Using a reliability-based analysis for evaluating soil liquefaction potential

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ABSTRACT
Simplified methods based on in-situ tests such as the standard penetration test (SPT) and the cone penetration test (CPT), are widely used by geotechnical engineers for assessing the liquefaction potential of soils. The four studied methods are those recommended by the NCEER and those proposed by Boulanger and Idriss. In our study, a reliability analysis, based on conventional probability theory, is used to calculate the relationship between the liquefaction probability and reliability index on one hand, and the traditional factor of safety on the other hand. The case study is in Qatar in the Persian Gulf. Hundreds of CPT and SPT tests have been carried out at this site. Uncertainty in the evaluation of peak horizontal earthquake induced ground acceleration is considered. It is observed that the probability of liquefaction drastically changes for a small change in the factor safety.

RÉSUMÉ
Des méthodes simplifiées basées sur les essais in-situ comme l'essai de pénétration standard (SPT) et l'essai de pénétration au cône statique (CPT), sont très utilisées par les ingénieurs géotechniciens pour l'évaluation du potentiel de liquéfaction de sols. Les quatre méthodes étudiées sont celles recommandées par le NCEER et celles proposées par Boulanger et Idriss. Dans notre étude, une analyse fiabiliste basée sur la théorie de probabilité conventionnelle est utilisée pour établir la relation entre la probabilité de liquéfaction et l'indice de fiabilité d'une part, et le coefficient traditionnel de sécurité d'autre part. L'étude de cas se trouve au Qatar dans le golfe persique. Des centaines d'essais SPT et CPT ont été réalisés. L'incertitude dans l'évaluation de l'accélération horizontale maximale induite est considérée. Il est observé que la probabilité de liquéfaction est modifiée significativement pour un léger changement du coefficient de sécurité.

1 INTRODUCTION

Over the past years, noteworthy attention has been given to the understanding of soil liquefaction induced by earthquake loading. Defining liquefaction under its different aspects was given by Ishihara (1993). Liquefaction problems have been of continuous interest since the 1964 earthquakes in Niigata (Japan) and Alaska (USA). Since that time, and after every earthquake namely the most recent and destructive ones (Chile 2010, New Zealand 2010 and 2011, Japan 2011) reconnaissance teams tried to investigate whether damages could be related to specific geotechnical failures namely liquefaction problems. Recently also, the analysis of soil liquefaction potential evaluation has been extensively discussed by Idriss and Boulanger (2008), Seed (2010), and Dobry and Abdoun (2011).

In 1999, the most important damages in Chi-Chi earthquake in Taiwan were due to the widespread liquefaction. Because of the fact that obtaining undisturbed soil samples is very costly, most of the liquefaction analysis is based on simplified methods considering the results of in-situ tests such as the standard penetration test (SPT) and the cone penetration test (CPT) (Seed and Idriss, 1982; Robertson and Wride, 1998; Idriss and Boulanger, 2004). A number of SPT-based and CPT-based procedures propose the calculation of liquefaction resistance (Seed et al. 2003). The evaluation of liquefaction potential of soils is affected by the uncertainty on earthquake parameters (magnitude, epicentre distance, duration) on one hand, and the uncertainty on soil properties (strength properties) on the other hand. Nevertheless, deterministic liquefaction potential evaluation yielding a traditional factor of safety does not clearly deal with the above mentioned uncertainties. In order to overcome the limitations of the deterministic analyses, probabilistic approaches have been used as early as in 1979 by Halder and Tang, and in 1982 by Fardis and Veneziano, who performed a first order second-moment analysis to take into consideration the variability of soil parameters that affect soil liquefaction; but these early simplified methods are seldom used nowadays. Chi and Ou (2003) developed a method for evaluating an average annual liquefaction probability based on seismic energy dissipation theory and limit state methodology (Juang et al., 1999 and 2000). In the deterministic analysis of soil liquefaction potential, fixed values of earthquake magnitude and peak acceleration are used; but without understanding the rate of occurrence of a specific event, it is not clear whether this performance is acceptable (Kramer et al. 2006). Lai et al. (2006) developed a simple model for evaluating liquefaction probability using CPT data and based on logistic regression analysis of 396 case histories. Kramer and Mayfield (2007) showed how the entire range of potential ground shaking can be considered in a fully probabilistic evaluation approach using a performance based analysis; the result was a return period of liquefaction rather than a simple factor of safety or a probability of liquefaction. It is important to note that the rationality of a probabilistic analysis mainly depends on the amount and quality of available data as far as

