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NEW INSIGHTS IN THE LIQUEFACTION POTENTIAL EVALUATION METHODS FOR SOILS

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Abstract. *The paper analyses the evaluation of the liquefaction potential based on in situ tests. The four studied methods are those recommended by the NCEER and those proposed by Boulanger and Idriss. These methods are based on the cyclic stress approach, which characterizes both earthquake loading and soil liquefaction resistance in terms of cyclic stresses. Liquefaction resistance is based on two field tests results, namely SPT and CPT. An extensive analysis is carried out to understand and compare the results obtained. The factor of safety against liquefaction is discussed in terms of many parameters particularly the depth, the percentage of fines and the number of blows N. The analyzed case study is a site in Qatar. Hundreds of CPT and SPT were performed at this site where the foundation soil is calcareous sand, this kind of soil being not commonly considered in the literature. As far as factors of safety against liquefaction are concerned, the results are very close in general, and correspond to the geological and geotechnical soil profile. Analysis shows that the CPT Boulanger and Idriss method is the most conservative. The case study wants to contribute to the understanding of the factors that influence the cyclic stress approach. The SPT resistance is most commonly used but the CPT resistance approach must be gaining more fields in future earthquake geotechnical engineering projects.*

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1 INTRODUCTION

Liquefaction is one of the most important and complex problems in soil dynamics. The occurrences of liquefaction as well as the deformations measured in case of cyclic loading, depend on the soil characteristics such as grain size and shape of particles. In fact, saturated soils for which the strength is developed by friction between particles under the effect of a confining pressure are the most susceptible to liquefaction. In the presence of fine particles, a cohesion or adhesion may exist between particles. Consequently, sands containing fines show a higher strength against liquefaction compared with clean sands. For a long period, sands containing a fraction of fines were considered as having a lower potential of liquefaction than clean sands. Nevertheless, in 1999, two strong earthquakes improved our understanding of the role of fines: The Kocaeli earthquake in the city of Adapazari in Turkey, and the Chi-Chi earthquake in the cities of Wu Feng, Yuang Lin, and Nanton in Taiwan. In these towns, significant damage due to liquefaction was observed: excessive settlement and/or loss of bearing capacity for structures on shallow foundations took place on sites with considerable cohesion in soils. As far as gravelly soils are concerned, they are more permeable than sandy soils, and any excess in pore water pressure generated by cyclic loading can be quickly dissipated.

The sequel of liquefaction may be quite severe: uniform and differential settlement, loss of bearing capacity for shallow and deep foundations, lateral spreading and particular damage to underground structures. After the Alaska earthquake (1964) and the Niigata earthquake (1964), a simplified procedure [1] based on in situ tests was developed to evaluate the liquefaction potential for a site. Later, this same procedure has undergone many modifications or improvements [2], [3] and [4]. In January 1996, a workshop was organized by the NCEER (National Center for Earthquake Engineering Research); the recommendations of this workshop were later analyzed [5]. Recently, the analysis of soil liquefaction has become a full research field [6], [7], [8] and [9]. This paper presents an analysis of the different methods available for the evaluation of the liquefaction potential based on SPT and CPT. The studied site is in Qatar in the Persian Gulf region where huge civil engineering projects are presently realized, and this highlights the importance of the carried out research. The analysis of the large existing database on CPT and SPT yields interesting conclusions both when comparing the liquefaction potential evaluation methods and the understanding of soil behavior under earthquake loading.

2 EVALUATION OF LIQUEFACTION POTENTIAL

2.1 Cyclic Stress Ratio CSR

A simple approach is proposed [1] to evaluate stresses induced by an earthquake. it estimated that the induced shear stress at a depth z was given by the following equation:

$$\tau_{max} = \frac{a_{max} \times r_d \times \sigma_v}{g} \tag{1}$$

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:

$$\tau_{cyclic} = 0.65 \times \tau_{max} \tag{2}$$

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 following equation:

$$CSR_{site} = \frac{\tau_{cyclic}}{\sigma'_{v0}} = \frac{0.65 \times a_{max} \times r_d \times \sigma_v}{g \times \sigma'_{v0}} \quad (3)$$

