2005 NBCC-based liquefaction assessment

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ABSTRACT
Seismic provisions of the 2005 NBCC have been developed based on a lower probability level. Thus, the related peak ground accelerations have significantly increased. Some developments of the seismic provisions introduced in structural design are evaluated and implemented into liquefaction assessment. Consequently, an "overstrength" factor of has been obtained to reduce the hazard levels to the design levels. This factor will maintain uniform performance in various parts of Canada, and will bring about an improvement to the safety level for the liquefaction assessment procedure.

RÉSUMÉ
Les considérations sismiques énoncées dans le Code national du bâtiment (CNB) 2005 ont été élaborées à partir d'un niveau de probabilite moins élevé. Les accélérations maximales sismiques ont augmenté significativement. Certaines considérations introduites dans le volet conception structurale sont évaluées et appliquées dans le cadre de l'évaluation du potentiel de liquéfaction. On obtient, conséquemment, un facteur de sur-résistance permettant de reduire le niveau de danger au niveau déterminé lors de la conception. Ce facteur permet une performance uniforme dans diverses régions du Canada et améliore la sécurité au niveau de la procédure d'évaluation du potentiel de liquéfaction.

1 INTRODUCTION

The new edition of the National Building Code of Canada (2005 NBCC) has introduced some significant changes. Lowering the probability level in seismic provisions from a 10% in 50 years to 2% in 50 years is one of the main changes. Because of these changes, the tabulated peak ground acceleration (PGA) has undergone a remarkable increase. For instance, the new reference (Class C) PGA for Ottawa is 0.42g, up from 0.2g in the 1995 NBCC. It should be noted that the seismic provisions of the 2005 NBCC have been implemented in most provincial codes in Canada such as 2006 Ontario Building Code (OBC).

PGA and earthquake magnitude are the main parameters used in traditional liquefaction assessment to account for the intensity of the seismic shaking. Conventional approaches used for liquefaction assessment utilize the new PGA to determine the new liquefaction susceptibility of soils, defined by the cyclic stress ratio (CSR). This approach typically leads to a significant increase in the CSR when compared to values obtained under the requirements of the 1995 NBCC and may lead to overly conservative assessments of the liquefaction potential. Thus, mitigation may be deemed necessary for previously nonliquefiable sites.

Such overly conservative assessments of liquefaction potential may impact economic drivers such as land development, as well as urban planning initiatives, as some sites may be deemed too costly to develop. Furthermore, the land value of existing properties may be reduced since they are now subject to a liquefaction hazard under the current interpretation of the 2005 NBCC.

Similarly, Kuan (2007) found that the potential growth and the economic well-being of a city, town or district would be impacted as a result of the application of the new provisions on slope stability design.

2 CONVENTIONAL LIQUEFACTION ASSESSMENT

The methodology for completing liquefaction assessments, referred to herein as the "simplified procedure", is still the main tool used to assess the liquefaction susceptibility of soils. The simplified procedure method was originally developed by Seed and Idriss (1971, 1982) following the 1964 earthquake in Niigata, Japan. It should be noted here that this methodology is empirical in nature. Additional improvements were later implemented in the simplified approach mainly by H.B. Seed and his colleagues such as Seed et al. (1983, 1985) and Seed and Harder (1990). The simplified procedure compares the cyclic resistance ratio (CRR), related to the relative density (strength) of the soil, and liquefaction potential of the soil measured with the CSR.

Technical workshops sponsored by the National Center for Earthquake Engineering Research (NCEE) were convened in 1996 in the United States to develop a consensus approach for liquefaction assessment. The workshop proceedings present a summary of consensus recommendations which represent the most used approach by practitioners (Youd et al., 2001). According to these recommendations, the CRR and CSR values can be calculated as follows:

2.1 Evaluation of Cyclic Resistance Ratio (CRR)

The general approach utilizes the empirical curve adapted by Seed et al. (1985) which is illustrated in Figure 1. This curve correlates the CRR with the normalized Standard Penetration Test (SPT), $(N_1)_{60}$, for a moment magnitude of 7.5 earthquake.