earthquake parameters and soil strength properties are concerned (Juang et al 2001 and 2002; Cetin et al. 2004; Hwang et al. 2005; Moss et al. 2006).

In this study, a practical reliability- based method is used for assessing the soil liquefaction potential. It is based on the conventional probability theory and involving the usual earthquake induced cyclic stress ratio (CSR) and the soil cyclic strength (CRR). Using CSR and CRR statistics and the first order second-moment, it becomes straightforward to calculate the liquefaction probability and reliability index, and compare results with the traditional factor of safety. The CRR is generally obtained by correlation to in-situ test results mainly SPT and CPT. In-situ testing data include the corrected blow counts number of SPT and the normalised tip resistance of cone penetration (CPT). This paper presents an analysis of four methods; the first two methods have been adopted by the NCEER (Youd et al. 2001): these are the CPT method from Robertson and Wride (1998) and the SPT method from Youd et al. (2001). The other remaining two methods are respectively an SPT method and a CPT method studied by Boulanger and Idriss (2004). Calculations following the four methods have been performed in the absence of any initial static shear stress ($K\alpha$ =1). The studied site is in Qatar in the Persian Gulf region where huge civil engineering projects are presently realized. The authors already performed an analysis of the existing database on CPT and SPT that yielded interesting conclusions when comparing the deterministic liquefaction potential evaluation methods (Rahhal and Zakhem 2008): In this paper, a reliability based analysis is undertaken to estimate the probability of failure as well as the reliability index based on the four adopted methods for liquefaction potential evaluation.

2 EVALUATION OF LIQUEFACTION POTENTIAL

2.1 Cyclic Stress Ratio CSR

Seed and Idriss (1982) proposed a simple approach to evaluate stresses induced by an earthquake. They estimated that the induced shear stress at a depth z was given by the following equation:

$$\tau_{max} = \frac{a_{max} \times r_d \times \sigma_v}{g} \qquad [1]$$

With a_{max} being the maximum horizontal acceleration of the earthquake as a function of gravity g, σ_v being the vertical stress at a depth z, and r_d being a reduction factor taking into account the deformability of the soil column located above the considered point. The equivalent uniform cyclic shear stress generated by the earthquake is given by:

$$\tau_{cyclic} = 0.65 \times \tau_{max} \qquad [2]$$

The uniform cyclic stresses are then normalized by the effective initial vertical stress. The obtained ratio is called the cyclic stress ratio (CSR) and is given by the equation:

$$CSR_{\text{in situ}} = \frac{\tau_{cyclic}}{\sigma'_{v0}} = \frac{0.65 \times a_{max} \times r_d \times \sigma_v}{g \times \sigma'_{v0}} \qquad [3]$$

2.2 Cyclic Resistance Ratio CRR

The cyclic resistance ratio (CRR) giving the resistance developed by soil against liquefaction is defined similarly to the CSR. As far as in situ tests are concerned, SPT (standard penetration test), and CPT (Cone penetration test) are the most used to obtain CRR. The CRR may be obtained through two approaches: First by correlating the N value (number of blows in SPT) or q_c (point or bearing resistance measured at the tip of the cone in the CPT) with the history of stresses in soil to know whether it has liquefied or not, the CRR being the limit separating liquefaction from non liquefaction. Second, by determining the CRR from laboratory tests and correlating it with N or q_c. Results of a traditional liquefaction potential evaluation for a site are generally presented in the form of factor of safety F_s defined by the ratio CRR / CSR. Theoretically, the occurrence of liquefaction is in the case where $F_s \leq 1$. This approach is known as the deterministic approach. However, due to uncertainties in the model or parameters used, a factor of safety $F_s > 1$ obtained in the deterministic approach does not always correspond to a non liquefaction condition. The same comment may be given to the situation where $F_s \leq 1$. Efforts are carried out to quantify these uncertainties by evaluating the liquefaction potential in terms of probability of liquefaction. This approach is designated by the probabilistic approach.

