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An experimental investigation into the effects of low plastic fines content on the behaviour of sand/silt mixtures

Nadia Benahmed^{a*}, Trung Kien Nguyen^a, Pierre Yves Hicher^b and Maxime Nicolas^c

^aIRSTEA d'Aix-en-Provence, Aix-en-Provence, France; ^bUMR CNRS 6183, Ecole Central Nantes, Nantes, France; ^cAix-Marseille Université, IUSTI UMR 7343, Marseille, France

This paper presents the results of an experimental investigation of the mechanical behaviour of silty sand regarding instability phenomena using conventional triaxial tests under drained and undrained conditions. First, a series of monotonic triaxial tests was carried out on reconstituted clean sand samples with emphasis placed on the effect of basic parameters such as relative density and confining pressure. To highlight the effect of fines content on the initiation of instability and on the critical state, a second series of monotonic triaxial tests was conducted on sand/silt mixture samples reconstituted with different percentages of fines. In this series, one group of samples was prepared with the same overall void ratio, while in the second one, all the specimens had the same intergranular void ratio. The effect of fines on the evolution of the maximum and the minimum void ratios was also investigated. The results obtained show that fines content has a significant influence both on resistance at the onset of instability and on the position of the critical state line of silty sands. In addition, they also revealed that the critical fine content for this soil is about 20%. We guess that, beyond this value, the effect of fines content on the mechanical response of the soil is reversed as reported by several authors in the literature.

Keywords: silty sand; triaxial test; fines; liquefaction; undrained instability; critical state; void ratio; intergranular void ratio

1. Introduction

In the delta of Rhône river (France), many dikes breaches occurred during the recent floods. Some of them cannot be attributed to any conventional failure mechanism of embankment dike such as internal erosion, overtopping or sliding. The constitutive materials of dikes are often silty sands. We know now that such a material can become unstable at stress states located well under the Mohr-Coulomb plasticity limit criterion. The instability of silty sand may be, consequently, one of the mechanisms of dike failure, which is still poorly known and ignored by engineering community.

While the instability of clean sands has been investigated in-depth over the past five decades, this phenomenon in silty sand, which could contain varying amounts of fines, has aroused a real interest in the scientific community only since the two last decades. The researches on this subject are quite numerous and have shown that this type of soil is more susceptible to liquefaction than clean sand. However, the reported results are still conflicting especially regarding the effect of fines content on the shear strength of silty sands.

*Corresponding author. Email: nadia.benahmed@irstea.fr



Figure 1. A breach of an embankment dike of Rhone river during flood in 2003.

The main purpose of this work, which is real-case study research, is to characterise the mechanical behaviour of natural silty sand, which was collected in the adjacent zone of a breach of the Rhône embankment dike during the 2003 flood (Figure 1).

Our emphasis was placed especially on the effect of fines content on the maximum and minimum void ratios, on the shear strength of silty sand at the onset of instability and on the critical state. The experimental results of drained and undrained triaxial tests performed on clean sand and sand/silt mixtures with different fine contents are presented and discussed.

1.1. Literature review on the effect of fines content

The effect of fines content is an important parameter when evaluating the mechanical behaviour of silty sands. However, contradictory results exist in the literature on the effect of fines content on the instability of soils. According to Pitman, Robertson, and Sego (1994), the addition of fines particles up to 40% leads to more strain hardening of sand-silt mixtures, whereas Huang, Huang, Kuo, and Tsai (2004) found that strain softening seems to be more pronounced as fines content increases up to 30%. Belkhatir, Arab, Della, Missoum, and Schanz (2010) reported a continuous decrease in peak deviator stress up to 50% fines. On the contrary, Yang, Lacasse, and Sandven (2006), and Dash and Sitharam (2011), observed that peak deviator stress decreases until the fines content reaches its threshold value and then increases with the further addition of fines particles.

