Assessing probability of liquefaction for Tangier soils by using Juang method and physically based reliability analysis modelling

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ABSTRACT

Approaches that are widely used to characterize propensity of soil liquefaction are mainly of empirical type. The potential of liquefaction is assessed by using procedures that are based on field tests. The standard and the cone penetration tests are widely used. They enable predicting liquefaction potential by means of correlation formulas. These correlations depend however on the site where they were derived. In order to adapt them to other sites such as soils existing in the northern Moroccan city of Tangier where seismic case histories are not available, further investigation is required. In this work, focus is done on Juang method which enables estimating the liquefaction potential without requiring excessive experiments. Predictions of Juang method are then compared to those obtained with a rigorous rational one-dimensional modelling of liquefaction phenomenon. Field tests consisting of core sampling and cone penetration testing were performed. They provided the necessary data for numerical simulations conducted by means of the physically based model which was developed under DeepSoil software package. Using reliability analysis, the probability of liquefaction was estimated and the obtained results were used to adapt Juang method to the particular case of sandy soils located in Tangier.

Keywords - Correlation, liquefaction, probabilities, seism, soil

I. INTRODUCTION

Loose saturated granular soils could experience great deformations during the occurrence of severe earthquakes. These could have destructive effects and may cause huge damage to buildings and infrastructures, as they are in general incompatible with the stability requirements. At the soil microstructural level, liquefaction is associated to the onset of excessive local pore water pressures. Loss of resistance and rigidity emerge, in case of insufficiently compacted and undrained soils, as the applied shear stresses are transferred spontaneously from total stresses to pore water pressures [1]. Dangerous reduction of the effective stresses occurs then. For sandy soils having a loose granular curve, pore pressure could jump to sufficiently high values that could compensate the total confining pressure, yielding the soil to undergo great strains as its behavior becomes of fluidic type.

In order to characterize vulnerability of soils to liquefaction phenomenon a lot of empirical methods have been developed. They all try to give estimation of liquefaction potential for a given soil. Most of these methods have been introduced by Seed and its fellows at Berkley during the seventies [2]. Empirical methods are based upon comparing results gathered form sites where earthquakes have occurred and examining among them those for which liquefaction took place and those where this event has not been observed. In these approaches, liquefaction potential is determined by correlating it to some given propriety of the considered soil which measures its capacity of resistance to liquefaction.

In Seed and Idriss [2], an empirical procedure for characterizing the propensity of loose sandy saturated soils to liquefaction was proposed. They have stated the conditions for the considered granular materials to liquefy by measuring the standard penetration test

(SPT) resistance and by evaluating, for a given seism, shear forces that were induced in the soil mass. The method was originally developed with SPT testing and was later modified to use the cone penetration test (CPT). This last enables to obtain the cone tip resistance and the sleeve friction. Seed and Idriss method consists in:

- Performing the SPT test in order to measure the penetration resistance, then correlating this result with the cone penetration test. Schmertmann correlations [3] or those given by Seed et al. [4] could be used.
- Estimating the initial stress state in the soil in terms of the shear stress caused by the seism. A classical formula is used for that. It gives the shear stress as a fraction of the seismic acceleration divided by the gravity acceleration and multiplied by the soil total vertical stress and then by a correction factor which is function of the soil depth.
- Defining a liquefaction limit state either empirically from historical case studies or by means of cyclic triaxial tests conducted on soil samples. Seed and Idriss [5] have given in this way a limit state curve that enables comparing the seismic demand represented by the ratio of shear stress over the soil vertical effective stress and the liquefaction resistance capacity defined as the modified SPT resistance.

Holzer et al. [6] have studied sandy soils located in a number of sites of the Californian imperial valley and have given their limit states of liquefaction. They have used a diagram having as axes the maximum registered soil surface acceleration and the measured shear wave velocity to represent liquefaction limit states.

Later Robertson [7,8] has proposed to compare the seismic demand to soil capacity expressed in terms of the shear wave propagation velocity. He has proposed liquefaction limit states under two forms: a curve in the diagram having as axes the cyclic shear stress and the shear wave velocity, and secondly a curve in the diagram compounded from the modified SPT resistance and the soil vertical effective stress.

It should be noted that the empirical methods discussed previously belong to what it is called the stress approach. Among these approaches, Juang method [9] which is considered in the next section is very interesting because it needs rather limited experimental resources. Other approaches were also proposed such as that of Dobry [10] who has introduced the strain approach and that of Park et al. [11] who have introduced the energetic approach. Some other empirical methods are reported in the literature [12].

Liquefaction of soils could also be stated by means of rational approaches based on analytic methods where mechanical models of the soil behavior are used. In this field three mean methods have been considered, they are recalled in the following:

- The first family of analytical methods is named the decoupled formulation. It uses the total stresses and consists in assuming a linear equivalent method for shear wave propagation [13,14,15]. The fundamental assumption in this approach relays on the fact that the non linear response could be always approximated satisfactorily if appropriate linear elastic and damping parameters are identified from the real soil behavior. A pair of curves representing the degradation of shear modulus with deformation and variation of damping with strain constitutes the input data needed in the context of the decoupled formulation. The soil is assumed to be governed by the one-dimensional Kelvin-Voigth equation that combines propagation and attenuation of shear waves. Values of shear modulus and damping are actualized at each iteration step to make them compatible with the reached level of strains.

