Ninth Canadian Conference on Earthquake Engineering
Ottawa, Ontario, Canada
26-29 June 2007

A PROPOSED APPROACH FOR LIQUEFACTION DESIGNS TO SOFTEN THE IMPACT OF THE NBCC 2005 REQUIREMENTS

T. Salloum[1] and K.T. Law[1]

ABSTRACT

The ground motion in the seismic provisions of building codes is often used inconsistently with the Seed-Idriss approach for liquefaction design. The inconsistency arises from combining the probabilistic ground motion and the deterministic curves compiled by Seed and Idriss in their approach. This inconsistency is particularly acute in the NBCC 2005. A simple and practical method, which harmonizes the Seed-Idriss approach with the NBCC 2005 requirements, is proposed in this paper. The proposed method stems from resolving the inconsistency in using the Seed-Idriss approach for estimating future liquefaction failures. Applications of the proposed method for various Canadian cities reveal that the proposed method results in uniform liquefaction performance across Canada. A comparison between the proposed method and the NEHRP approach suggests that the latter tends to underestimate liquefaction performance in some cities and hence, does not meet the desired return period recommended by the NBCC 2005.

Introduction

The seismic loading and design provisions of the 2005 edition of the National Building Code of Canada (NBCC 2005) have undergone various amendments for implementation in structural and geotechnical designs. As the changes and requirements introduced have been the direct outcome of the recent advances in structural engineering and seismology, structural designers have accepted the new changes with minimal implications to their designs. However, serious implications to geotechnical designs (liquefaction in particular) have made the new changes impractical and in many cases cause confusion.

A brief description of the new changes of relevance to liquefaction designs is first presented in this paper followed by discussing the implications of the new changes on liquefaction design across Canadian cities. Then, the inconsistency between the Seed-Idriss deterministic approach and the NBCC 2005 probabilistic requirements is presented. Recognition of the inconsistency is vital in harmonizing the Seed-Idriss approach with the NBCC 2005 requirements. A simple and practical approach to resolve the inconsistency and reduce conservatism in liquefaction design, primarily caused by lowering the hazard level from 10%/50 years to 2%/50 years, is then presented and illustrated for selected Canadian cities. Finally, a comparison between the proposed approach and the approach recommended by the National Earthquake Hazard Reduction Program (NEHRP, 1997 Edition) is given.

[1] Dept. of Civil & Environmental Engineering, Carleton University, 1125 Colonel By Drive, Ottawa, Ontario K1S 5B6, Canada. Email: tsallou2@connect.carleton.ca

Changes and New Requirements

Site Classification and Foundation Factors

Effects of local soil conditions on ground motions are usually expressed through the foundation factors. In the NBCC 1995, a foundation factor was primarily defined as a function of the soil condition and its thickness, resulting in a somewhat ambiguous four-category site classification system. Table 1 shows the four site categories and their corresponding foundation factors which vary from 1.0 to 2.0. The NBCC 1995 approach for developing the foundation factors has two flaws: first, there exists some ambiguity pertaining to which category a soil should belong and second, it does not take into consideration the nonlinear behavior of soils. The nonlinear behavior of soils causes amplification factors to be dependent on the intensity of shaking (Jarpe et al. 1989).

Therefore, the NBCC 2005 adopts a more comprehensive approach for the development of the foundation factor (Finn and Wightman 2003). Site classification is quantitatively accomplished through either the use of mean shear wave velocity V_{30}, standard penetration resistance $(N_1)_{60}$, or undrained shear strength S_u. This approach for soil classification simplifies and reduces ambiguity in the classification process. Borcherdt (1994) has shown that the increase in amplification with decreasing mean shear wave velocity is distinctly less for short period motion than for intermediate or long period motion. Therefore, to reflect this observation in characterizing the site response and to account for the nonlinear behavior of soils, the NBCC 2005 has defined the foundation factors as a function of frequency and intensity of shaking. Table 2 shows the NBCC 2005 site classification and Tables 3 shows the NBCC 2005 foundation factors, where the intensity of the shaking is defined by the short-period ($T=0.2$ s) and the long-period ($T=1.0$ s) spectral accelerations $S_{0.2}$ and $S_{1.0}$, respectively.

Based on the spectral acceleration values for various Canadian cities; which are mapped for 2% probability of exceedance in 50 years, and with the aid of Table 3, short-period foundation factors for *Site Classes E* and *D* can be easily interpolated (Table 4, columns 1-4). The reason behind choosing *Site Classes E* and *D* is because soils under these two categories would be susceptible to liquefaction. It should also be emphasized that for liquefaction designs, the foundation factors associated with only the short-period spectral accelerations need to be obtained because only the short-period motion controls the amplitude of *PGA* (Adams and Halchuk 2004).