Several methods were described [10] to determine the coefficient r_d. First, a variation of r_d was proposed as a function of depth [1]. The following equations were adopted [11] to determine r_d for non-critical projects:

$$r_d = 1 - 0.00765 \quad z(m) \qquad z \leq 9.15 \text{ m} \quad (4)$$

$$r_d = 1 - 0.0267 \quad z(m) \qquad 9.15 \text{ m} < z \leq 23 \text{ m} \quad (5)$$

More recently, a relation of r_d to the depth z was proposed [12], expressed in meters, and to the moment magnitude of the earthquake; these relations are appropriate for a depth z< 34m:

$$\text{Ln}\,(r_d) = \alpha(z) + \beta(z)M \quad (6)$$

$$\alpha\,(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right) \quad (7)$$

$$\beta\,(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right) \quad (8)$$

2.2 Cyclic Resistance Ratio CRR

On the other hand, the cyclic resistance ratio (CRR) giving the resistance developed by soil against liquefaction is defined similarly to the CSR. Methods used to measure CRR are based either on laboratory tests or on situ tests. Laboratory tests, particularly cyclic shear and cyclic triaxial tests present the inconvenience of dealing most often with remolded specimen. In fact, it is very difficult to get undisturbed samples especially under water table; Disturbed samples do not reproduce accurate in situ conditions in the laboratory. Techniques such as special tubing or freezing are quite expensive for the time being. 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.

2.3 SPT and CPT Tests

The Standard penetration test (SPT) is a simple in situ test used worldwide. This test gives a basic idea about soil identification (mostly disturbed samples) and soil strength. It can be easily used in fills and granular soils but is not recommended for clayey soils. Moreover, many liquefaction cases during earthquakes are documented based on SPT results. On the other hand, the SPT presents some disadvantages: absence of continuous reading (interval of 50cm between readings), and numbers of blows (N values) are function of the size of the particles, the confining stress and the relative density. As far as the cone penetration test is concerned (CPT), the latter is more advantageous than the SPT. It allows continuous measurements in soil and detects the presence of loose or dense thin layers. A new CPT allows the measurement of the pore pressure in soils. The problem with CPT is that it cannot penetrate dense soils and does not permit any sampling even for simple identification.

2.4 Corrections

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. A magnitude scaling factor (MSF) is used to correct the CSR value measured for an earthquake with a magnitude different from 7.5. This MSF is calculated based on correlations between the number of equivalent uniform cycles and the magnitude on one hand, and on the other hand based on relations obtained in the laboratory between the CSR required to cause liquefaction and the equivalent number of uniform cycles. For an earthquake magnitude greater than 7.5, the following upper limit of MSF is recommended [5]:

$$\mathrm{MSF} = \left(\frac{\mathrm{M}}{7.5}\right)^{-2.56} \tag{9}$$

Equation (10) for calculating MSF is recommended [12]. When used in conjunction with equations linking r_d with z and with the magnitude M (Equations (6), (7), (8)) and not only with z, equation (10) will yield conservative values especially at shallow depths:

$$\mathrm{MSF} = 6.9\,\mathrm{e}^{\left(-\frac{\mathrm{M}}{4}\right)} - 0.058 \leq 1.8\ \mathrm{r} \tag{10}$$

The correlations giving CRR used to evaluate the liquefaction potential of a soil deposit under earthquake loading correspond to horizontal cases only (reference magnitude 7.5 and effective vertical stress of 100 kPa). It is possible to consider [13] the effect of an initial horizontal static stress due to the sloping surface, and cases where the effective vertical confining stress is greater than 100 kPa by correcting the value of CRR as follows:

$$\mathrm{CRR}_{\alpha,\sigma} = \mathrm{CRR}_{\alpha=0,\sigma'_{v0}<100\ \mathrm{kPa}} \times \mathrm{K}_{\alpha} \times \mathrm{K}_{\sigma} \tag{11}$$