Because the linear equivalent model is not directly applicable to saturated soils for which pore water pressures could develop, a modified version of the decoupled linear equivalent method integrating generation and dissipation of pore pressure was proposed [5]. This method consists in coupling the soil model consisting of the Kelvin-Voigth equation with cyclic undrained triaxial experimental results performed on soil samples extracted from the considered site. The procedure relays on determining time histories of shear stresses, under undrained conditions, and comparing them to cyclic shear stresses causing liquefaction of soil samples during laboratory triaxial testing. This modelling is however not fully satisfactory because strains could be erroneously estimated due to the various approximations that are stated in the framework of the linear equivalent model approach.

- The second approach is termed partially coupled formulation. It uses also total stresses and is based on considering soil dynamical equations of equilibrium with a nonlinear soil constitutive law under the presence of viscous damping. Empirical law describing generation of pore pressures as function of total stresses and strains under undrained conditions is also proposed along with a diffusive mechanism describing dissipation of pore water pressures. Several finite element codes using this approach have been developed [16,17]. This approach is different from the uncoupled formulation in that a nonlinear constitutive soil law is used with variable bulk and shear modules that are actualized at each step. Pore water pressure generation is also well described by means of an empirical law using up to seven parameters that are identified from laboratory cyclic shear and consolidation tests. At each step, the pore pressure is used to compute the effective stress which is used to actualize the soil elastic constants. Though this approach is more complete than the previous one, it suffers some limitations such as the analysis is purely elastic since it does not include hysteric damping resulting from plastic behavior during the loading and unloading cycles. Besides, as coupling is formulated at the global level, the model could not predict satisfactorily pore water pressure dissipation and hence soil displacement history.

- The third family of approaches consists of coupled formulations in effective stresses. These formulations are based on Biot assumption [18]. The first model of this kind was proposed by Ghabousi and Wilson [19]. Later, Zienkiewicz and Shiomi [20] have proposed a modified version of the fully coupled formulation. An extensive comparative study of all the proposed variants of the coupled formulations is presented in Smith [21]. This review focuses on aspects such as solution strategy, the used assumptions regarding mass and damping matrices, the variety of proposed elements and the constitutive equations introduced to model soil behavior.

A comparative study between coupled and partially coupled formulations has been conducted by Arulanandan et al. [22]. Despite the fact that the calculated acceleration spectra as predicted by the two approaches was not exactly the same, the conclusion was that the partially coupled approach is sufficient to analyze liquefaction by means of the simplified procedure as the obtained soil accelerations are conservative. This motivated use of the partially coupled approach in this study in order to assess analytically liquefaction potential of soils. It should be noticed also that this approach needs only a limited number of experimental results in comparison with the complete coupled approach where a huge experimental work is necessary to identify the soil behavior parameters; otherwise its accuracy would be degraded and its use would not be effective.

As seen before, liquefaction resistance is usually evaluated by using procedures that need in situ tests. The standard penetration test (SPT) and the cone penetration test (CPT) are the mean known standard tests that are used. These tests enable estimating the liquefaction potential by means of correlation formulas. However, these correlations depend on the site where they were derived. Their use for other sites is questionable. In this work, we study how these correlations can be adapted to predict liquefaction in the particular case of sandy soils that are located in the northern region of Morocco, near from the city of Tangier.

As Juang method [9] had proved to be well suited because it uses less experimental information than the others methods and also because it is less sensitive regarding stochastic variations affecting soil parameters, it will be used in the following. The objective is to investigate to what extent the empirical Juang method can predict correctly liquefaction for Tangier soils by comparing its predictions with the results of a rational physical one-dimensional modelling of the problem. This modelling is based on the partially coupled formulation which is provided by the open source software package DeepSoil. This will be assessed in terms of probability of liquefaction as obtained by reliability analysis conducted on DeepSoil results. It will provide a way to adapt the empirical method of Juang for predicting liquefaction of Tangier soils. A case study is examined in this assessment. It consists of the site where the complex Tangier City Centre was built. This site shows a high liquefaction risk due to the particular composition of its foundation soil.

II. Evaluating liquefaction potential by means of Juang method

The mean factors controlling liquefaction of cohesion less saturated soils are the duration and the intensity of the earthquake motion, as well as soil density and the confining effective pressure. In order to characterise soil response under the action of cyclic seismic acceleration, a lot of methodologies were developed [2,10,11,12]. Methods that are based on cyclic stresses and strains were developed from laboratory tests. Due to the fact that the cyclic response of a soil is controlled by factors such as the nature of soil, existing pre-strains, the loading history and some others altering effects that could not be reproduced exactly during laboratory tests, empirical relationships that are based on in-situ measurements seem to be more effective. These are obtained from the well known standard tests such as the CPT which characterises the quasi-static resistance of a soil to penetration action and the SPT test which gives the dynamic soil resistance. Some precautions should be considered while using these tests as the rod can be subjected to bucking problem for depths exceeding $30\,m$, the domain of validity of these tests is then limited to depths that do not exceed this limit. In addition, these tests do not apply properly for soils containing grains having diameters greater than $2\,cm$.

