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UNCERTAINTIES IN A DETERMINISTIC AND PROBABILISTIC APPROACH ON LIQUEFACTION SUCEPTIBILITY

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SUMMARY

Evaluation of liquefaction susceptibility is affected by different sources of uncertainties. These uncertainties may be due to the applied models as well as the limited availability of data from soundings and differences in quality. In the context of this paper, the uncertainties will be discussed based on both a deterministic and a probabilistic approach, using state-of-the-art methods for evaluation of liquefaction susceptibility in combination and stand alone. Firstly, deterministic procedures using laboratory criteria and the simplified procedure are discussed based on their sensitivity to the input parameter. Second, the sensitivity of the input parameter is studied in a probabilistic analysis, while uncertainties of the soil parameters have been incorporated into a Bayesian Probabilistic Network. Third, the results of the deterministic analysis are compared with the probabilistic analysis based on available data from the city of Adapazari in Turkey, where examples of damage caused by liquefaction were numerous. Both approaches are difficult to compare due to controversial results and the different procedures used. Additionally, the deterministic methods adopted might not be consistent in their terminology for initiation of liquefaction using the terms “susceptible to liquefaction”, “liquefaction possible” and “liquefaction very likely”. Conclusions for further activities in this field will be drawn.

INTRODUCTION

The evaluation of the behaviour of the near surface soil layers is one of the key issues in earthquake risk management and microzonation. Liquefaction of these layers is caused by a combination of the loading function from the incoming earthquake and the subsoil strata (type and parameters of the different soil layers). If an earthquake occurs, liquefaction might lead to large deformations on the surface and subsequently this deformation may imply significant settlements or tilting of buildings and subsequent structural damages.

Waves are propagated through the rock and soil from the seismic centre to the structure. In terms of the behaviour of a structure, the soil behaviour in the upper 30 m, where the stresses of the structure are released (called subsoil) are of main interest. Here flat surfaces are considered, whereas inclined surfaces with potential liquefaction induced slope failures is not considered here. Due to the earthquake shaking, the expected soil response will be usually soil collapse due to liquefaction or soil compaction both leading to settlements. The settlements have a direct impact on the structure. In this contribution, the main focus is on soil failure due to soil liquefaction. (Other reasons for soil failures under earthquake loading such as exceeded bearing capacity, grain crushing, etc. will be dealt with in an upcoming stage.)

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The determination of liquefaction susceptibility is subject to different sources of uncertainties like the incoming earthquake, soil strata and soil parameters as well as uncertainties in the applied analysis models. The definition of uncertainties is one of the major challenges when dealing with the evaluation of liquefaction susceptibility. These uncertainties include criteria for the assessment of liquefaction based on the variability of sounding results as well as the applied methodical framework. The criteria for evaluation of liquefaction susceptibility considered in the present study are based on the state-of-the-art used in practical applications. They consist of a combination of the recent version of the modified Chinese criterion [Andrews and Martin 2000], the criterion of the German nuclear safety standards [KTA 1990] and the so-called "simplified procedure" [Seed and Idriss 1971]. In terms of a deterministic approach this simplified procedure is mostly applied as a stand alone approach as summarized after the NCEER workshop [Youd et al. 2001]. In addition, criteria using laboratory information were selected complementary to the traditional analysis, with known material characteristics. Ground accelerations needed for both deterministic and probabilistic considerations are estimated using a linear-equivalent approach for the soil behaviour based on a simple one dimensional modelling [e.g. Schnabel et al. 1972].

In the probabilistic approach used here, the criteria described above are incorporated in a framework for earthquake risk management. The probabilistic approach is based on the use of a Bayesian Probabilistic Network (BPN) [Pearl 1988;Faber et al. 2002]. The structural part of the ongoing ETH project in earthquake risk management has been clearly formulated [see e.g. Bayraktarli et al. 2005]. The BPN of it forms a basis for consistently integrating all aspects affecting the damage on structures located within a region subjected to the same earthquake exposure. The uncertainties influencing the functional chain of an earthquake from the source mechanism, the site effects, the soil behaviour, the structural response, the damage as well as the indirect consequences can be handled consistently. With new information at hand a consistent updating of the results can be performed. This facilitates the extension of the approach to decision problems related to risk management during and after earthquakes. A key element in the approach pursued is the quantification of the effect of various types of information (condition indicators) on the elements of the functional chain of an earthquake. These condition indicators have very different characteristics and their identification and characterization necessitates that different levels of expertise are integrated into the project.