$$\alpha = \frac{\tau_{\text{horizontal static}}}{\sigma'_{v0}} \tag{12}$$

K_α and K_σ are respectively correction coefficients taking into account the initial horizontal static shear and the effective vertical confining stress. These coefficients vary according to the soil nature and should be determined for each site if possible. The evaluations of K_α have been discussed [14]. Subsequently, K_α and K_σ values were revised [15]. Of course, CRR values increase as a function of fines content. The practice today is to increase the values of N or q_c, by introducing ΔN or Δq_c, to take into account the amelioration of soil resistance due to the presence of fines content, or increase the CRR by a factor depending on soil plasticity. Nevertheless, it is necessary to distinguish between soils having a sandy like behavior for which the corrections are applicable, and a soil having a clayey like behavior where specific criteria should be examined.

2.5 Normalized N and q_c

In general, number of blows N_{60} and bearing resistance q_c are normalized for a confining stress σ'_{v0} of 100 kPa, in the aim of obtaining values depending on the relative density of sand, whatever the depth. Normalized values for N_{60} and q_c are designated respectively by $(N_1)_{60}$ and q_{c1}. Moreover, an additional correction is suggested for the values of q_{c1} in order to become dimensionless; this correction consists in normalizing q_{c1} by a reference pressure of 100 kPa. The value of q_{c1N} is hence obtained. A procedure to calculate the equivalent tip bearing resistance for a clean sand $(q_{c1N})_{cs}$, was proposed [16]. More recently, CRR curves as a function of $(q_{c1N})_{cs}$ have been reviewed [12].

3 METHODOLOGY OF ANALYSES AND CASE STUDY

In this paper, four methods to evaluate soil liquefaction potential will be applied to a site in Qatar in the Persian Gulf. The first two methods have been adopted by the NCEER [5]: these are the CPT method [16] and the SPT method [17]. The other remaining two methods are respectively an SPT method and a CPT method studied [15]. Calculations following the four methods have been performed in the absence of any initial static shear stress ($K\alpha$ =1).

The studied site is part of a larger zone where residential as well as industrial projects are under construction. This area is located on the Persian Gulf southern coast in the city of Doha in Qatar. 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. In total, 6 boreholes were executed and 18 CPT profiles were measured. In the case of 4 CPT profiles, measurements were made after 6, 12 and 30 passes of the compactor in order to check the depth reached by the compaction effort. A total of 8417 CPT values were obtained for different depth.

3.1 Geology of the Region

From a geological perspective, the peninsula of Qatar is part of the Persian Gulf basin. The basin extends from the Arabian plate to the east, to the Zagros fold in Iran, over a width of 1200 km. The formations of the peninsula of Qatar are all of Tertiary to Quaternary age. The surface of the Doha area (city were the studied site is located) is generally made of surface deposits of Holocene and Pleistocene ages, consisting of aeolian sand and silt. The major underlying tertiary formations are divided into: Limestone Simsima (Upper Dammam Formation); argillite Midra (Lower Dammam Formation), and finally the formation of Rus. The majority of the land of Qatar consists of a calcareous horizon uniform Middle Eocene age, limestone Simsima, which is part of the formation of upper Dammam. The limestone of this formation is of poor to moderate resistance, and occasionally strong. We often encounter in this formation thin layers or lenses of silty carbonated rock and marl, interconnected voids and cavities filled with silt, marl, calcite, or gypsum. The argillite Midra is part of the lower Dammam formation, Middle Eocene age. It consists of laminated argillite, brown/yellow and occasionally greenish. These clays contain thin discontinuous beds of beige limestone chalky or crystalline. Fossil shark teeth are found in this formation. The formation of Rus, overlain by argillites Midra, is of Lower Eocene age. It generally consists of dolomitic soft chalky limestone. Thin intercalations of greenish brown clay are occasionally encountered.

3.2 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 sandy cover nature. Contrary to the majority of sands normally encountered and that are made of silica, the sands of this site are calcareous, given their origin from the underlying limestone. On the other hand, the calcareous sand which breaks more easily has a lower resistance to penetration and crushing than silica sand. The strength characteristics of the sand cover, and its behavior before and after compaction should be considered with caution.

