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LIQUEFACTION SUSCEPTIBILITY OF SILTY SAND LAYERS IN THE GRENOBLE BASIN: INVESTIGATION BY IN SITU TESTING AND NUMERICAL SIMULATION

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ABSTRACT

Seismocone and piezocone penetration tests have been carried out in the superficial layers of two sites on the Grenoble Universities campus in order to assess the liquefaction susceptibility at these sites. The data enabled the identification of layers of fine, loose silty sand of varying thicknesses between 2 and 10 metres depth. Using the cone penetration data, a liquefaction study was carried out at each site based on the simplified method developed by Seed and Idriss (1971) for a seismic event of magnitude 5.25. The simplified method predicted liquefaction risk zones in the silty sand layers. These predictions were supplemented by methods using seismic wave speed V_s and methods using piezocone data. Undisturbed core samples of silty sand were obtained at the piezocone site in order to carry out cyclic undrained triaxial tests at confining pressures corresponding to the depths of interest, and thus to compare the laboratory liquefaction resistance with that estimated empirically from cone penetration data. The parameters obtained from the laboratory and field results will enable us to model the in situ response of the silty sand layers at the piezocone site, using a nonlinear multi-mechanism model capable of simulating cyclic liquefaction of sands (Iai et al., 1992).

Keywords: Liquefaction, silty sand, piezocone, seismocone.

INTRODUCTION

The Grenoble basin, located in the French Alps, is filled with hundreds of metres of soft, post-glacial sediments. A typical configuration consists of, from the bedrock to the surface, 400 metres of lacustrine silts and clays, followed by 10-20 metres of fine silty sand (sablons) and 20 metres of alluvial sediments and river silts in the uppermost layer. The content of these surface layers varies significantly depending on their location in the Grenoble basin, and previous site investigations have illustrated the existence of loose silty sands and sandy silts in the upper 10 metres at sites on the Grenoble University campus. The city of Grenoble is located 15km from a fault capable of producing a magnitude M_W=5.5 earthquake. Although low compared with other regions in the world, this level of seismicity, combined with possible site effects associated with the thick sedimentary basin, is sufficient to warrant a study of the liquefaction susceptibility of these surface layers on the university campus. To this end, an in situ testing campaign has been carried out on the campus. Two different test sites are presented in this paper: a site at the Physics department known hereafter as the Physics site, and the site of the engineering school of Hydraulics and Mechanics of Grenoble, known hereafter as the HMG site. A seismocone test was carried out at the Physics site, and a piezocone test and Cone

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Penetration Test (CPT) were carried out at the HMG site. Boring with undisturbed sampling was also carried out at the HMG site. Liquefaction potential was evaluated using different in situ test based methods. The results of the in situ tests and corresponding liquefaction analyses are presented in this paper, and laboratory testing of the core samples is also discussed. In situ and laboratory results will contribute towards the numerical modeling of these superficial layers, using a multi-mechanism model capable of describing liquefaction behaviour.

Evaluating liquefaction potential using the cone penetration test

The "simplified procedure" for evaluating liquefaction resistance is an empirical procedure first developed by Seed and Idriss (1971), who published an approach in which the cyclic stress ratio (CSR) caused by a given earthquake was estimated from the following equation:

$$CSR = \frac{\tau_{av}}{\sigma'_{v0}} = 0.65\left(\frac{a_{\max}}{g}\right)\left(\frac{\sigma_{v0}}{\sigma'_{v0}}\right)r_d \tag{1}$$

Where τ_{av} is the average cyclic shear stress; a_{max} is the maximum horizontal acceleration at the ground surface; g is the acceleration due to gravity; σ_{v0} and σ'_{V0} are the total and effective vertical overburden stresses, respectively; and r_d is a stress reduction factor. The CSR profile for a given earthquake can then be compared to the estimated cyclic resistance ratio (CRR) profile for the soil deposit, adjusted to the magnitude of interest. In this paper, the method described by Robertson and Wride (1998) is applied, in which the CPT cone tip resistance, q_c and friction ratio, FR are used to estimate soil grain characteristics and consequently a "soil behaviour type index", I_c using an iterative procedure. I_c is used to estimate the fines content in the soil, and is incorporated into a correction factor in order to obtain the "clean sand" equivalent tip resistance. The CRR itself is calculated from the relation derived by Idriss and Boulanger (2004):

$$CRR = \exp\left\{\frac{q_{c1N}}{540} + \left(\frac{q_{c1N}}{67}\right)^2 - \left(\frac{q_{c1N}}{80}\right)^3 + \left(\frac{q_{c1N}}{114}\right)^4 - 3\right\} \tag{2}$$

Where qc1N is, for a given depth, the dimensionless normalised cone penetration resistance corrected for a vertical effective stress of 100 kPa. The q_{c1N} in equation (2) refers to clean sand data (fines content < 5%). In this paper, the magnitude-dependent stress reduction factor, r_d and magnitude scaling factor MSF were included in accordance with Idriss and Boulanger (2004), to account for the effects of earthquakes with magnitudes different from 7.5.

Evaluating liquefaction potential using shear wave velocity

Andrus and Stokoe (2000) established a V_s-based procedure for CRR estimation using field correlations and databases of liquefaction and non-liquefaction case histories. Empirical curves estimating CRR from Vs1 values (shear wave velocity corrected for an overburden stress of 100 kPa) were established. This method is particularly useful for sites difficult to penetrate using the CPT. However, V_s values alone give information only on the relative density, D_r, of a soil and not on fines content. Estimation of cyclic liquefaction potential from V_s requires that fines content information be available from either core samples or CPT data. The use of the seismocone incorporates the three measurements q_c, sleeve friction (f_s) and V_s. Details of this test are outlined in the next section.