DETERMINISTIC APPROACH

There are different criteria to assess the liquefaction potential of the soil. The most common analysis is the simplified procedure. Other methods are based on laboratory results such as the Modified Chinese Criterion (MCC) or the guidelines of the German nuclear safety standard [KTA 1990]. However, applying all criteria might not lead to a better liquefaction assessment, which could be expected since the results can be conflicting or data may be not available for all criteria (see example application). Using only one criterion might already exclude from the beginning other procedures, which would lead to a different result. In order to profit from the knowledge of the laboratory tests, the following procedure was selected.

Procedure

Firstly, the clay content and the liquid limit according to the MCC and the grain size distribution and density according to the KTA is investigated and secondly, the simplified procedure is run. For fine-grained soils MCC is best to use and for coarse soils the KTA is best to use. Still there is a wide range of soils that consist of both fine and coarse grains so that both of the laboratory criteria are also applied to these soils.

The MCC is presented in Table 1. The MCC is applied to the usual soil types such as gravel, sand, silt and clay (without organic soil or stones; in Table 2). The categorization is based on mean values taken [from VSS 1999]. Clean gravels and sands (GW, GP, SW, SP) are excluded since their clay content is below 5% and no liquid limit is determinable. The shaded soil types SM-SC, ML, CL and CL-ML cannot be categorized uniquely. In those cases, the clay content of a material is not exactly known and the liquid limit is equal or higher than 32%, the soil is more likely not to liquefy.

Table 1: Liquefaction susceptibility of silty and clayey sands [Andrews and Martin 2000]

	Liquid Limit, LL<32%	Liquid Limit, LL≥32%
Clay Content <10%	Susceptible for liquefaction	Further studies required
Clay Content ≥10%	Further studies required	Not susceptible for liquefaction

Table 2: Liquefaction susceptibility for USCS classified soils based on typical classification parameters. For shaded soil types controversial results can potentially obtained

	Liquid Limit, LL<32%	Liquid Limit, LL≥32%
Clay Content <10%	GW-GM, GP-GM GW-GC, GP-GC GM, GM-GC SW-SM, SP-SM SW-SC, SP-SC SM ,SC, SM-SC ML	
Clay Content ≥10%	GC SM-SC CL-ML CL	MH, CH ML CL-ML CL

The grain size distribution curve is plotted in the diagram of the KTA (Figure 1). If the principal part of the grain size distribution curve is outside zone 1 and 2 liquefaction should be excluded from further considerations. If the principal part of the grain size distribution curve is within zone 1 and 2 liquefaction has to be assessed by means of the stress ratio induced by the earthquake and the density of the soil. The density can be derived from the corrected standard penetration blow counts $(N_1)_{60}$ [VSS 1997].