Based on cyclic triaxial tests, the conflicting results are similar. As fines content increases, the cyclic resistance of sand silt mixtures can: (i) increase (Amini & Qi, 2000); (ii) decrease (Belkhatir et al., 2010; Stamatopoulos, 2010); (iii) decrease up to a certain fines content value and then increase (Koester, 1994; Papadopoulou & Tika, 2008; Polito & Martin II, 2001; Xenaki & Athanasopoulos, 2003).

1.2. Threshold fines content

It is now considered that with a content of fines particles (F_c (%)) below a certain value, the mechanical behaviour of a soil is governed only by the skeleton of the sand

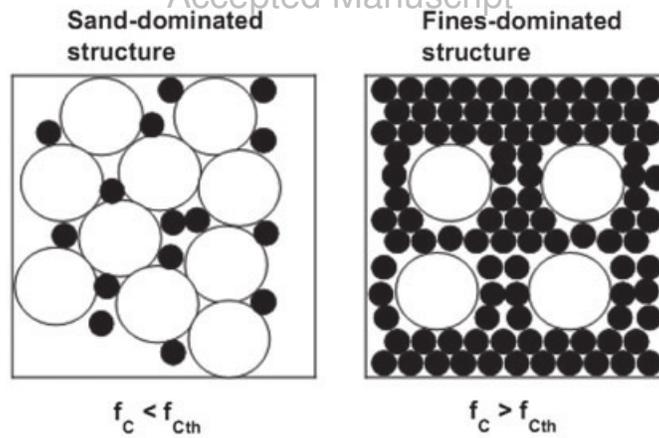


Figure 2. Two structure configurations of sand containing fines.

grains. The fines are non-active and do not contribute to shear strength. However, when the fines content exceeds this limit, the mechanical response is dominated by fines particles and the sand grains are considered as the void (Figure 2). This limit value is called “threshold fines content $f_{c_{th}}$ ”.

Polito and Martin II (2001) used a term of “limiting fines content” to refer to the maximum amount of fines that can be contained in the void by keeping the skeleton of sand unchanged. Yang et al. (2006), Yang, Sandven, and Grande (2006a, 2006b) used another term, i.e. “transitional fines content”, which is actually the amount of fines above which the effect of fines on the peak strength of the soil changes from positive to negative. All these terms were presented differently but they seem to refer to a single physical parameter.

1.3. Intergranular void ratio

Through an analysis of the mechanical behaviour of silty sand (sand-dominant structure), Kuerbis, Negusse, and Vaid (1988) proposed the concept of intergranular void ratio, where the volume of fines content is considered as voids, since the fines particles simply fill the gap between the coarse particles. Also, they do not participate in chain forces or in granular structure resistance.

It should be mentioned that the intergranular void ratio has also been called the sand skeleton void ratio by Dash and Sitharam (2011); Kuerbis et al. (1988); Polito and Martin II (2001) and the granular void ratio by Dasari, Ni, Tan, and Hight (2004).

The intergranular void ratio can be calculated easily by the formula proposed by Thevanayagam (1998), by assuming that the specific gravity of coarse particles is equal to that of the fines particles:

$$e^g = \frac{e + f_c}{1 - f_c} \quad (1)$$

where e is the overall void ratio and f_c is the fines content ($f_c = F_c/100$).

Using this approach, Kuerbis et al. (1988); Papadopoulou and Tika (2008); Stamatopoulos (2010); Xenaki and Athanasopoulos (2003) showed that cyclic resistance increases with the addition of fines particles at the same intergranular void ratio. Polito and Martin II (2001) tested two different sands and showed that for a given intergranular void ratio, the cyclic resistance of Yasteville sand increases slightly as the percentage of fines increases while that

of the Monterey sand is constant. Based on monotonic tests, Dasari et al. (2004) pointed out that peak deviator stress could either increase or decrease, depending on whether the fines used is non-plastic or plastic, respectively, at the same intergranular void ratio.