Evaluating cyclic liquefaction using the piezocone

The piezocone is also used for liquefaction studies. In addition to measurements of q_c and f_s, two important parameters are obtained: Δu, the over pore water pressure due to penetration (=u-u_0, where u_0 is the hydrostatic water pressure), and t_{50}, the time in which 50% of the over pore water pressure is dissipated during a dissipation test. The value of Δu is linked to the relative density of finer grained soils such as sands and clays, and the value of t_{50} is related to the permeability and consolidation characteristics and thus to the soil type. Authors such as Robertson (1990), Senneset and Janbu (1985) and Jones and Rust (1982) have proposed soil behaviour type classification charts based on q_c and Δu data. Senneset and Janbu defined a pore pressure parameter ratio, B_q:

$$B_q = \left(\frac{u - u_0}{q_t - \sigma_{v0}} \right) \tag{3}$$

Where q_t is cone tip resistance corrected for pore water pressure effects; $u-u_0$ is the over pore pressure generated by the piezocone; and σ_{v0} is the total vertical overburden pressure. In this paper, two approaches are used to evaluate liquefaction potential using the piezocone. The first approach follows that presented by Berthelot and Durand (2004), in which B_q and q_t are plotted on a classification chart. The second approach, proposed by Berrill et al. (1992) also involves the use of a dimensionless parameter, ΔU:

$$\Delta U = \frac{\Delta u}{\gamma_w V_f t_{50}} \tag{4}$$

Where V_f is the penetration rate for the CPT (normalised to 2cm/s) and γ_w is the bulk unit weight of water. ΔU and q_c are plotted on a diagram consisting of iso-relative density and iso-confining pressure curves, in which a “liquefiable zone” has been delineated from results on New Zealand sand.

The in situ tests undertaken at the Physics site and the HMG site are outlined in the next section, as well as interpretation of the results to identify the nature of the soils present. Then, site liquefaction potential is evaluated using the methods described above.

IN SITU TESTING

Physics site

A seismic cone penetrometer, or seismocone, test was carried out at the Physics site on the university campus in October 2003. In addition to the usual q_c and f_s measurements, 3 geophones are positioned behind the cone tip, orientated in the x, y and z directions respectively. Penetration was to a depth of 9.8 metres, with downhole tests being performed every 0.5 m of penetration. The downhole tests consisted of a hammer impact on a metallic mass, known as the seismic source. The impact was made in such a way as to generate vertically propagating, horizontally polarized shear waves. 19 downhole tests were carried out during the penetration.

Results

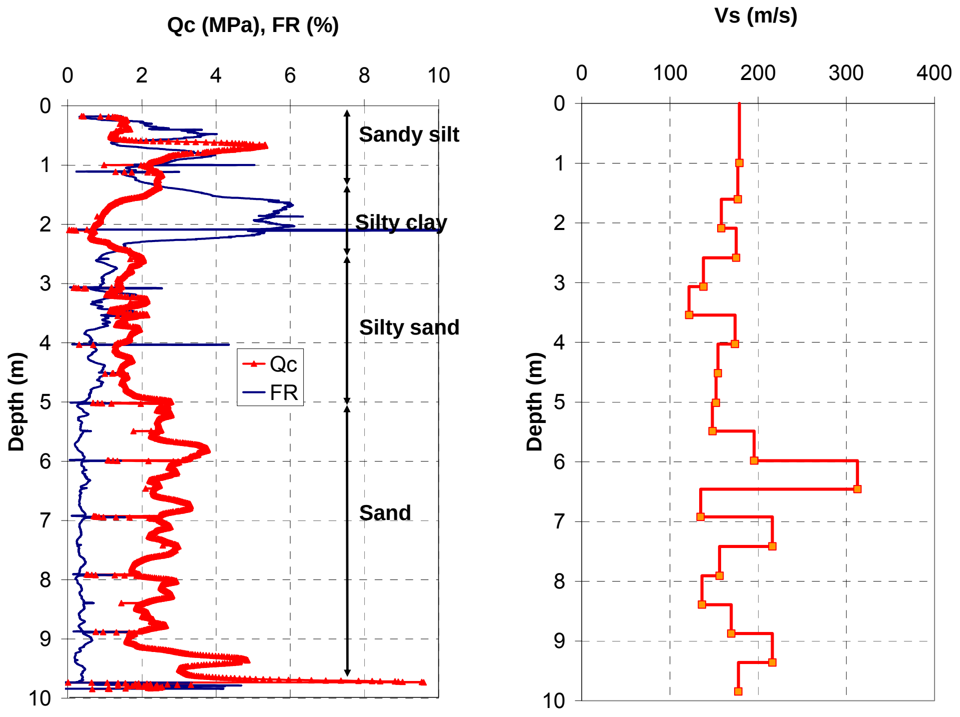


Figure 1. Seismocone test results (a) q_c and FR (b) V_s

Figure 1 presents the seismocone test results, in terms of (a) the cone tip resistance, q_c, and friction ratio, FR, and (b) shear wave velocity, V_s. The q_c-FR plot was used to identify the layers of soil present in the profile.