The measured SPT, N_m, needs to be corrected to obtain the normalized SPT (corrected), $(N_1)_{60}$, used by Seed et al. (1985). The corrections should be applied according to the following formula:

$$(N_1)_{60} = N_m\, C_N\, C_E\, C_B\, C_R\, C_S \qquad [1]$$

The procedure to apply the correction factors were discussed and summarized in detail by the NCEER workshop (Youd et al. 2001).

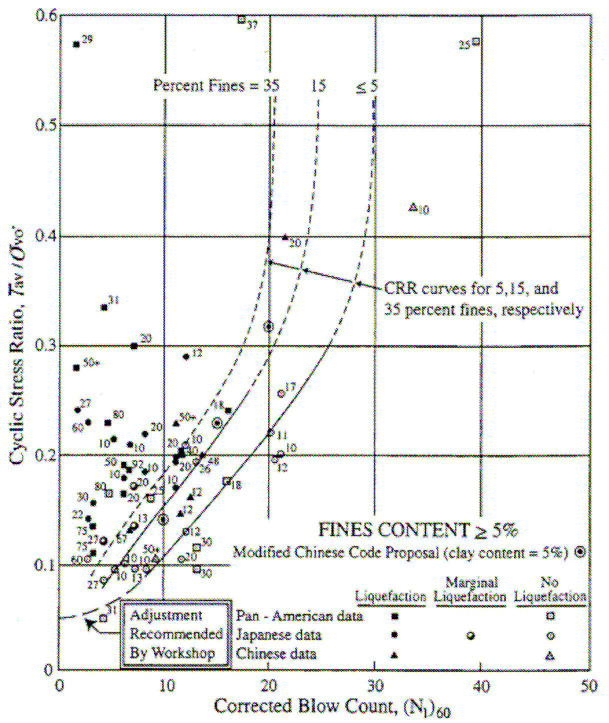

Figure 1: Base Curve for Determination of CRR from $(N_1)_{60}$ for Moment Magnitude 7.5 with data from case histories (Youd et al. 2001 modified from Seed et al. 1985)

For sites with other moment magnitudes, a magnitude scaling factor (MSF) was introduced to account for the different shaking duration, which is traditionally represented by the number of (uniform) stress cycles at the site of interest. It should be noted that it is more suitable to use a site-specific moment magnitude. The MSF factor was implemented in the safety factor relationship against liquefaction as given below:

$$Fs = (CRR_{7.5}/CSR)\ MSF \qquad [2]$$

It should be noted that the MSF factor illustrated in Figure 2 takes values less than 1 for sites of moment magnitudes over 7.5 and more than 1 for sites less than 7.5. NCEER workshop, however, has recommended a narrow range for MSF as shown in Figure 2.

2.2 Evaluation of Cyclic Stress Ratio (CSR)

The simplified approach (Seed and Idriss, 1971) outlines a simple equation to calculate the CSR which is the same equation adopted the NCEER workshop and used by most practitioners today. The CSR is calculated as follows:

$$CSR = (\tau_{av}/\sigma_v') = 0.65\ (a_{max}/g)\ (\sigma_v/\sigma_v')\ r_d \qquad [3]$$

where a_{max}: Peak horizontal ground surface acceleration (PGA), g: Acceleration of gravity, σ_v: Total vertical stress, σ_v': Effective vertical stress, r_d : Shear reduction coefficient.

