A Critical State Evaluation of Fines Effect on Liquefaction Potential

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ABSTRACT

Published results from laboratory tests show that an increase in the percentage of fines generally leads to a reduction of the cyclic liquefaction resistance of a sand, while empirical correlations from in-situ tests consider the presence of fines as beneficial. In order to study this seemingly not univocal effect of fines content, this paper involves the integrated framework of Critical State Soil Mechanics. For this purpose, firstly the effect of fines on the location of the Critical State Line (CSL) is studied through statistical analysis of a large data set of triaxial element tests. Results show that fines affect the CSL location in the (e-lnp) space, but not its slope in (p-q) space. In particular, an increase of fines content practically leads to a clockwise rotation of the CSL in (e-lnp) space. Introducing this finding as a mere change in parameter values of an appropriate Critical State constitutive model, simulations of cyclic undrained triaxial tests were performed. These simulations show that the presence of fines is beneficial at relatively small effective stresses, i.e. the stresses prevailing at liquefiable layers in-situ. Furthermore, these simulations show that the effect is reversed at relatively large effective stresses, i.e. the stresses usually considered in laboratory tests.

INTRODUCTION

The effect of fines content f(%) on the liquefaction potential of sands has been studied extensively in geotechnical literature. Emphasis has been also set on whether the fines are plastic or not. Despite the amount of related research, results seem somewhat contradictory. Namely, empirical correlations from in-situ tests show that the presence of fines increases liquefaction resistance (e.g. Seed et al. 1985), while liquefaction laboratory tests mostly show the opposite trend, at least for $f \leq 30\%$ (e.g. Troncoso 1990, Koester 1994). For sands with $f > 30\%$, laboratory results mostly show increase in liquefaction resistance (e.g. Troncoso 1990, Koester 1994).

Published research results regarding the effect of plasticity of the included fines on liquefaction resistance seem also contradictory. For example, Koester (1994) claims that the plasticity index I_P of the included fines is less important than the fines content f(%) in estimating liquefaction resistance, contrary to Prakash et al. (1999) who claim that fines with high plasticity fundamentally change the mechanism of excess pore pressure buildup. This paper focuses on the effect of f(%) alone, which is in itself a relatively complicated issue.

Available interpretations for the effect of fines content on liquefaction resistance are based on mechanisms of deformation in the particle size level. Factors such as soil fabric and aging have been considered responsible for the increase of resistance of undisturbed specimens compared to reconstituted (e.g. Yoshimi et al. 1989). This may be the reason why empirical correlations from in-situ tests consider the presence of fines as beneficial for liquefaction resistance. When the effect of aging is being removed by using reconstituted specimens, micromechanical interpretations of laboratory test results suggest that fines in small percentages ($f<30\%$) mostly take up the space between sand particles without contributing to soil strength. This leads to a decrease of void ratio e without any particular change in soil behavior. In this manner, the decrease in liquefaction resistance with fines percent f(%) is considered an artifact of considering soils with the same e value, and not the same void ratio of the sand skeleton e_{SK}, a more representative index of behavior (e.g. Vaid 1994, Thevanayagam 1998). Similarly, when f increases to values greater than 30%, sand behavior depends greatly on the included fines, which contribute their share in soil strength leading to the measured increase in liquefaction resistance (e.g. Vaid 1994, Thevanayagam 1998).