From 0 to 1.2 metres, the soil is predominantly silty. At 0.6 metres, there is a peak in the q_c corresponding to a denser, 20 centimetre sand layer. The average fines content is estimated to be about 30% for this first layer. From 1.2 to 2.4 metres, the low q_c and high FR values indicate a softer, more cohesive layer. According to the Robertson and Wride procedure, it is a layer of silty clay, with an average fines content of 50%. From 2.4 to 5 metres, a homogeneous, loose silty sand layer is encountered, indicated by the low and relatively constant q_c value combined with a low FR value. The estimated average fines content is 30%. From 5 to 9 metres, the value of q_c is slightly higher than for the previous layer, and the FR value is lower, indicating a loose silty sand layer. The average fines content is estimated at 20%. Beyond 9.8 metres, a much denser, possibly gravel layer is encountered. The water table was found to be at 4 metres.

The shear wave velocity profile was obtained from the wave arrival times recorded on the x and y component geophones, except for the first two downhole test depths, at 1 and 1.6 metres, where the shear wave arrival times were estimated by extrapolating from the deeper shear wave trains. In figure 1(b) it can be seen that the shear wave results confirm to some extent the cone penetration test results. The general V_s trend follows the q_c profile, and quite homogeneous Vs values are observed in the silty sand layer from 2.4 to 5 metres (average V_s = 148 m/s). Between 5 and 6 metres, the increase in V_s corresponds to the q_c profile, indicating the denser sand layer. The average V_s measured in this denser layer is 186 m/s.

HMG site

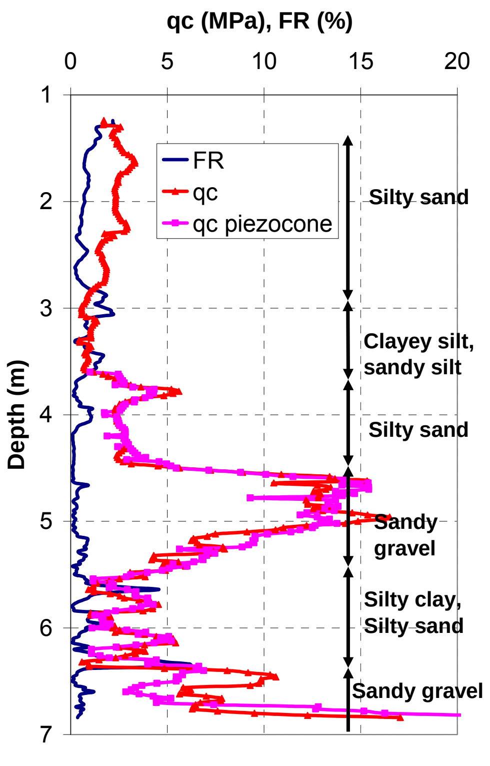
CPT and piezocone tests were carried out at the HMG site in September 2006 to 6.8 metres depth. A dense gravel layer beyond this depth prevented further penetration. The piezocone profile begins at 3.6 metres, which was the initial assumed water table position. 11 dissipation tests were carried out. A

CPT was performed subsequently at 0.5 metres horizontal distance from the piezocone, in order to obtain a complete q_c-FR profile from as near to the surface as possible. In parallel to the penetration tests, drilling with intact sampling took place. The drilling went to 9 metres depth, with intact samples being taken from 1.7 to 2.8 metres, and from 3 to 4.3 metres. Cuttings from the drilling were obtained at all other depths.

Results

Figure 2 presents the results of the piezocone and cone penetration tests; in figure 2(a), q_c and FR are plotted for the CPT, as well as the q_c profile obtained from 3.6 metres during the piezocone test. As the CPT and piezocone q_c profiles are almost identical from 3.6 metres, it is supposed that the CPT q_c profile can be used to represent the piezocone site, and thus the liquefaction studies using the Robertson and Wride approach (q_c, FR) and the piezocone approaches can be compared. In figure 2(b), the total pore water pressure generated, u, and the hydrostatic pore water pressure, u_0, are plotted with depth. The pore water pressure dissipation test positions are also plotted. From backcalculating final u values for the dissipation tests, the water table level was found to be at 2.9 metres.

Soil layers were at first identified using the soil type index I_c as well as examining the general profile of figure 2(a). From 1.3 to 2.9 metres, there is a loose silty sand, with an average fines content of about 20%. From 2.9 to 3.6 metres there is a heterogeneous layer consisting of sandy silt as well as small thicknesses of clayey silt from 2.9 to 3.1and 3.5 to 3.6 metres. This is followed by a loose silty sand layer from 3.6 to 4.5 metres, with an average fines content of 15%. A dense sandy gravel layer underlies the silty sand, from 4.5 to 5.4 metres. Another heterogeneous layer is then encountered, with loose silty sand (average fines content 18%) and silty clay stratifications, from 5.4 to 6.4 metres. Finally, the beginning of another, thicker, dense sandy gravel layer is found from 6.4 metres until the end of penetration. Examination of the intact samples, as well as cuttings from the boring between 0 and 7 metres, confirms the types of soil estimated by the I_c method.