In order to represent in a simple manner soil motion resulting from an earthquake, by using only a single parameter, an effective procedure was developed by Seed and Harder [23]. The liquefaction potential is evaluated in this context by comparing a normalized index which is related to the cyclic soil resistance capacity R_{CR} to the ratio of the cyclic stress demand R_{CS} being applied to the soil.

At a given site, the R_{CS} is essentially a function of peak ground surface acceleration a_{max} and moment magnitude M_w. The R_{CR} is determined from a limit state curve that is obtained by calibration of the available case histories consisting of R_{CS} and in situ test data such as normalized SPT blow count $N_{1,60}$ or CPT resistance q_{c1N}. The limit state curve can then be given as an empirical equation where the R_{CR} is a function of $N_{1,60}$ or q_{c1N}. This enables evaluating the security factor F_S as

$$F_S = \frac{R_{CR}}{R_{CS}} \qquad (1)$$

The demand ratio R_{CS} is defined as

$$R_{CS} = \frac{\tau_{ave}}{\sigma'_{vo}} = 0.65\frac{a_{max}}{g}\frac{\sigma_{vo}}{\sigma'_{vo}} r_d \qquad (2)$$

where τ_{av} is the average shear stress resulting form the earthquake at the given depth, a_{max} is the maximum acceleration at the soil surface, g the acceleration of gravity, σ_{vo} the total vertical stress at the considered depth, σ'_{vo} the effective vertical stress at the considered depth and r_d a reduction stress factor. Factor r_d is given as function of the depth *z*. Seed and Harder [23] had given an explicit formula which enables evaluating the mean value of this factor as function of *z* which is expressed in *m*.

The earthquake magnitude influences the seism duration and may increase significantly the number of stress cycles. The amplitude effect of an earthquake is not included in (2). In order to take this effect into account, a scaling factor denoted *MSF* (*Magnitude Scaling Factor*) has been introduced. The reference amplitude for a stress based approach was fixed at degree 7.5 according to Richter scale. Various formulas were presented in the literature to give the MSF coefficient. When this factor is calculated as function of the seism magnitude M_w which is retained in the analysis of liquefaction risk, the normalisation of the R_{CS} ratio is performed according to the following equation

$$R_{CSM\,7.5} = \frac{R_{CS}}{MSF} = \frac{\tau_{ave}}{\sigma'_{vo}\ MSF} = \frac{\tau_{M\,7.5}}{\sigma'_{vo}} \qquad (3)$$

Evaluation of the cyclic resistance ratio R_{CR} depends on the performed test. Various methods were proposed to estimate the capacity coefficient R_{CR}. In case of Juang method, R_{CR} is evaluated by using the following formula [6]

$$R_{CR} = \left(-0.016\left(\frac{\sigma'_v}{100}\right)^3 + 0.178\left(\frac{\sigma'_v}{100}\right)^2 - 0.063\left(\frac{\sigma'_v}{100}\right) + 0.903\right)$$
$$\exp\left(-2.957 + 1.264\left(\frac{q_{c1N,CS}}{100}\right)^{1.25}\right) \qquad (4)$$

with

$$q_{c1N,CS} = \frac{q_c}{Pa}\left(\frac{Pa}{\sigma'_v}\right)^{0.5}$$
$$\left(2.429 I_C^4 - 16.943 I_c^3 + 44.551 I_c^2 - 51.497 I_c + 22.802\right) \qquad (5)$$

where q_c is the tip cone resistance, I_C is the index of soil behaviour which is computed according to the method described in Mitchell and Tseng [12] and Pa is a constant reference pressure ($Pa = 1\,atm$).

III. Numerical modelling of liquefaction by means of DeepSoil

DeepSoil [24] is a one-dimensional site response analysis program that can perform both equivalent linear and nonlinear analyses in the framework of partially coupled formulation approach to soil liquefaction problem. To create a new analysis under DeepSoil, the user should indicate the number of layers to be used in modelling the soil profile. Then he indicates the analysis method (frequency or time-dependent) and the type of inputs for shear properties. In the next step he specifies the variables to be used in the analysis: total stresses, effectives stresses with pore water pressure generation only or with also its dissipation. He enters then the method to define the soil constitutive curve. At the final stage, the user enters the boundary conditions (fully permeable or impermeable).

For the case study considered here, soil behaviour is assumed to be described according to a hyperbolic dependent pressure curve. It enables considering the reduction of soil shear modulus under the following form

$$\tau_N = \frac{\bar{G}_N \gamma}{1 + \beta\left(\frac{\bar{G}_N}{\bar{\tau}_N}|\gamma|\right)^s} \qquad (6)$$

where $\bar{G}_N$ is the initial shear modulus, $\bar{\tau}_N$ the normalised shear strength and γ the shear strain. Parameters β and s are material constants. Estimations of material parameters for sandy soils are given by $\beta = 0.8$ and $s = 0.7$.