Return Period (Hazard Level)

The NBCC 2005 has proposed a new hazard level (2%/50 years instead of 10%/50 years in NBCC 1995) to be used for structural and geotechnical designs. The rationale behind lowering the hazard level was two folds: the overstrength inherent in structures and the seismic hazard dissimilarities between Eastern and Western North America. During the design and construction processes, many sources may contribute to the safety of the structures leading to actual strengths of about 1.5 greater than their designed strengths; in other words, it would take one and a half times the design earthquake force to exhaust all the overstrength inherent in the structure before it collapses. Therefore, past designs for 475-year earthquake have strengths to withstand larger earthquakes of longer return periods.

Eastern North America (ENA) and Western North America (WNA) have different seismic hazard characteristics, the West being more active than the East in terms of seismicity. This difference has led to a much more rigorous seismic design process (detailing) in the West than the East. Now, if the overstrength factor is considered in the well established process in WNA, the return period of the earthquake that would deplete the overstrength is about 2475 years. However, if the same overstrength factor is considered in ENA, the corresponding return period would be about 1500 years. As such the level of protection against collapse in ENA is different from that in WNA.

Therefore, to unify the safety margin against collapse nationwide, seismic designs are anchored to the well established process in the West by considering the 2475-year earthquake as the basis for design.

However, as the design forces corresponding to that earthquake would be much higher than those corresponding to a 475-year earthquake (particularly in ENA), the overstrength concept is now incorporated into the design process to mitigate the "would be" drastic increases in seismic design forces. By doing so, two implicit performance objectives are achieved (DeVall 2006):

- the essential service objective where a structure designed in accordance with this current code, would *resist* all minor earthquakes (those correspond to less than 2475-year seismic event) without damage,
- the basic objective where the same structure would *survive* a rare earthquake that corresponds to 2475-year return period at a near collapse state.

Implications on Liquefaction Designs

Appreciation of the new hazard levels in the NBCC 2005 requirements is best achieved through the comparison of the liquefaction designs for selected Canadian cities obtained in accordance with both editions of the NBCC. Liquefaction designs will be based on the Seed-Idriss approach, which will be briefly explained for the sake of clarity and completeness.

Seed-Idriss Approach (Seed and Idriss 1971)

Liquefaction designs and evaluations have been largely based on the Seed-Idriss deterministic approach using the standard penetration resistance (SPT)-based correlations. The cyclic stress ratio *(CSR)* causing soil liquefaction is often obtained using the Seed-Idriss approach involving site specific peak ground acceleration (*PGA)*, recommended by NBCC, together with a representative earthquake magnitude, conventionally chosen as the maximum magnitude predicated for the governing seismic source zone. The *CSR* is expressed as:

$$CSR = 0.65\frac{a_{\max}}{g}\times\frac{\sigma_{vo}}{\sigma'_{vo}}\times\frac{r_d}{MSF} \tag{1}$$

where: a_{max} = maximum design acceleration at the ground surface, g = acceleration of gravity, σ_{vo} = in-situ vertical total stress, σ'_{vo} = in-situ vertical effective stress, r_d = depth reduction factor, and *MSF* = magnitude scaling factor, which is a function of earthquake magnitude and can be expressed as $MSF=10^{2.24}/M^{2.56}$ (Yould and Idriss 1997). The depth reduction factor incorporates the fact that the actual shear stresses induced in the soil due to a particular earthquake (ground surface acceleration) would be less than those developed in a rigid body for the same earthquake, and the magnitude scaling factor accounts for the duration of the earthquake. It should be noted that two pieces of ground motion information - a_{max} and earthquake magnitude – are required for estimating the cyclic stress ratio. The above equation is used in conjunction with the Seed-Idriss liquefaction curves for liquefaction designs and evaluations.

Implications on *CSR*

There are two terms in the Seed-Idriss equation, which are influenced by the changes introduced in the NBCC 2005: the maximum design acceleration at the ground surface (a_{max}) and the magnitude scaling factor (*MSF*). There are different ways for obtaining a_{max} values (Youd and Idriss 1997). Preferably a_{max} could be obtained through a site specific analysis taking into account the correlations between ground motions, earthquake magnitude and distance. The purpose of this section is to illustrate the impact of the NBCC 2005. Therefore, a simpler method is used here. This is done by multiplying the mapped *PGA* with the short period foundation factor (F_a). The other terms will be set to practical values (if needed) throughout the discussion. The mapped *PGA* corresponding to the new hazard level ranges from 1-4.6 times those in the NBCC 1995, and therefore, resulting in doubling and in some cases quadrupling the a_{max}, particularly in low seismicity areas such as Toronto. The resulting CSR from the Seed-Idriss equation may be reduced by the new magnitude scaling factor. However, the net effect is still an increased level of *CSR* in most Canadian cities. The increased level of the *CSR* either conflicts with the Seed-Idriss approach as the Seed-Idriss liquefaction curves were developed with seismic data lacking in values where *CSR>0.25* (Cetin 2000), or leads to conservative liquefaction designs.