The main corrections affecting the cyclic resistance ratio CRR may be grouped in four categories: The corrections for thin layers, for earthquake magnitude, for surcharge and initial static shear, and finally for percentage of fines. In a CPT test, the bearing resistance q_c may be influenced by the presence of soft soil layers above and beneath the liquefiable layer; there is an obligation to correct the value of bearing point resistance q_c measured for such thin layers. The simplified approach to measure CRR is based on a reference earthquake magnitude of 7.5; the equivalent number of uniform cycles being proportional to earthquake magnitude, the minimum stress ratio (CSR minimum) required to cause liquefaction, that is equal to the CRR, decreases when magnitude M increases.

3 RELIABILITY-BASED APPROACH

Methods based on the traditional factor of safety don't take into account the variability of neither the strength, nor the stress. This means that the probability of failure cannot be estimated. The probabilistic method considers a performance function defined by:

$$Z = CRR - CSR \qquad [4]$$

Further clarifications on the value of Z can be given as follows: when Z < 0, failure has occurred (liquefaction in our case); when Z > 0, it is the safe zone; and finally

when Z = 0, this corresponds to the limit state. Because of the uncertainties involved in the estimation of CSR and CRR, the latter could be considered as random variables, and Z will also become a random variable. The liquefaction probability P_f is defined as the probability of having a negative performance function value Z. A simplified method would be to consider the statistics of random variables CRR and CSR to calculate the statistics of Z. The reliability index β is defined as the ratio of μ_z to σ_z, being respectively the mean value (μ_z) and standard deviation (σ_z) of the limit state function (Z). Probabilistic distributions of the geotechnical parameters are generally lightly skewed in such a way that they can be practically represented by lognormal distribution functions. So, the CSR and CRR follow lognormal distributions functions. The performance function Z can be presented by a lognormal distribution function if CSR and CRR are independent variables. Based on this hypothesis, the probability of failure and the reliability index may be calculated using equations [5] and [6], with δ being the coefficient of variance:

$$\beta = \frac{\mu_Z}{\sigma_Z} = \frac{\mu_{\ln CRR} - \mu_{\ln CSR}}{\sqrt{\sigma^2_{\ln CRR} + \sigma^2_{\ln CSR}}} = \frac{\ln\left[\frac{\mu_{CRR}}{\sigma_{CSR}}\left(\frac{\delta^2_{CSR}+1}{\delta^2_{CRR}+1}\right)^{\frac{1}{2}}\right]}{\ln\left\{\left(\delta^2_{CRR}+1\right)\left(\delta^2_{CSR}+1\right)\right\}^{\frac{1}{2}}} \quad [5]$$

$$P_f = \phi(-\beta) = 1 - \phi(\beta) \quad [6]$$

δ is defined as the coefficient of variance equal to the ratio of σ (standard deviation) over μ (mean). ϕ is the cumulative normal probability (the area under the standard normal distribution curve).

The simplified probabilistic method consists in considering the CSR and CRR values for a specific depth, calculating the mean, standard deviation and coefficient of variance, as given in equations [5] and [6]. For every factor of safety calculated, the reliability index and the probability of failure may be obtained.