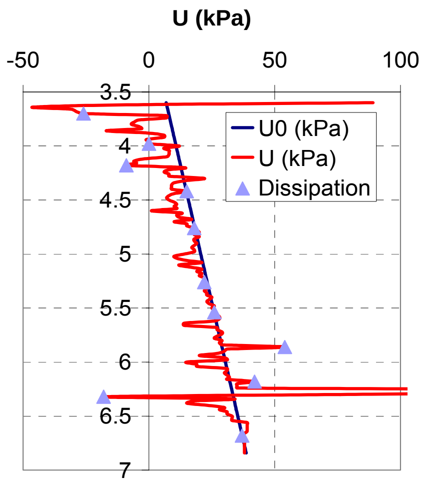


Figure 2. (a) q_c, FR from the CPT and q_c from the adjacent piezocone test; (b)U and U_0 with pore water pressure dissipation tests during the piezocone test, HMG site

In the layer between 3.6 and 4.5 metres, excess pore water pressures are consistently negative, indicative of a dilative soil. Looking more closely, an exception is found to these observations at around 4.2 to 4.3 metres, where small positive pore water pressures are generated. In the gravel between 4.5 and 5.4 metres, the excess pore water pressures remain negative and approach zero as expected of such a permeable layer. In the layer of silty sand and silty clay from 5.4 to 6.4 metres, there is more fluctuation of the excess pore water pressure. At 5.66, 6 and 6.32 metres, excess pore pressures are negative, while at 5.87 and 6.26 metres there are positive excess pore water pressures. The negative values at 5.66 and 6 metres seem to correspond to points at which q_c rises and FR decreases relative to a thin, soft, silty layer above. The positive spike at 5.87 metres corresponds to a thin, 10 centimetre layer of sandy silt, while the large positive spike at 6.19 metres corresponds to another thin 10cm layer of a more clayey silt before the probe encounters the second gravelly layer. At this point (6.32 metres) excess pore water pressures become strongly negative because of the dense gravel layer and the relative impermeability of the clayey layer just above, but the values then decrease with depth to become close to hydrostatic pressure due to the overall permeability of this layer.

The negative excess pore water pressures found from 3.6 to 4.5 metres may be due to the reasonably low confining pressure, which causes even a loose soil to dilate during the penetration-induced shear. In the zone between the two gravel layers (5.4 to 6.4 metres), interfaces between softer, clayey layers and sandy layers are marked by a decrease in the excess pore water pressure from positive to negative, although these pressures are not maintained with depth, probably due to the permeability of the surrounding gravels. Classification of this profile using piezocone data and a classification chart such as that of Jones and Rust (1982) gives the same types of soil as the Robertson and Wride method using the CPT data.

The parameter t_{50} was estimated from each of the 11 dissipation tests, with values varying between 0 and 3 seconds except for at 5.87 metres (15 seconds) and 6.19 metres (8.4 seconds). These results are consistent with the permeable nature of the profile, due to the gravel layer. Figure 3 shows some typical results. The dissipation tests were performed at an average spacing of about 30 centimetres, which may not be sufficient in order for full pore water pressures to be developed between tests.