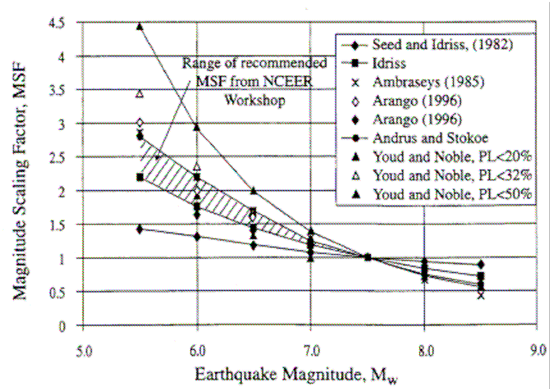

Figure 2: Magnitude Scaling Factors Derived by Various Investigators (Youd and Noble 1997)

The value a_{max} is the PGA as defined for the site in the NBCC and adjusted as necessary to account for site-specific ground conditions (i.e., site class conditions which may differ from Site Class C). Hence the CSR is proportional to the specified PGA, and the factor of safety is inversely proportional to the PGA.

Since the PGA in the 2005 NBCC has been increased significantly, the CSR will increase significantly and the factor of safety will decrease. Table 1 lists the ratios of the 2005 PGA to the 1995 PGA. Based on a simplified approach, these ratios will be the same as the ratios of the increase in CSR from the 2005 NBCC PGA to the 1995 NBCC. The average increase is about 2.7 with the minimum of 1.95 for Québec City.

Table 1: Ratio of 2005 NBCC PGA to 1995 NBCC PGA

PGA(g)	1995 NBCC	2005 NBCC	Ratio
Vancouver	0.23 (g)	0.48 (g)	2.09
Calgary	0.019	0.088	4.63
Toronto	0.056	0.20	3.57
Ottawa	0.20	0.42	2.1
Montréal	0.18	0.43	2.39
Québec City	0.19	0.37	1.95
Halifax	0.056	0.12	2.14

On the other hand, seismic forces in structural design such as base shear have not appreciably changed by implementing the new code, 2005 NBCC, as discussed by Heidebrecht (2003), through the consideration of the overstrength factor. Therefore, the increases listed in Table 1 may not be needed to achieve uniform geotechnical performance intended in the 2005 NBCC. In order to achieve a design that is consistent with the 2005 NBCC, the developments of the 2005 NBCC seismic provisions have to be analysed and incorporated into liquefaction assessment, where applicable.

3 DEVELOPMENT OF THE 2005 NBCC SEISMIC PROVISIONS

As discussed by Mohamad (2006) and Mohamad and Law (2007), the 2005 NBCC seismic provisions had undergone several developments pertaining to the seismological, geotechnical, and structural aspects. In the next section, two of the main developments which are somewhat applicable to liquefaction assessment will be briefly discussed:

3.1 New Probability Level

Probability level is one of the main changes in the seismic hazard maps. Geological Survey of Canada (GSC) has introduced several updates to the former seismic maps. The new hazard level is set at 2% in 50 year event while it was 10% in 50 year event in the 1995 NBCC. This change has significantly increased the PGAs in the country as listed in Table 1. The performance achieved in the 1995 NBCC design was not uniform in different parts of the country (the geographical variation in the slopes of the hazard curves present across Canada would preclude achieving a constant level of safety according to Adams and Atkinson 2003) as illustrated in Figure 3. There are several reasons for this difference, which include the distance from the source of the governing earthquakes.

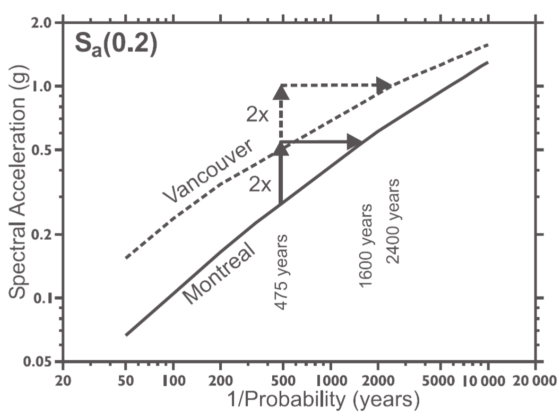

Figure 3: Seismic hazard curves of Montréal and Vancouver (after Adams and Atkinson 2003)

