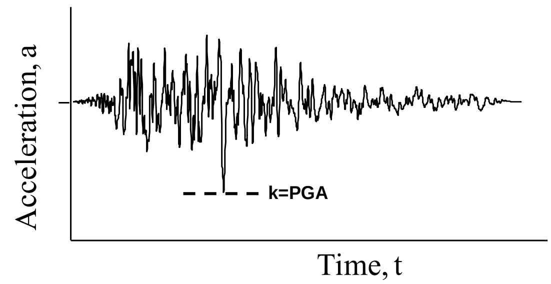

Fig.2. k = PGA, can be a very conservative estimate of k.

SLOPE PERFORMANCE DURING SHAKING

Newmark (1965) revolutionized concepts of seismic slope stability by pointing out that just because the factor of safety occasionally fell below FS = 1.0 during earthquake shaking, it did not necessarily mean slope failure. He proposed that the total displacement accumulated during the times when the factor of safety was less than FS = 1.0 be used as the index of slope performance during an earthquake and he developed simple procedures for calculating the displacements.

Permanent displacements can occur in a slope during an earthquake only if the shear stresses generated by the earthquake exceed the shearing resistance of the slope. The horizontal force required to bring the slope to the condition of incipient displacement is shown in Fig. 3 as $F = k_yW$ where k_y is the seismic yield coefficient, a special value of the seismic coefficient that just allows slip or yielding in the slope. The yield coefficient, $k_y = a_y/g$, where, a_y = yield acceleration and g = the acceleration of gravity.

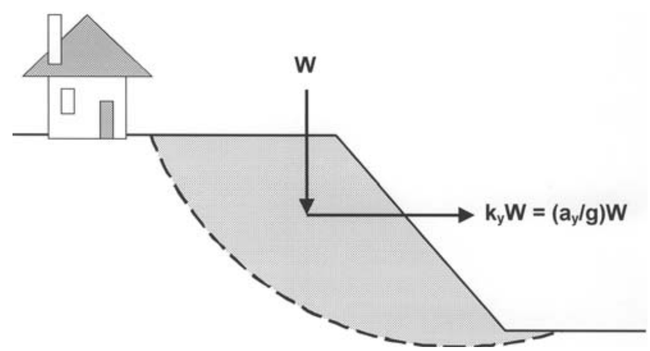

Fig.3. The condition of incipient displacement under k_y.

Figure 4 is a segment of a typical earthquake shaking record to an enlarged scale. Slope displacements can occur whenever the ground acceleration, ‘a’, exceeds the yield acceleration, a_y. In Fig. 4, the time intervals during which displacements are initiated have been shaded. The total slope displacement at the end of earthquake shaking is the sum of the incremental slope displacements generated by the ground acceleration being greater that the yield acceleration. Newmark (1965) calculated these displacements by considering the sliding mass of soil to be rigid. He also provided charts for estimating the maximum displacements. These charts were based on the small selection of strong ground motion records available at the time. In present practice, slope displacements are also estimated by direct calculation using design ground motions as input to the Newmark (1965) sliding rigid block computational model or by using a model that takes the flexibility of the slope into account.

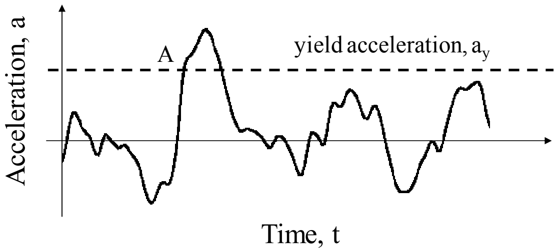

Fig.4. Displacement is initiated when ground acceleration exceeds yield acceleration.

Makdisi and Seed (1978) improved the Newmark model for application to embankment dams by taking into account the flexibility of the embankment and the amplification of ground motions on passing up through the embankment. They developed charts relating slope displacement to earthquake magnitude and the ratio of the seismic coefficient k to yield coefficient k_y. On the basis of Makdisi and Seed (1978) data, Seed (1979) recommended values of k in the range 0.1-0.15 depending on earthquake magnitude, M, for the analysis of the slopes of earth dams. For example the Seed procedure calls for k = 0.15 and a factor of safety FS ≥ 1.15 for an earthquake with M = 8.25. This value of k is associated with a maximum allowable displacement of 100 cm. Los Angeles County subsequently modified this procedure to a single k value with k = 1.15 and FS ≥ 1.0 (Blake et al. 2002). Hynes-Griffin and Franklin (1984) recommended using k = 0.5(PGA). This value of k is also based on a maximum allowable slope displacement of 100 cm. It is important to note that these generally accepted methods for selecting a seismic coefficient in U.S. practice for embankment dams are based on slope displacement criteria. The two procedures recommended here by TFSSS are also based on a criterion of acceptable slope response during an earthquake expressed in terms of allowable displacement. These methods, to be acceptable for general use, had to be conceptually simple and easy to apply.

SLOPE DISPLACEMENT (METHOD 1)

The TFSSS reviewed recent developments in methods of seismic slope stability analysis and selected a new approach based on the concept of tolerable displacements. The method is based on the work of Bray and Travasarou (2007). They conducted approximately 55,000 Newmark type slope displacement analyses involving eight different slope configurations, ten different yield accelerations for each slope configuration, and 688 different recorded ground motions from the PEER (2005) data base. From a regression analysis of the resulting slope displacements, they developed an equation for estimating the median slope displacement along a slip surface with a conditional probability of exceedance of 50%, if the design ground motion occurs. When this probability is combined with 2% probability of exceedance of the ground motions in 50 years, the absolute probability of the median displacements being exceeded is 1% in 50 years (approximately 1/5000). The median displacement is selected as the controlling slope displacement because of the low absolute probability of exceedance.