As the soil is sandy, pore water pressure is assumed to be given by the law of Dorby et al. [10]. This law which is implemented in DeepSoil, predicts the inter-pore pressure u_N to be given as

$$u_N = \frac{2\,p\,N_c\ f\,F\left(\gamma_{ct} - \gamma_{tup}\right)^s}{1 + 2\,N_c\ f\,F\left(\gamma_{ct} - \gamma_{tup}\right)^s} \qquad (7)$$

where N_c is the number of cycles, γ_{tup} the shear limit strain and γ_{ct} the last known shear undergone before sign changing. Coefficient γ_{tup} is comprised between 0.01% and 0.02% for most of sands. Parameters f, p and F enable adjusting the model to experimental results. Triaxial non drained cyclic tests are necessary for that. Estimations of these parameters for sandy soils are given by $f = 1$, $p = 1.1$ and $F = 2.6$.

Because of the hydro-mechanical coupling taking place in the porous medium, the generated excess pore water pressure modifies the effective shear stress-strain behavior curve. This last is assumed to be given by the Matasovic model [25] as follows

$$\tau_N = \frac{\sqrt{1-u_N}\,\bar{G}_N \gamma}{1+\beta\left(\dfrac{\sqrt{1-u_N}\,\bar{G}_N}{\left(1-u_N^{\nu}\right)\bar{\tau}_N}|\gamma|\right)^s} \qquad (8)$$

where ν is a material constant. Estimation of this material parameter for sandy soils is given by $\nu = 3.8$.

According to DeepSoil modelling, the soil is assumed to be a vertical column that is formed by a given number of layers. Parameters for each layer are identified from laboratory and in situ tests. When the boundary conditions are specified and the water table level is entered, the seismic acceleration which reproduces a typical seismic motion is imposed at the soil substratum.

IV. RELIABILITY ASSESSMENT OF LIQUEFACTION BY USING THE PARTIALLY COUPLED APPROACH UNDER DEEPSOIL

Analysis of the liquefaction potential for a given depth is directly performed on the obtained results by examining the ratio of inter-pore pressure over the effective soil stress. If this ratio is close to the unity then the liquefaction risk is high. To give a further rational description of liquefaction in terms of probabilities, reliability analysis approach is performed in the following.

Let recall first that in the classical deterministic approach, a soil will liquefy if the security factor satisfies $Fs \le 1$. It will not liquefy otherwise. However, in reality a soil could liquefy even if $Fs > 1$. The big question that every body can make is how to evaluate the risk of liquefaction. Building Seismic Safety Council (BSSC) considers that the limit state to be taken is given by $Fs = 0.83$ for ordinary buildings and $Fs = 0.75$ for high security buildings. Chen and Juang [26] have proposed in to evaluate liquefaction propensity in terms of probabilities. They have introduced the probability of liquefaction P_L defined as function of F_s under the following form

$$P_L = \frac{1}{1+\left(\dfrac{F_s}{A}\right)^B} \qquad (9)$$

Juang et al. [9] have calculated the coefficients for the limit state $g(F_s) = 0.83 - F_s$ and have obtained $A = 0.96$ and $B = 4.5$.

In the present study, reliability analysis approach is applied to assess the probability of liquefaction on a rational basis [27,28]. An interpolation formula having the form of equation (9) is then derived to get directly the probability associated to the limit state defined as pore water pressure over effective vertical stress equal to 83%.

A campaign of tests was conducted in the site of Tangier city centre. 16 core sampling tests and 18 CPT tests were performed, [29]. Among the CPT results, test number 14 was the most severe with regards to liquefaction. Figure 1 gives the CPT resistance profile q_c (MPa) associated to this test.

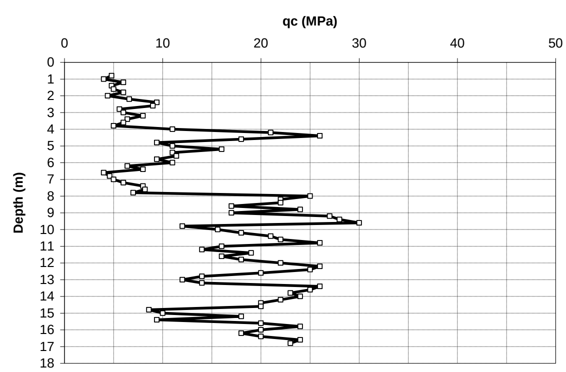

Figure 1: CPT resistance profile for test # 14

To perform reliability analysis a design of experiment (DOE) full factorial table was used to derive surface response based models giving pore water pressure as function of four factors. These last include the dimensionless seismic surface acceleration $A = a_{max} / g$, the seismic fundamental frequency F, the water table level H and the shear modulus G. Three levels were selected for each factor. They were defined by choosing a mean value and by using the coefficient 0.8 and 1.2 to obtain the low and high levels. Table 1 gives the levels chosen for each factor at depth $z = 7m$.