3.3 Seismic Context of the Region

The Persian Gulf is located in the area of a major collision, the Alpine-Himalayan belt. The gradual movement of the tectonic Arabic plate towards the Eurasian plate resulted in a compression of the earth crust between them. This movement is about 20mm a year now. Compression affects mainly Iran, either in the form of a subduction of the Arabian plate under the Eurasian plate at depths greater than 50 km, or in the form of surface compression affecting the crust up to 50 km deep, resulting into major folds and faults (Zagros Fold Belt). Heading towards the south of this region, the seismicity becomes smaller. The Arabian plate is considered tectonically stable, and it is generally assumed that no major seismic activity has occurred in the last 10 million years in the South. The extremely low intra-plate seismic activity reflects a smooth fold in the Arabian plate. Summing up, 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 0.02 g and 0.07 g 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 0.06 g and 0.21 g 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. On the other hand, an evaluation of the effect of the earthquake that struck western Iran on March 31st 2006, conducted by USGS confirmed these results, giving peak accelerations ranging from 0.02 to 0.04g in the south and 0.08 to 0.16 g in the north of Qatar. The city of Doha and therefore the studied site is located in the east and middle of Qatar, a maximum acceleration of 0.15 g is suggested for calculation. The maximum magnitude of 6.5 is also considered.

4 RESULTS AND DISCUSSION

The four methods of calculation presented in the previous section are used for the CPT profiles available in the studied site. As an illustration, Figure 1 shows a representative CPT soil profile CPT311, as measured in situ. Figure 2 shows the corrected values in terms of q_{C1N} and $(q_{C1N})_{CS}$ of CPT311 soil profile. Obviously these corrections are made according to the two studied CPT methods (1998 and 2004; [16], [12] and [15]). Lastly, Figure 3 shows the SPT profile N60 obtained by calculation from the profile CPT311 measured at the same place. The corrections shown $(N_1)_{60}$ and $(N_1)_{60CS}$ in Figure 3 are done according to SPT methods (1997 and 2004; [17], [12] and [15]) studied in this work. For the analyzed case study, the results of calculations of the safety factor Fs, made based on the four methods are given in Figure 4. Figure 5 shows the measured in situ SPT profile N_{60} at the same place; the measurement points in the sampled SPT borehole are linked to show the trend, it is the profile P4-03. Figure 6 in turn presents the calculations of the safety factor obtained following the two SPT methods.

The safety factors calculated by different methods indicate a risk of liquefaction in the layer of calcarenite, located around the depth of 3m, and for sands located below. Similarly, the safety factors calculated from borings indicate values less than unity for this layer and the underlying sand. These results indicate that the site is prone to liquefaction phenomena between depths of 3 and 7 m, where an earthquake of magnitude 6.5 and characterized by a maximum acceleration of 0.15 g is considered to hit the region. Moreover, the results show that the CPT and SPT profiles reveal an analogy, and outline the presence of the soft calcarenite layer at a 3m depth. Thus, the general trend of safety factors derived from the CPT is consistent with the safety factors calculated from the SPT carried out at the same location. Of course, the CPT profiles are continuous and can detect variations and thin layers, unlike SPT profiles that provide readings at intervals of 1 to 1.5 m. As for compaction, the results of CPT carried out in the soil in its initial state and then after each compaction step indicates that compaction does not affect the soil strength (in terms of q_c and therefore CRR below a depth of 2 m. Of course the soil strength increases with compaction energy increasing, but the depth of the influence compactor will not exceed 2m below surface.