Bray and Travasarou's equation slope displacement, D, greater than 1cm is:

$$\begin{aligned} \mathrm{Ln}(D) = &-1.10 - 2.83 \ln(k_y) - 0.333 (\ln(k_y))^2 \\ &+ 0.566 \ln(k_y) \ln (S(1.5T_s)) + 3.04 \ln(S(1.5T_s)) \\ &- 0.244 (\ln(S(1.5T_s)))^2 + 1.5T_s + 0.278(M-7) \pm \varepsilon \quad (1) \end{aligned}$$

The displacement D is due to shearing along the slip surface and has both vertical and horizontal components.

T_s is the initial fundamental period of the potential sliding mass prior to the seismic event, ($0.05s < T_s < 2.0s$) and, for a slope such as shown in Fig. 1, is estimated by:

$$T_S = 4H/V_S \quad (2)$$

where H is the average height and V_s is the average shear wave velocity of the potential sliding mass. Site investigations for most residential developments do not typically include measurements of shear wave velocity, but estimates can be inferred from standard penetration test or cone penetration test data (Sykora and Koester, 1988).

In Eq. 1, ε is a normally distributed random variable with a mean of zero and a standard deviation σ = 0.66 and M the moment magnitude of the earthquake under consideration. The term $S(1.5T_s)$ is the spectral acceleration at the site for the period of $(1.5T_s)$. It is given by $S(1.5T_s) = F{*}S_a(1.5T_s)$ where $S_a(1.5T_s)$ is the spectral acceleration for firm soil conditions and F is the amplification factor for the site class. Values of F, as a function of site class and period, and $S_a(1.5T_s)$, for periods T = 0.2, 0.5, 1.0 and 2.0, are provided in NBCC 2005. Values of $S_a(1.5T_s)$ for other periods can be interpolated linearly from the values provided in NBCC 2005. Bray (2007) suggested that a value of T_s = 0.33, giving a spectral period ($1.5T_s$) of 0.5, would be adequate for general use. The TFSSS recommends this value but an engineer is not precluded using a slope specific T_s, when he considers it more appropriate. S decreases with increasing values of period and therefore the general value S = 0.5 will become more conservative as the slope period increases beyond Ts = 0.33. For periods shorter than 0.33s, S(T) increases. In such cases the designer may wish to use a slope specific period.

The ground motions specified by NBCC 2005 are probabilistic. Therefore the PGA is not associated with any particular earthquake magnitude but reflects the contributions of all earthquake magnitudes considered in the probabilistic seismic hazard analysis. The designer has to select an appropriate magnitude. The TFSSS recommends using the modal magnitude.

This is the magnitude making the largest contribution to the PGA. Site specific values of modal magnitudes may be obtained from the Geological Survey of Canada, (GSC 2008). Since the modal magnitudes for BC sites are rarely much larger than M = 7.0, it is suggested that M = 7.0 may be used for all sites.

The parameter k_y is the yield coefficient ($0.01 < k_y < 0.5$) and is best determined by iterative analyses using commercially available computer programs. Simplified equations for calculating k_y may be found in Bray et al. (1998). Bray and Travasarou (2007) point out that "the primary issue in calculating k_y is estimating the dynamic strength of the critical strata within the slope." Since k_y is assumed to be a constant during earthquake shaking, the earth materials in the slope cannot undergo significant strength loss. The selection of appropriate shear strength should follow best current practice. An extensive discussion of the dynamic strength of soil, may be found in Blake et al. (2002) and Duncan and Wright (2005).

The TFSSS recommends a displacement of 15cm or less as a tolerable slope displacement along the slip surface for use with the Bray and Travasarou (2007) method in most cases. This guideline is based on experience with wood frame construction and is predicated on the residential building being located back from the slip surface. The objective is to avoid the slip surface daylighting within or behind the building.

As examples of the use of Eq. 1, displacements were estimated for soil slopes located in Nanaimo, Duncan, and Victoria, BC, being considered for development. Slope properties were provided by the geotechnical engineers involved in the projects. As shown in Table 2, the calculated median slope displacements (D) are relatively small (2 cm to 13 cm). Using a maximum allowable displacement of 15 cm, these slopes may be considered suitable for residential development. Note that, in these examples, site specific site periods, T_s, are used rather than the general value of 0.33. The applicable values for $S(1.5T_s)$ are obtained from the listed NBCC 2005 values in Column 6 by interpolation.

Table 2. Displacements estimated using Equation 1

Slope Location	H (m)	M	T (s)	PG (g	S_a (T) NBCC			k_y	D (cm)
					0.	0.	1.		
Nanaim	3	7	0.3	0.5	1.	0.6	0.3	0.1	1
Dunca	2	7	0.3	0.5	1.	0.7	0.3	0.4	2
Victoria	1	7	0.2	0.6	1.	0.8	0.3	0.5	2

Conventional pseudo-static slope stability analysis, with 2% in 50 year ground motions and k = PGA, shows all three slopes to have FS < 1.0 and therefore, typically would be considered unsuitable for residential development. Even for k = 0.5(PGA), the Nanaimo slope would have a k = 0.25 which is greater than the yield acceleration, and would be considered unsuitable for residential development. The use of displacement analysis in conjunction with a criterion for tolerable displacement provides a more flexible and less conservative approach to evaluating slope stability for residential development than the factor of safety approach.

PSEUDO-STATIC ANALYSIS USING A SLOPE DISPLACEMENT-BASED SEISMIC COEFFICIENT (METHOD 2)

To continue to allow the use of pseudo-static slope stability analysis and yet retain the advantages of using a displacement criterion, the TFSSS asked Bray (2007) to provide a seismic coefficient that would be compatible with the recommended limiting displacement of 15 cm of displacement, k_{15}, (Fig. 5).