On the other hand, an additional analysis will be carried out to assess the effect of variability and uncertainty in the evaluation of maximum horizontal earthquake acceleration considered. In this scope, a coefficient of variance δ = 0,10 will be considered for a_{max}. Since only the variation of a_{max} is taken into account, and since calculations are carried out for a specific depth, all other parameters entering in the equations are constants. CSR may then be described by equation [2]. Let c be a constant for a given depth:

$$c = \frac{0.65 \times r_d \times \sigma_v}{g \times \sigma'_{v0}} \quad [7]$$

Hence CSR = c $\times$ a_{max}, then the mean of CSR is expressed by: μ_{CSR} = c μ_{amax}. As far as the standard deviation σ_{CSR} is concerned, it may be obtained by $\sigma_{CSR} = \delta_{CSR} \times \mu_{CSR}$. In this paper, two different scenarios have been analyzed in the calculations: First, a horizontal variation by choosing a specific depth for analysis is considered; all CSR and CRR corresponding to the same depth are analyzed. Second, a vertical variation considering a CPT profile is considered.

4 CASE STUDY

The case study area is located on the Persian Gulf southern coast in the city of Doha in Qatar. Many vital civil engineering projects are being now executed in the area. Geotechnical works consisted in filling the coast, hence enlarging the area by gaining into the sea using material from nearby dredging. The fill material was compacted using dynamic methods and repeated passes of compactors (25 tons). In order to insure that the compaction was carried out correctly, many boreholes were realized in order to obtain coring to be examined in the laboratory. SPT and CPT tests were realized as well. Available soil investigation data helps in the assessment.

4.1 Geotechnical Context of the Site

The obtained core samples are consistent with the regional geology of Doha. There is a surface layer of dense silty sand, fine to coarse, containing shell fragments and coarse calcarenite gravel. The approximate thickness of this layer is 3.5 m. This layer is mainly made of materials imported during backfilling part of ongoing activities. This layer covers in most of the borings a lower thin layer of calcarenite very to moderately soft, whose thickness, when this layer exists, varies between 0.3 and 1m at most. It is characterized by the presence of voids up to 150mm in size, and horizontal to sub-horizontal fractures closely spaced, and is found mainly to the North of the studied site. After this layer or directly after the first layer, we find loose silty sand, fine to coarse, containing shells, and over a thickness of about 3-4m.

The bedrock follows; it consists of limestone conglomerate, moderately to strongly altered, overlying the calcisiltite, and generally soft to moderately soft. It is characterized by a fractured state. Water levels measured during drilling indicate the presence of groundwater at a depth of approximately 2-3m. It should be noted that the originality of this site resides in its calcareous sandy cover nature.

4.2 Seismic Context of the Region

In the Persian Gulf, seismic activity increases when approaching the active Zagros fold belt in Iran. A study was conducted to assess the seismic hazard in two areas of Qatar, Ras Laffan in the north-east and Umm Said in the south-east, taking into account the various possible sources and applying suitable attenuation equations. The following results were obtained: the maximum probable earthquake magnitude is to be more likely from 6.0 to 6.5. For the area of Umm Said, accelerations of 0.02g and 0.07g corresponding respectively to standard operating

basis earthquake (OBE, return period of 500 years) and safety shutdown earthquake (SSE, return period of 10,000 years) were calculated. For the area of Ras Laffan, accelerations of 0.06g and 0.21g corresponding respectively to standard operating basis earthquake (OBE, return period of 500 years) and safety shutdown earthquake (SSE, return period of 10,000 years) were calculated. The city of Doha (the studied site) is located in the east and middle of Qatar, a maximum acceleration of 0.15g is taken for calculation. The maximum magnitude of 6.5 is also considered.

4.3 Analysis and Discussion

For the above mentioned site, Rahhal and Zakhem (2008) compared the methods adopted by the NCEER based on CPT (Robertson et Wride, 1998), and SPT (Youd et al. 2001), and those more recently developed by Boulanger and Idriss in 2004 for CPT and SPT as well. In the presentation of results these four methods are referred to respectively: CPT-1998, SPT-1997, CPT-2004 and SPT-2004. It has been shown that the method of Boulanger and Idriss for CPT (CPT-2004) is the most conservative, in other words it yields the lowest factors of safety. Furthermore, it has been possible to outline a linear dependence between the factors of safety of the different calculation methods.