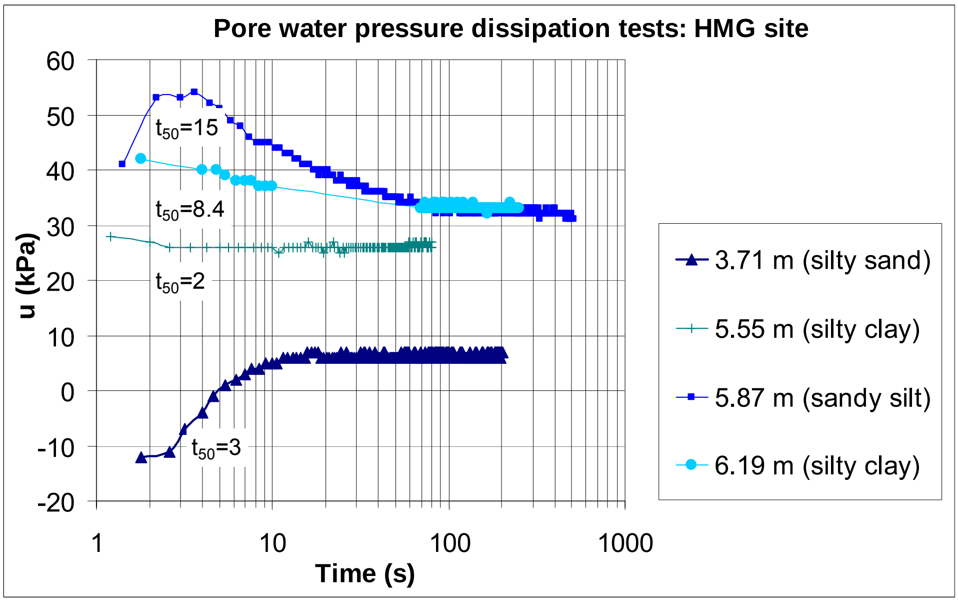


Figure 3. Dissipation tests from different depths

LIQUEFACTION ANALYSIS

Physics site

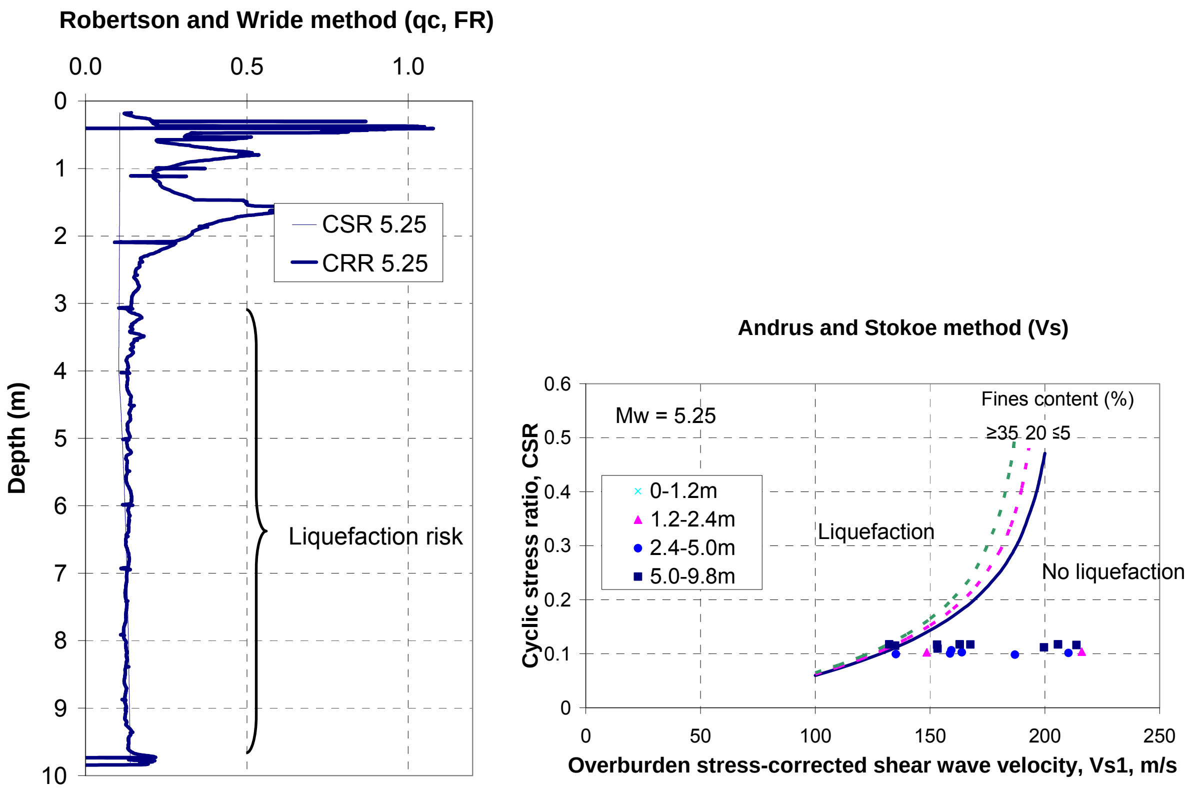


Figure 4. Application of (a) Robertson and Wride method and (b) Andrus and Stokoe method for the Physics site profile

The cyclic resistance ratio (CRR) with depth was calculated for the Physics site in accordance with the Robertson and Wride method. CRR was plotted against cyclic stress ratio (CSR), estimated using equation (1), with an estimated a_{max} for the Grenoble region of 0.16g. The results are shown in figure 3(a). As expected, there is no liquefaction risk from 0 to 2.4 metres depth. However, beyond 2.4 metres, the liquefaction risk increases with depth up until the end of the profile. It is thus the silty sand layers, particularly the one from 5 to 9.8 metres, which have considerable liquefaction potential in the event of a Grenoble region earthquake.

Figure 3(b) shows the results of the applied Andrus and Stokoe method (2000), which makes use of the V_s values obtained during the seismocone test. Values of V_{s1}, the V_s measurements corrected to a reference overburden stress of 100 kPa, were plotted against the estimated CSR from equation (1). Liquefaction resistance (CRR) curves for different fines contents were then plotted for an earthquake of magnitude 5.25. Most of the data points plot well into the "no liquefaction" part of the diagram. The exceptions are two points from the lowest, loose silty sand layer, and one point from the loose silty sand layer just above (2.4 to 5 metres), which plots very close to the limit line for less than 5% fines. The V_s method for evaluating liquefaction resistance shows a similar trend to the CPT method, with the greatest risk of liquefaction occurring in the lowest silty sand layer between 5 and 9.8 metres. However, the discrete nature of the Vs measurements does not allow for a global view of the liquefaction risk with depth, and, according to the Andrus and Stokoe method, there are only two critical points as opposed to an entire critical layer by the Robertson and Wride method. The V_s method thus seems less conservative, and should only be used as a basis for more in-depth study of the depths shown to be "liquefiable".