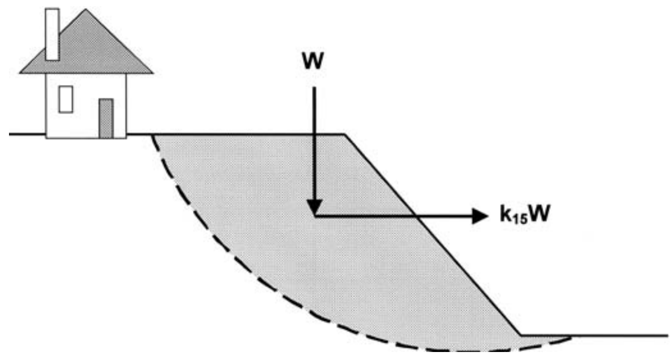

Fig.5. Pseudo-static analysis with a slope displacement-based seismic coefficient.

Bray estimated this value of k to be that given by:

$$k_{15} = (0.006 + 0.038\ M)\ S(0.50) - 0.026 \qquad (3)$$

with $S(0.50) < 1.5g$

This regression equation is valid only for a spectral period of 0.5s. Therefore slope specific periods cannot be used with this equation.

M is the moment magnitude of the earthquake. As in the case of Eq. 1, modal magnitude, M = 7.0, and spectral acceleration, S(0.50), are acceptable for general use but the designer is not precluded from selecting a site specific period in determining the period of the spectral acceleration S(T) and using a site specific modal magnitude, obtainable from the Geological Survey of Canada (GSC).

Values for k_{15} were estimated for the three slopes in Table 3. The k_{15} values for 2% in 50 year ground motions, and k values for k = 0.5(PGA) for 10% in 50-year ground motions, are also shown in Table 3. The values of the slope displacement-based seismic coefficient (the k_{15} values) corresponding to 2% in 50-year ground motions are slightly larger, and therefore somewhat more conservative, for these cases, than the seismic coefficient used in association with 10% in 50-year ground motions, when k = 0.5 PGA.

If the pseudo-static analysis, using the slope displacement-based seismic coefficient k_{15} (Fig. 5) gives a factor of safety FS ≥ 1.0, the slope may be considered suitable for residential development.

Table 3. Comparison of k_{15} with k = 0.5(PGA)

Slope Location	H (m)	k_{15} 2% in 50 yrs	k=0.5(PGA) 10% in 50 yrs
Nanaimo	30	0.16	0.11
Duncan	22	0.18	0.15
Victoria	13	0.20	0.18

ANALYSIS WITH UNCERTAINTY IN VARIABLES

Slope displacement D>1cm is given by Bray and Travasarou (2007) as

$$\ln D = f\,[S(T), k_y, T_s, M] \pm \varepsilon \tag{4}$$

where S (T) = spectral acceleration at the period $T = 1.5T_s$; k_y = yield coefficient; T_s = initial period of the potentially sliding mass and M = earthquake magnitude. These variables are treated as deterministic by Bray and Travasarou (2007) in their Eq.1 for evaluating D. The error term, ε, is the uncertainty in the displacements for deterministic values of the other independent variables and has a normal distribution with a mean of 0.0 and a standard deviation of 0.66.

The performance function for probabilistic analysis of the likelihood that some limiting displacement D_{lim} is exceeded for specified uncertainties in the variables is given by the performance function:

$$G = \mathrm{Ln}\, D_{lim} - \mathrm{Ln} D \tag{5}$$

D_{lim} is some specified limiting displacement and D is the displacement calculated using the Bray and Travasarou (2007) Eq. 3, taking into account the probabilistic variations in the controlling parameters.

Reliability analysis of the Austrian Dam in California was conducted using the program RELAN (Foschi et al. 2007), in which the performance function G had been inserted. This dam was one of many case histories analyzed by Bray and Travasarou (2007) in validating their method for estimating seismic displacement of slopes.

The variations in slope parameters were prescribed as follows. The spectral values in NBCC 2005 have a lognormal distribution with a standard deviation of 0.7. A standard deviation of 0.3 was assumed for magnitude M. Standard deviations equivalent to 20% of the deterministic values used by Bray and Travasarou (2007) in their analysis of the dam were assumed for the other variables, k_y and T_s, reflecting the difficulty in defining shear strength and slope period accurately. The latter three variables were assumed to have normal distributions.

The analyses were conducted using both first and second order statistical analysis to assess the impact of uncertainty in the independent variables controlling D. In this application the difference between first and second order analyses was negligible.

The observed displacement of the slope of the Austrian Dam was 50cm during the 1989 Loma Prieta Earthquake. The results of the RELAN analysis are shown in Fig. 6 which shows the conditional probabilities of exceedance of prescribed displacements D_{lim}. The observed displacement of 50cm is predicted to have a probability of exceedance of 38% for the specified variations in seismicity and slope parameters. Bray and Travasarou (2007) estimated the 84% displacement to be 70cm. RELAN estimates that this displacement has a probability of exceedance of 30% for the specified uncertainties in the controlling parameters.

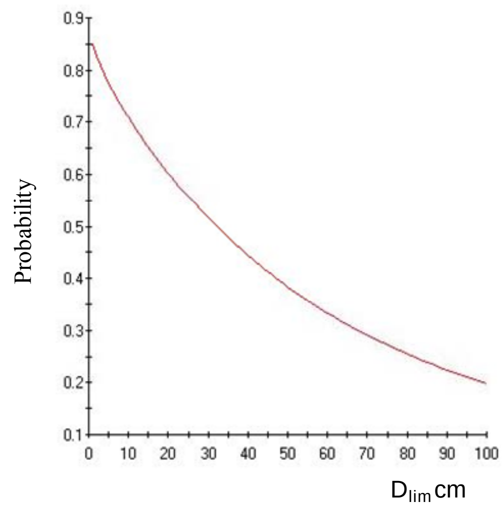

Fig.6 Probability of exceeding displacement D_{lim}

SUMMARY OF RECOMMENDATIONS

Two methods for determining whether a slope is suitable for residential development, when subjected to 2% in 50-year design ground motions are recommended.