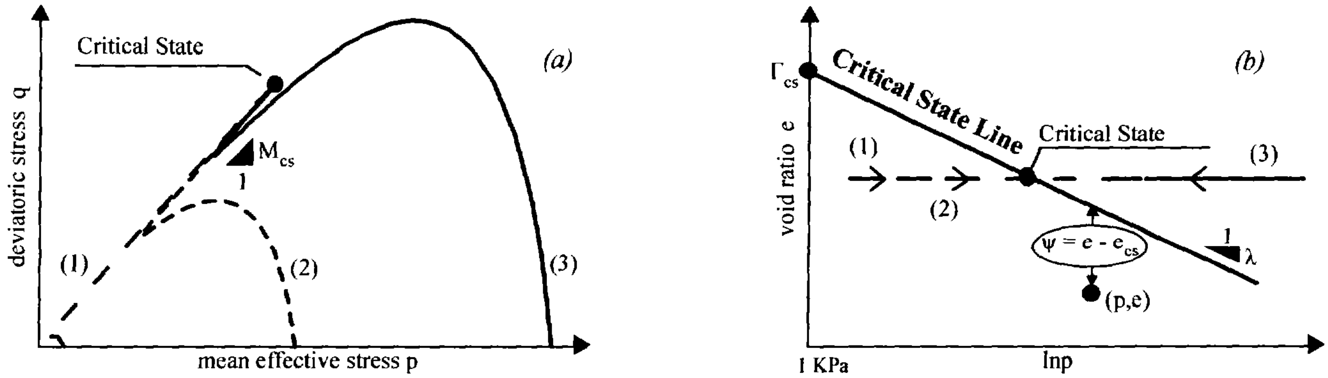

Fig. 1. Effect of initial conditions (e, p) in monotonic undrained triaxial response of sand in CSSM context

These micromechanical explanations provide insight to the phenomenon, but fail to draw the full picture. For example, laboratory results for tests on reconstituted specimens performed at relatively small mean effective stresses (i.e. similar to those expected in potentially liquefiable layers at generally shallow depth) show increase of liquefaction resistance with f(%), which can't be attributed to aging (e.g. Yasuda et al. 1994). This is an indication that the phenomenon is very complicated to be treated merely on the micromechanical level and that it should also be interpreted on the basis of an integrated framework of mechanical behavior.

Aiming to fill this gap, this paper involves the integrated framework of Critical State Soil Mechanics (CSSM, e.g. Roscoe et al. 1958) in an attempt to view this practical issue from a different perspective.

OVERVIEW OF CRITICAL STATE CONCEPTS

For ease of interpretation, the analysis is performed in triaxial stress-strain space. In other words, all equations and results are presented in terms of the mean effective and deviatoric stresses: $p=(\sigma_v+2\sigma_h)/3$, $q = (\sigma_v - \sigma_h)$. It is noted, that subscripts v and h denote the vertical and horizontal planes respectively. As a good approximation, the Critical State Lines (CSL) of a specific soil in the (e – lnp) and the (p – q) space can be assumed as straight and unique, at least for stresses not causing particle crushing (e.g. Been et al. 1991). Uniqueness refers to independence of the CSL from drainage conditions, sample preparation method and strain rate, i.e. the testing conditions. In this sense, the terms Steady State and Critical State are considered interchangeable.

Figures 1a & 1b present the Critical State Lines (CSL) in the (e – lnp) and the (p – q) spaces respectively. These lines are described by the following equations:

$$e_{CS} = \Gamma_{CS} - \lambda \ln p \qquad (1)$$

$$q = M_{CS}\, p \qquad (2)$$

where, e_{CS} is the void ratio at Critical State under mean effective stress p. Furthermore, Figures 1a & 1b show examples of how monotonic undrained behavior of sand is influenced by initial conditions, i.e. the values of p and e. In an attempt to fully interpret the effect of initial conditions with a single parameter, Been & Jefferies (1985) introduced the State Parameter ψ, as:

$$\psi = e - e_{CS} \qquad (3)$$

From Figure 1 it becomes evident, that when $\psi < 0$ sand behavior is dilative, while the opposite is observed (i.e. contractive behavior) when $\psi > 0$.