HMG site

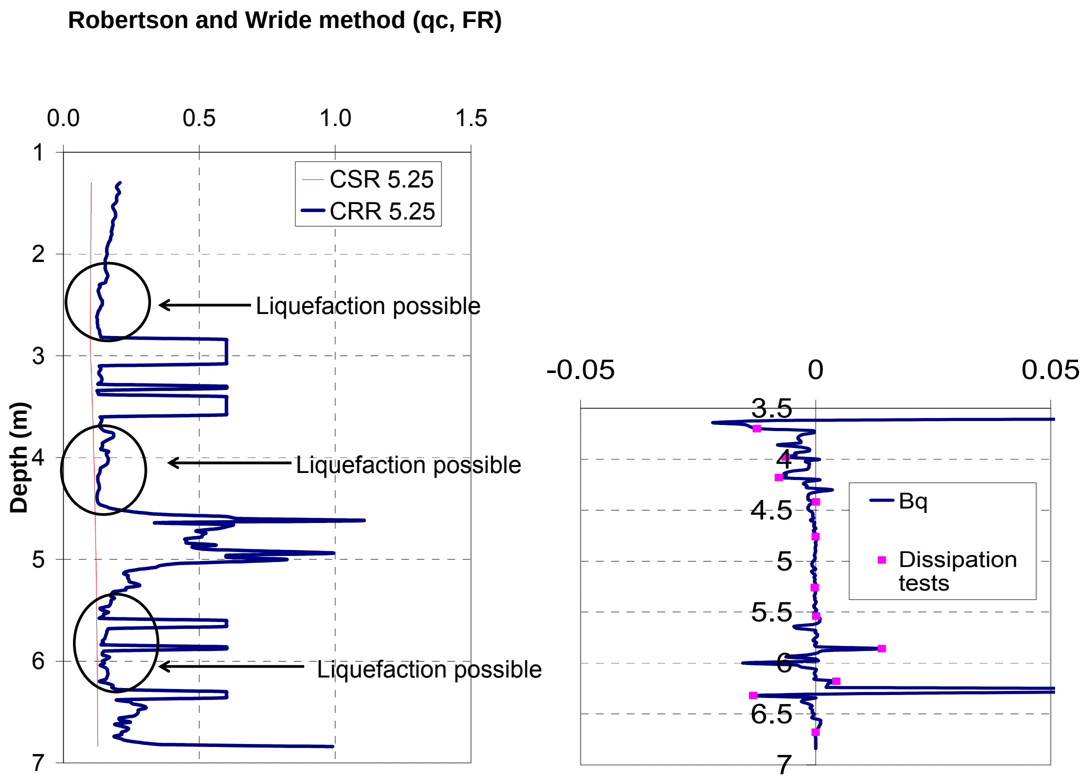


Figure 5. (a) liquefaction resistance evaluated for the HMG site using CPT data; (b) dimensionless parameter Bq with depth from piezocone data

In figure 5(a), the results of the liquefaction analysis using cone penetration data (method Robertson and Wride) are presented. 3 potentially liquefiable zones are identified: from 2.3 to 2.9 metres, from 3.6 to 4.5 metres and from 5.5 to 6.2 metres. Along these depth intervals, the CRR becomes very close to the CSR. In figure 5(b), the dimensionless coefficient B_q (calculated from equation (3)) and the dissipation test locations are plotted as a function of depth.

Liquefaction analysis as a function of piezocone data was undertaken using two different approaches: firstly, the B_q values were plotted against q_c values and situated on the chart from Berthelot and Durand (2004). Secondly, 11 data sets corresponding to the dissipation test locations and containing Δu, q_c, and t_{50} were analysed in order to calculate the dimensionless parameter ΔU, from equation (4), and plot it as a function of q_c. Figure 6(a) shows the results of the first approach. The liquefaction risk zone consists of the soils defined as loose sand, fine sand and silty sand. Points from all 4 layers penetrated by the piezocone plot in this zone, with a particular concentration of points from 5.4 to 6.4 metres and 3.6 to 4.5 metres in the fine and silty sand part of the zone. A cluster of points from the layer 3.6 to 4.5 metres plot just outside of the apparent liquefaction risk zone (low, negative B_q and low q_c) although this depth interval (from 4.2 to 4.5 metres) is considered to be potentially liquefiable by previous analysis.

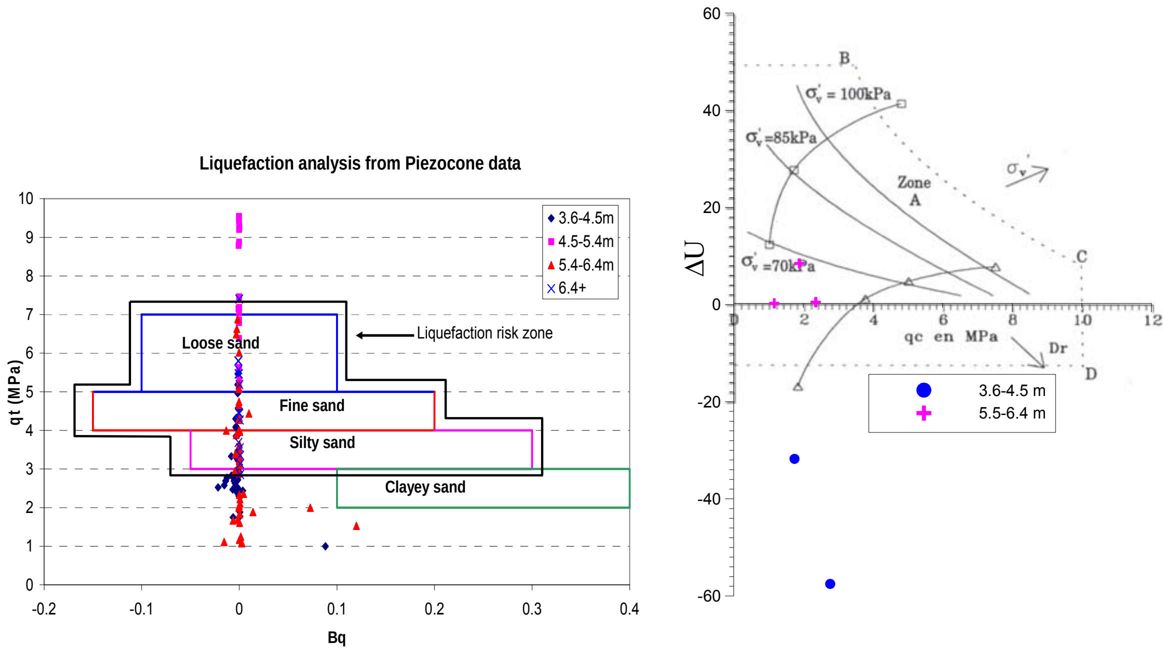


Figure 6. Liquefaction analysis from piezocone data using criteria by (a) Durand and Berthelot; (b) Bouguerra (with dimensionless ΔU from Berrill (1992))