1.4. Equivalent intergranular void ratio e^*

In analyzing in detail the role of fines content in the material structure, Thevanayagam, Shenthan, Mohan, and Liang (2002) argued that when the fine content is very small, all the fines particles stay in the voids created by coarse grains and consequently, do not contribute to material resistance. These fines particles are entirely inactive. However, with the addition of fines content, there will be a part of fines particles that begin to touch and separate the coarse grains, in other words, to become active. So, to take into account either active and inactive fines particles, Thevanayagam et al. (2002) proposed a new density parameter: equivalent intergranular void ratio which is calculated as follows:

$$e^* = \frac{e + (1 - b)f_c}{1 - (1 - b)f_c} \quad (2)$$

To calculate the equivalent intergranular void ratio, it is necessary to determine the b parameter value. Some authors identified this value by back-analysis in order to obtain the best fit to their experimental data (Dasari et al., 2004; Yang et al., 2006b). It means that the value of b was chosen arbitrarily and it is a constant for a certain range of fines percentage. Thereafter, Rahman, Lo, and Gnanendran (2008) introduced the first formula to identify the b parameter varying with fines content:

$$b = \left\{ 1 - \exp \left[-0.3 \frac{(f_c/f_{thre})}{k} \right] \right\} \left(r \frac{f_c}{f_{thre}} \right)^r \quad (3)$$

where $r = d_{50}/D_{10}$ (d_{50} is mean diameter of fines particles, whereas D_{10} is effective diameter of sand particles), $k = 1 - r^{2.5}$ and $f_{thre} = .30$ which may be assumed.

Similar to intergranular void ratio, equivalent intergranular void ratio is also called by several different names like equivalent granular void ratio (Ni et al., 2004) or corrected void ratio (Yang et al., 2006, 2006a). This new density parameter was reported as a pertinent choice to describe the mechanical behaviour of silty sand.

2. Experimental procedure

2.1. Material used

The soil used in this study is poorly graded natural Camargue silty sand, which was taken from flood protection dikes. The clean sand was extracted by sieving the silty sand. The particles smaller than 80 μm mesh size (fines) were separated in order to prepare different mixtures. The sand was uniform with angular and rounded grains, a uniformity coefficient $C_U = 2.09$ and an effective diameter $D_{50} = .25$ mm. Its minimum and maximum void ratio was equal to .610 and 1.100, respectively. The fines used were low plastic fines with a liquid limit $W_L = 32$ and a plasticity index $I_p = 8$. The specific gravity of both sand grains and silt grains was $\rho = 2.67$ g/cm³.

The host sand and host silt were blended together to obtain the different mixtures with the different desired target fines contents (by weight) varying from 0, 5, 10, 15 and 20%. The grain size distributions of all the materials used in this study are

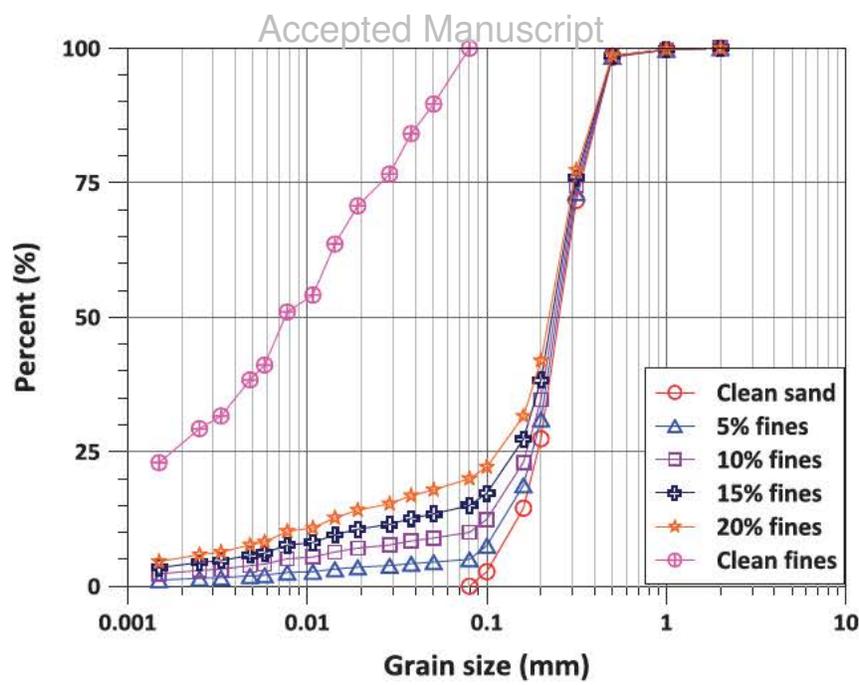


Figure 3. Grain size distribution.