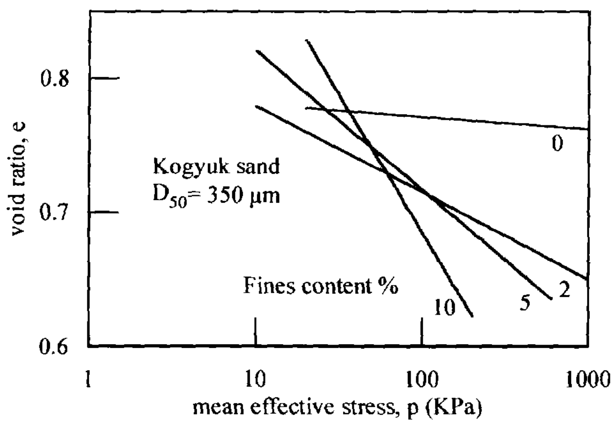

Fig.2 Effect of fines content on CSL location of Kogyuk Sand (After Been & Jefferies, 1985)

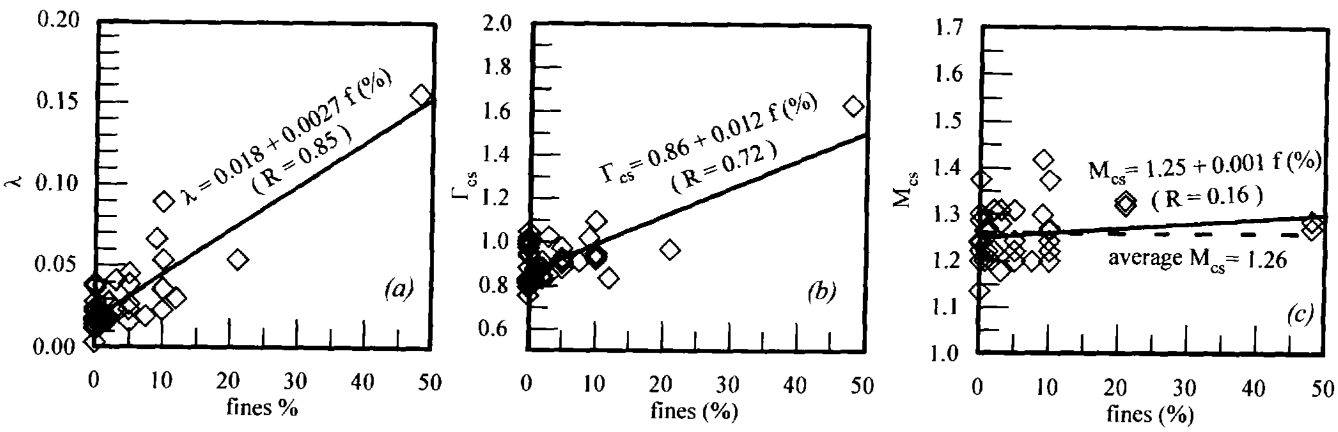

Fig.3 Results of statistical analysis regarding the effect of fines content on Critical State Line location

EFFECT OF FINES CONTENT ON CSL LOCATION

Before attempting to interpret analytically the effect of f(%) on liquefaction resistance, it is crucial to study how f(%) affects the corner stone of the analysis, i.e. the Critical State Line location. More specifically, while the Critical State Line in both (e-lnp) and (p - q) spaces has been measured independent of testing conditions, laboratory evidence show dependence on the value of f(%). For example, Figure 2 shows the Critical State Lines for Kogyuk sand with different f (%) values (Been & Jefferies 1985). This plot shows an example of increasing λ and Γ_{CS} as a function of f (%). The issue of whether this commonly measured trend of increasing λ and Γ_{CS} values is representative for all sands has not been fully established in the literature, since there is some evidence to the opposite (e.g. Chen & Liao, 1999). On the other hand, the potential effect of fines content on M_{CS} is an issue that has not drawn much attention in the literature.

In this study the above issues are addressed in detail with the aid of experimental data from 42 different soil types. The raw data include Γ_{CS}, λ and M_{CS} values for different sands with various values of f(%), and were communicated to the authors by Dr. M. G. Jefferies. It should be noted that soils consisting of the same host sand with different values of f(%) are treated as independent soils in the analysis.