In the first analyses, Tables 1, 2 and 3 present the probabilities of failures associated with the factor of safety obtained through the four methods, for depths of 2,5m, 3,0m and 3,5m respectively. In these tables, probabilities of failure are given with both a coefficient of variance equal to 0 and equal to 0,1 for maximum horizontal acceleration a_{max}. Analysis of the results shows that probabilities of failure are very much affected even when there is slight change in the factor of safety. Based on the 4 methods, for a factor of safety close to 1, the probability of failure by liquefaction is 1 (100%); and for a factor of safety of 1,8 the probability of failure becomes very small.

As far as the reliability indices are concerned, a factor of safety of 1 yield a reliability index of 0, while for a factor of safety of 1,8 the reliability index may be as high as 7. It is shown that deterministic analysis is very conservative as far as results interpretation is concerned. In the safe zone, while the increase in the reliability index may be quite high, the corresponding enhancement in factor of safety remains very small.

Table 1. Factor of safety and probability of failure using different methods at a depth of 2,5m.

Method of Calculation		Coefficient of variance for a_{max} = 0	Coefficient of variance for a_{max} = 0,1
	FS	P_f	P_f
CPT-1998	0,81	0,948	0,897
	1,28	0,018	0,053
	1,76	9,72E-13	4,32E-06
	2,24	7,71E-42	1,64E-12
	2,74	3,71E-97	3,32E-20
	3,12	0	8,04E-29
CPT-2004	0,78	0,942	0,906
	1,25	0,025	0,068
	7,71	2,10E-10	2,13E-05
	2,26	3,24E-45	6,29E-13
	2,76	1,60E-62	1,56E-18
	3,28	2,12E-154	6,53E-28
SPT-1997	0,90	0,841	0,764
	1,26	0,035	0,076
	1,75	2,77E-14	3,17E-06
	2,20	4,22E-30	3,42E-11
	2,67	1,40E-86	4,65E-19
SPT-2004	0,91	0,894	0,770
	1,27	0,023	0,061
	1,72	3,50E-11	1,30E-05
	2,24	1,13E-31	1,32E-11
	2,70	1,02E-83	2,88E-19
	3,26	2,44E-178	2,72E-28

Table 2. Factor of safety and probability of failure using different methods at a depth of 3,0m.

Method of Calculation		Coefficient of variance for a_{max} = 0	Coefficient of variance for a_{max} = 0,1
	FS	P_f	P_f
CPT-1998	0,76	0,957	0,926
	1,20	0,074	0,127
	1,73	7,86E-11	1,33E-05
	2,21	8,12E-34	1,27E-11
	2,73	1,33E-71	6,46E-19
CPT-2004	0,45	1	1
	0,75	0,964	0,937
	1,20	0,053	0,110
	1,73	1,25E-10	1,50E-05
	2,24	1,27E-35	5,04E-12
	2,72	1,48E-57	7,95E-18
SPT-1997	0,79	0,935	0,898
	1,25	0,017	0,059
	1,71	7,18E-12	1,21E-05
	2,19	1,64E-35	1,38E-11
	2,74	1,99E-80	1,87E-19
SPT-2004	0,82	0,947	0,893
	1,20	0,074	0,126
	1,71	1,31E-08	5,29E-05
	2,24	5,77E-30	2,01E-11
	2,73	1,12E-96	4,23E-20

Table 3. Factor of safety and probability of failure using different methods at a depth of 3,5m.