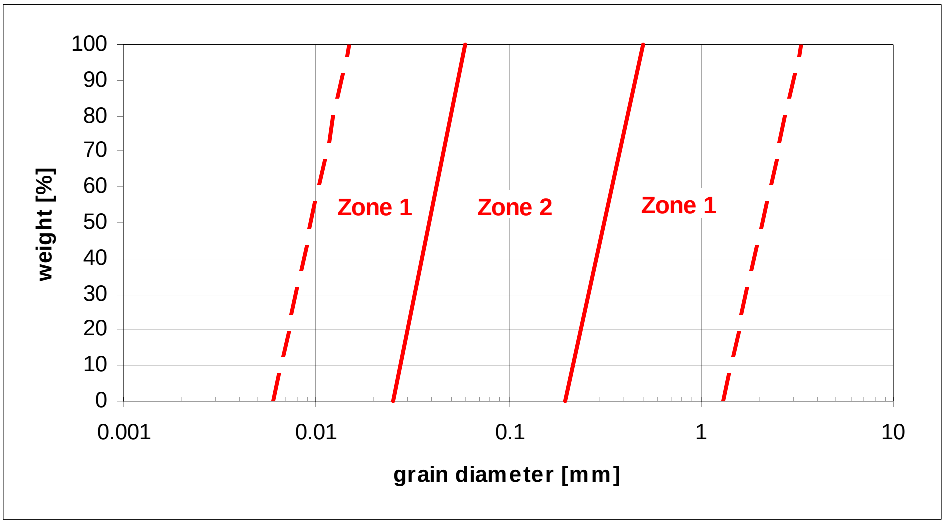

Figure 1: Liquefaction susceptible grain size areas in zone 1 and 2 [after KTA 1990]

In a second step, the liquefaction potential is assessed using the simplified procedure. In addition to the soil type the blow counts $(N_1)_{60}$ indicating the density of the soil layer, the depth of the soil layer, the ground water level and the maximum horizontal acceleration from the earthquake are needed to establish the cyclic stress ratio (CSR) and the cyclic resistance ratio (CRR). A comparison of CRR with CSR for which the soil will liquefy can be interpreted in terms of a factor of safety (F.S.). The simplified procedure, which has been established in different microzonation approaches [Ansal 2004] describes free field conditions. The influence of the stress states in the subsoil due to a building or due to topography (including slopes) is not taken into account, and hence will not be considered here. The influence of bi-directional loading [see Buchheister and Laue 2006, paper nr. 1006] is not considered in this approach and only the maximum horizontal acceleration of the earthquake is taken into account.

The CSR is given as:

$$CSR = 0.65 \cdot \frac{a_{\max}}{g} \cdot \frac{\sigma_{vo}}{\sigma'_{vo}} \cdot r_d \quad (1)$$

where a_{max} is the maximum horizontal acceleration in the soil layer, σ_{vo} and σ'_{vo} are the total and effective stresses in the middle of the soil layer respectively and r_d is a depth reduction coefficient.

The CRR is given as:

$$CRR_{7.5} = \frac{1}{34-(N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \cdot (N_1)_{60} + 45]^2} - \frac{1}{200} \quad (2)$$

where $CRR_{7.5}$ is the cyclic resistance ratio for magnitudes of approximately 7.5 and $(N_1)_{60}$ is the normalized SPT blow count. Equation (2) is valid for $(N_1)_{60}$ values less than 30. For larger values, clean granular soils are classified as non-liquefiable.

Based on the simplified procedure, the limit state function for liquefaction triggering can be defined by CSR and CRR. Based on Equations (1)-(2) the limit state function can be derived as:

$$g(x) = CRR_{7.5} \cdot MSF \cdot K_\sigma - CSR = 0 \quad (3)$$

where MSF is a magnitude scaling factor to correct for magnitudes smaller or larger than 7.5 and K_σ is a correction factor to consider overburden pressures different from 100kPa.

The procedure described will be applied for each soil layer below the ground water table without the top layer.

Use of the soil parameters

To be able to apply all three criteria, the parameters needed are: liquid limit w_L, clay content (< 0.002 mm), grain size distribution curve, soil type USCS classified, unit weight, water content, void ratio, saturated unit weight, fines content, relative density of the soil in terms of blow counts $(N_1)_{60}$, groundwater level, depth of soil layer and peak ground acceleration.

Grain size distribution, clay content, fines content, unit density, water content, void ratio and w_L can be determined in the laboratory. If a grain size distribution is made, the fines content is known. Additional to the sieving method, the sedimentation method is essential to determine the clay content, which is not often done in practice. Based on the laboratory results, the soil can be classified according to the USCS system. Since the determination of the void ratio in the laboratory is complex, it is herein taken from the USCS classification of test [VSS 1999] for deriving the saturated unit weight.

