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CRITICAL SHEAR AMPLITUDE AND LIQUEFACTION IN CYCLIC LOADING

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ABSTRACT

The behaviour of sand under cyclic loading conditions has been investigated. A series of cyclic triaxial tests with various shear stress amplitudes has been carried out on a loose sand for the same initial conditions. The existence of a critical cyclic shear amplitude which separates contractant and dilatant behaviour is shown. Below this value, and for a given density, liquefaction cannot occur irrespective of the initial consolidation pressure. For higher values, the material exhibits contractant behaviour only, and after a finite number of cycles the threshold collapse surface defined by Sladen (1985) is reached. The number of cycles needed for liquefaction depends on the initial consolidation stress and on the cyclic shear amplitude. A sand behaviour diagram is suggested.

KEYWORDS

Triaxial tests, sand, cyclic liquefaction, cyclic mobility,

INTRODUCTION

A large number of studies have been reported concerning the failure of saturated sandy soils which are subjected to a shear load in undrained conditions. According to the steady state concept (Castro, 1969; Poulos, 1981), the behaviour of soils under any loading condition is dominated by its initial state relative to the steady state locus. Castro (1969) has shown that, for a given initial consolidation pressure, a sand in undrained monotonic compression triaxial tests, could show three different types of behaviour. Depending on the initial void ratio, liquefaction, limited liquefaction and dilatant behaviour can occur. Sladen and Oswell (1989) summarised the effect of the initial soil state on the behaviour by an illustration in a normalised stress space. The parameters of this normalisation are the effective stresses in the steady state (p'_{cs}, $M\,p'_{cs}$). All stress paths then end at the point (p'_{cs}, $M\,p'_{cs}$) = (1,1). Soils which are initially very loose ($p'_0/p'_{cs} >> 1$, where p'_0 is the initial effective mean stress) are completely contractive. In this case, Sladen *et al* (1985) have shown that the peaks of the stress paths obtained from the same void ratio and from different consolidation pressures in undrained monotonic compression triaxial tests, form a fairly straight line through the steady state point. For different void ratios this line becomes a surface called the collapse surface. This collapse surface represents the locus of points in the stress space where liquefaction is triggered. If the soil has an initial ratio p'_0/p'_{cs} smaller than one, it shows a small contractive phase, followed by significant dilatant phase when the stress path crosses the phase transformation line (Ishihara *et al,* 1975). If this ratio is slightly larger than one, the soil shows the limited behaviour as described by Castro. In this case the stress path shows a peak deviator, a post-peak minimum, followed by a dilative phase. The locus of the post-peak minima form the quasi-steady state (Alarcon-Guzman *et al,* 1988; Ishihara, 1993).

As has been suggested by Roscoe and Poorooshab (1963) for cohesive and granular media, and by Been and Jefferies (1985) for sandy soils, the soil behaviour in monotonic loading can be directly related to the difference between the initial void ratio and the void ratio at the critical state for the same mean stress level. This difference has been called the state parameter ψ by Been and Jefferies (1985). If ψ is positive, the sand is contractant and liquefaction can occur. If ψ is negative, dilatancy dominates the sand behaviour. The state parameter approach has also been used by Ishihara (1993) to define his state index Is. However, this parameter is related to the quasi-steady state and not to the steady state. It also takes into account the consolidation and the fabric effects by using a second reference line in the void ratio-effective confining stress space (e, p'). This reference line is the isotropic consolidation line in the loosest state obtained by a given deposit method.

Using detailed triaxial test results, this paper will show that, for cyclic loading conditions, these parameters are not sufficient to describe the cyclic behaviour of soils, and will show the existence of a critical cyclic shear resistance which bounds the domain of liquefaction.

EXPERIMENTAL PROCEDURE AND SAND CHARACTERISTICS

Triaxial tests have been performed. In order to show that, with the same initial conditions, two different types of sand behaviour can be observed, the initial void ratio and the isotropic consolidation pressure have been fixed (and therefore the state parameter). As the tests are stress-controlled, the amplitude of the deviatoric stress is taken as the variable test parameter. The tests were conducted using a cyclic triaxial shear apparatus on loose samples 70 mm in diameter and 140 mm in height, under force controlled conditions. The samples were prepared by the moist tamping method which achieves high void ratios in the laboratory. After saturation and consolidation of the sample, it was subjected to a regular sinusoïdal load with a period of 30 seconds. The sand used was a fine Hostun sand with properties summarised in Tab. 1. In general, it is a silicon subangular sand with relatively uniform granulometry.