When designing against liquefaction, a representative earthquake magnitude needs to be selected so that the duration of the earthquake (or the number of cyclic shear stress induced by the earthquake) is taken into consideration. In current practice, the representative earthquake magnitude is selected as the maximum earthquake experienced or the maximum predicted earthquake in the governing seismic source zone (Table 4, Column 5). However, for liquefaction designs, using a single earthquake magnitude in conjunction with the probabilistically-obtained *PGA* is not quite rational (Idriss 1985). Typically, the *PGA* is obtained through a probabilistic seismic hazard evaluation, where different earthquake magnitudes contribute differently to the *PGA*. Therefore, a more rational selection of the representative magnitude would be based on its contribution to the *PGA*. The modal earthquake, defined as the earthquake that contributes the most to *PGA* and the most probable earthquake to occur in a return period of interest (2475 years), may be a reasonable representative earthquake magnitude for use in liquefaction designs and evaluations.

The modal earthquake is usually obtained through the deaggregation of seismic hazard at a return period of interest. Table 4 (columns 1, 5 & 6) shows the modal earthquake magnitudes, associated with a return period of 2475 years, for various Canadian cities (obtained using the EZ-FRISK software), the results being very close to those suggested by Halchuk et al. (2007).

Liquefiable site classes of comparable characteristics from the NBCC 1995 and NBCC 2005 are compared with regard to the design CSR values. In NBCC 1995, Site Class 3 with a foundation factor of 1.5 is selected. In NBCC 2005, Site Class *E* and *D* are chosen. The F_a values for different cities are interpolated from the code data and summarized in Table 4. Figures 1 and 2 compare the *CSR* values in both editions of NBCC for selected Canadian cities. It can be readily seen from both figures that there is an increase in the *CSR* values. This increase is more pronounced in low seismicity areas such as Toronto.

The Standard Penetration Resistance, $(N_1)_{60}$, calculated based on the *CSR* values shown in Figures 1 and 2 will always be in the range of *25-30*. These results indicate that there is an obvious conservatism as past experience and observations have shown that sandy sites with $(N_1)_{60}=25\text{-}30$ seldom liquefy as there are almost no liquefaction records of soils having $(N_1)_{60}\geq 28$. It can also be seen from the Seed-Idriss figure that the liquefaction curves become parallel to the *CSR* axis starting at $(N_1)_{60}$ values of 20 and 28 (depending on fines content), suggesting that there is a very slim chance that the soil will liquefy beyond these $(N_1)_{60}$ values.

The conservative results are due to an inconsistency arising from combining the probabilistically-obtained *PGA* and the deterministic Seed-Idriss curves.

Inconsistency between PGA and Seed-Idriss Curves

The mapped peak ground acceleration (*PGA*) is usually evaluated probabilistically. Conventionally, the *PGA* is computed corresponding to a particular probability of exceedance in a given time period.

The NBCC 2005 evaluated ground motion parameters (including *PGA*) for Canadian cities based on the 2% probability of exceedance in 50 years (0.000404 per annum or 2475-year return period). The ground motion parameters corresponding to this hazard level are to be used in structural and geotechnical designs across Canada.

However, as it has been shown above, using the NBCC 2005 probabilistically-based *PGA* (associated with 2475 years return period) with the Seed-Idriss deterministically-based liquefaction curves leads to conservative liquefaction designs because the resulting liquefaction return period will be much longer than 2475 years as explained in the following.

Although the Seed-Idriss liquefaction curves are perceived by geotechnical engineers as deterministic, they suffer from a great deal of uncertainties as they were developed based on a limited number of seismic events and they have not included the increasing body of field case history data from seismic events that have occurred since 1984 (Cetin 2000). They are also lacking in data from cases with high peak ground shaking levels ($CSR > 0.25$), an increasingly common design range in regions of high seismicity. Other sources of uncertainties in the Seed-Idriss approach stem from the estimation of the depth reduction and the magnitude scaling factors, both being essential parts in the Seed-Idriss approach.

Therefore, the return period of liquefaction occurrence obtained with the Seed-Idriss approach will be different from that (2475 years) of the *PGA* used as the input for the approach. Let P_{PGA} be the probability of occurrence of the seismic event (P_{PGA} = 0.0004040, which is the probability of *PGA* based on NBCC 2005). Let P_{Seed} be the probability that quantifies the uncertainty associated with the Seed-Idriss curve. Then the total probability of liquefaction will be the product of P_{PGA} and P_{Seed}. As P_{Seed} is less than 1, it will always reduce the total probability of liquefaction failure and therefore, lengthen the liquefaction return period.