presented in Figure 3. We should point out that the grading curve of natural silty sand is the same as the mixture of the sand with 10% of fines content.

To determine the maximum and minimum void ratios of the mixtures linked to the loosest and densest soil structures, respectively, we used both French and Japanese standard procedures to obtain reliable values. The measures for each mixture were repeated five times. The evolution of maximum and minimum void ratios is reported in Figure 4. It can be observed that they tend to decrease initially as fines are added up to a certain amount, and then to increase. The evolution of the minimum void ratio indicates that the value of threshold fines content of the soil tested was approximately 20%. The properties of the used materials are summarized in Table 1.

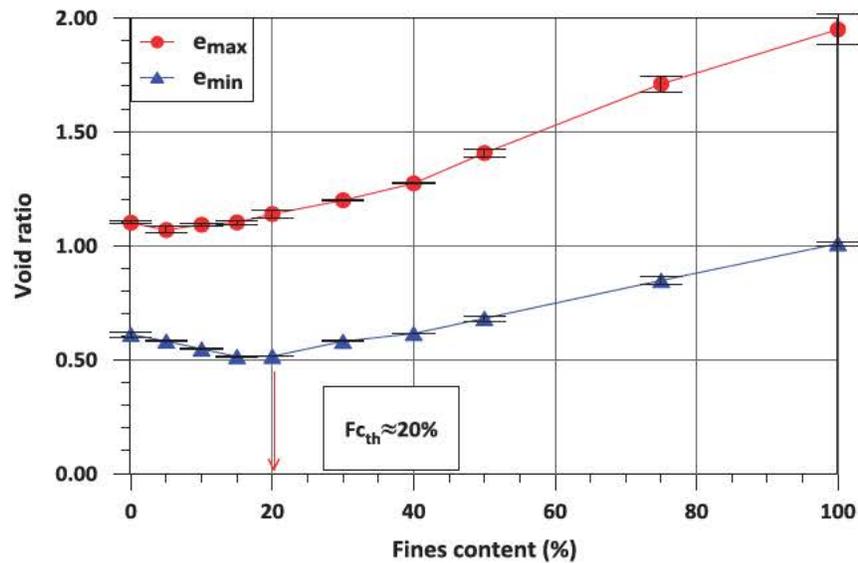


Figure 4. Evolution of maximum and minimum void ratios with varying fines content.

Table 1. Physical properties of the materials used.

Materials	Fines content Fc (%)	Mean diameter D_{50} (mm)	Coefficient of uniformity C_U	Coefficient of gradation C_C	Maximum void ratio e_{max}	Minimum void ratio e_{min}
Clean sand	0	.25	2.09	1.13	1.100	.610
Mixture 1	5	.25	2.47	1.27	1.068	.580
Mixture 2	10	.24	3.34	1.57	1.091	.546
Mixture 3	15	.23	16.06	6.79	1.101	.513
Mixture 4	20	.22	33.53	11.51	1.138	.514
Clean silt	100	.01	–	–	1.948	1.008

It is worth noting that no standard exists for determining the maximum and minimum void ratio applicable for the entire range of fines content (0–100%). The American standards ASTM-4254-91 and ASTM-4253-93 are applicable to materials with fines content up to 15%, while the French standard NF-P94-059 has a fines content limit of about 12%. It has been reported that the values of maximum and minimum void ratios strongly depend on the procedure used, especially with high fines content. As for the minimum void ratio, some authors still use the ASTM procedure, like Thevanayagam (1998), Amini and Qi (2000), Xenaki and Athanasopoulos (2003), Huang et al. (2004), Stamatopoulos (2010), whereas others combine the ASTM procedure and the Proctor test to obtain consistent values, like Polito and Martin II (2001), Yang et al. (2006) or Papadopoulou and Tika (2008). In order to determine the maximum void ratio, the ASTM procedure (Amini & Qi, 2000; Thevanayagam, 1998), or simply pouring material into the mould with a funnel (Yang et al., 2006) or a combination of these two methods (Papadopoulou & Tika, 2008) were used.