As the shear modulus is variable, the same proportionality coefficient was applied for the whole profile. The level of seismic acceleration was derived by using Trifunac [30] equation which gives the surface seismic acceleration as correlated to the seismic magnitude M_w and the distance from the seism epicentre D under the following form

$$a_{max} = 13 \frac{\exp(0.67 M_w)}{(25 + D)^{1.6}} \qquad (10)$$

where a_{max} is the surface seismic acceleration in $m.s^{-2}$ and D the distance in km. In case of Tangier the distance to the nearest geological fault is $D = 13 km$, varying the magnitude M_w according to the following values $7.15, 7.48, 7.75$ the accelerations shown in the first line of table 1 are obtained. The mean frequency was computed from that of Kobe earthquake that is shown in figure 2. Figures 3 and 4 give the shear modulus and shear velocity as function of the depth.

	Low	Medium	High
$A\,(m.s^{-2})$	4.63	5.79	6.94
$F\ (Hz)$	1.343	1.679	2.0145
$H\ (m)$	2.74	4.24	5.74
$G\ (MPa)$	311.8	389.7	478.4

Table 1: Levels of the factors used to derive response surface models

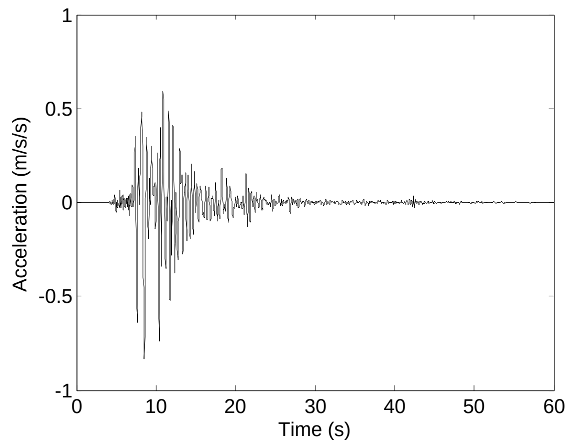

Figure 2: Kobe earthquake accelerogram

The profile of shear velocities is introduced as soil behaviour data in DeepSoil. Damping was fixed to the value 0.5%. The following parameters values which correspond to those of sandy soils were entered: $\beta = 0.8$, $s = 0.7$, $f = 1$, $p = 1.1$, $F = 2.6$, $\gamma_{tup} = 0.015\%$ and $\nu = 3.8$.

DeepSoil performs the analysis and returns the results in terms of the acceleration, strain, shear stress over effective vertical stress versus time of strain, pore water pressure over effective vertical stress versus time, response spectra,...

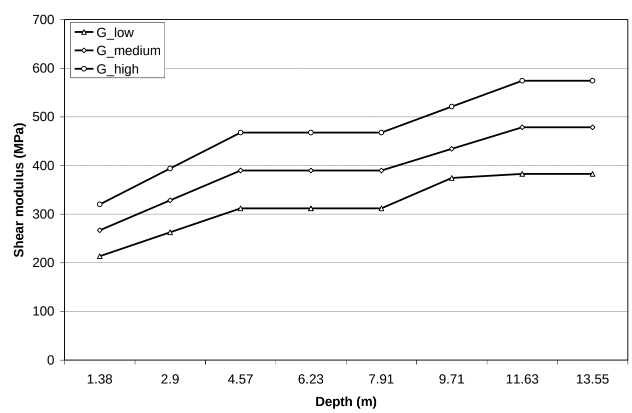

Figure 3: Shear modulus profile for CTP number 14

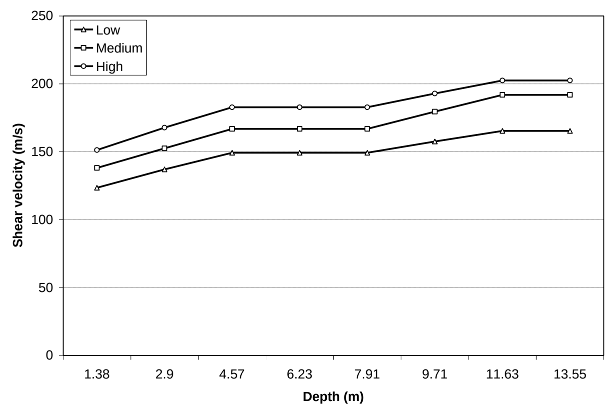

Figure 4: Shear velocity profile for CPT number 14

A total number of $4^3 = 81$ calculations were performed. The results giving pore water pressure over effective vertical stress (PWP) as function of time for depths $z = 5m$, $z = 7m$ and $z = 9m$ were obtained.

Figure 5 gives, for the combination corresponding to medium levels of factors, the reduced PWP as function of time for the three depths $z = 5m$, $z = 7m$ and $z = 9m$.

Figure 6 gives, for the combination corresponding to high levels of factors, the reduced PWP as function of time for the three depths $z = 5m$, $z = 7m$ and $z = 9m$.