Just to illustrate the idea, let us assume that $P_{Seed} = 0.5$, meaning that there is a fifty-fifty chance that liquefaction will occur based on the Seed-Idriss approach, then the total probability of liquefaction would be: $P_{Liq} = P_{PGA} \times P_{Seed} = 0.0004040 \times 0.5 = 0.0002020$.

The liquefaction return period corresponding to that probability level will be 4950 years (1% in 50 years) which is twice as long the NBCC 2005 proposed return period.

In terms of performance, a structure near collapse (at an earthquake event of 2475-year return period) is similar to soil liquefying. Therefore, it is more logical to design for liquefaction at a return period of 2475 years as mandated in the NBCC 2005. That is, one has to select a 1237-year *PGA* for the above case to give a liquefaction return period of 2475 years (Salloum and Law 2006 and Law and Salloum 2006).

The assumption of $P_{Seed} = 0.5$ is merely an example. In fact, as P_{seed} is less than 1.0, the liquefaction return period is always longer than the *PGA* return period. In the following, a method is proposed to meet the hazard level recommended by the NBCC 2005 by incorporating the probabilistic nature of both the Seed-Idriss curve and the *PGA*.

Proposed Method

There have been various methods proposed by others such as Finn and Wightman (2006) to soften the impact of the NBCC 2005 requirements. However, the authors attempt to tackle the problem from a different point of view. Resolving the inconsistency should stem from quantifying all sources of uncertainties involved in the Seed-Idriss approach so that the Seed-Idriss liquefaction curves can be viewed within a probabilistic point of view rather than deterministic. Therefore, the liquefaction design boils down to combining the probabilistically-evaluated *PGA* (and in turn *CSR*) with the probabilistically-presented Seed-Idriss liquefaction curves. As a result, no inconsistency arises.

Fortunately, many researchers such as Liao et al. (1988), and Toprak et al. (1999), have recognized the uncertainties associated with the Seed-Idriss approach and tried to develop similar approaches but in probabilistic forms. The most comprehensive study was done by Cetin (2000) and Cetin et al. (2002), where all field case histories employed in the previous studies were used in addition to other data sets in the development of a stochastic model. The model has been developed within a Bayesian framework. In the course of developing the model, all relevant uncertainties have been addressed, which include (a) measurement/estimation errors, (b) model imperfection, (c) statistical uncertainty, and (d) those arising from inherent variables. Contours for the probability of liquefaction values P_L=*5, 20, 50, 80,* and *95%* are presented in Figure 7, and the Seed-Idriss deterministic curves are superimposed on the probability contours for comparison. It should be emphasized that the studies by Cetin (2000) and Cetin et al. (2002)

were not done for the very problem the authors are presenting herein. However, the results from their studies can be used in the present study.

With the aid of the liquefaction probability contours superimposed on the Seed-Idriss liquefaction curves, the uncertainty (probability) associated with the Seed-Idriss curves can be estimated.

In order to meet the NBCC 2005 seismic requirements, the probabilistically-determined ground motion and the probabilistically-represented Seed-Idriss curves can be combined and the solution is obtained with an iterative procedure as described in the following:

1- Start off with a *PGA* that corresponds to 2475-year return period, i.e., $P_{PGA} = 0.000404$
2- Calculate the corresponding *CSR* using the Seed-Idriss equation
3- With the aid of Figure 7, determine the probabilistic value, P_{Seed}, associated with the Seed-Idriss curve

4- Calculate the liquefaction performance (return period) T_{Liq}, where: $T_{Liq} = \dfrac{1}{P_{PGA} \times P_{Seed}}$

5- If T_{Liq} computed in Step 4 lies within 2475±100 years, the set of values computed in the last iteration (T_{Liq}, *CSR*, P_{PGA}, and P_{Seed}) are considered solution that satisfies the NBCC 2005 seismic requirements

6- If T_{Liq} computed in Step 4 is outside the range of 2475±100 years, redo steps 2 to 4 using a new *PGA* where $P_{PGA} = \dfrac{1}{P_{Seed}} \times 0.000404$, and P_{Seed} is the probability value of the Seed-Idriss curve in the last iteration.

It must be emphasized that the frequency of exceedance of the PGA, λ_{PGA}, is different from the frequency of occurrence of PGA, P_{PGA}. However, the difference of the two values tends to be negligible as the frequency of exceedance gets smaller. In addition, when carrying out the iterative procedure, the modal magnitude corresponding to a different hazard level may change when moving up and down on the hazard curve. However, it was observed that this change was small enough to be neglected.