In the United States, NEHRP (National Earthquake Hazards Reduction Program) recommended provisions have also undergone significant developments recently. In earlier editions of the NEHRP provisions, seismic hazard maps for the United States were developed uniformly at 10% in 50 year event. The slopes of the seismic hazard curves in different seismic zones are not the same, especially in the range of 10% in 50 years, as illustrated in Figure 4. Therefore, designs in different zones had achieved different levels of performance.

Figure 4 shows two general slopes of hazard curves, one for the east represented by the dashed curves, and another for the west by the solid curves. Different performances between different regions were the main drive for the recent probability revisions of seismic provisions (Adams 2006, personal communications). The gentle slope curve in the west (i.e., Vancouver or Los Angeles) would have higher performance for the same safety margin.

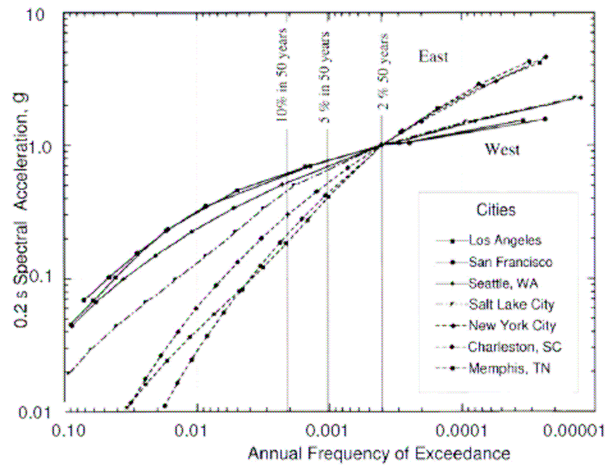

Figure 4: Normalized seismic hazard curves (after NEHRP 2003 Commentary)

The implications of the new concept of uniform performance may point out that the philosophy of seismic design has been changed in the recent American and Canadian building codes from designing a structure to resist 475 year earthquake events with large margin of safety (1995 NBCC) to designing a structure that will just survive 2500-year earthquake events (2005 NBCC). Therefore, to achieve uniform performances for another specific design type, such as liquefaction assessment, a similar philosophy should be implemented.

3.2 Overstrength Factor R_O

The overstrength factor accounts for the tendency of structures designed according to a specific standard (such as Concrete Design Handbook, CSA A23.3 1994) to reserve some strength beyond the design level. The 2% in 50 year event deemed suitable for an ultimate performance or near-collapse conditions will be achieved under extreme load effects (Mitchell et al. 2003). Thus, the near-collapse will occur at the performance level which is beyond the design level (safe level).

The new design approach implemented in the 2005 NBCC is to specify a target performance (2% / 50 years) for the whole country, and then reduce it back to obtain the design level using a uniform overstrength factor for designing the same type of structure as shown in Figure 5. In other words, the overstrength factor represents the overall safety factor inherent in the design.

Within one country, the design of an engineering system follows the same standard specifications like CSA in Canada. Therefore the reserved strength which depends on analysis methods, environmental factors, construction techniques, and material properties, should be identical nationwide. Seismic forces, however, do not increase uniformly nationwide when the probability of exceedance decreases as illustrated earlier in Figures 3 to 5. Consequently, in different regions, the design will not maintain the same performance.