The results of the statistical analysis of the raw data for Γ_{CS}, λ and M_{CS} as a function of f(%) are presented in Figure 3. More precisely, Figures 3a & 3b show that both Γ_{CS} and λ increase with f(%), while Figure 3c shows that M_{CS} could be practically considered constant. Average values of Γ_{CS} and λ for clean sands are 0.86 and 0.018 respectively, while an average value of M_{CS}=1.26 is considered appropriate for all sands with f(%) values ranging from 0 to 48%.

The empirical relations between Γ_{CS}, λ and fines content (%), that are derived from the statistical analysis, are the following:

$$\lambda = 0.018 + 0.0027\, f(\%) \qquad (4)$$

$$\Gamma_{CS} = 0.86 + 0.012\, f(\%) \qquad (5)$$

If empirical relations (4) & (5) are combined, they lead to the following relation between Γ_{CS} and λ:

$$\Gamma_{CS} = 0.78 + 4.45\, \lambda \qquad (6)$$

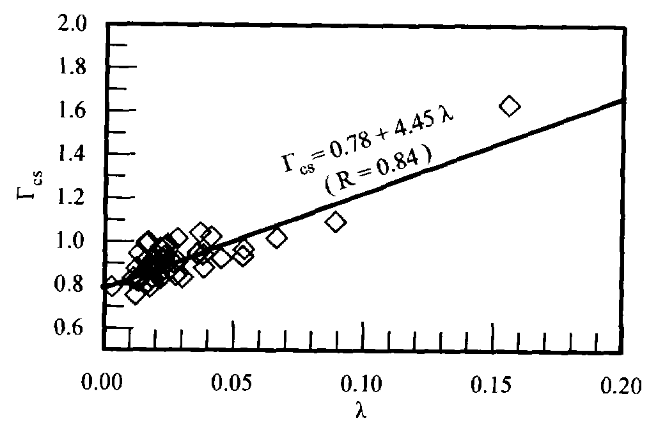

Fig.4 Correlation between Γ_{cs} & λ

As shown in Figure 4, empirical relation (6) compares well with the raw data (R = 0.84) and is thus considered reliable and representative for silty sands. Combining empirical relation (6) with equation (1) leads to the following equation:

$$(e - 0.78) = \lambda\,(4.45 - \ln p) \qquad (7)$$

Equation (7) yields the CSL locations for all considered soil types as a function of the corresponding slope λ. Moreover, Equation (7) shows that the CSL of all considered soil types pass through point ($p[kPa]=e^{4.45}$, $e=0.78$), independently of the value of λ. Figure 5 shows exemplary CSL for sands with different f(%) values, based on Equation (7). It is deduced, that the effect of increasing fines content f(%) on the CSL location in (e-lnp) space can be alternatively viewed as a clock wise rotation around this pivot point (p, e) = (85.6 KPa, 0.78). This pivot point is not considered as some universal constant and is clearly a function of the available data set. Nevertheless, the performed analysis provides insight in the phenomenon and yields reliable estimates of average CSL locations that could be used in lack of actual test data.