Application of the Proposed Method on Selected Canadian Cities

The above procedure is carried out for selected Canadian cities for Site Classes *E* and *D*. Shown in Figures 3 and 4 are the computed *CSR* values from this procedure compared with those from Figures 1 and 2. The comparison shows that there is a reduction in *CSR* based on the procedure proposed here while reaching the desired liquefaction performance of 2475-year return period.

Comparison with the NEHRP Approach (the American Approach)

The National Earthquake Hazard Reduction Program (NEHRP 1997 Edition) recommended that the design earthquake ground motion should be obtained using a reduction ratio (ρ_{NEHRP}) of 2/3 (1/1.5) applied to the maximum considered earthquake ground motion. However, no compelling argument or explanation was given on why that value was chosen. It is most likely that the level was chosen based on accumulated experience. The NEHRP foundation factors are interpolated for the selected Canadian cities and tabulated in Table 5 alongside with the NBCC 2005 foundation factors. As well the NEHRP and the proposed *PGA reduction* ratios (ρ_{NEHRP} and $\rho_{Proposed}$) and the corresponding a_{max} are shown in the same table, where $\rho_{Proposed}$, is the ratio of the *PGA* that gives a liquefaction return period of 2475 year to the 2475-year *PGA*. It should be noted that the de-amplification foundation factor for *Site Class E* in Vancouver resulted in a lower maximum acceleration ($\mathbf{a_{(max)Proposed}}$) as oppose to the other cities. Figures 5 and 6 compare the *CSR* values that are used for purposes of liquefaction designs for Site Classes *E* and *D* based on the proposed procedure and the NEHRP procedure. It can be seen that both approaches are in relatively good agreement for Toronto, Ottawa, and Montreal; however, the NEHRP approach generally underestimates liquefaction performance. Using a similar procedure to the one proposed above, one can readily show that the NEHRP approach does not yield the desired performance recommended by the NBCC 2005 of 2475 years in three cities out of the four studied.

Summary and Conclusions

The new changes and requirements introduced by the NBCC 2005, which are of relevance to liquefaction designs, are discussed alongside with their implications in selected Canadian cities. Conventionally, the ground motion in the seismic provisions of building codes is often used inconsistently with the Seed-Idriss approach for liquefaction designs. The inconsistency arises from combining the probabilistically-evaluated ground motion and the deterministically-based liquefaction curves compiled by Seed and Idriss in their approach. This inconsistency is particularly acute in the NBCC 2005 and has caused conservative liquefaction designs especially in low seismicity areas such as Toronto. A proposed method has been introduced, which takes advantage of the work done by Cetin (2000) and Cetin et al. (2002), to resolve the inconsistency by quantifying all sources of uncertainties inherent in the Seed-Idriss approach. The proposed procedure meets the liquefaction performance of 2475 years and results in eliminating unnecessary conservatism in the design process. Illustration of the proposed procedure has been carried out for selected Canadian cities and a comparison with the NEHRP approach suggests that NEHRP approach does not yield the desired performance recommended by the NBCC 2005 for three cities out of the four studied. The new *PGA* reduction ratios, $\rho_{Proposed}$, might be used for design against liquefaction across Canadian cities to meet the uniform hazard level required by the NBCC 2005.

References

Adams, J., and Halchuk, S., 2004. A review of NBCC 2005 Seismic Hazard Results for Canada - the interface to the ground and prognosis for urban risk mitigation. The 57th Canadian Geotechnical Conference, Session 6C, pp1-9.

Borcherdt, R.D., 1994. Estimates of site-dependent response spectra for design (methodology and justification). Earthquake Spectra, 10(4): 617–653.

Cetin, K.O., 2000. Reliability-based assessment of seismic soil liquefaction initiation hazard. dissertation submitted in partial satisfaction of the requirements for the degree of Doctor of Philosophy. University of California at Berkeley.

Cetin, K.O., Der Kiureghian, A. and Seed, R.B., 2002. “Probabilistic models for the initiation of seismic soil liquefaction.” Structural Safety, 24(1), 67–82.

DeVall, R.H., 2006. The “1 in 2475” earthquake and how it affects the way structural engineers think about seismic structural design of buildings and is this thinking applicable to aspects of geotechnical engineering: a personal perspective

EZ-FRISK 7.1, 7.11, 7.12, 7.14, User Manuals. Risk Engineering Incorporated.

Finn, W.D.L. and Wightman, A., 2003. Ground motion amplification factors for the proposed 2005 edition of the National Building Code of Canada. Canadian Journal of Civil Engineering, Volume 30, Number 2, Pages272-278, April 2003, Special Issue: Proposed Earthquake Design Requirements of the National Building Code of Canada, 2005 edition.