Method of Calculation		Coefficient of variance for a_{max} = 0	Coefficient of variance for a_{max} = 0,1
	FS	P_f	P_f
CPT-1998	0,71	0,989	0,972
	1,24	0,029	0,074
	1,74	2,76E-13	5,09E-06
	2,25	1,38E-31	1,10E-11
	2,80	2,72E-142	4,44E-22
CPT-2004	0,45	1	1
	0,71	0,957	0,938
	1,22	0,039	0,092

	1,73	7,13E-11	1,37E-05
	2,23	7,17E-25	1,03E-10
	2,68	8,12E-81	5,98E-19
SPT-1997	0,76	0,970	0,939
	1,27	0,019	0,056
	1,73	8,93E-12	9,47E-06
	2,21	1,12E-33	1,31E-11
	2,70	5,53E-65	3,31E-18
SPT-2004	0,79	0,966	0,922
	1,28	0,015	0,050
	1,71	2,88E-10	2,25E-05
	2,25	7,25E-38	2,72E-12
	2,82	7,23E-158	1,14E-22

Tables 4, 5 and 6 present the reliability indices associated with a factor of safety around 1,2 obtained through the four methods, for depths of 2,5m, 3,0m and 3,5m respectively.

Table 4. Factor of safety and reliability index using different methods at a depth of 2,5m.

Method of Calculation		Coefficient of variance for a_{max} = 0	Coefficient of variance for a_{max} = 0,1
	FS	β	β
CPT-1998	1,28	2,11	1,62
CPT-2004	1,25	1,97	1,49
SPT-1997	1,26	1,81	1,43
SPT-2004	1,27	1,99	1,55

Table 5. Factor of safety and reliability index using different methods at a depth of 3,0m.

Method of Calculation		Coefficient of variance for a_{max} = 0	Coefficient of variance for a_{max} = 0,1
	FS	β	β
CPT-1998	1,20	1,44	1,14
CPT-2004	1,20	1,62	1,23
SPT-1997	1,25	2,11	1,56
SPT-2004	1,20	1,45	1,15

Table 6. Factor of safety and reliability index using different methods at a depth of 3,5m.

Method of Calculation		Coefficient of variance for a_{max} = 0	Coefficient of variance for a_{max} = 0,1
	FS	β	β
CPT-1998	1,24	1,89	1,45
CPT-2004	1,22	1,76	1,33
SPT-1997	1,27	2,07	1,59
SPT-2004	1,28	2,17	1,64

It is observed that a lower reliability index is obtained in the presence of a coefficient of variance for a_{max}. Tables 1, 2 and 3 show also that a higher probability of failure is calculated in the presence of a coefficient of variance for a_{max}. The only observed discrepancies are in the case of a factor of safety less than 1; in this case, failure already occurred and the probability of failure is anyway close to 1.

Afterwards, a CPT profile was considered as shown in Figure 1 to analyse the vertical variation. Factors of safety were calculated as a function of depth following the four deterministic methods; results are shown in Figure 2 for the deterministic calculations. The probability of failure by liquefaction has been calculated both with and without considering a coefficient of variance for a_{max}. Figure 3 presents the probability of failure (liquefaction) in the absence of a coefficient of variance for a_{max}, while Figure 4 gives the results in the presence of a coefficient of variance. No major difference can be denoted when analysing the results of Figures 3 and 4.

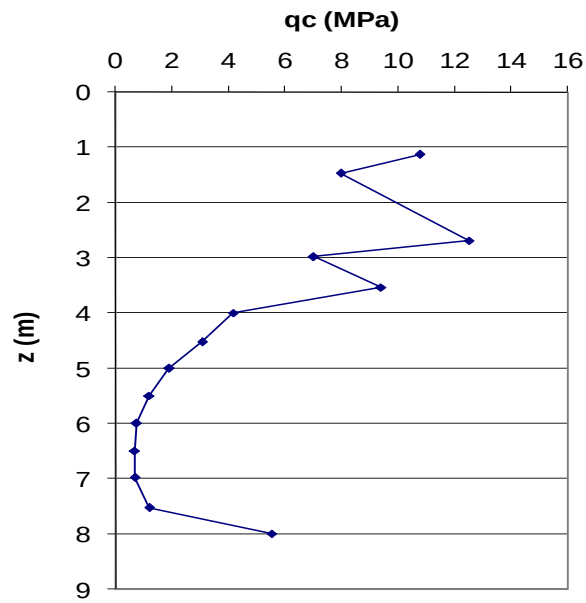

Figure 1. CPT- qc soil profile.