PROBABLISTIC PROCEDURE USING A BAYESIAN PROBABILISTIC NETWORK (BPN)

The basic features of BPNs may be described by following steps (Figure 2):

- Formulation of causal interrelations of events leading to the events of interest (consequences). This is graphically shown in terms of nodes (variables) connected by arrows. Variables with ingoing arrows are called children. Variables with outgoing arrows are called parents.
- Assigning to each variable a number of discrete mutually exclusive states.
- Assigning probability structures (tables) for the states of each of the variables (conditional probabilities in case that the variables are children).

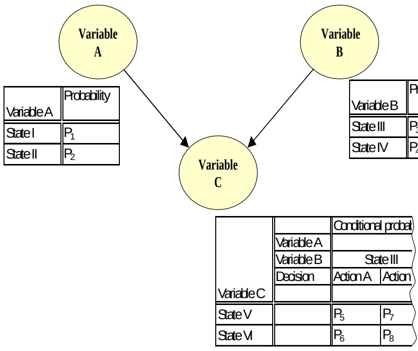

Figure 2: A principle BPN with states and probability tables

Having developed the BPN and the required probability tables, the risk assessment is straightforward. BPNs provide a tool to update information in a more or less precise form as condition indicators. The general idea will be explained in Figure 3, whereas details are provided in the subsequent example application.

Procedure

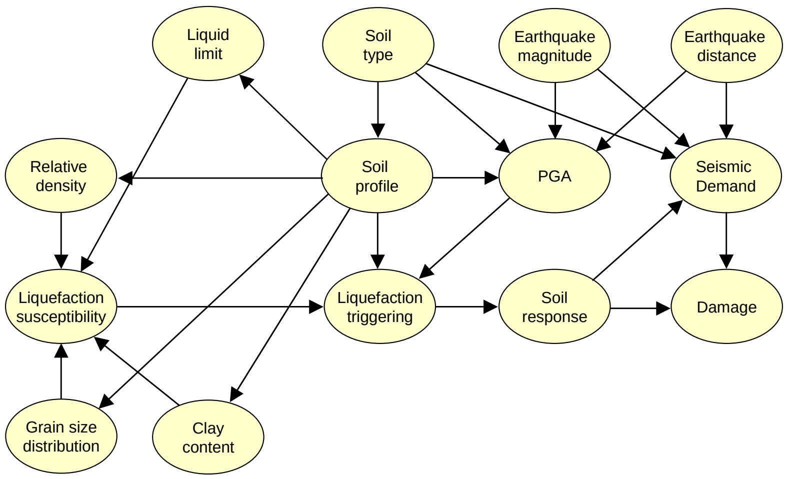

Figure 3: Bayesian Probabilistic Network for assessing liquefaction of soil

The seismic activity related to the source zone is considered as a point source. The probabilities for the occurrence of earthquakes with specific magnitudes are considered in the ‘Earthquake Magnitude’ node. The probability distribution of the distances of the structures is presented by the ‘Earthquake Distance’ node. The prevailing soil conditions at the considered location are represented by the ‘Soil type’ node. The ‘Seismic demand’ node is conditioned on these three nodes. The node ‘PGA (Peak Ground Acceleration)’ is conditioned on the ‘Earthquake Magnitude’, ‘Earthquake Distance’, ‘Soil Profile’ and ‘Soil type’ nodes. In the node ‘Liquefaction susceptibility’ the logical connection based on MCC and KTA is given. These nodes have dependencies on the nodes ‘Liquid limit’ and ‘clay content’ as well as ‘relative density’ and ‘grain size distribution’, which are conditioned on the node ‘Soil profile’. The conditional probabilities computed based on the simplified procedure (Eq. 3) form the node ‘Liquefaction triggering’. Conditional on the ‘Seismic demand’ and ‘Soil response’, the probabilities of being in a predefined damage state form the conditional probability tables in the ‘Damage’ node.