Figure 6(b) shows results of the second approach, based on a delineated "liquefiable" zone (within dotted lines) in q_c-ΔU space, with curves of constant vertical confining pressure, σ'_v, and constant D_r. This diagram was developed in the PhD thesis of Bouguerra (1997), who used the results of calibration chamber piezocone tests on New Zealand sand. Points in gravel layers, with t_{50}=0, could not be plotted. Of the two non-gravel layers, the only liquefaction risk identified is between 5.4 and 6.4 metres. Points from the upper layer (silty sand) plot well outside the "liquefiable" zone, in contrast with the analysis using q_c and FR in which this layer appears to be critical in terms of liquefaction potential. The three points plotting within the "liquefiable" zone correspond to depths of 5.54, 5.86 and 6.18 metres. 5.86 metres corresponds to a thin layer of sandy silt, probably non-liquefiable. However, 5.54 and 6.18 metres are located in loose silty sand layers, and are considered equally critical by the q_c and FR analysis. The overall lower permeability of this layer has allowed it to be considered using the present criteria, as it is the only layer where t_{50} values are significant and Δu values are positive.

The liquefaction analysis using piezocone results has resulted in a more detailed study of the HMG profile. The Berthelot and Durand method indicates that a significant part of the 3.6 to 4.5 metre layer is non-liquefiable, despite contrary evidence shown by the CPT method. The liquefaction resistance should be confirmed by laboratory tests, and eventually an extension of the "liquefaction risk zone" in figure 6(a) to include $2 < q_c < 3$ MPa could be envisaged. As regards Bouguerra's method, the major disadvantage is the smaller number of points that can be plotted, since the parameter t_{50} is incorporated, and can only be obtained at discrete points in the profile. The current diagram does not take into account vertical overburden pressures smaller than 70 kPa, whereas in the layer 3.6 to 4.5 metres the σ'_v is 60-65 kPa. In this layer the lower overburden pressure has resulted in negative excess pore pressures, and, with t_{50} of the order of 1-3 seconds, the calculated ΔU becomes very sensitive to even small negative pressures. Points from this layer are thus plotted outside of the "liquefiable" zone. Finally, the presence of gravel layers and the frequency of dissipation tests in a piezocone profile of only 3.2 metres depths means that Δu and t_{50} values may not represent well enough the contractive, dilative or permeable nature of the silty sands.

Potentially liquefiable layers identified at the Physics and HMG sites
Combined CPT and Vs liquefaction analysis at the Physics site shows that liquefaction potential exists in the loose silty sand from 3 metres down to 9.8 metres, and that this potential increases with depth. Liquefaction prediction by V_s proved to be the less conservative method in this case. Combined CPT and piezocone liquefaction analysis at the HMG site shows liquefaction potential in silty sand layers between 2.3 and 2.9 metres, 3.6 and 4.5 metres and 5.5 and 6.2 metres.

LABORATORY TESTING: HMG SAMPLES

The depths of the intact core samples taken beside the piezocone test at the HMG site correspond to potentially liquefiable depth intervals identified during the liquefaction analysis. A series of undrained cyclic triaxial tests using these samples has been programmed. Samples will be taken from the critical depths and consolidated isotropically to their in-situ vertical confining pressure. Cyclic tests will then be performed at different levels of cyclic stress ratio in order to enable the construction of liquefaction resistance curves, and thus the triaxial cyclic resistance ratio $(CRR)_{triax}$ for a magnitude 5.25 earthquake (equivalent to 3 cycles). After applying the appropriate correction factors, the equivalent field CRR can be plotted with the q_{c1} value obtained from the CPT, representing a true, non-empirical liquefaction resistance to be compared with the empirical curves currently used for liquefaction analysis. The testing program is currently delayed due to repairs on equipment.

Grain size analyses have been performed on the cuttings (disturbed samples) obtained during the drilling between 2.8 and 2.9 metres, and 4.3 and 4.5 metres, which correspond to potentially liquefiable depth intervals. The grain size distribution curves are situated with respect to the limit grain size distribution curves for liquefaction after Iwasaki et al. (1986), in figure 7.

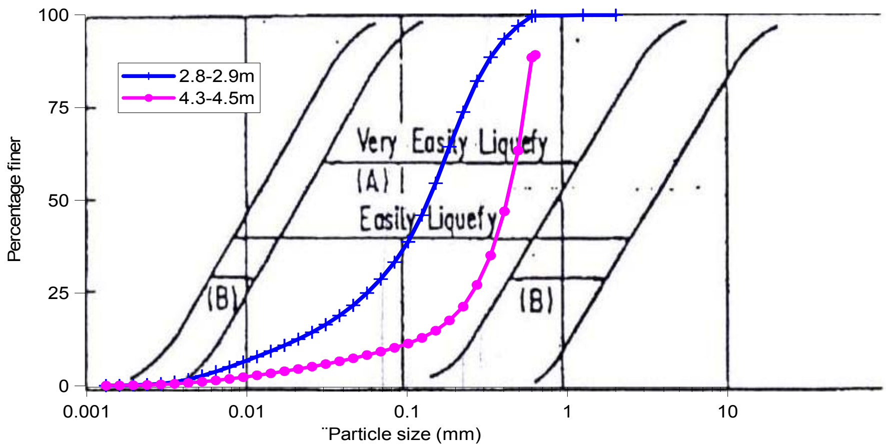


Figure 7. Grain size distribution curves for HMG cuttings, plotted on chart of grain size limits for liquefaction by Iwasaki et al. (1986)