- Method 1 involves calculating the median slope displacement with parameters that reflect slope properties and local seismicity (Eq. 1). This slope displacement has an absolute probability of exceedance of approximately 1/5000 for BC design ground motions.

- Method 2 is based on pseudo-static (limit equilibrium) seismic slope stability analysis, similar to current practice, but uses a slope displacement-based seismic coefficient, k_{15}, given by Eq. 3, that is equivalent to a tolerable slope displacement of 15cm, when the slope is subjected to design ground motions.

- Both methods provide the Qualified Professional with a basis for exercising his/her judgment as to whether the slope is suitable for residential development.
- Based on experience with wood frame residential construction a displacement of 15 cm is considered an acceptable slope displacement, when the sliding surface is between the building foundation and the face of the slope.

The results of the above methods, when used in conjunction with 2% in 50-year design ground motions (NBCC 2005), appear to be comparable to the results obtained by the current methods using 10% in 50-year ground motions (Provincial Regulation M268, December 2006) and k = 0.5(PGA).

The use of k = PGA with a factor of safety FS > 1.0 as a basis for final judgment on slope stability is considered by the TFSSS as too conservative and is recommended only as a preliminary screening tool. The limiting displacement of 15 cm is proposed as a guideline and is not intended to preclude the engineer of record from selecting any other value that he judges appropriate. The engineer should strive for a balance between desirable locations for a building and the associated seismic displacements.

EVALUATION OF LIQUEFACTION POTENTIAL

The impact on the triggering of liquefaction is examined here and suggestions are made for determining the appropriate compatible input parameters (magnitude and acceleration) for evaluating the potential for liquefaction, when probabilistic ground accelerations are used. These methods are shown to reduce significantly the seismic demand in some environments. The new seismic parameters are consistent with the hazard level for seismic design of 2% in 50 years specified in NBCC 2005.

The generally accepted procedure in Canada for evaluating the potential for triggering liquefaction is the updated Seed-Idriss (1971) procedure described by Youd et al. (2001). Whether liquefaction occurs or not depends on the balance between the resistance to liquefaction of the soil and the seismic demand on the site represented by the intensity and duration of shaking. The intensity of shaking is defined by the peak ground acceleration and the duration is represented by earthquake magnitude. Adopting the notation recommended by Youd et al. (2001), the seismic demand at a site is termed CSR, the cyclic stress ratio, and is defined by:

$$CSR = \tau_{av}/ \sigma'_{vo} = 0.65\ (a_{max}/g)\ (\sigma_{vo}/\sigma'_{vo})\ (r_d) \tag{6}$$

where a_{max} = peak horizontal ground acceleration at the ground surface; g = the acceleration due to gravity; σ_{vo}, σ'_{vo} = total and effective vertical overburden stresses respectively, r_d = stress reduction coefficient, and τ_{av} = average cyclic shear stress. The inherent resistance to liquefaction is represented in the Seed-Idriss method by either penetration resistance or shear wave velocity. Liquefaction potential may be determined from a liquefaction assessment chart such as that shown in Fig. 7. Here the seismic demand is represented by the cyclic stress ratio, CSR, and the resistance by the normalized Standard Penetration Resistance, $(N_1)_{60}$. The curves shown in Fig. 7 separate liquefiable from non-liquefiable sites for a given percentage of fines in the sand for a duration corresponding to M = 7.5. Stress ratios on these lines are called cyclic resistance ratios, CRR. The factor of safety against liquefaction is given by CRR/CSR.

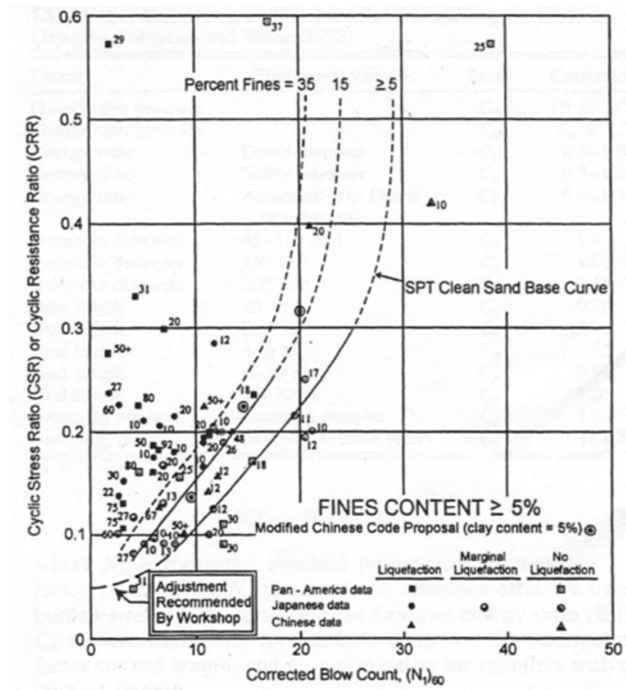

Fig.7. Liquefaction chart (from Youd et al, 2001)

The simplified method was originally used with scenario earthquakes in California. The design earthquake was usually located on a fault and the outcrop acceleration at the site to be used for site response analysis was determined by an attenuation relationship. There was a direct link between the design earthquake magnitude and the outcrop acceleration at the site. With the advent of probabilistic ground motion parameters, the direct link between site acceleration and design earthquake magnitude was lost because the probabilistic site acceleration is composed of the contributions of many different earthquakes. For liquefaction assessment in Canada, the site acceleration has been assigned to one, somewhat arbitrarily selected, single earthquake magnitude, without any assessment of how well the acceleration–magnitude pair simulated the combined effects of all the earthquakes affecting the site. As will be shown later, this procedure results often in the probability of triggering liquefaction being lower than the probability of the structural design motions being exceeded and therefore there may be an unintentional conservatism in evaluating the potential for triggering liquefaction. The degree of conservatism depends on the seismic environment.