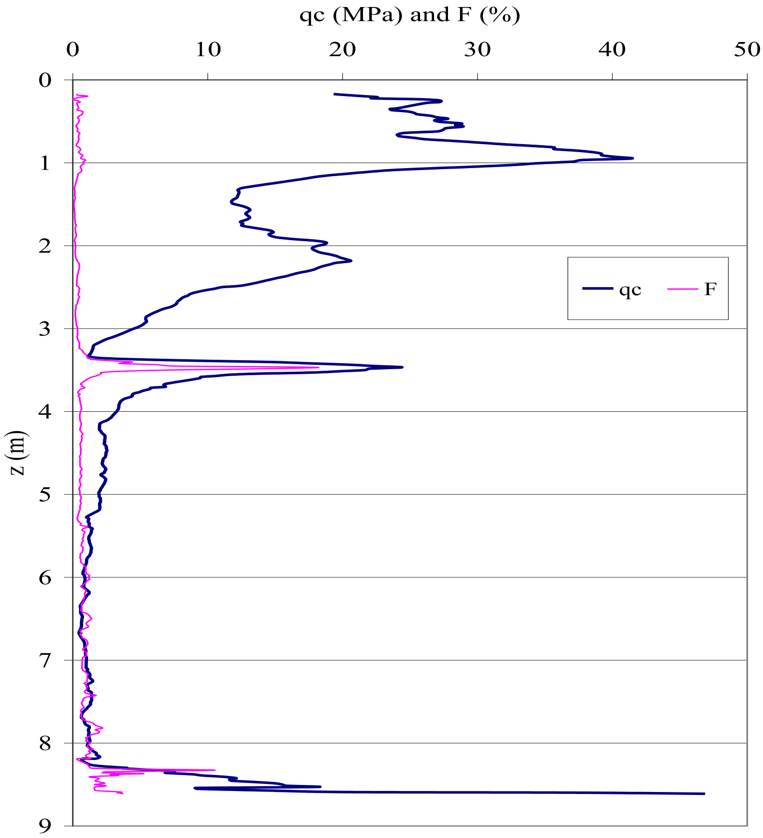

Figure 1. CPT-311 Profile typical of the site in Qatar

By observing the results of calculations, we note that for the majority of CPT profiles, the CPT method of Boulanger and Idriss (2004) for evaluating the liquefaction potential is the most conservative, providing the lowest factors of safety. Note that all calculations were performed neglecting the presence of initial shear stress ($K\alpha = 0$). But in the case of presence of such stresses, the values of CRR will be increased for $(N_1)_{60cs}$ above 12, and decreased otherwise. This implies that the factor of safety for calcarenites and sands more than 3m deep will decline further, knowing that beyond this depth the factors of safety calculated are already less than 1 indicating an area susceptible to liquefaction, even in the absence of initial shear stress. The variation in safety factors obtained for each method versus other methods is presented in Figure 7. In addition, a linear regression analysis was performed to derive the linear trend between the various safety factors calculated. This analysis was based on the method of least squares.