2.2. *Triaxial apparatus*

A fully computer controlled Dynamic Triaxial System from Wykeham Farrance was employed. The axial load was monitored by an internal submersible load transducer, while the axial displacement was measured by an LVDT mounted on the top platen. The volume variation of the sample was collected by an automatic volume change device. All the tests were conducted in strain controlled mode at a constant rate of 1% per minute. The barrel effect can be solved using the section correction formula of French standard NF 094-74.

2.3. *Specimen reconstitution and procedure*

Vaid and Negussey (1988) pointed out that if the sand contains a large amount of fines, preparing specimens using the pluviation method will lead to segregation between coarse grains and fines particles, and thus to a non-uniform sample. Huang et al. (2004) studied the homogeneity of silty sand specimens reconstituted by the moist tamping technique. After saturation and consolidation, the specimens were

removed from the triaxial cell, cut horizontally into five equal slices, and then the fines content in each slice was checked by wash sieving. The results showed that there were no signs of fines migration in the specimens. Therefore, the moist tamping method was used to prepare all the specimens in this study. The height of the samples was about 140 mm and the diameter about 70 mm. The soil was arranged in seven layers, each 20 mm thick. First, the oven-dried mixture of sand with fines was divided into seven equal parts; each part was mixed thoroughly with de-aired water to a water content of 2%. Particular care was taken to avoid any potential segregation, especially in the case of high fines content. Subsequently, the moist sand was placed in a split mould by spooning to obtain loose conditions and then compacted carefully at a predetermined height to achieve the target density. Compaction was performed by a hand tamper with a graduated vertical rod and a small removable horizontal tab to control the desired height by abutting against the edges of the mould. To saturate the specimens, CO₂ was flushed under a pressure of 20 kPa for 30 min, followed by de-aired water of about three times the specimen volume. Back pressures ranging from 400 to 500 kPa were then applied to ensure full saturation of the specimens and to obtain a Skempton coefficient B higher than .96.

3. Results and discussion

3.1. Undrained behaviour of clean sand

To investigate the undrained behaviour of the clean sand, a series of undrained monotonic triaxial tests was conducted, focusing on the influence of two key parameters, namely the density index I_D (or, equivalently, the void ratio) and the confining pressure P_c . Three initial density indexes, I_{Di} .10, .20, and .30 and three confining pressures, P_c 100, 200 and 400 kPa were selected.

3.1.1. Effects of void ratio

Figure 5 shows the results obtained from tests carried out at the same confining pressure P_c 100 kPa but with different initial density indexes I_{Di} equal to .10, .20 and .30. A typical mechanical response for loose sand was observed for the three tests. The deviator stress for all the specimens reached a peak resistance at very low strains and then decreased rapidly as axial strain increased until reaching a very low or null shear resistance value. It can be seen that the peak shear strength increased as the density index increased. In other words, the denser the specimen was, the higher its resistance at the initiation of liquefaction. Furthermore, we observed that the specimen collapsed earlier when its density was low. The specimens with a density index after consolidation of I_{Dc} .22 and .30, collapsed at 6% and 18% of axial strain, respectively, while the specimen with an I_{Dc} .39 still resisted at residual state (Figure 5(a)). The excess pore water pressures (Δu) increased steadily and reached the value of the confining pressure at the moment the sample collapsed (Figure 5(b)). It could be seen in Figure 5(d), which represents the evolution of excess pore pressure rate per percent of deformation, that the lower the density index, the faster the pore pressure rose at the beginning of the shearing.