The duration of shaking depends on the magnitude of the earthquake as was recognized by Seed and Idriss (1982) when they introduced Magnitude Scaling Factors, MSF, to relate the

contributions of different magnitudes in generating liquefaction relative to the base magnitude, M = 7.5, which anchors the widely used liquefaction assessment chart shown in Fig. 7.
These scaling factors can be applied in two different ways; either to the liquefaction resistance or the seismic demand, when assessing the potential for triggering liquefaction. Youd et al. (2001) described a range of magnitude scaling factors that geotechnical engineers may adopt for use in practice. In this paper the factors recommended by Idriss as reported in Youd et al. (2001) are used. These factors are a lower bound to all the factors recommended by Youd et al. (2001) and their use is more conservative. These factors for magnitudes M are given in Eq.7 in terms of magnitude M = 7.5 which is the base magnitude in Fig. 7.

$$MSF = 10^{2.24} / M^{2.56} \qquad (7)$$

Some examples of MSF are shown in Table 4.

Table 4. Idriss magnitude scaling factors (Youd 2001)

Mag.	5.5	6.0	6.5	7.0	7.5	8.0
MSF	2.2	1.76	1.44	1.19	1.0	0.84

In this paper the seismic demand is scaled using the magnitude weighting factor, MWF, where MWF is the inverse of the scaling factor.

The effect of the magnitude weighting factor on the CSR for a given magnitude is given by Eq.8.

$$CSR = 0.65\,(a_{max}/g)\,(\sigma_{vo}/\sigma'_{vo})\,(r_d)\,(MWF) \qquad (8)$$

When dealing with a scenario earthquake of magnitude M which has a direct link to the PGA at the site, the MWF for M can be applied directly in Eq.8 without any ambiguity. However, if a probabilistic PGA is used, which is the result of the contributions of many magnitudes, what magnitude and hence what MWF should be used? In current practice a single magnitude is often selected which may be the maximum experienced earthquake or tends towards the maximum magnitude expected in the governing seismic source zone and its weighting factor is used with the NBCC 2005 PGA. Does this single magnitude represent adequately the collective effects of the many different magnitudes contributing to the probabilistic PGA? The answer to this question is sought using two methods that logically include the effects of weighting on the contributions of all magnitudes to the probabilistic PGA. These methods are: (1) a probabilistic seismic hazard analysis using weighted magnitudes and (2) a weighted magnitude procedure based on a magnitude deaggregation for the hazard level in NBCC 2005. The weighted magnitude probabilistic analyses were conducted using the computer program EZ-FRISK 4.3 (Risk Engineering, 1997). Earthquake magnitudes in Eastern Canada are Nuttli magnitudes, mN and are scaled to moment magnitudes M for liquefaction hazard analysis.

This paper is an update of two previous reports (Finn and Wightman, 2006a and 2006b) and incorporates updated deaggregation data for Vancouver and Toronto supplied by Halchuk and Adams (2006) of the Geological Survey of Canada (GSC).

WEIGHTED MAGNITUDE HAZARD ANALYSIS

The weighted magnitude probabilistic analysis approach was first proposed by Idriss (1985). He demonstrated the need for weighting the magnitudes and showed how for the same acceleration level the return period for the weighted response could be much longer depending on the seismic environment. As noted above, the weighting factors, MWF, used in the present study are the inverse of the MSF proposed by Youd (2001) and listed in Table 4.

The weighted magnitude probabilistic analysis is accepted in California as a procedure for implementing the requirements of the Division of Mines and Geology guidelines in DMG SP 117 and the Seismic Mapping Act for projects requiring review under the Seismic Mapping Act of California. DMG SP 117 states "The alternative approach calculating "magnitude-weighted accelerations" is considerably easier and it provides a unique magnitude to be used with the probabilistically derived accelerations" (SCEC 1999).

The weighted magnitude probabilistic analyses reported in this paper were conducted to obtain the magnitude–acceleration pair for evaluating liquefaction potential. In this context, the weighted hazard curves are called liquefaction hazard curves. The seismic hazard curve for Vancouver and the liquefaction hazard curve weighted for magnitude M = 7.5 are shown in Fig. 8.

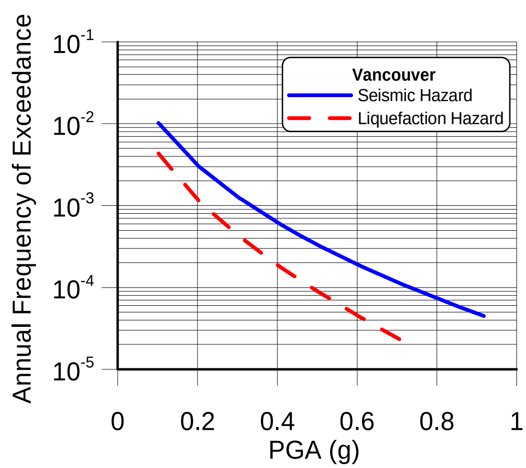

Fig.8. Seismic hazard curves for Vancouver

The acceleration for assessing liquefaction potential for an exceedance rate of 2% in 50 years is 0.30g for M = 7.5 and the site factor C = 1.0. For other values of C, the compatible acceleration is 0.30Cg. The liquefaction hazard acceleration should be used directly with the liquefaction resistance curve for magnitude M = 7.5 without further scaling. As pointed out by Idriss (1985) the weighted probabilistic analysis can be done for any normalizing earthquake magnitude other than M = 7.5 but then the appropriate magnitude weighting factor for the chosen magnitude must be used in assessing liquefaction resistance using Fig.8. Therefore, when calculating liquefaction triggering only, the magnitude-acceleration pair to be used is the normalizing magnitude and the associated weighted acceleration.