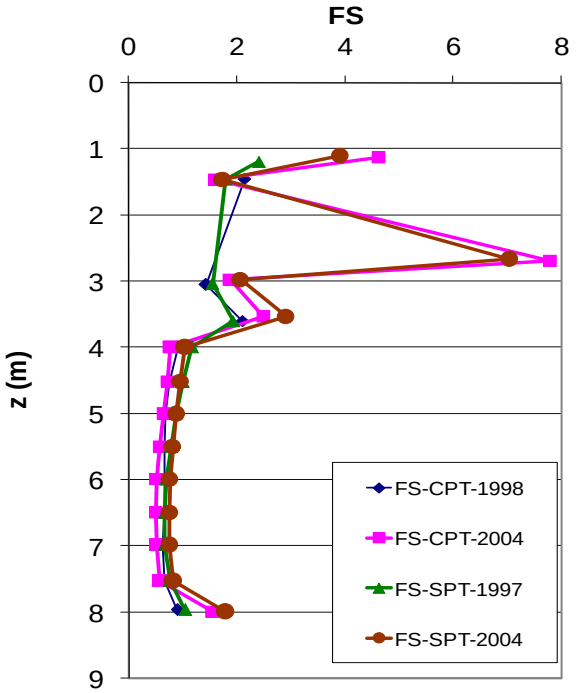

Figure 2. Factor of Safety calculated following the four methods as a function of depth.

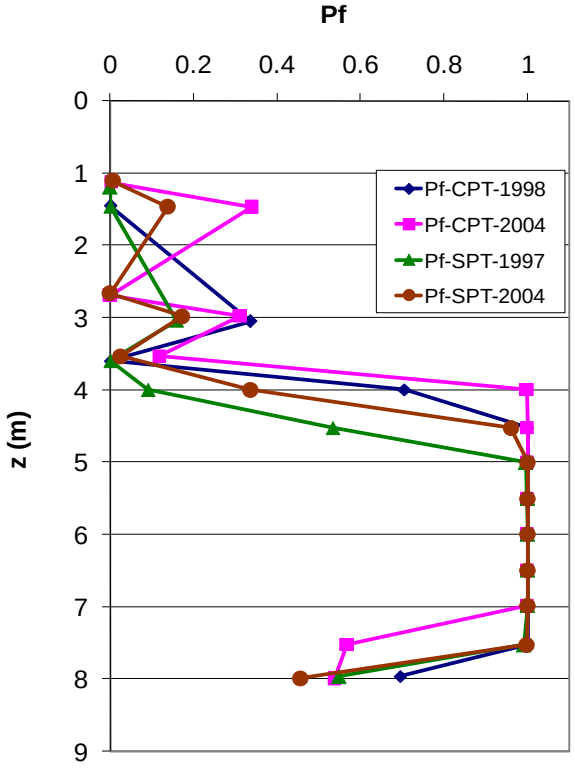

Figure 3. Probability of failure calculated following the four methods as a function of depth, with a coefficient of variance=0 for a_{max}.