Table 1 Index properties of RF Hostun sand

Gs*	D50(mm)	Cu**	e_{min}	e_{max}
2.65	0.38	2	0.656	1

*Specific density

**Uniformity coefficient

TESTING PROGRAM AND RESULTS

Loose sand samples with a void ratio e = 0.897 (Id = 0.30) have been consolidated at σc = 400 kPa and have been subjected to a non alternating cyclic loading with a controlled deviatoric stress varying between $q_{min} = 0$ and different values of q_{max}. The table 2 summarises the tests conditions.

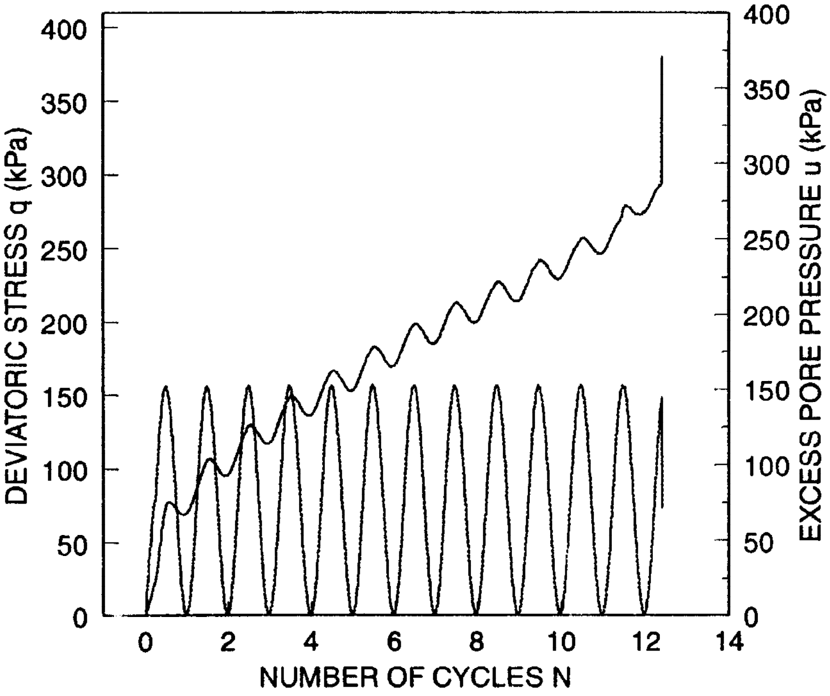
Tab. 2. Test characteristics

Essai	q_{max}	N_{crit}*	Behaviour
CYC 1	200	1	Liquefaction
CYC2	180	3	Liquefaction
CYC3	160	6	Liquefaction
CYC4	157	12	Liquefaction
CYC5	140	28	Liquefaction
CYC6	130	33	Liquefaction
CYC7	120	44	Liquefaction
CYC8	100	63	Liquefaction
CYC9	90	66	Liquefaction
CYC10	80	82	Liquefaction
CYC11	75	100	Liquefaction
CYC12	70	490	Cyclic mobility
CYC13	65	2186	cyclic mobility
CYC14	50	2070	Adaptation
CYC15	40	2900	Adaptation

*Ncrit is the number of cycles needed for liquefaction, for accommodation in cyclic mobility or for the occurrence adaptation.

Typical Cyclic Liquefaction Test

Figure 1 shows a typical result of a liquefaction test (the test CYC4). Liquefaction, as it has been described by Castro (1969) and reproduced in this test, occurs during the last loading cycle. Progressive pore pressure build-up preceding liquefaction is caused by the loose sand's contractive nature. After liquefaction, which occurs at cycle 13 in this test, the pore pressure increases suddenly to a value close to the consolidation pressure and the effective stress path reaches the residual point which corresponds to the stress minimum.