Using interpolation of the obtained results response surface models giving the reduced PWP can be derived. A total number of 9 RSM models were identified. All the obtained correlation coefficient R^2 were greater that 80% showing that the RSM models are quite good for representing the maximum PWP.

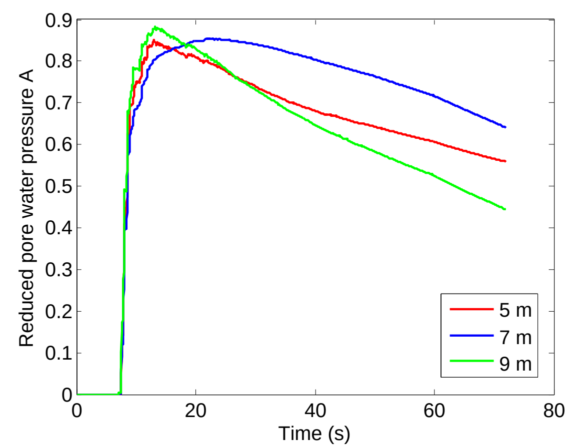

Figure 5: Reduced pore water pressure for G_{Medium}, A_{Medium}, F_{Medium} and H_{Medium}

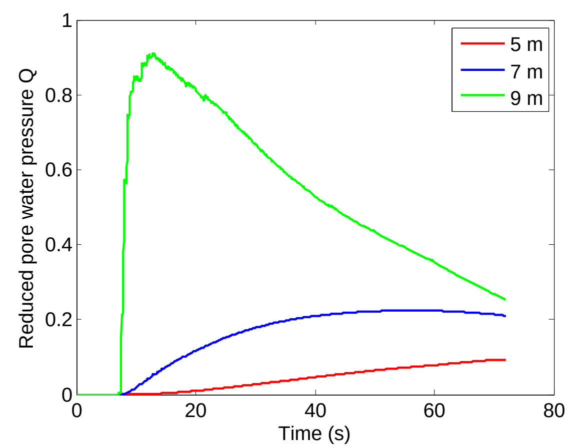

Figure 6: Reduced pore water pressure for G_{High}, A_{High}, F_{High} and H_{High}

For a given value of the shear modulus G, parameters A, F and H are assumed to be random and distributed normally. It is assumed that their means are the values defining each combination and that their standard deviations are chosen according to table 2.

	Standard deviation
A	10%
F	10%
H	17.5%

Table 2: Standard deviations of the random factors A, F and H

The limit state is assumed to be given in the following form

$$g(A,F,H) = 0.83 - Q(A,F,H) \qquad (11)$$

G	$z\,(m)$	P_f	P_L
G_{Low}	5	0.6238	
	7	0.0106	
	9	0.0186	
G_{Medium}	5	0.3315	0.5646
	7	0.2989	0.7332
	9	0.0243	0.0287
G_{High}	5	0.8824	
	7	0.7886	
	9	0.1228	

Table 3: Probability of failure as function of shear modulus and depth

Using reliability analysis for each case of the nine combinations between shear modulus and depth, table 3 gives the obtained results in terms of the calculated probability of failure P_f. The last column of table 3 gives Juang probability of liquefaction as obtained form equation (9).

V. Modification of Juang probability of liquefaction using DeepSoil simulations and reliability analysis

Predictions of probabilities provided by Juang empirical approach and those obtained by simulation results obtained from physical modelling under DeepSoil and reliability analysis are compared in the following. The site where the complex Tangier City Center was built is chosen for this comparison, [29].

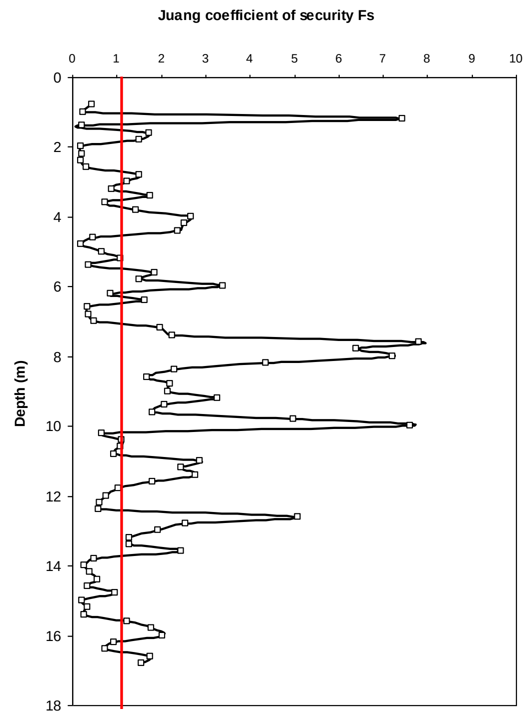

Figure 7: Variation of Juang coefficient of security as function of depth

Figure 7 gives Juang coefficient of security F_s obtained from (1) as function of the depth $z\,(m)$. One can notice the existence of points for which the security factor is less that unity, this indicates that the soil is likely to liquefy under the action of a seism having the magnitude 7.5 in Richter scale. Liquefaction occurs for depths located between 2m and 7 m, and for depths between 10 m and 15 m.