Finn, W.D.L. and Wightman, A., 2006. The application of NBCC2005 probabilistic ground motions to liquefaction assessments. The 59th Canadian Geotechnical Conference, October 1-4, 2006, Vancouver, BC.

Halchuk, S., Adams, J. and Anglin, F., 2007. REVISED DEAGGREGATION OF SEISMIC HAZARD FOR SELECTED CANADIAN CITIES. Ninth Canadian Conference on Earthquake Engineering Ottawa, Ontario, Canada 26-29 June 2007. Paper 1188.

Idriss, I.M., 1985. Evaluating seismic risk in engineering practice, Proc. 11th Int. Conf. on Soil Mech. And Found. Engrg., Vol 1, 255-320.

Jarpe, S., Hutchings, L., Hauk, T. and Shakal, A., 1989. Selected strong- and weak-motion data from the Loma Prieta earthquake sequence. Seismological Research Letters, 60: 167–176.

Kramer, S.L., and Mayfield, R.T., 2005. Performance-Based Hazard Evaluation. American Society of Civil Engineers, Geotechnical Special Publications 130-142 and GRI-18. January 24-26, 2005, Austin, Texas.

Law, K.T. and Salloum, T., 2006. Probabilistic Evaluation of Past Liquefaction Hazards, Part II: The Spatial and Temporal Distributions of Seismic Parameters. The 59th Canadian Geotechnical Conference, October 1-4, 2006, Vancouver, BC.

Liao, S.S.C., Veneziano, D., Whitman, R.V., 1988. "Regression Models for Evaluating Liquefaction Probability", Journal of Geotechnical Engineering, ASCE, Vol. 114, No. 4, pp. 389-409.

NEHRP, 1997. The National Earthquake Hazard Reduction Program. NEHRP recommended provisions for seismic regulations for new buildings and other structures, 1997 Edition.

Salloum, T. and Law, K.T., 2006. Probabilistic Evaluation of Past Liquefaction Hazards, Part I: The Statistical distributions of Soil and Seismic Parameters. The 59th Canadian Geotechnical Conference, October 1-4, 2006, Vancouver, BC.

Seed, H.B. and Idriss. I.M., 1971. Simplified procedure for evaluating soil liquefaction potential. J. Geotech Engrg. Div. ASCE, 97(9).

Toprak, S., Holzer, T.L., Bennett, M.J., Tinsley, J.C., 1999. "CPT- and SPT-based Probabilistic Assessment of Liquefaction Potential, Proceedings of Seventh U.S.-Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures Against Liquefaction.

Youd, T.L. and Idriss, I.M., 1997. Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Technical Report NCEER-97-0022.

Table 1. NBCC 1995 Site Classification and Foundation Factors.

Category	Type and depth of soil measured from the foundation or pile cap level	**F**
1	Rock, dense and very dense coarse-grained soils, very stiff and hard fine-grained soils, compact coarse-grained soils and firm and stiff fine-grained soils from 0 to 15 m deep	**1**
2	Compact coarse-grained soils, firm and stiff fine-grained soils with a depth greater than 15 m; very loose and loose coarse-grained soils and very soft and soft fine-grained soils from 0 to 15 m deep	**1.3**
3	Very loose and loose coarse-grained soils with depth greater than 15 m	**1.5**
4	Very soft and soft fine-grained soils with depth greater than 15 m	**2**

Table 2. NBCC 2005 Site Classification.

Site Class	Soil Profile Name	Average Properties in Top 30 m		
		Soil Shear Wave Average velocity, V_s(m/s)	**Standard Penetration Resistance, N_{60}**	**Soil Undrained Shear Strength, S_u**
A	Hard Rock	$V_s > 1500$	Not applicable	Not applicable
B	Rock	$760 < V_s \leq 1500$	Not applicable	Not applicable
C	Very Dense Soil and Soft Rock	$360 < V_s < 760$	$N_{60} > 50$	Su > 150 kPa
D	Stiff Soil	$180 < V_s < 360$	$15 \leq N_{60} \leq 50$	$50 < S_u \leq 100$ kPa
E	Soft Soil	$V_s < 180$	$N_{60} < 15$	$S_u < 50$ kPa
E		• Any profile with more than 3 m of soil with the following characteristics: • Plastic Index PI > 20 • Moisture Content w ≥ 40%, and • Undrained shear strength $S_u < 25$ kPa		
F	Others	Site specific evaluation required		

Table 3. NBCC 2005 Foundation Factors.