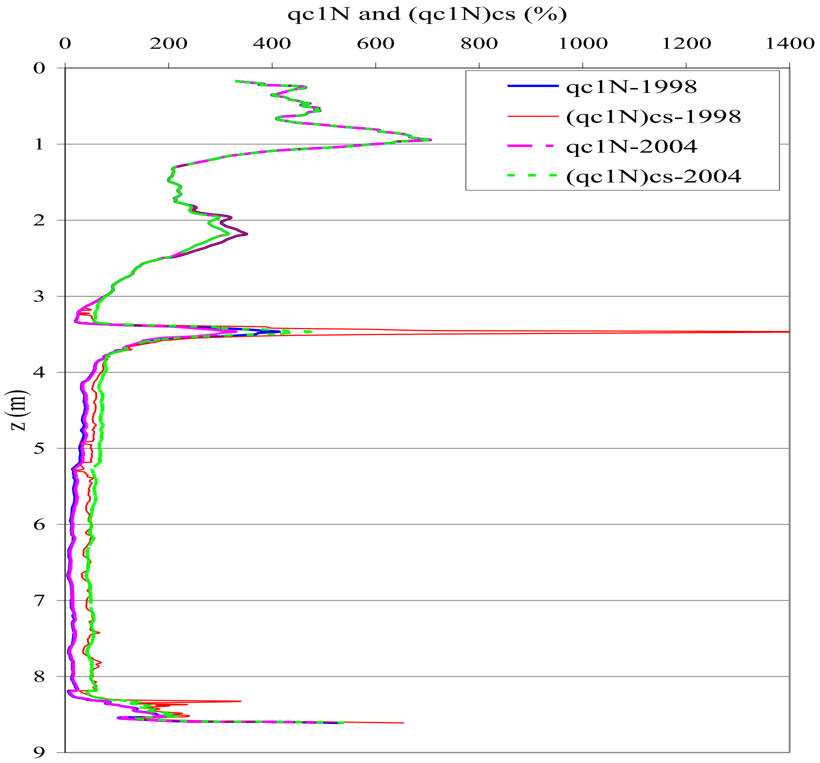

Figure 2. q_c corrected according to CPT-1998 and CPT-2004 methods for CPT-311 profile

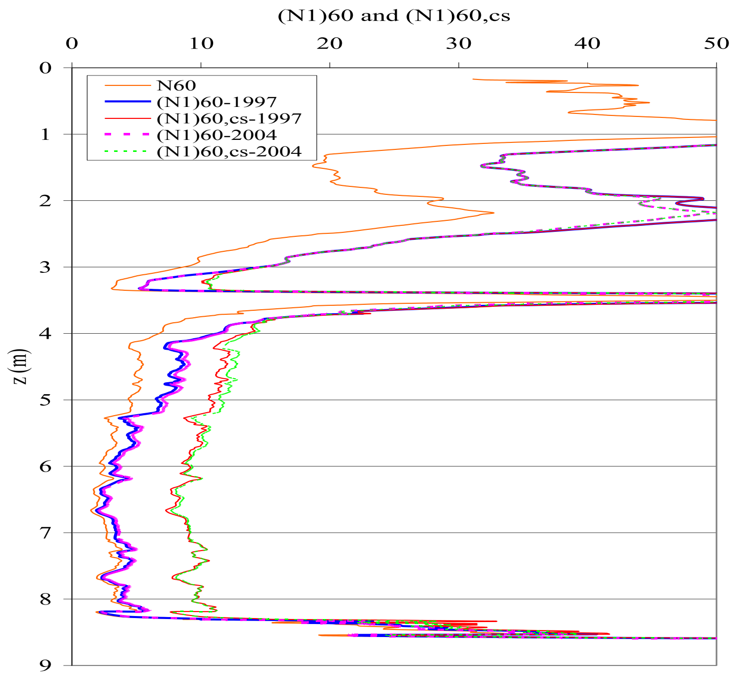

Figure 3. $(N_1)_{60}$ and $(N_1)_{60cs}$ profiles based on SPT-1997 and SPT-2004, forCPT-311 profile

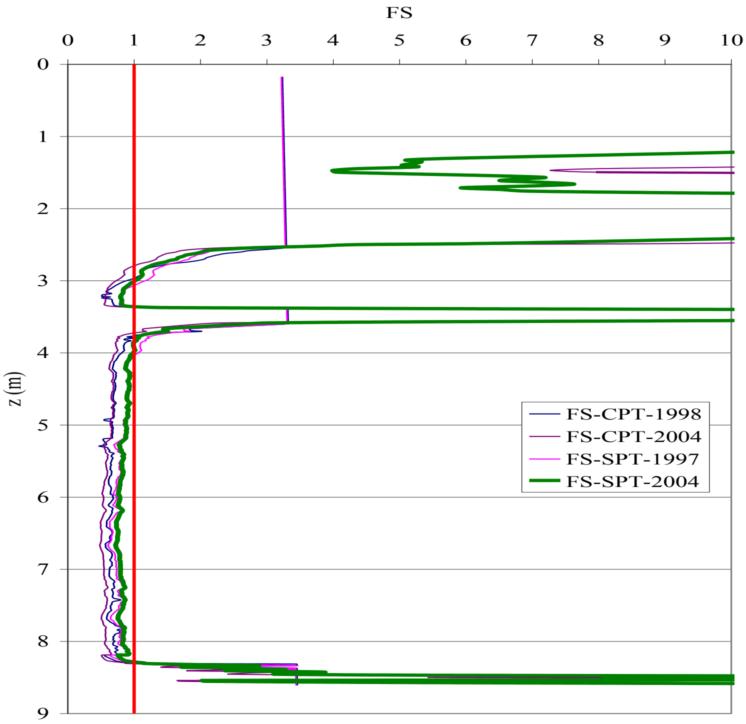

Figure 4. The Factor of safety function of depth according to the 4 methods, CPT-311 profile