The unweighted and weighted PGA are for firm ground and, depending on the intensity of shaking, will be amplified or deamplified at the surface by a site factor C on propagating through the softer soils often associated with liquefaction. The site factor C is often estimated from generalized amplification data such in Idriss (1990), the short period amplification factors in NBCC 2005 or from ground motion attenuation relations for different soil types. Site response analysis should not be used to get PGA for use with the simplified Seed-Idriss method. It would be more reliable in this case to use the computed cyclic stress ratios from the analysis directly with Fig. 8 to assess liquefaction potential. The factors of safety against liquefaction presented in the following table were calculated by the simplified method for a range in $(N_1)_{60}$ values using the magnitude-acceleration pair from the weighted magnitude probabilistic analysis. Generic site conditions were assumed, consisting of sand, with unit weight 20 kN/m3, a water table at 2 m, and a range of $(N_1)_{60}$ values at 6 m depth. For these analyses the site factor was assumed to be C = 1.0. The factors of safety are shown in Table 5.

Table 5 .Factors of safety against liquefaction in Vancouver site with $(N_1)_{60}$ = 18 at 6m depth

$(N_1)_{60}$	Vancouver Liquefaction Safety Factors	
	Current Practice	Weighted Magnitude Analysis
	M7.3: 0.46g	M7.5: 0.30
10	0.28	0.40
13	0.35	0.49
15	0.39	0.57
18	0.47	0.67
20	0.53	0.76
25	0.72	1.02
30	1.15	1.64

MAGNITUDE DEAGGREGATION METHOD

The magnitude deaggregation method will be explained with reference to the magnitude-deaggregation for Vancouver shown in Fig. 9 (Halchuk and Adams, 2006). In this case the magnitudes are collected in bins 0.25M wide and the central magnitude value is assigned to each of these bins. For example, the bin labeled M = 5.125 contains all earthquakes in the range 5.0 ≤M <5.25. Contributions of the various bin magnitudes are magnitudes are collected in bins 0.25M wide and the central magnitude value is assigned to the bin. For example the bin labeled M = 5.125 contains all earthquakes in the range 5.0≤M<5.25. The contributions of the bin magnitude are sampled at various distances from the site. These contributions are shown by the row numbers in the deaggregation matrix in Fig. 10.

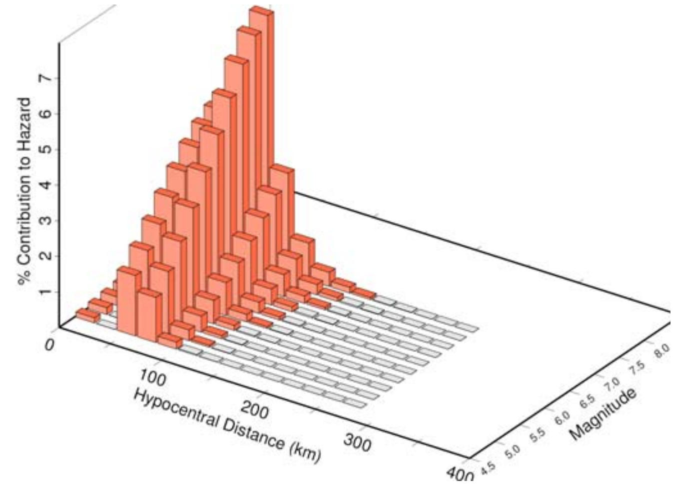

Fig.9. Magnitude-distance deaggregation for NBCC 2005 PGA in Vancouver

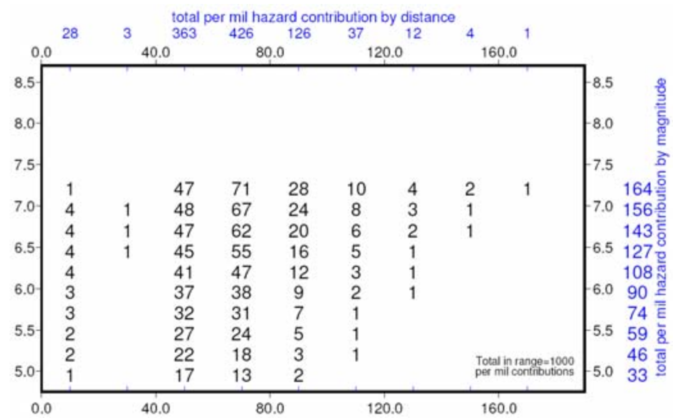

Fig.10. Deaggregation matrix for NBCC 2005 PGA in Vancouver

The total bin contributions to the NBCC 2005 PGA are given by the row numbers outside the matrix boundary in Fig. 10. These contributions per magnitude bin are shown in the 2-D plot in Fig.11. The sum of the bin contributions is 100%.

The factor of safety against liquefaction at a site, taking into account the magnitude weighting factors is calculated as follows. The factor of safety of the site at the code acceleration level is computed for each binned magnitude and then multiplied by the contribution of the magnitude to give the contribution to factor of safety. The sum of all the bin contributions to the factor of safety gives the global factor of safety for the site. The calculation process for Vancouver is shown by the example in Table 6.