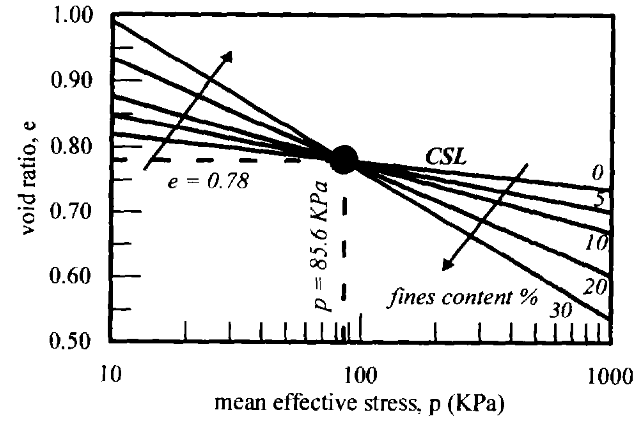

Fig 5. Effect of fines content on CSL in (e-lnp) space

EFFECT OF FINES CONTENT ON CYCLIC UNDRAINED RESPONSE

From Figure 5 it is deduced, that two soil samples containing the same clean sand but with different f(%) values, if consolidated to the same initial conditions (p_o, e_o) correspond to different ψ values. For example, a sample of silty sand ($f \neq 0\%$) with $p_o < 85.6$ kPa is expected to behave in a more dilative manner than a clean sand sample, consolidated to the same initial conditions. As a result, the clean sand sample is expected to portray lower cyclic liquefaction resistance due to the relatively smaller $|\psi|$ value. In other words, liquefaction resistance is expected to increase with f(%) for samples with relatively small mean effective stresses ($p_o < 85.6$ kPa). In this context, the effect of f(%) on liquefaction resistance is reversed for samples with relatively large mean effective stresses ($p_o > 85.6$ kPa).

Laboratory evidence of this reversing trend is presented in Figures 6a & 6b, where the effect of f(%) is studied by element tests consolidated at $p_o = 49$ and 196 kPa respectively. More specifically, in Fig. 6a tests with low mean confining stress (p_o=49 KPa) show than increasing f(%) leads to an increase of the cyclic liquefaction resistance, while in Fig. 6b tests with large mean effective stress (p_o=200KPa) show the opposite effect. In this way, the beneficial effect of f(%) on liquefaction resistance from in-situ test correlations can also be interpreted, since these results originate from liquefiable layers close to the surface (with p_o rarely exceeding 100 kPa).

These preliminary conclusions are verified from analytical simulations obtained from an appropriate Critical State model. For this purpose, the constitutive model recently proposed by Papadimitriou et al. (1999) has been employed. This model has been shown to provide accurate cyclic liquefaction test simulations for both triaxial and simple shear tests consolidated under various p_o and e_o values with the same set of parameters (Papadimitriou, 1999). Furthermore, the selection of this model was based on the fact that the parameters governing the CSL location, i.e. parameters Γ_{CS}, λ and M_{CS}, are user specified.

This model has been used to perform parametric simulations of the undrained cyclic triaxial behavior of two soils, a clean sand (f = 0%) and the same sand with f =5, 10, 20, 30 %. More specifically, the values of Γ_{CS}, λ and M_{CS} used in the analysis are provided in Table 1:

Soil	Γ_{CS}	λ	M_{CS}
Sand with f = 0%	0.86	0.018	1.26
Sand with f = 5%	0.92	0.032	1.26
Sand with f = 10%	0.98	0.045	1.26
Sand with f = 20%	1.10	0.072	1.26
Sand with f = 30%	1.22	0.099	1.26

Table 1. Model parameters for simulating the effect of f(%)

Two sets of analyses are performed for each soil type, one with p_o=50kPa, i.e. smaller than that of the pivot point ($p_o \leq$ 85.6kPa) and another with p_o=200kPa, i.e. larger than that of the pivot point ($p_o >$ 85.6kPa). The same void ratio is used for all analyses, e = 0.70, a value corresponding to relatively loose, potentially liquefiable, sand. As an example, Fig. 7 compares the analytical simulations in terms of the excess pore pressure Δu , as a function of number of cycles N. In particular, Figures 7a and 7b compare the simulated behavior of a clean (f=0%) and a silty sand (f=20%), for p_o = 50 and 200 kPa, respectively. For completeness, Fig. 8 compares the liquefaction curves derived from analytical simulations of triaxial tests on sands with f = 0, 5, 10, 20, 30 % under initial conditions (p_o = 50 & 200 kPa, e = 0.70). It is noted that stress amplitude q_c in Figure 8 refers to the applied $|\Delta\sigma_v|/2$ within each load cycle of the simulated tests.