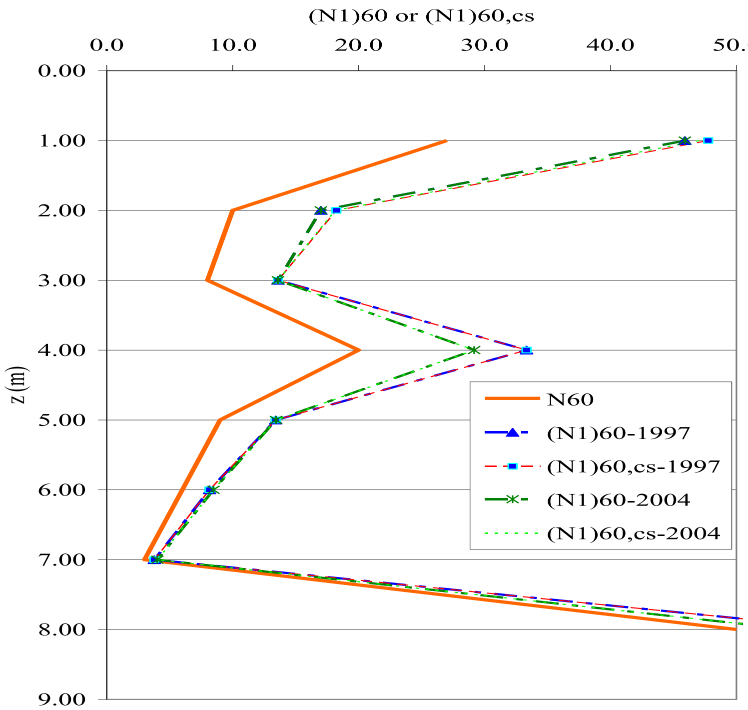

Figure 5. The Values of N using methods SPT-1997 and SPT-2004, borehole P4-03

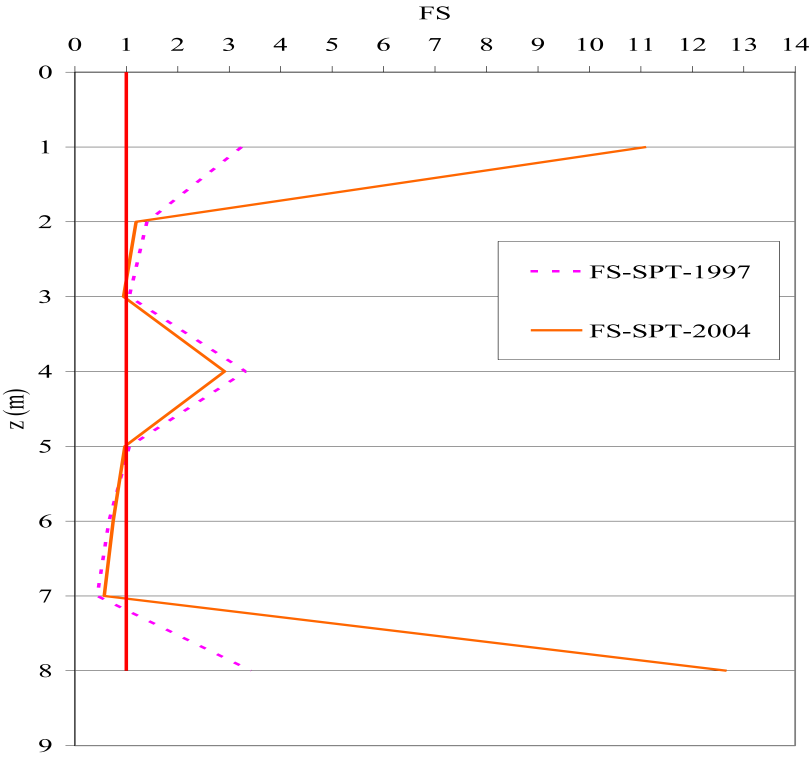

Figure 6. The Factor of safety, methods SPT-1997 and SPT-2004 for borehole P4-03

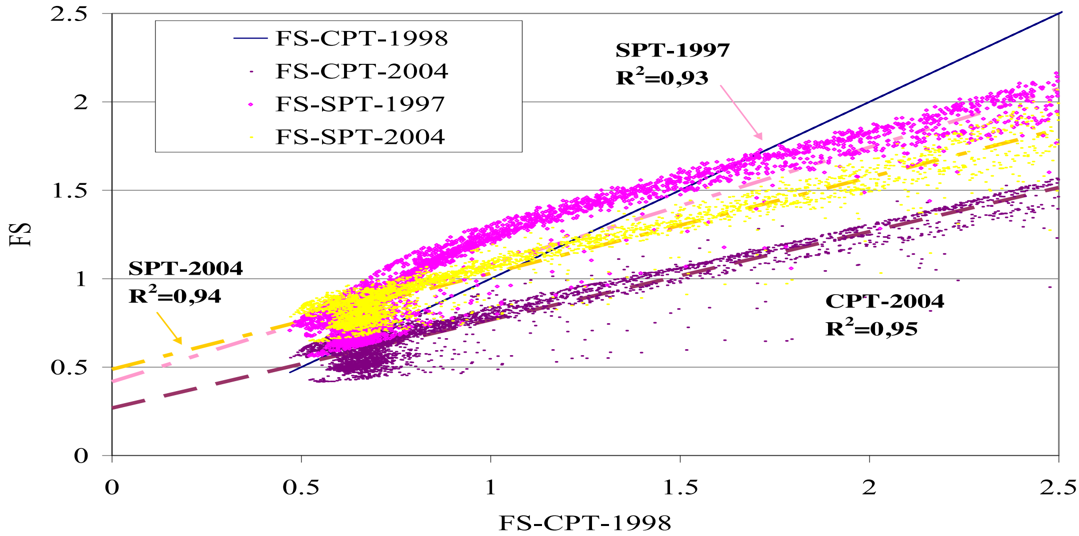

Figure 7. Example of relation (linear regression) between factors of safety obtained according to the different calculation methods

The regression analysis in Figure 7 shows that all methods are linearly dependent, with R^2 values ranging between 0.86 and 0.96. In general, the R^2 values vary between 0 and 1 and give an indication on the dispersion of the original data from the estimated linear curve. This curve is most reliable when R^2 is close or equal to 1. In addition, the R^2 values obtained for the various safety factors indicate that the linear regressions represent the experimental values deduced with a high degree of reliability. Additionally, the R^2 values show that the safety factors of Boulanger and Idriss for CPT and SPT (2004; [12] and [15]) are approximately linearly dependent ($R^2 = 0.86$). Also, they indicate that the safety factors based on CPT methods ([15] and [16]) on one hand, and those based on SPT methods ([17] and [12]) on the other hand, show perfectly linear dependencies ($R^2 = 0.95$ and 0.96).

5 CONCLUSIONS

Many methods have been developed to evaluate the soil liquefaction potential. In the present paper, the comparison of methods adopted by the NCEER based on CPT [16], and SPT [17], and those more recently developed by Boulanger and Idriss in 2004 for CPT and SPT ([12] and [15]), has shown that the method of Boulanger and Idriss for CPT 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. The present study forms a basis for subsequent developments to be carried out by the authors in the evaluation of liquefaction potential field, especially that very interesting research is actually going on in the same domain ([8] and [9]). Having obtained variable results for methods based on CPT, the authors strongly recommend additional comparative analyses to be realized for different sites before proposing conclusions on the convergence of these methods. Furthermore, in order to validate the CPT and SPT methods, laboratory tests need to be undertaken. A main contribution of the present paper is that it has analyzed the behavior of calcareous sand. A specific attention should be given to the effect of the presence of fines on the potential of liquefaction of sandy soils.

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