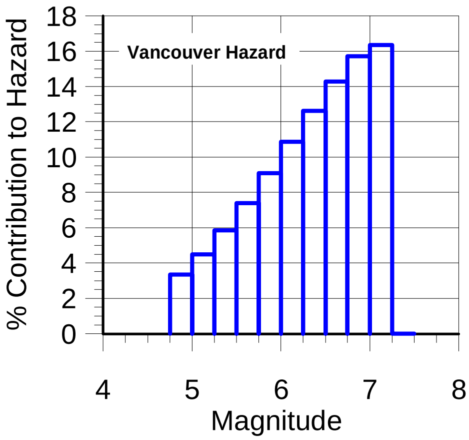

Fig.11. Magnitude Contributions to NBCC 2005 PGA Hazard in Vancouver

Table 6. Sample calculation of factor of safety against liquefaction for Vancouver site: $(N_1)_{60}$=18 at a depth of 6m .

Magnitude Bins	Central Magnitude	Contribution Factor	Liquefaction F.S.	F.S. Contribution
4.75 - 5.0	4.875	0.033	1.33	0.044
5.0 - 5.25	5.125	0.045	1.17	0.052
5.25 - 5.5	5.375	0.058	1.03	0.06
5.5 - 5.75	5.625	0.074	0.92	0.068
5.75 - 6.0	5.875	0.091	0.82	0.075
6.0 - 6.25	6.125	0.109	0.74	0.08
6.25 - 6.5	6.375	0.126	0.67	0.084
6.5 - 6.75	6.625	0.143	0.6	0.086
6.75 - 7.0	6.875	0.157	0.55	0.086
7.0 - 7.25	7.125	0.163	0.5	0.082
	Σ =	1.00	Total F.S. =	0.717

The factors of safety from the deaggregation method are compared in Table 7 with the factors obtained using the magnitude-acceleration pair from the magnitude weighted probabilistic analysis. The factors given by current practice in Vancouver and those arising from using mean and modal magnitudes with the code acceleration are also shown. The weighted magnitude probabilistic method and the deaggregation method give factors of safety within an average of 2% of each other. Note that the mean magnitude combined with the NBCC 2005 peak ground accelerations gives results very similar to the weighted magnitude probabilistic analysis in this seismic environment.

Deaggregation gives additional information on the statistics of the seismic environment. Of particular interest are the mean and modal magnitudes. For Vancouver these are M = 6.32 and M = 7.125 respectively. The mean magnitude in conjunction with the NBCC 2005 accelerations gives the same factors of safety as the other methods described above for Vancouver. The modal magnitude is the event that contributes most to the hazard even though it usually contributes less than 25%. For Vancouver, for example, it contributes about 16%. The modal magnitude is close to the M = 7.3 used in Vancouver practice and it also underestimates the factors of safety by about the same amount.

Table 7. Factors of safety against liquefaction in Vancouver for various triggering options

$(N_1)_{60}$	Liquefaction Triggering Safety Factors for Vancouver				
	Current Practice	Modal Magnitude	Mean Magnitude	Deaggregation Method	Weighted Mag. Analysis
	PGA = 0.46g				PGA = 0.30g
	M=7.3	M=7.1	M=6.3	M=7.25-4.75	M=7.5
10	0.28	0.30	0.40	0.41	0.40
13	0.35	0.37	0.50	0.51	0.49
15	0.39	0.42	0.57	0.58	0.57
18	0.47	0.50	0.68	0.69	0.67
20	0.53	0.56	0.77	0.78	0.76
25	0.72	0.76	1.04	1.05	1.02
30	1.15	1.22	1.66	1.69	1.64

A deaggregation study was also conducted for Toronto. The GSC magnitude deaggregation for Toronto is shown in Fig. 12 and the associated deaggregation matrix is shown in Fig. 13 (Halchuk and Adams, 2006). The equivalent 2-D plot is shown as Fig. 14.

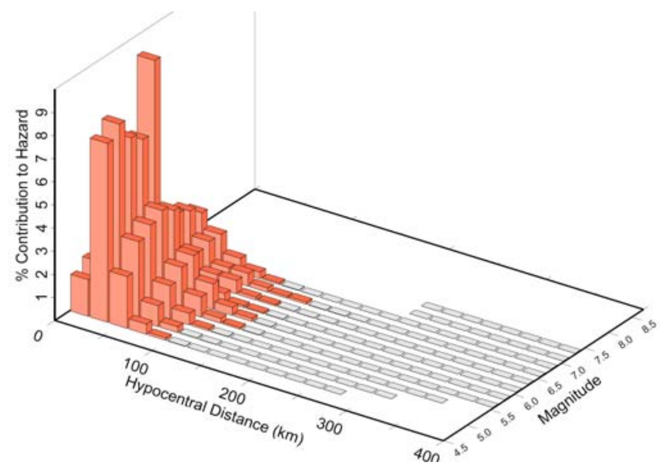

Fig.12. Magnitude-distance deaggregation for NBCC 2005 PGA in Toronto

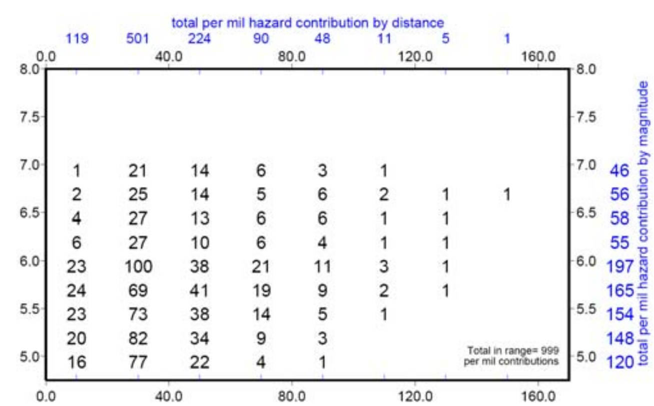

Fig.13. Deaggregation matrix for NBCC 2005 PGA in Toronto

The factor of safety for each binned magnitude was calculated for the previously prescribed range in $(N_1)_{60}$ values using the Seed-Idriss simplified method. The contribution of each magnitude bin to the total factor of safety was calculated using the contribution data given in Fig. 14.