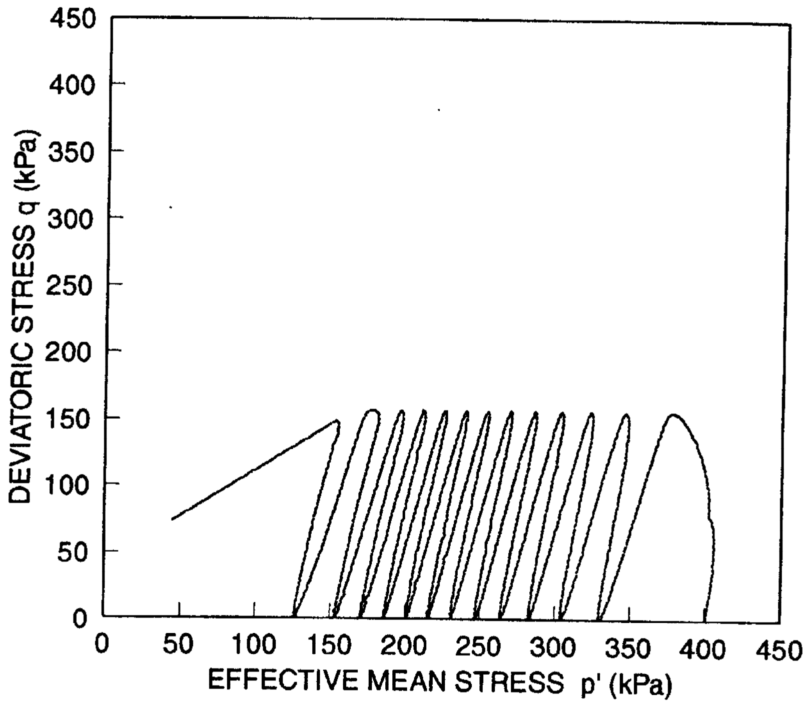


Fig. 1. Typical results of a cyclic liquefaction test on RF Hostun sand
a- (q, N) and (u,N) curves
b-stress path in the (q,p') plane

Collapse Surface

For tests where liquefaction occurs, the stress peaks and the residual state points are shown in Fig. 2. The scatter of the points presents a fairly straight line . As has been proposed by Sladen (1985), this line defines the collapse line which represents the locus of points in the (q, p') space which trigger liquefaction.

Remarks

1- Many studies have shown that the collapse line (or similar criteria) given by monotonic and cyclic liquefaction tests can be considered as unique (Vaid and Chern, 1985; Konrad, 1993; Canou *et al.*,1994). Accordingly, no difference is made in this study.

2- Several discussions have been reported about the uniqueness of the residual state (e. g. Konrad, 1990; Ishihara, 1993). The test results shown above also give different residual points. The origin of such scatter will not be discussed in the present paper. We will simply assume that the residual deviator for the fixed void ratio is around 50 kPa.

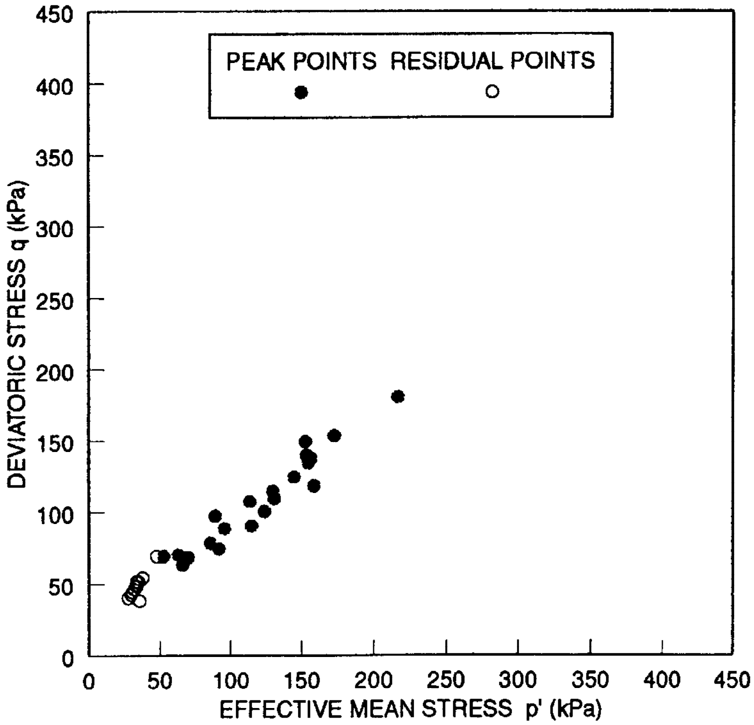


Fig. 2. Collapse line for RF Hostun sand at a given density (Id = 0.30)