Uncertainties of the soil parameters

In deterministic approaches, characteristic values for soil parameters are chosen to cover a specific fractile in the distribution of the single parameter. Since the probabilistic models account for the uncertainties in the properties of the condition indicators a distribution of the different parameters is needed. For the present analyses, the uncertainties in the following condition indicators are considered: unit weight, water content and void ratio

(Table 3). For the layer thickness, the values given in boring logs are taken as mean values with a coefficient of variation of 15%.

Table 3: Chosen soil parameters [VSS 1999]

Soil Type*	Unit Weight [t/m^3]		Water content [%]		Void Ratio [-]	
	Mean	Stdv	Mean	Stdv	Mean	Stdv
A	2.04	0.15	5.5	2.5	0.26	0.05
SP-SM	2.03	0.18	33	9.3	0.60	0.25
ML	1.99	0.20	36	19.7	0.77	0.51
ML-CL	2.11	0.11	37	5.3	0.55	0.15
CL	2.13	0.10	44	5.4	0.55	0.14
MH-CH	1.76	0.26	44	29.3	1.33	0.63
CL-CH	2.02	0.24	37	16.6	0.80	0.43

*A: Fill, SP: poorly graded sand, SM: medium graded sand, ML: Low plasticity silt, CL: Low plasticity clay, CH: High plasticity clay

For all soil types, these random variables can be assumed to be normally distributed [Lacasse and Nadim 1996]. For fill, no data was available and for clay characterized as CL-CH one single data set was considered not meaningful. For the other soil types, parameters of the distribution are taken from the Swiss Standard SN 670 010b [VSS 1999] published by the Association of Swiss Road and Traffic Engineers, where a set of 6000 laboratory tests were analyzed and the parameters for each soil type were classified after the Unified Soil Classification System.

EXAMPLE APPLICATION

An area affected by the 17th August, 1999 Kocaeli M_w7.4 earthquake is chosen. Many buildings in the city center of Adapazari/Turkey suffered damage due to liquefaction induced ground settlement during that earthquake. Field investigations in Adapazari were performed by different groups [Kiku et al. 2001;Tsukamoto et al. 2001;PEER 2000]. Figure 4 illustrates the liquefied area within the investigated area in the city centre with emphasis on borehole SPT A-2 [Bray et al. 2001] representing a surely liquefied site. Unfortunately, it is not known which layer liquefied at the site.