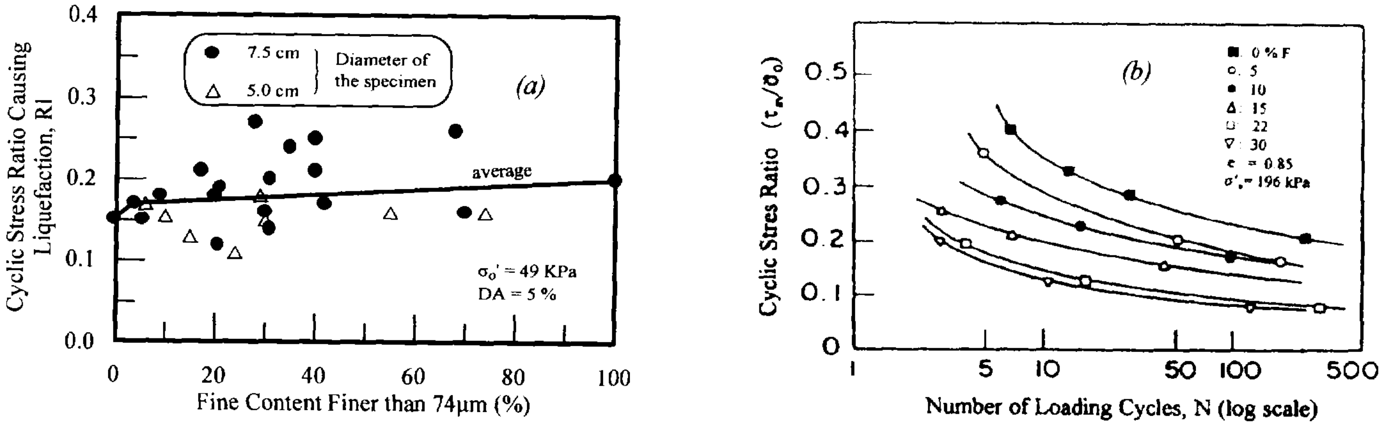

Fig 6. Effect of fines content on cyclic liquefaction resistance:
a) based on Yasuda et al. (1994), b) Troncoso(1985)

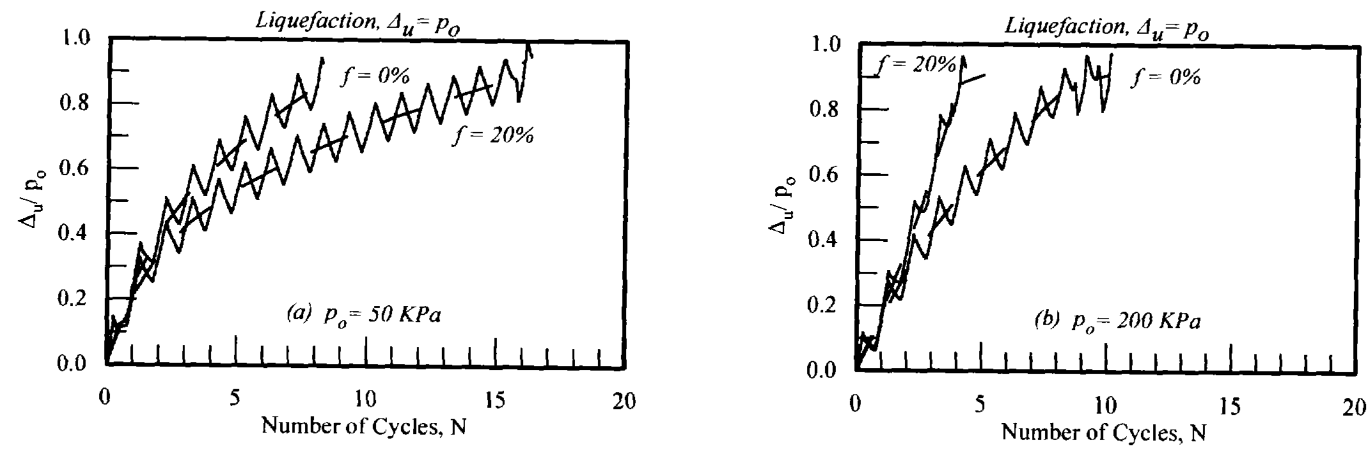

Fig 7. Excess pore pressure buildup during simulated cyclic undrained triaxial tests on sands with f=0% and f=20%