Both curves have the majority of their points within the "very easily liquefy" limit. The sample from 2.8 to 2.9 metres is classified as a well graded, very silty sand with a mean grain diameter, d_{50}, of 0.12 mm, whilst the sample from 4.3 to 4.5 metres is a uniform silty sand with $d_{50} = 0.41$ mm. These results are in keeping with the in situ liquefaction analyses performed. Finally, the percentage of fine-grained material (diameter < 0.074mm by the Unified Soil Classification System) obtained during the grain size analysis is compared with the percentage of fines estimated using the I_c of Robertson and Wride. For the sample between 2.8 and 2.9 metres, the fines content obtained from the grain size distribution is 32%, compared with 37% estimated by the I_c method. For the sample between 4.3 and 4.5 metres, the fines content is 10%, compared with 12% estimated by the I_c method. These results confirm to some extent the valid use of the index I_c for estimating soil grain characteristics in situ.

NUMERICAL STUDY OF LIQUEFACTION BEHAVIOUR

The program of field and laboratory testing is to be completed by numerical modeling of the Grenoble university campus in situ profiles. The model used is that of Iai et al. (1992), defined in strain space and capable of describing the undrained cyclic behaviour of cohesionless materials, including cyclic mobility and liquefaction. The model incorporates the concept of a "liquefaction front" to describe liquefying behaviour. Laboratory-based correlations between cumulated shear work and pore water pressure rise during undrained cyclic loading (Towhata and Ishihara, 1985) have been used to describe the progression towards liquefaction. An intrinsic parameter in this formulation is S_0, the "liquefaction front" parameter, correlated to the normalised shear work through 5 parameters. These 5 parameters, known as the "dilatancy" parameters, must be obtained by calibration of the model with undrained cyclic laboratory test results, and therefore proper use of the model requires the carrying out of such tests on intact samples taken from the soil profile of interest. On completion of the triaxial testing program for the HMG intact samples, numerical modeling of the profile will be carried out and the dilatancy parameters capable of describing liquefaction will be obtained for layers from 2.3 to 2.8 metres and 3.6 to 4.5 metres. A typical earthquake signal for the Grenoble region will be input at the base of the profile to model the dynamic response.

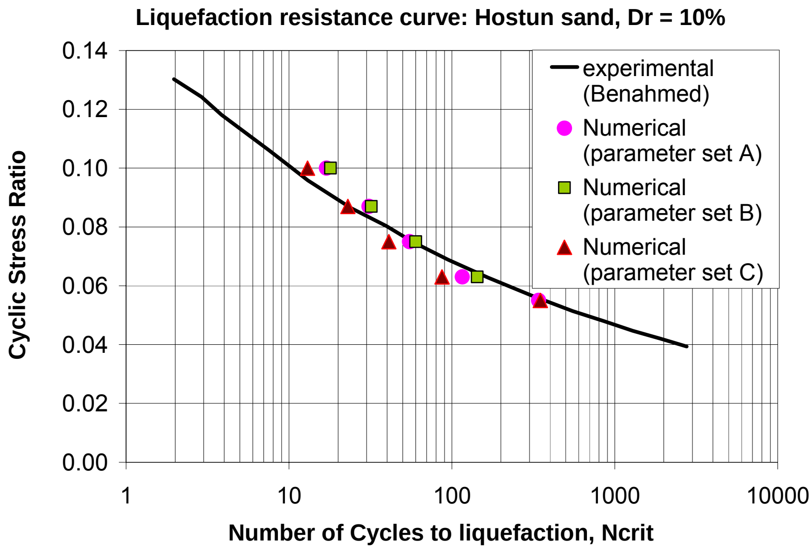


Figure 8. Calibration of parameters for cyclic model describing liquefaction using Hostun sand triaxial test results from Benahmed (2001)

An example of dilatancy parameter calibration is illustrated in figure 8; the liquefaction resistance curve for a series of undrained cyclic triaxial tests on Hostun sand with D_r = 10% was obtained from the PhD thesis of Nadia Benahmed (2001). A best fit for this curve was attempted using different sets of dilatancy parameters to model the cyclic triaxial tests performed at different levels of τ_{cyc}/σ'_0 , in each case aiming to obtain the number of cycles to liquefaction, N, as close as possible to the experimental case. The model was used in the finite difference code NOAH (Bonilla 2000), and the parameter set chosen in this case was set A. The same method will be applied to the triaxial test results for the intact HMG samples for modeling of the profile.

CONCLUSION

This paper presents a liquefaction study based on CPT, seismocone and piezocone data from two different sites on the Grenoble university campus, the Physics site and the HMG site. Liquefaction potential was identified in loose silty sand layers between 2.3 and 9.8 metres at both sites by the Robertson and Wride method. Piezocone data for the HMG site was more precise in identifying thin silty sand layers, but liquefaction analyses using piezocone data were shown to require further development in order to be implemented in a wide range of soil profile types. Layers of loose silty sand exhibited negative excess pore water pressures, and values of t_{50} were consistently low due to the presence of permeable gravel layers. Laboratory testing on soil samples obtained at the HMG site has so far been restricted to grain size analyses on disturbed samples at depths of interest; the results provide further evidence for the liquefaction susceptibility of this profile. This study will be completed by the imminent cyclic triaxial testing program on intact HMG samples, and the laboratory results will also be used for numerical modeling of the profile in order to carry out a non-empirical liquefaction analysis.

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