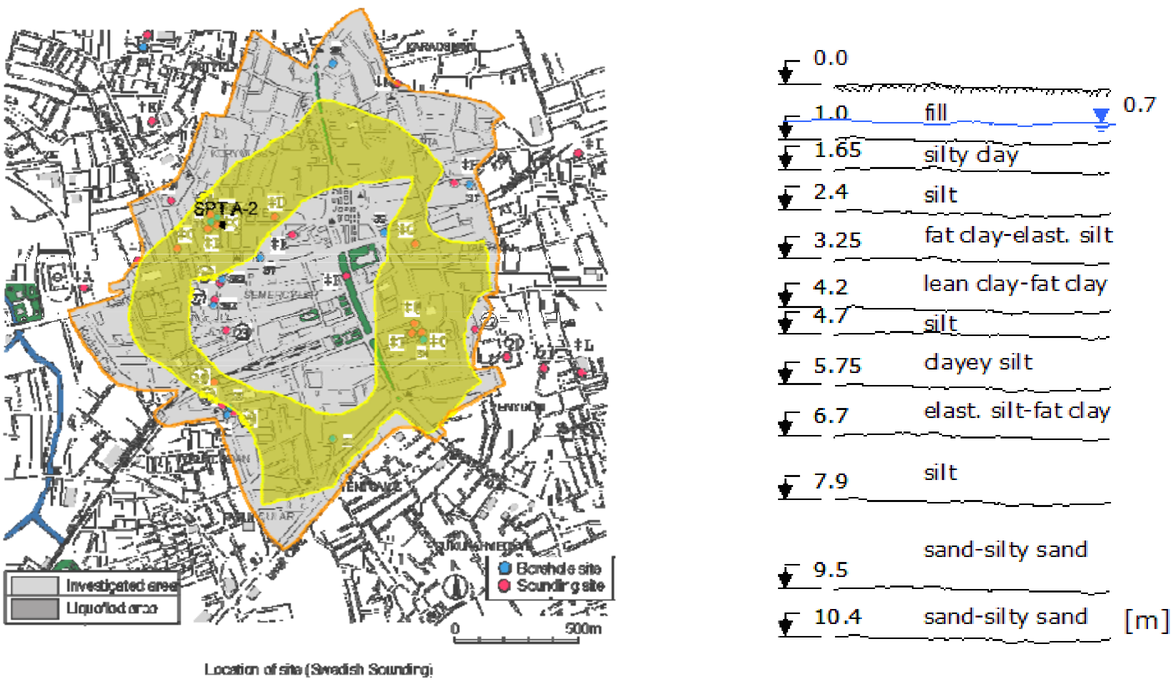

Figure 4: Boring profile SPT A-2 [PEER 2000] in the liquefied area [investigated area after Yasuda in Ansal 2004]

Deterministic Analysis

As first step, the criterion of MCC and KTA was applied. As second step the simplified procedure was performed. Maximum accelerations (between 1.37 m/s^2 and 2.15 m/s^2) were assigned for each layer using the one dimensional site response analysis of EERA [Bardet et al. 2000]. Soil will most likely liquefy, when the F.S. is below 1.25 (according to Eurocode 8-5 [DIN 1997]) pointing out that the effective stresses must not be zero to start the liquefaction process. The results of all three deterministic methods are summarized in Table 4:

layer	USCS soil type	MCC	criterion of KTA	F.S. of simplified procedure
1	Fill	-	-	-
2	ML/CL	x	-	1.09 liquefaction
3	ML	x	-	1.13 liquefaction
4	CH/MH	not susceptible	x	
5	CL/CH	x	-	0.71 liquefaction
6	ML	not susceptible	liquefaction possible	1.28 no liquefaction
7	CL	x	-	
8	MH/CH	not susceptible	no liquefaction to be expected	0.67 liquefaction
9	ML	not susceptible	liquefaction possible	0.85 liquefaction
10	SP-SM	-	liquefaction possible	
11	SP-SM	-	liquefaction possible	1.17 liquefaction

Table 4: Summary of the results of the deterministic approach for the SPT A-2,
“-“ = no sample, “x” = insufficient data

The top layer (fill) is not of interest for liquefaction. Layers 10 and 11 are defined as sand and have no liquid limit. Unfortunately w_L is not known for all fine grained layers. Grain size distribution curves were available for all coarse grained soils. The blow counts respectively the relative density are not known for all soil layers, which has an influence on the use of the criterion of KTA as well as the simplified procedure.

From Table 4, it can be seen that the MCC criterion indicates always no liquefaction (for the layers where MCC is applicable) and the simplified procedure indicates mostly liquefaction. The criterion of KTA has a strong influence on the decision of the liquefaction susceptibility in the deterministic analysis. The three deterministic methods can be compared using layers 6, 8 and 9: Even though the MCC indicates no liquefaction for the silt layer 9 (ML) the other criteria indicate liquefaction. Also layer 6 is a silt layer (ML), but two out of three criteria indicate no liquefaction. This points out the difficulty in identifying liquefiable silt layers. Layer 8 (high plastic silt and clay) is assumed not to liquefy, which is based on two out of three criteria. This comparison shows the difficulties to judge liquefaction susceptibility based on the selected criteria and explores the risk of relying on a standardized chain of approaches.

Probabilistic Analysis

The probabilistic analysis has been conducted in order to prepare the probability tables for the node liquefaction triggering (Figure 3).

The annual probabilities for each magnitude were calculated using the Gutenberg-Richter magnitude recurrence relationship with recurrence rate parameters corresponding to Anatolian Trough source zone [Erdik et al. 1985]. The occurrence of strong earthquakes is assumed to follow a stationary Poisson process. These probabilities form the probability tables for the ‘Earthquake Magnitude’ node and are the preconditions for Table 5. The sites are assumed to be uniformly distributed in the region with epicenter distances of R=10 km, 20 km, 40 km and 80 km. The soil types rock, gravel, sand, silt and clay are assumed to be uniformly distributed in the considered locations.