Site Class	Values of F_a					Values of F_v				
	$S_a(0.2) \leq 0.25$	$S_a(0.2) = 0.50$	$S_a(0.2) = 0.75$	$S_a(0.2) = 1.00$	$S_a(0.2) \geq 1.25$	$S_a(1.0) \leq 0.1$	$S_a(1.0) = 0.2$	$S_a(1.0) = 0.3$	$S_a(1.0) = 0.4$	$S_a(1.0) \geq 0.5$
A	0.7	0.7	0.8	0.8	0.8	0.5	0.5	0.5	0.6	0.6
B	0.8	0.8	0.9	1.0	1.0	0.6	0.7	0.7	0.8	0.8
C	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
D	1.3	1.2	1.1	1.1	1.0	1.4	1.3	1.2	1.1	1.1
E	2.1	1.4	1.1	0.9	0.9	2.1	2.0	1.9	1.7	1.7
F	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)

(1) To determine F_a and F_v for Site Class F, site specific geotechnical investigations and dynamic site response analysis shall be performed.

Table 4. Spectral Acceleration, Foundation Factors for Site Classes E and D, Maximum magnitude used in practice, Modal Magnitude for Canadian Cities Based on Deaggregating the Seismic Hazard Peak Ground Acceleration for 2%/50 Years and the Corresponding Magnitude Scaling Factors.

City	**Sa(0.2)**	**Foundation Factor**		**$M_{Max1995}$**	**MSF_{1995}**	**Modal M**	**MSF_{2005}**
		Site Class E	**Site Class D**				
Vancouver	0.96	0.93	1.10	7.3	1.07	7.05	1.17
Toronto	0.28	2.02	1.29	6.0	1.77	5.90	1.85
Ottawa	0.67	1.20	1.13	6.9	1.24	5.90	1.85
Montreal	0.69	1.17	1.12	6.5	1.44	5.90	1.85

Table 5. NEHRP and the proposed PGA reduction ratios and their corresponding a_{max} values.

City	PGA	ρ_{NEHRP}	$\rho_{Proposed}$		F_{NEHRP}		F_{NBCC}		$a_{(max)NEHRP}$		$a_{(max)Proposed}$	
			E	D	E	D	E	D	E	D	E	D
Vancouver	48	0.67	0.60	0.60	0.90	1.02	0.93	1.10	29	33	27	32
Toronto	20	0.67	0.75	0.85	1.70	1.40	2.02	1.29	23	19	28	22
Ottawa	42	0.67	0.71	0.71	0.90	1.08	1.20	1.13	25	30	36	34
Montreal	43	0.67	0.88	0.88	0.90	1.07	1.17	1.12	26	31	44	43

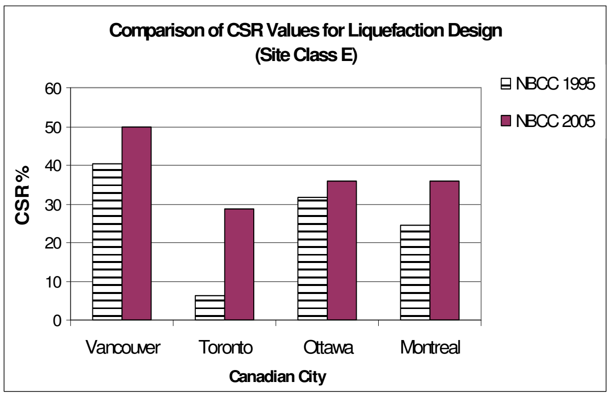

Figure 1. Comparison of CSR values for NBCC for for Site Class E.

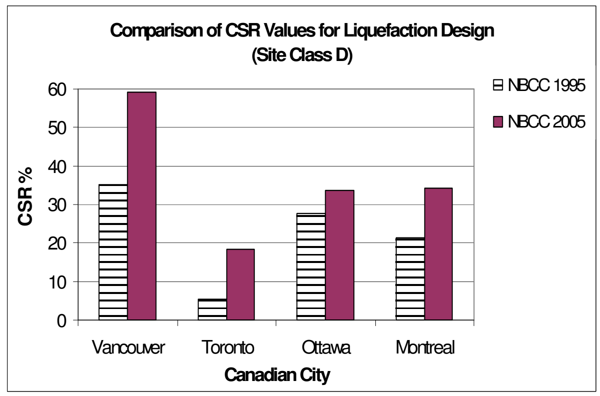

Figure 2. Comparison of CSR values for NBCC Site Class D.

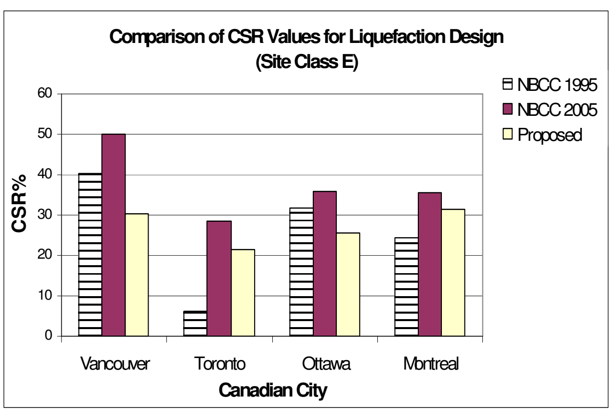

Figure 3. Comparison of CSR values for NBCC editions and the proposed procedure for Site Class E.