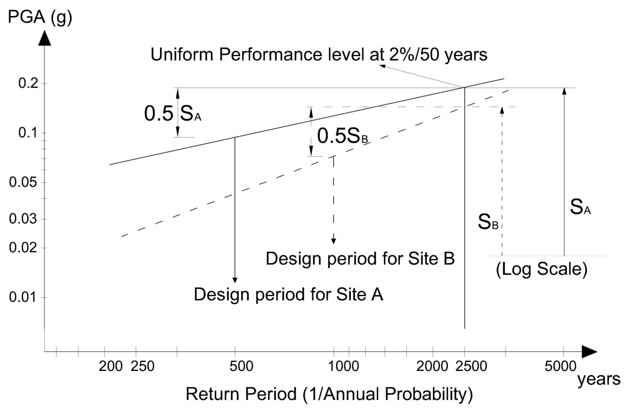

Figure 5: The concept of uniform performance but different design levels similar to 2005 NBCC

4 OVERSTRENGTH IN LIQUEFACTION ASSESSMENT

To transpose the overstrength factor concept to the liquefaction assessment, it must be recognized that the definition of ultimate performance or near collapse conditions recognized in structural systems would be comparable to the state of near triggering liquefaction.

Some components such as the conventional factor of safety used to assess liquefaction (see equation 2) and the fact that it is now recognized that the deterministic curve (Figure 1) used to assess non-liquefiable versus liquefiable sites would contribute also to the overstrength factor. In addition, the various magnitude-distance combinations to the overall seismic hazard would also add to the overstrength factor. The following sections discuss these factors further.

4.1 R_{FS} (Factor of Safety)

Since the uniform performance level proposed by the 2005 NBCC represents the ultimate performance or near collapse conditions, then it is more realistic to use safety factor of 1.0 when assessing liquefaction at the performance level. Prior standards of practice for assessing liquefaction potential use the 475 year return period earthquake and incorporated factors of safety greater than 1.0 and therefore would reserve some overstrength.

According to the 1997 California Department of Conservation, Division of Mines and Geology (CDMG) guidelines (Special Publication 117), a factor of safety against liquefaction greater than about 1.3 can be considered an acceptable level of risk based on ground shaking levels which have a 10% probability of being exceeded in 50 years. This factor of safety assumes that high-quality, site-specific penetration resistance and geotechnical laboratory data were collected. However, the guidelines acknowledge the possibility of using lower factors of safety only after evaluating the severity of the hazard associated with potential liquefaction. The 2000 State of Nevada guidelines for evaluating liquefaction recommend that a factor of safety in the range of about 1.1 is generally acceptable for single family dwellings, while a higher value in the range of 1.3 is appropriate for more critical structures. The 2007 Draper City (Utah, USA) Geologic Hazards Ordinance requires a site-specific liquefaction investigation to be performed using safety factors ranging from 1.2 to 1.3 depending on the land use and/or liquefaction potentials.

Thus, considering land use implications and the potential consequences of triggering liquefaction, the factors of safety against liquefaction were typically between 1.1 and 1.3 using the 475-year earthquake and provide some overstrength to survive stronger or longer shaking and to account for variability in the subsurface models and material properties.

4.2 R_{CRR} (Conservative Deterministic vs. Probabilistic CRR)

The liquefaction curves were originally plotted originally by Seed et al. (1985) to provide a reasonable, if not conservative, boundary between liquefiable and non-liquefiable conditions. In an ultimate performance level or near collapse state, it is more appropriate to use criteria which represent the mean values (50% probability of liquefaction). Several probabilistic studies, some by using the expanded (up-to-date) case histories, conducted by Liao and Whitman (1986), Liao et al. (1988), Youd and Noble (1997), Liao and Lum (1998), Toprak et al. (1999) and recently by Cetin et al. (2004) and Salloum and Law (2007) have indicated that Seed's curves do not represent the 50 % probability of liquefaction. Thus it appears that Seed's curves also had some amount of overstrength built in that might provide some capacity to survive higher magnitude earthquakes as well as reflect uncertainties in the empirical approach.