Considering the case $G = G_{Medium}$, table 4 gives comparison of results due to DeepSoil reliability analysis based method and those of Juang method. One can notice that the results are quite different in case of depth $z = 7\,m$. This could be attributed to the constants A and B appearing in equation (9). We propose here to identify these constants by assuming that the probability of liquefaction is equal to P_f. The obtained results are: $A = 0.824$ and $B = 3.06$.

$z\ (m)$	P_f	P_L
5	0.3315	0.5646
7	0.2989	0.7332
9	0.0243	0.0287

Table 4: Probabilities of liquefaction for DeepSoil reliability based method P_f and Juang P_L

Figure 8 gives a comparison between probabilities obtained directly from (9) with the initial values of coefficients A and B (discontinuous line) and those computed with the new identified constants. One can notice that the original Juang method overestimates the probability of liquefaction in the useful domain $Fs \geq 1$.

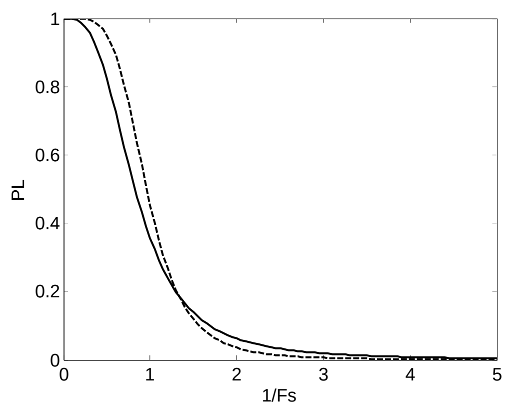

Figure 8: Comparison between Juang probability of liquefaction (discontinuous line) and modified probability of liquefaction (continuous line)

VI. CONCLUSION

Physical modelling of liquefaction phenomenon was performed according to the partially coupled formulation provided by DeepSoil software. Then, reliability analysis was performed in order to calculate the probability of liquefaction associated to the limit state defined by pore water pressure over effective vertical stress greater than 83%. In this analysis four factors were assumed to be random variables and normally distributed. These include seismic surface acceleration, seismic fundamental frequency, water table level and shear modulus. Comparison of predictions of probability of liquefaction as obtained by the empirical Juang method and the more accurate physical modelling was performed. The obtained results were found to be quite different. Assuming that the probability of liquefaction is given by the DeepSoil reliability analysis based method, a correlation enabled to modify the coefficients giving the probability of liquefaction according to Juang method. This work has shown how to use analytical modelling in order to correct the empirical Juang method with the objective to adapt it to the particular context of soils located in sites that are different from those used to derive it initially. Further verifications are needed to assess validity of the modified Juang probability of liquefaction that was proposed in this work.

References

[1] A. Casagrande, Liquefaction and cyclic deformation of sands - a critical review, In S.A.G. (Ed.), *Fifth Panamerican Conference on Soil Mechanics and Foundation Engineering*, Buenos Aires, Argentina, 1975, pp. 79-133.

[2] B. Seed and I. Idriss, Ground Motions and Soils Liquefaction During Earthquakes, Technical report, Earthquake Engineering Research Institute, 1982.

[3] J. Schmertmann, A method for determining the friction angle in sands from Marchetti Dilatometer test (DTM), *Proc. 2nd European Symp. on Penetration Testing*., 2(ESOPT II, Amsterdam.), 1982, 853-861.

[4] B. Seed, K. Tokimatsu, L. Harder and R. Chung, The Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations, Technical Report 84, Earthquake Engineering Research Centre, University of California, 1984.

[5] H. Seed and I. Idriss, Simplified procedure for evaluating soil liquefaction potential, *Journal of Soil Mechanics and Foundations Division, ASCE,* 97(9), 1971, 1249-1273.

[6] T. Holzer, T. Youd and T. Hanks, Dynamic of liquefaction during the superstition hills earthquake (M=6.5) of november 14, 1987, Post Presentation, *ASCE Geotechnical Engineering Division Specialty Conference, Earthquake Engineering and Soils Dynamics II*, 1988.

[7] D. Robertson, P. Woeller and W. Finn, Seismic cone penetration test for evaluation liquefaction potencial under seismic loading, *Canadian Geotechnical Journal*, 24, 1992, 686-695

[8] P. Robertson, S. Sasitharan, I. Cunnig and D. Sego, Shear wave velocity to evaluate flow liquefaction,

Journal of Geotechnical Engineering, ASCE., 121(3), 1995, 262-273.

[9] C.H. Juang, H. Yuan, D.H. Lee and P.S. Lin, Simplified cone penetration test-based method for evaluating liquefaction resistance of soils, *J. Geotech. Geoenviron. Eng.*, 129(1), 2003, 66–80.

[10] R. Dobry, Soil properties and earthquake ground response, Volume 4, *Proceedings of the Tenth European Conference on Soil Mechanics and Foundation Engineering*, 1994, 1171-1187.