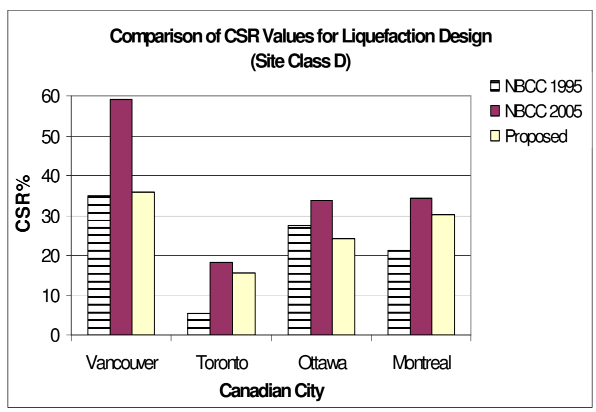

Figure 4. Comparison of CSR values for NBCC editions and the proposed procedure for Site Class D.

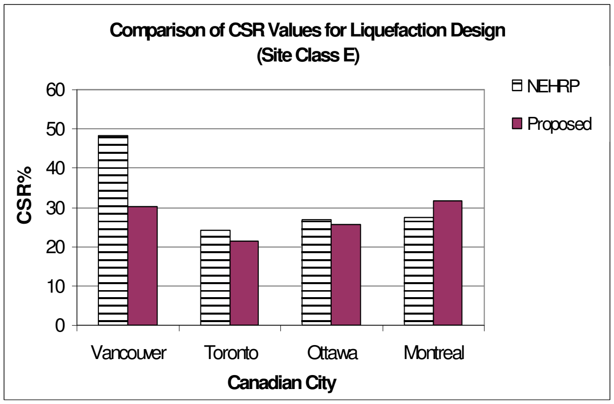

Figure 5. Comparison between NEHRP and the proposed approach for Site Class E.

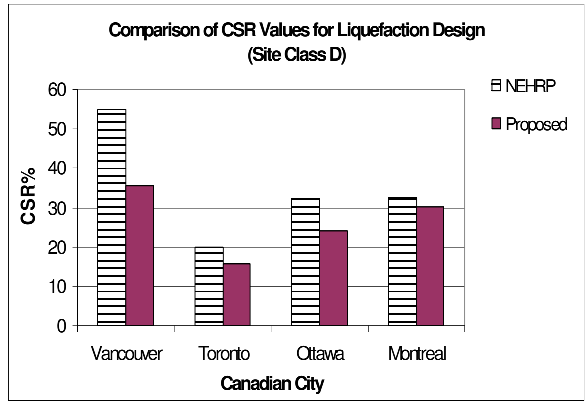

Figure 6. Comparison between NEHRP and the proposed approach for Site Class D.

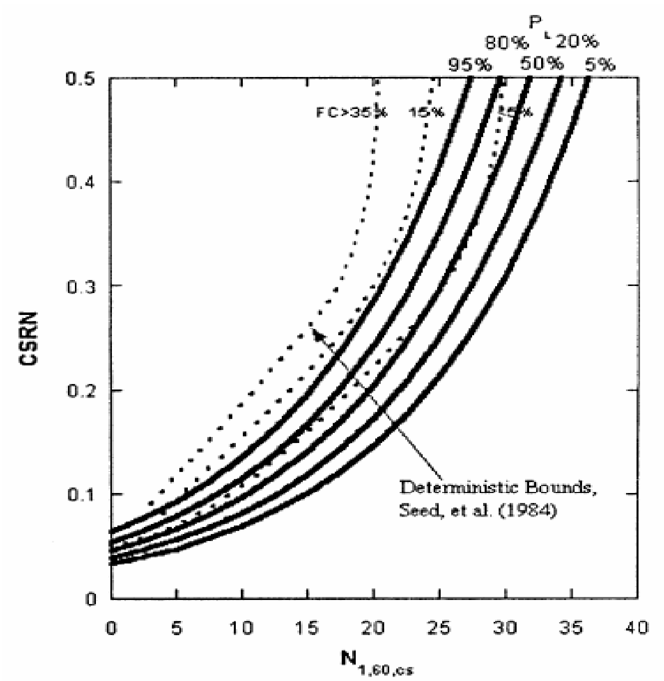

Figure 7. Probability of liquefaction curves and Seed-Idriss deterministic bounds (after Cetin 2000).