The effective stress paths are shown in Figure 5(c). They can be seen to move towards the origin of the axes or zero effective stress state, which is linked to the strong contractive tendency of loose sand. It can be seen that the peak of the effective stress paths can be joined to the origin of the axes by a straight line called the instability line,

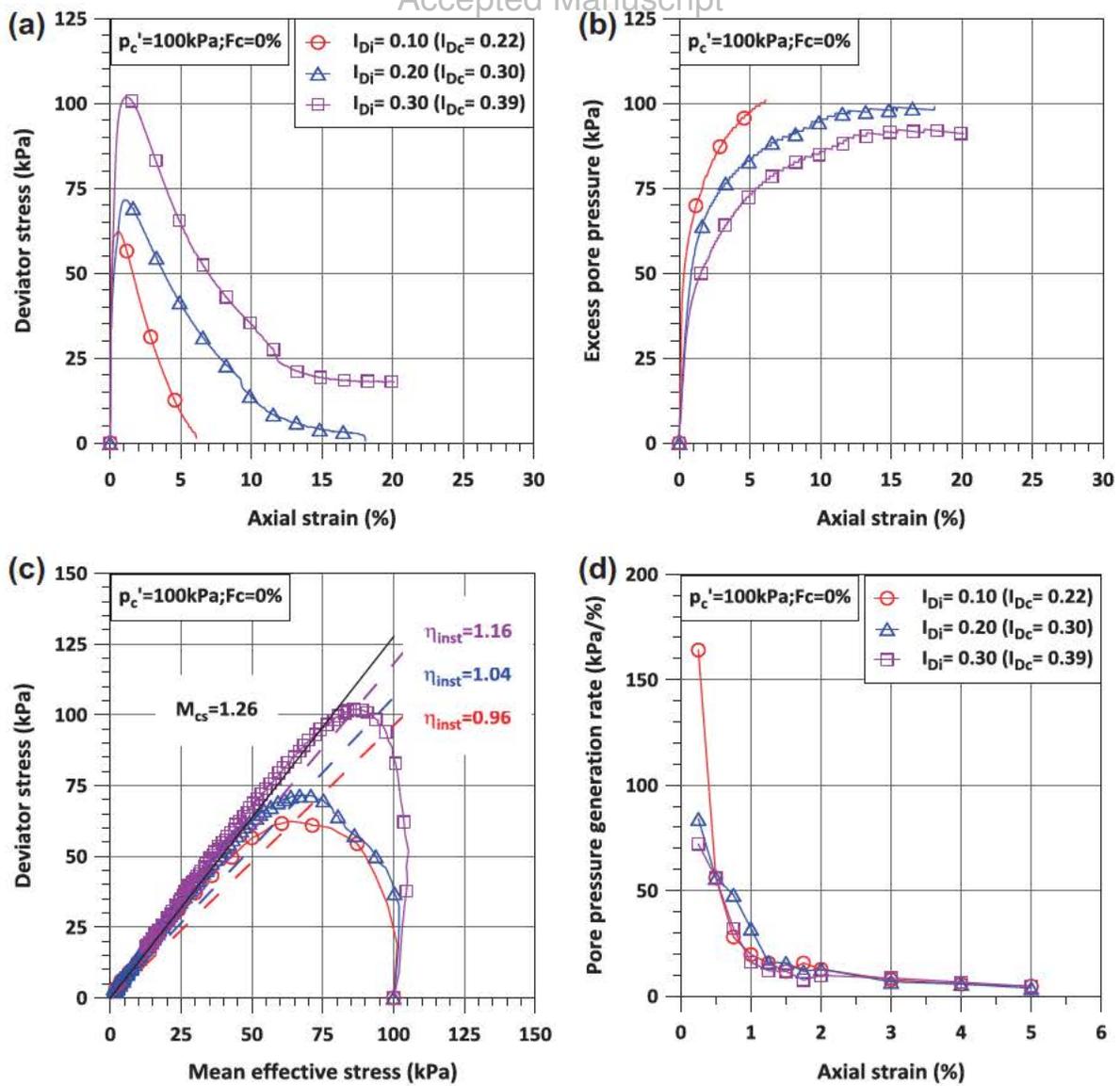


Figure 5. Effect of void ratio on the undrained behaviour of clean sand (a) $q - \varepsilon_a$; (b) $\Delta u - \varepsilon_a$; (c) $q - p'$; (d) pore pressure generation rate.