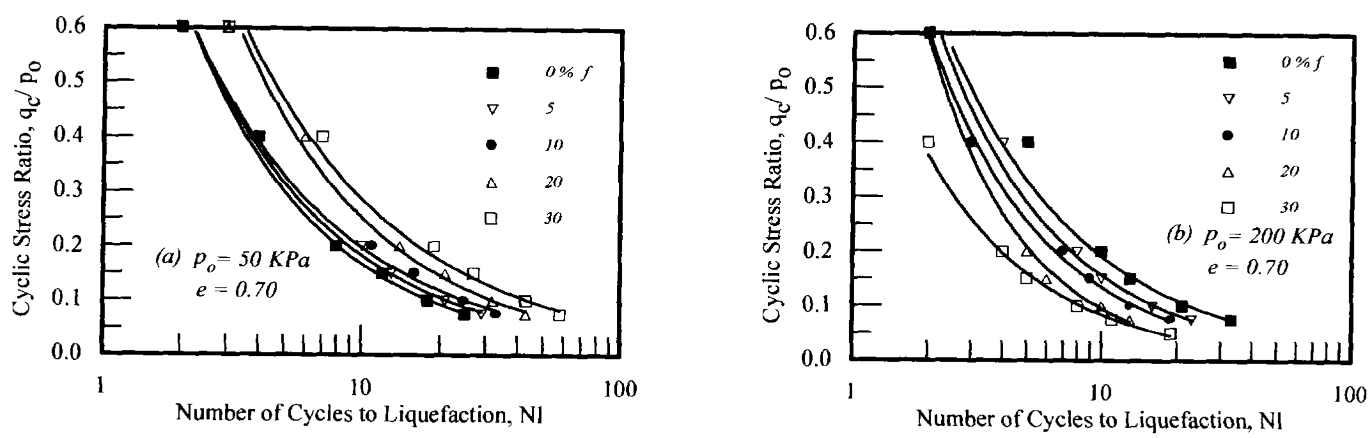

Fig.8 Analytically estimated liquefaction curves for sands containing different percentages of fines f (%)
(Symbols correspond to performed triaxial test simulations)

From Figures 7 & 8, it is deduced that the non-univocal trend regarding the effect of fines on cyclic liquefaction resistance that was inferred earlier, is fully verified by analytical simulations. Namely, the simulations for low p_o values show that cyclic liquefaction resistance increases when fines are added to the clean sand. On the contrary, simulations for relatively high p_o values show that the effect of fines is reversed.

CONCLUSIONS

This paper analyses the effect of fines content f(%) on cyclic liquefaction resistance in a Critical State Soil Mechanics context. The effect of the plasticity of the included fines is not taken into consideration. Results show that the effect of f(%) is not univocal, but depends on initial conditions. Moreover, the influence of initial conditions is attributed to the parallel effect of f(%) on the Critical State Line location, an issue studied statistically in this paper as well.

More specifically, the statistical analysis of laboratory results shows that the average expected location of the Critical State Line for silty sands practically rotates clock-wise with increasing f(%) around a pivot point in (e – lnp) space. This pivot point is estimated approximately at (p, e) = (85.6 kPa, 0.78). On the other hand, the slope of the Critical State Line in (p - q) space does not seem to be affected by f(%).

By modelling the presence of fines as a mere rotation of the Critical State Line location in the (e-lnp) space, analytical simulations of undrained cyclic triaxial tests were performed. These simulations showed that the presence of fines improves liquefaction resistance under mean effective consolidation stress p_o less than approximately 86 kPa, i.e. stress levels usually prevailing on potentially liquefiable layers at shallow depth. In parallel, the analytical simulations showed a decreasing liquefaction resistance with increasing f(%) for higher consolidation stress levels. This finding gives an integrated interpretation of the effect of fines content and is supported by laboratory test results as well as field evidence.

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