Using pre-1989 USGS (US Geological Survey) and Noble and Youd (1998) databases, Toprak et al. (1999) proposed that Seed's curve is characterized by a probability of approximately 20, 30 and 40% for $(N_1)_{60cs}$ below 10, between 10 and 15, and above 15 blows per foot, respectively. The logistic regression equation obtained for the previous database is given in the Equation 4 where P_L is the probability of liquefaction will occur.

$$\ln[P_L/(1-P_L)]=10.4459-0.2295(N_1)_{60cs}+4.0573\ln(CSR/MSF) \quad [4]$$

Figure 6 illustrates the SPT probabilistic curves for 20, 30, 40, and 50% probabilities of triggering liquefaction along with the deterministic curve of Youd and Idriss (1997). It should be noted that all the above curves correspond to an earthquake of magnitude 7.5.

By examining the probabilistic data, it appears appropriate to choose corresponding overstrength factors of 1.4, 1.3, and 1.1 for sites of $(N_1)_{60cs}$ lower than 10, between 10 and 15, and over 15 blows per foot, respectively.

4.3 R_{ME} (Magnitude Effect)

Geological Survey of Canada has adopted a procedure to develop the seismic hazard curves (such as PGA). This probabilistic procedure considers contributions of all possible earthquakes from different distances and more importantly from different magnitudes. Therefore, even relatively minor earthquake would contribute to the 2 % in 50 year PGAs as shown in Figure 7.

As stated by Finn and Whitman (2006), there may be an unintentional conservatism in evaluating the potential for triggering liquefaction as a consequence of the new procedure. The degree of the conservatism depends on the seismic environment.

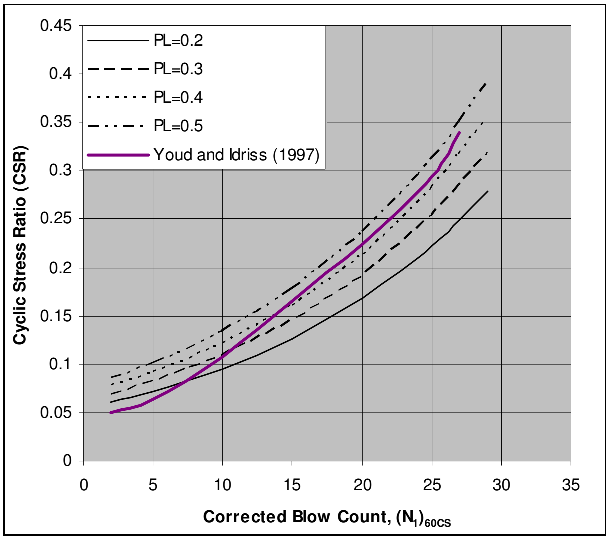

Figure 6: SPT Probabilistic Curves after Toprak et al. (1999)

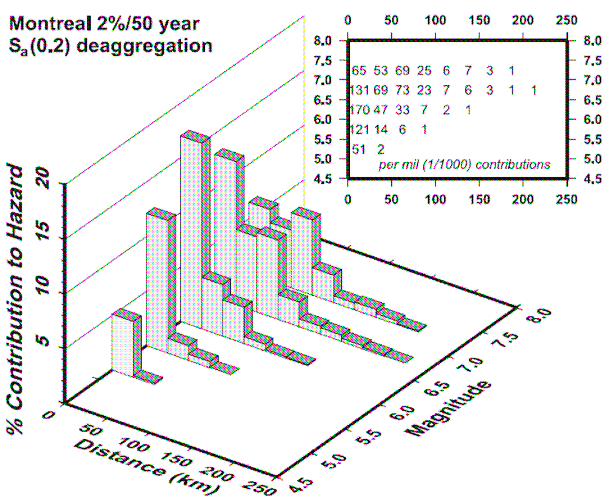

Figure 7: Deaggregation of the Hazard Curve of Montréal (Adams and Atkinson 2003)

Finn and Whitman (2006) investigated the use of modal, mean, or weighted magnitudes and proper deaggregation of total hazard. It was concluded that the safety factors against liquefaction from the weighted magnitude probabilistic are about 8% to 37% higher than the factors given by current practice (conventional assessment) in Vancouver, Toronto, Ottawa, and Montreal.