A set of acceleration time histories is generated for the soil response analyses. For this purpose an attenuation relationship [proposed by Boore et al. 1997] is used for estimating the pseudo-acceleration response spectra and the random horizontal component at 5% damping. 16 pairs of 4 moment magnitudes and 4 epicentre distances on the site rock were selected. Using a modified version of SIMQKE [Gasparini and Vanmarcke 1976; from Lestuzzi 2000] 10 samples of accelerogram time histories for each pair were generated, resulting in 160 simulations. These 160 acceleration time histories were applied at the bedrock level and propagated vertically through the soil layers in a one-dimensional analysis program Shake [Schnabel et al. 1972;Iyisan 1996]. The maximum acceleration a_{max} in each layer is then used in the further analysis. The limit state function given in Eq. 3 is evaluated to obtain the probability of liquefaction by a Monte Carlo simulation (N=1’000’000) using the uncorrelated normal distributed random variables layer thickness, unit weight, water content and void ratio.

The node ‘Soil profile’ constitutes the different layers of the considered boring profile. The modified Chinese criterion and the criterion according to KTA is implemented as a logical connection in the node ‘Liquefaction susceptibility’. The node ‘Liquefaction triggering’ comprises the conditional probabilities for liquefaction from Table 5, whereby the states of the node ‘PGA’ are taken from the mean of the ten simulations for each pair of (M_w, R).

Table 5: Probabilities of liquefaction for each layer and for each simulated ground motion, R =epicentre distance, no SPT data was available = n.a., n.liq.= not liquefiable layer

M	R	probability of liquefaction for layer i [%]										
		1	2	3	4	5	6	7	8	9	10	11
5.5	10	n.a.	0	0.005	n.a.	0.036	0.02	n.a.	0.039	0.03	n.liq.	0.035
	20	n.a.	0	0.001	n.a.	0.012	0.015	n.a.	0.017	0.022	n.liq.	0.028
	40	n.a.	0	0.003	n.a.	0.005	0.005	n.a.	0.008	0.013	n.liq.	0.035
	80	n.a.	0	0.001	n.a.	0.003	0.015	n.a.	0.01	0.018	n.liq.	0.025
6.5	10	n.a.	0	0.024	n.a.	15.01	0.041	n.a.	99.93	0.076	n.liq.	0.109
	20	n.a.	0	0.017	n.a.	0.157	0.031	n.a.	0.087	0.048	n.liq.	0.047
	40	n.a.	0	0.010	n.a.	0.039	0.022	n.a.	0.034	0.03	n.liq.	0.035
	80	n.a.	0	0.003	n.a.	0.022	0.024	n.a.	0.016	0.03	n.liq.	0.037
7.0	10	n.a.	11.19	2.417	n.a.	99.93	0.084	n.a.	99.95	0.113	n.liq.	9.009
	20	n.a.	0.210	0.078	n.a.	98.60	0.056	n.a.	99.88	0.093	n.liq.	0.077
	40	n.a.	0	0.017	n.a.	0.104	0.039	n.a.	0.123	0.035	n.liq.	0.043
	80	n.a.	0	0.011	n.a.	0.035	0.023	n.a.	0.031	0.028	n.liq.	0.028
7.5	10	n.a.	99.94	99.98	n.a.	99.95	99.93	n.a.	99.94	99.93	n.liq.	99.21
	20	n.a.	7.930	3.905	n.a.	99.93	0.095	n.a.	99.94	0.108	n.liq.	0.095
	40	n.a.	0	0.025	n.a.	5.35	0.046	n.a.	60.31	0.046	n.liq.	0.047
	80	n.a.	0	0.006	n.a.	0.083	0.019	n.a.	0.039	0.035	n.liq.	0.031

As result of Table 5 the probability of liquefaction depending on the soil type and the influencing earthquake can be assessed. It is obvious, that the probability of liquefaction is high only in some layers. The underlined row is the comparable row to the deterministic approach.