as defined by Lade (1992). It is important to point out that this line is not unique and varies as a function of density index variations; the higher the latter, the steeper the instability line becomes and the larger the mobilized friction angle at instability is. Moreover, it should be noted that this angle is much smaller than the internal friction angle of the sand, which is about 31.3° .

3.1.2. Effects of confining pressure

The influence of confining pressure has been investigated by comparing tests on specimens having the same initial density index $I_{D_i} = 0.10$ and being carried out at different confining pressures $p'_c = 100, 200$ and 400 kPa (Figure 6). It can be seen that soil strength increases with confining pressure. In fact, for a given initial density index, increasing the confining pressure significantly improves the peak shear resistance. A similar effect can be observed at residual state (Figure 6(a)). The sample confined at 100 and 200 kPa collapsed totally at 6 and 13% of axial strain, respectively, while the sample confined at 400 kPa had residual shear strength of about 18 kPa at the end of the test.

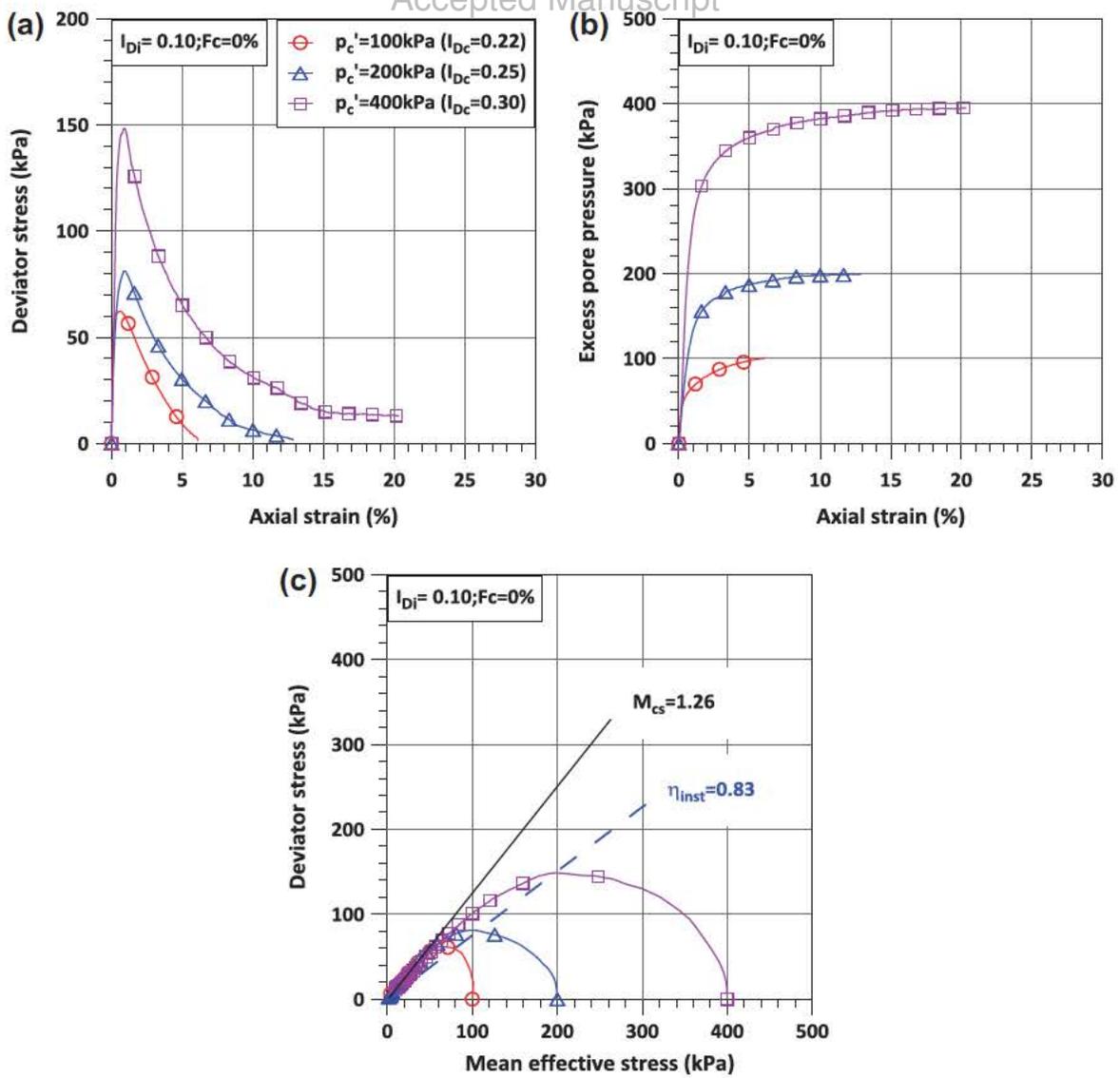


Figure 6. Effect of confining pressure on the undrained behaviour of clean sand (a) $q - \varepsilon_a$; (b) $\Delta u - \varepsilon_a$; (c) $q - p'$.

Concerning the generation of excess pore water pressure, it is shown that for the higher confining pressure, there was a slight lag in the development of pore water pressure.

The effective stress paths are shown in Figure 6(c). They appear to be homothetic, and the contractive trend seems to increase with increasing confining pressure. Furthermore, it can be seen that for the three specimens with a constant density index but different confining pressures, the peaks of the effective stress paths fell at the same straight line passing through the origin of $q-p'$ space. These results are in good agreement with the concept of the instability line (Lade, 1992).

3.2. Drained behaviour of clean sand

To study the drained mechanical behaviour of clean sand, a second series of monotonic triaxial tests was carried out under drained conditions with the same initial conditions as for the undrained tests in terms of density and confining pressures.