This conservatism level varies between different cities, and therefore, it will not reserve the same overstrength, and ultimately the same performance which was the intent of the new seismic provision developments.

Any conservatism that existed in any engineering arrangement could be translated to an overstrength factor, and since the overstrength is proportional to the factor of safety, the overstrength component from the magnitude effect will be about 8% to 37%, accordingly.

In summary, the total combined overstrength existed in liquefaction assessment can range as shown in Table 2.

Table 2: Overstrength Factors

	Liquefaction	Gravity Retaining Walls	Structural design (2005 NBCC)
R_{FS}	1.1-1.3	-	-
R_{CRR}	1.1-1.4	-	-
R_{ME}	1.08-1.37	-	-
R_O	1.3-2.5	1.66- 1.91[1]	1.28-1.63[2]

[1]Mohamad and Law (2007)
[2]Mitchell et al. (2003)

5 OVERSTRENGTH IN NEHRP RECOMMENDED PROVISIONS

Recent NEHRP provisions were developed based on a low probability hazard level (2%/50 years), which is different from the design level. The corresponding overstrength in a structural design has been estimated to vary from 1.5 to 3.5 for different lateral resistant systems.

An overstrength factor of 1.5 is suggested to reduce the spectral values. The 1.5 estimate is supported by Kennedy et al. (1994) who evaluated structural design margins (NEHRP Commentary 2003). Further, Kennedy has proposed four levels of qualitative seismic performance goals varying in their probability of exceedance (Categories 1 to 4) and up to 100,000 year return period in Category 4.

6 PRACTICAL IMPLICATIONS

The intent of this paper was mainly to introduce the concept of the overstrength in liquefaction assessment as the most suitable manner to deal with the new hazard level. One should be extremely cautious when considering a suitable value for overstrength factor in liquefaction assessment. In addition, under no circumstances should an overstrength factor exceed the ratio of the tabulated NBCC PGA in 2005 to 1995 listed in Table 1.

In keeping with the provisions of the 2005 NBCC and achieving an improved safety level, it is suggested that the PGAs of the 2% in 50 year event be adopted along with a suitable overstrength factor. The resultant PGAs after the application of the overstrength factor can be used in directly in liquefaction assessment (simplified approach).

The use of the corresponding overstrength factor will maintain uniform performance in various parts of the country, and will bring about an improvement of the safety level for liquefaction assessment since the new design PGAs will moderately increase.

Thus, the design PGAs are equal to the PGAs of the 2500-year return period event divided directly by corresponding overstrength factor. The 2005 NBCC based liquefaction assessment will commence by using the design PGAs and follow the same simplified procedure currently used in practice.

7 CONCLUSIONS

The following conclusions can be drawn from this study:

- The philosophy of seismic design has been changed from a design to resist a 475-year event (1995 NBCC) to a design that will survive a 2500-year event (2005 NBCC).
- In the 2005 NBCC provisions, the philosophy is to apply a uniform performance level across the country and to make use of the overstrength of the design structures that is consistent with the performance of the structures.
- There is also overstrength inherent in the liquefaction assessment.
- After examining the various aspects of the overstrength associated with liquefaction assessment, an overstrength factor ranging from 1.3 to 2.5 is proposed. The use of this factor will maintain uniform performance in various parts of the country, and will bring about an improvement to the safety level of liquefaction assessment.
- In keeping with the provisions of the 2005 NBCC and to achieve an improved safety level, it is therefore suggested that for liquefaction assessment, the PGAs of the 2% in 50 years event should be used with the corresponding overstrength factor. The design PGAs will equal to the PGAs from the 2005 NBCC divided by the overstrength factor. These PGAs can be used directly in the simplified approach.

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