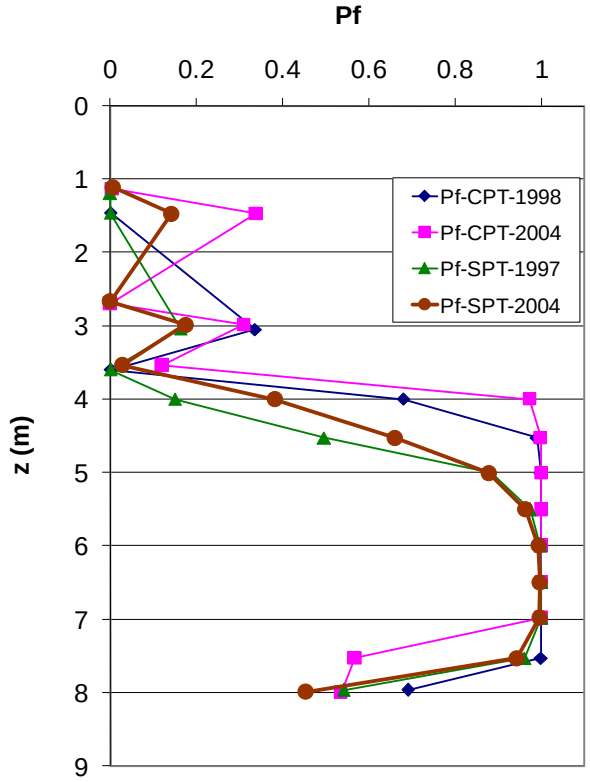

Figure 4. Probability of failure calculated following the four methods as a function of depth, with a coefficient of variance=0,1 for a_{max}.

Probabilistic calculations have revealed the following results: The variations of probabilities of failure and reliability indices are very close following the results of the four deterministic methods used. Of course, the reliability increases with the safety factor for all methods and variations of a_{max}. A safety factor of 1 corresponds to a reliability index of zero. This can be justified by the fact that the reliability index defines the difference between the CSR and CRR, and the safety factor of 1 indicates equality between action (CSR) and reaction (CRR). For safety factors less than one, and thus indicating the failure by liquefaction in the deterministic method, the reliability indices obtained are negative, indicating the absence of reliability. On the other hand, the probability of failure by liquefaction decreases with increasing safety factors. In fact, this probability drops rapidly from 0,9 for safety factors of 1, reaching values of 0,06 to 0,12 for safety factors of 1,2, then tends almost to 0 for safety factors around 1,8. This points out that the adoption of a minimum safety factor permitted equal to 1 in the deterministic calculations involves a huge risk and a high probability of failure as well, and the design of reliable structures requires the adoption of safety factors of about 1,2 to 1,3 or more, depending on the importance of the structure concerned and the degree of allowable risk. Considering a coefficient of variance for a_{max} implies a decrease in reliability and an increase in the likelihood of

failure. Note that for the CPT example presented, the reliability indices are significantly lower than those obtained by considering the horizontal distribution of results. This is because the latter method was performed by subdividing the results for each factor of safety. Whereas for CPT example, all results obtained at a certain level of depth have been considered without taking into account the variability of the corresponding safety factors. Thus, the greater variability in the case of the CPT example led to lower reliability. Finally, the change in the probability of failure as a function of reliability index, especially in the case of the horizontal study is perfectly consistent with published results in the geotechnical literature.

Another point that has not been considered is the effect of parameter r_d which is a reduction factor taking into account the deformability of the soil column located above the considered point. It is clear that the uncertainty on r_d affects the liquefaction potential analysis. This parameter is actually being today the center of many exchanges between researchers working in the evaluation of liquefaction potential, and in fact many equations for this parameter are available in the geotechnical literature (Rahhal and Zakhem, 2008; Seed et al., 2003; Seed, 2010). The authors shall be considering the role of r_d in a reliability based liquefaction analysis in the near future, after more agreement within the geotechnical engineering community is reached on the right r_d to consider starting with the deterministic analysis.

5 CONCLUSION

Application of probabilistic liquefaction analysis is not yet a common practice in the geotechnical engineering community. Still, reliability calculations provide a means of evaluating combined effects of uncertainties, and a means of distinguishing between conditions where uncertainties are high or low. In the case study covered by this paper, the probability of failure drops from 0,9 for safety factors of 1, and tends almost to 0 for safety factors around 1,8. Reliability should be made familiar to most geotechnical engineers, and it should not be perceived as requiring more data, time and effort than deterministic traditional calculations. The authors are actually trying to integrate reliability based approaches in the sate of practice. Finally, a continuation of the work presented in this paper is going on. More results related to the role of r_d parameter are forthcoming.

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