Typical Cyclic Mobility Test

Figure 3 represents the results of the test CYC12. During the first part of the test, the axial strain is limited ($\varepsilon_a < 1\%$) and the pore pressure increases gradually. After a certain number of cycles, a large axial strain rate takes place in loading, but no collapse in terms of liquefaction is registered. At this point, the pore pressure decreases in loading and increases in unloading. In the stress path diagram, the effective mean pressure p' decreases continuously because of the sand's contractive nature. At the state described above, p' increases in loading. However, as soon as unloading takes place, the pore pressure increase makes the stress path bend towards the origin. The stress path shows an accommodation after a large number of cycles (about 500 cycles). This phenomenon is called the cyclic mobility and has been studied by several authors (Seed and Lee, 1966; Ishihara *et al*, 1975). It is known as typical dense sand behaviour under these loading conditions. The fact that the rate of the pore pressure increase can be negative in loading points to the existence of a dilative zone even for a loose sand. Such a dilative zone has been shown by Ishihara (1993) for monotonic undrained triaxial liquefaction tests. It is the region between the quasi-steady state and the steady state. To unify dense and loose sand behaviour, we will assume that the quasi-steady state and the phase transformation state represent the same state.

Adaptation

In tests CYC13 to CYC 15, the pore pressure increases very slowly and after a large number of cycles, the reloading process traces the same path as that along which unloading has been executed. No excess pore pressure is registered and the effective mean pressure remains very large (around p' = 300 kPa). Liquefaction does not occur.