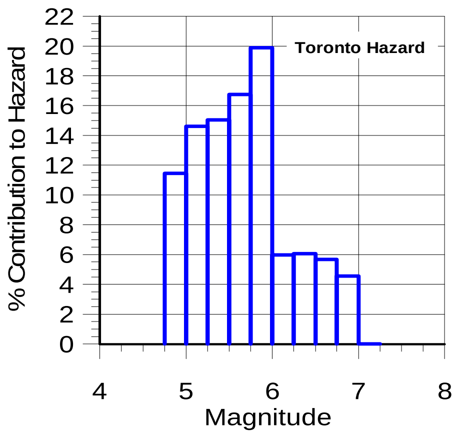

Fig.14. Magnitude Contributions to Toronto NBCC 2005 PGA Hazard

The resulting factors of safety are given in Table 8. The magnitude deaggregation method gives factors of safety on average 4% greater than the weighted magnitude analysis. The mean magnitude in combination with the NBCC 2005 PGA gives factors of safety for the Toronto site that are similar to the factors given by the deaggregation and weighted magnitude methods. In the Toronto seismic environment, the modal magnitude also gives 15% - 20% less to liquefaction potential. These analyses were conducted with an amplification/deamplification factor C = 1.0 as in the case of Vancouver.

Table 8. Factors of safety against liquefaction in Toronto for various triggering options

$(N_1)_{60}$	Liquefaction Triggering Safety Factors for Toronto			
	Modal Magnitude	Mean Magnitude	Deaggregation Method	Weighted Mag. Analysis
	PGA = 0.20g			PGA = 0.11g
	mN=5.875 Mw=5.47	mN=5.67 Mw=5.204	mN=7.0-4.75 Mw=7.0-4.75	mN=7.5 Mw=7.78
10	1.33	1.52	1.59	1.5
13	1.67	1.9	1.96	1.89
15	1.9	2.16	2.24	2.2
18	2.27	2.59	2.68	2.58
20	2.54	2.91	3.01	2.92
25	3.46	3.94	4.08	3.93
30	4.61	5.05	5.29	4.92

ASSESSMENT OF LIQUEFACTION RESULTS

The factors of safety given by weighted magnitude analysis and deaggregation analysis are approximately the same. The minor differences result primarily from the different approaches to sampling data in the two methods. The weighted magnitude analysis does not account for epistemic uncertainty because it can not be included directly in the EZ-FRISK analyses. Therefore "best estimate" seismic parameters given by Halchuk and Adams (2006) are used and the resulting data on liquefaction hazard for Toronto are scaled by the proportion that the acceleration hazard needs to be scaled to agree with code values. It was not necessary to scale the results for Vancouver.

The deaggregation method is based on site deaggregations given by the Geological Survey of Canada (Halchuk and Adams, 2006). The analyses leading to these deaggregations include the effects of epistemic uncertainty through the use of three sets of seismic parameters, the best estimates and upper and lower bounds on these estimates. The results from using these three sets are weighted and summed to give the code values for PGA and the associated deaggregations. The effects of epistemic uncertainty vary with the seismic environment.

CONCLUDING COMMENTS ON LIQUEFACTION

There are two logical methods for incorporating probabilistic ground accelerations into the Seed-Idriss simplified method for evaluating liquefaction potential at a site. The most direct method is a probabilistic seismic hazard analysis using weighted magnitudes. The weighting factors quantify the contributions of different magnitudes to liquefaction potential for a given ground surface acceleration relative to a normalizing magnitude M. The normalizing magnitude is usually taken as M = 7.5. The weighting factors for liquefaction assessment may be any of the sets recommended by Youd et al. (2001) as determined by the geotechnical engineer. In the analyses conducted for this study, the weighting factors recommended by Idriss are used. These factors are a lower bound on the factors available in Youd et al. (2001).

The weighted magnitude probabilistic analysis gives a unique magnitude-acceleration pair for use with the Seed-Idriss simplified method. In this study the normalizing magnitude was taken to be M = 7.5. Any other normalizing magnitude can be selected and a compatible magnitude-acceleration pair can be determined by simple proportion of the relative scaling factors for the magnitudes. All compatible magnitude-acceleration pairs determined by the weighted probabilistic analysis will yield the same factor of safety against liquefaction. The probabilistic acceleration from the weighted magnitude analysis must be multiplied by the site amplification/deamplification factor, C, to give the magnitude-acceleration pair to be used in evaluating liquefaction potential.

The second logical approach is based on a magnitude-distance deaggregation of the seismic hazard at a site. Here a 2-D magnitude deaggregation is developed which gives the contribution of each magnitude to the probability of exceeding the NBCC 2005 PGA. The code PGA is first multiplied by the amplification/deamplification factor C. Then the factor of safety against liquefaction for each magnitude bin is calculated for the modified acceleration and scaled by the contribution of that magnitude to the hazard. The scaled contributions to the factor of safety are summed to give the total factor of safety against liquefaction. This process gives safety factors that are on average 4% greater than the factors from weighted magnitude probabilistic analysis. The differences are attributable primarily to different approaches to sampling the relevant seismic parameters.

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