[11] K. Park, S. Kim, J. Lee and I. Park, Energy-based evaluation of excess pore pressure using damage potential, *International Journal of Offshore and Polar Engineering*, 2007.

[12] J.K. Mitchell and D.J. Tseng, Assessment of liquefaction potential by cone penetration resistance, *Proc., H. Bolton Seed Memorial Symposium*, Vol. 2. BiTech Pub., Richmond, British Columbia, May, 1990, pp. 335-350.

[13] I. Idriss and J. Sun, Users Manual for SHAKE 91. Technical report, Center for Geotechnical Engineering Modeling, Departament of Civil Engineering, University of California, Davis, U.S.A, 1992.

[14] I. Idriss, R. Lysmer, R. Hwang and B. Seed, QUAD4 - A Computer Program for Evaluating the Seismic Response of Soil Structures by Variable Damping Finite Element Procedures, Technical Report EERC 73-16, Earthquake Engineering Research Center, University of California, Berkeley. U.S.A., 1973.

[15] J. Lysmer, T. Ukada, C. Tsai and B. Seed, FLUSH - A Computer Program for Aproximate 3-D Analysis of Soil - Structure Interaction Problems, Technical Report EERC 75-30, Earthquake Engineering Research Center, University of California, Berkeley, U.S.A., 1975

[16] M. Lee and W. Finn, DESRA - Program for the Dynamic Effective Stress Response Analysis of Soil Deposites, Including Liquefaction Evaluation, Technical Report 36, Soil Mechanics Series. Departament of Civil Engineering, University of British Columbia, Vancouver, Canada, 1975.

[17] R. Siddharthan and W. Finn, TARA2 - Two Dimensional Nonlinear, Static and Dynamic Response Analysis, Technical Report 74, Soil dynamics Group, University of British Columbia, Vancouver, Canada, 1982.

[18] M.N. Biot, Theory of propagation of elastic waves in a fluid saturated porous solid i. low frequency range, *Journal of the Acoustic Society of America*, 28 No 2, 1956, 168-178.

[19] J. Ghabousi and E. Wilson, Variational formulation of dynamics of fluid-saturated porous elastic solids, *Journal Engineering Mechanics Division*, 98, 1972, 947-963.

[20] O.C. Zienkiewicz and T. Shiomi, Dynamic behavior of saturated porous media: The generalised Biot formulation and its numerical solution, *International Journal for Numerical and Analytical Methods in Geomechanics* 8, 1984, pp. 71-96.

[21] I.M. Smith, An overview of numerical procedures used in the VELACS project. verification of numerical procedures for the analysis of soil liquefaction. In K. Arulanandan and R. Escott (Eds.), *Proceedings of the International Conference on the Verification of Numerical Procedures for the Analysis of Soil Liquefaction* 1994, pp. 1231-1340, Balkema A. A.

[22] K. Arulanandan, K. Muraleetharan and C. Yogachandran, Seismic response of soil deposits in San Fransisco marina district, *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, 123, No 10, 1997, 965-974.

[23] H.B. Seed and L.F.Jr. Harder, SPT-based analysis of cyclic pore pressure generation and undrained residual strength, *Proc., H. Bolton Seed Memorial Symposium*, Vol. 2, BiTech Pub., Richmond, British Columbia, May, 1990, pp. 351-376.

[24] Y.M.A Hashash, D.R. Groholski, C.A. Phillips and D. Park, DEEPSOIL V3.7beta, User Manual and Tutorial, University of Illinois at Urbana-Champaign, USA, 2009, 88 p.

[25] N. Matasovic and M. Vucetic, Cyclic Characterization of Liquefiable Sands, *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, 119(11):1805-1822, 1993.

[26] C.J. Chen and C.H. Juang. Calibration of SPT- and CPT-based liquefaction evaluation methods. In: Mayne P., Hryciw R. (Eds.), *Innovations Applications in Geotechnical Site Characterization. Geotechnical Special Publication*, vol. 97. ASCE, Reston, VA, 2000, pp. 49- 64.

[27] T. Iwasaki, T. Arakawa and K. Tokida, Simplified procedures for assessing soil liquefaction during earthquakes, *In Proceedings of the Conference on Soil Dynamics and Earthquake Engineering*, Southampton, UK, Balkema, Rotterdam, the Netherlands, 1982, pp. 925–939.

[28] C.H. Juang, C.Y. Ou, C.C. Lu and Z. Luo, Probabilistic framework for assessing liquefaction hazard at a given site in a specified exposure time using standard penetration testing, *Can. Geotech. J.* 47, 2010, 674-687.

[29] N. Touil, A. Khamlichi, A. Jabbouri and M. Bezzazi, Adaptation d'approches en vue d'évaluer le potentiel de liquéfaction des sols de Tanger, *CFM'2009*, 23-28 Août 2009, Marseille, France.

[30] M.D. Trifunac and A.G. Brady, On the correlation of seismic intensity scales with the peaks of recorded ground motion, *Bull. Seism. Soc. Am*. 65, 1975, 139-162.