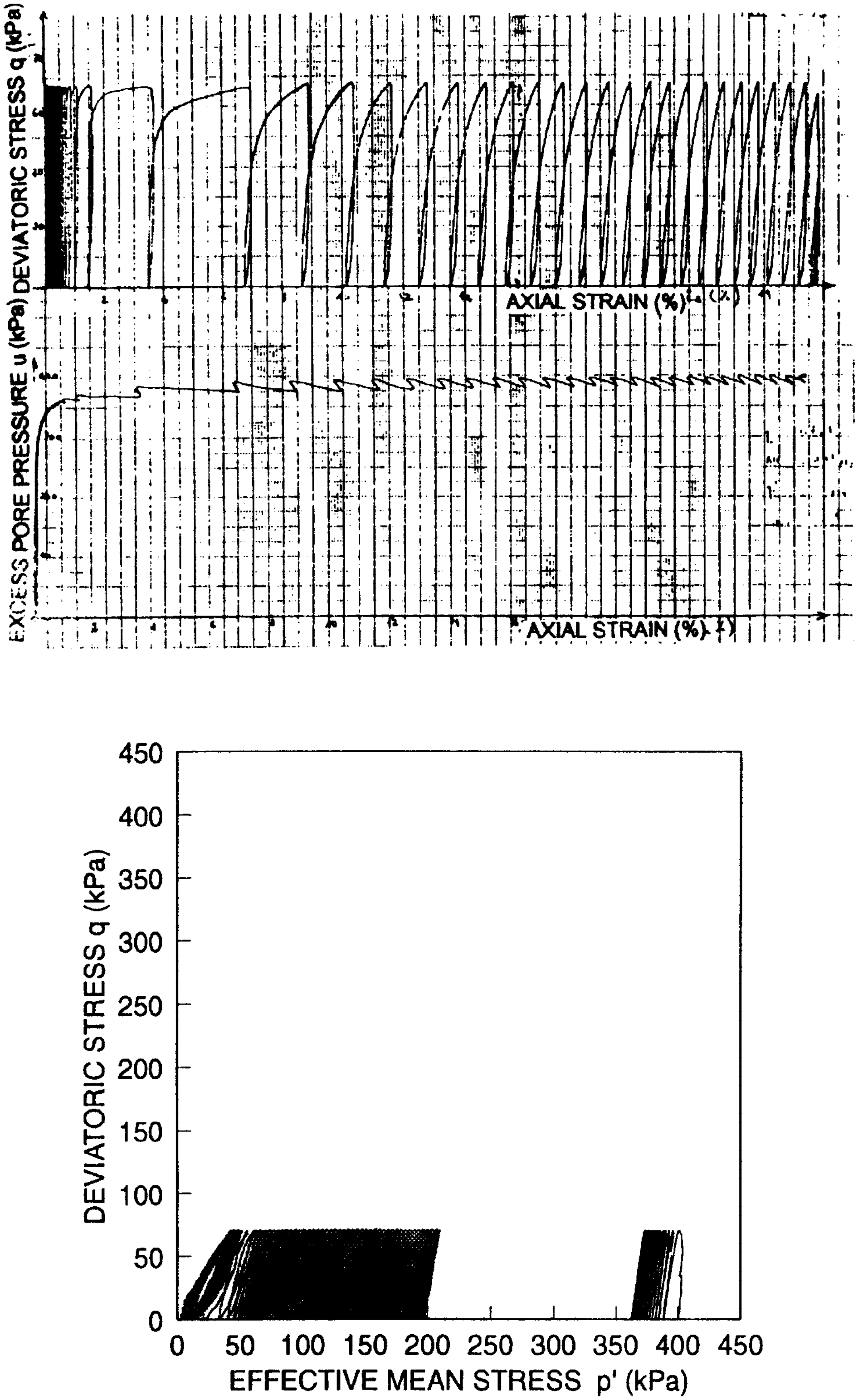


Fig. 3. Typical cyclic mobility test results
a-(q,ε_a) and (u, ε_a) curves
b-stress path in (q,p') plane

SHEAR STRESS CURVE AND LIQUEFACTION POTENTIAL

The results obtained above can be summarised using the cyclic shear stress curve (Fig. 4). This type of presentation is interesting for the evaluation of liquefaction potential. It represents the evolution of the cyclic shear stress as function of the number of cycles. It gives the shear stress in monotonic loading as well (N=1). The curve becomes asymptotic below a certain value of shear resistance. A critical shear stress can then be defined as the smallest value of cyclic shear stress for which liquefaction can still occur.

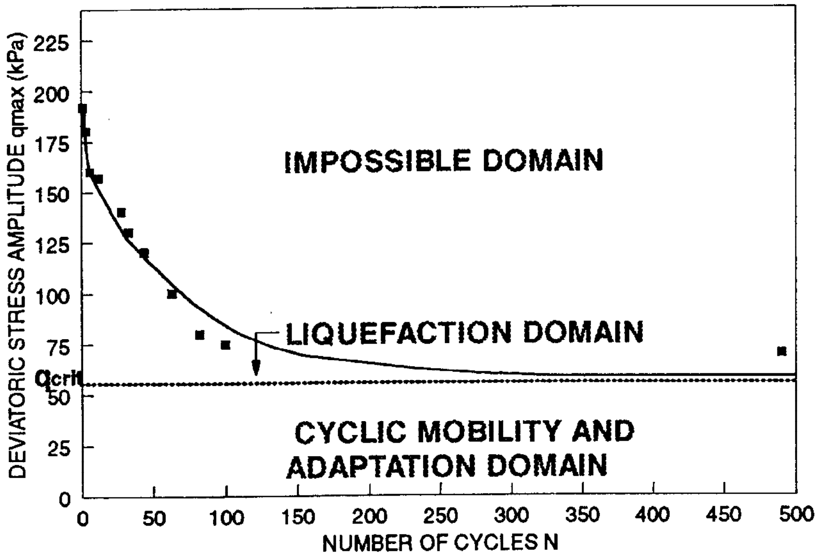


Fig. 4. Shear stress curve

Figure 4 presents three regions related to the behaviour of sand. The first region is an impossible domain. The second domain is where liquefaction can occur. It is bounded by the locus of collapse points. This curve represents the number of cycles needed for occurrence of liquefaction for a given cyclic shear resistance. In the third zone, the sand can show a cyclic mobility behaviour. In this case, the collapse is characterised by a fixed strain rate, but no liquefaction occurs, irrespective of the consolidation pressure. If the cyclic shear resistance is low enough, the sand can manifest an adaptation, so that the strains and the excess pore pressure remain limited.

Consequently, the cyclic liquefaction domain is reduced because of the existence of the q_{rit} asymptote. This result comfirms the schematic diagram which represents the effect of soil sate on the liquefaction potential, suggested by Sladen *et al.* (1985) and Alarcon-Guzman *et al.* (1988), on RF Hostun sand.

CONCLUDING REMARKS

Undrained cyclic behaviour of a loose sand has been studied. The knowledge of the initial conditions in terms of state parameters is not sufficient to fully characterise the sand behaviour. A sand sample under the same initial conditions can show different types of behaviour as liquefaction and cyclic mobility, depending of the shear stress amplitude. The existence of a critical shear stress has been pointed out. Such a parameter is particularly interesting for the evaluation of liquefaction risk. In fact, it has been pointed out that there is no risk of liquefaction below a given value of this parameter. The critical shear stress seems to be nearly equal to the residual shear stress.

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