Comparison

The outcome of the probabilistic results is difficult to compare with the results from the deterministic procedure. For the deterministic procedure the same maximum accelerations were used as in the probabilistic procedure for an earthquake with a magnitude of M=7.5 and a distance of R=20 km (shaded pink in Table 5), which seems reasonable for the city of Adapazari. The probability of liquefaction for layers 2, 3 and 11 is low respectively below 8%. However, the simplified procedure indicates soils with a F.S. between 1.1 and 1.2 presumably liquefy. For layers 5 and 8 the simplified procedure and the probabilistic procedure show comparable results, this yields a liquefiable clayey soil whereas the two other deterministic material criteria (MCC and KTA) show the opposite result. No liquefaction is predicted based on the available data. In this case, partial agreement between the deterministic and probabilistic methods is reached. Layer 6 shows a partially comparable result, too, this is a silty soil not susceptible to liquefaction including in this case contradicting results based on the deterministic material criteria.

The example study is based on an artificial acceleration time histories for different magnitudes and epicentre distances. The probabilistic results for selected magnitudes and distances are shown in Table 5. It is noticeable that most values are close to zero or to one but rare in between these values. The deterministic procedures were executed for most of the selected magnitudes and epicentre distances. Especially for the minimum and maximum acceleration of the generated time history the result shows the same tendency as that represented by the probabilities and described in more detail in the example above.

CONCLUSIONS

Deterministic assessment of liquefaction susceptibility using three methods has been compared with a probabilistic procedure using BPN applied for one borehole. This includes the deterministic application of the modified Chinese criterion (MCC), the criterion proposed by KTA and the analysis based on the simplified procedure. Probabilistically, the uncertainties of important soil parameters, e.g. unit weight, water content, void ratio and layer thickness were taken into account. Probabilities of liquefaction triggering were calculated using a Monte Carlo simulation.

The application shows the different uncertainties, which have to be covered in such an analysis. The approaches adopted based on the state-of-the-art are mostly empirically developed and thus lack a direct connection to the mechanical description of the soil behaviour. As shown previously by other authors [e.g. by Iyisan 1996], those methods have to be reworked for special local conditions, to be able to cover local (and thus spatial) variability.

Purely deterministic analysis shows the influence of the sequence of applications of different methods on the results. Exclusive use of one of the criteria, might lead to strict borderlines influencing potential decisions. A soil could be excluded by one criterion and therefore not been regarded by the other. In a probabilistic approach it is possible to describe this sharp borderline, which in terms of soil parameter is most meaningful as the standard deviation of the different influencing soil parameters. This requires a more detailed study of the different variability in the description of the soil parameters (as the state to be found in a borehole) as well as spatial distribution of parameters, which both have to be formulated in respect of the physically possible boundaries. In addition also methodological uncertainties need to be assessed and probably to be included into the probabilistic analyses. Some of these uncertainties might be reduced by replacing empirical based formulations with more mechanical based condition indicator. So that the uncertainties will be reduced to the distribution of the soil characteristics in addition to the fact that using known criteria - important condition indicators like number of equivalent cycles of pulse periods and/or permeability of soil - are rarely taken into account in such an analysis.

At present state the indicators used for the BPN are based on epistemic uncertainty [definition after Jones et al. 2002]. In future these indicators should also account for aleatory uncertainty. For this task, a larger amount of high quality data of a region should be available. Even though plenty of quality data is available for the city centre of Adapazari, the amount of data seems to be small for a probabilistic analysis as soon as the spatial variability of different soil types is taken into account.

In the further work, effort will be focused on the liquefaction susceptibility of silty soils and bi-directional earthquake loading to establish condition indicators based on physical mechanisms. Furthermore, spatial variability of the soil parameters and modelling uncertainties of the procedures will be considered. Considering spatial variability will give a better estimate for the geometric spread of the liquefaction event. Implementing modelling uncertainties might overcome the above stated controversy in the results. Next, the sensitivity of the different indicators will be taken into account.

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