3.2.1. Effects of void ratio

Figure 7 shows the results of three tests on samples reconstituted at different initial density indexes I_{Di} .10, .20, .30 and consolidated at the same confining pressure $p'_c = 100$ kPa. The deviator stress and volumetric strains increased continuously as axial strain increased. The deviator stress-axial strain curves show that increasing the density index has the effect of improving the mechanical characteristics of the sand, particularly in terms of initial stiffness (Figure 7(a)). It can also be seen that the three shear curves tend almost to the same ultimate value under large deformations, which corresponds to the critical state.

The evolution of volumetric strains was mainly positive in all the tests and the effect of density is very clear (Figure 7(b)). This figure shows that decreasing the density index causes the specimens to contract strongly. This trend is consistent with the observation on the effect of void ratio on the generation of pore pressure resulting from the undrained series mentioned above.

3.2.2. Effects of confining pressure

As with the density index, confining pressure improves the shear strength and the rigidity of the material. In addition, the shear curves are almost proportional to the confining pressures applied (Figure 8(a)). It is interesting to note that the deviator stress reaches the critical state as fast as the confining pressure increases from 100 to 400 kPa.

Regarding the evolution of volumetric strains, it was found that the confining pressure considerably increased the contractiveness of the soil (Figure 8(b)).

3.3. Undrained behaviour of sand with fines

Four series of triaxial undrained tests were performed to investigate the effect of fines content on the behaviour of silty sand. In the two first series, the overall void ratio was kept constant by reducing the quantity of sand and adding the fines. In the second two series, the intergranular void ratio was kept constant, which means that the quantity of sand was maintained constant while the percentage of fines was changed. In this case, the overall void ratio varied. The difference between “constant initial void ratio” and “constant initial intergranular void ratio” is illustrated in Figure 9.

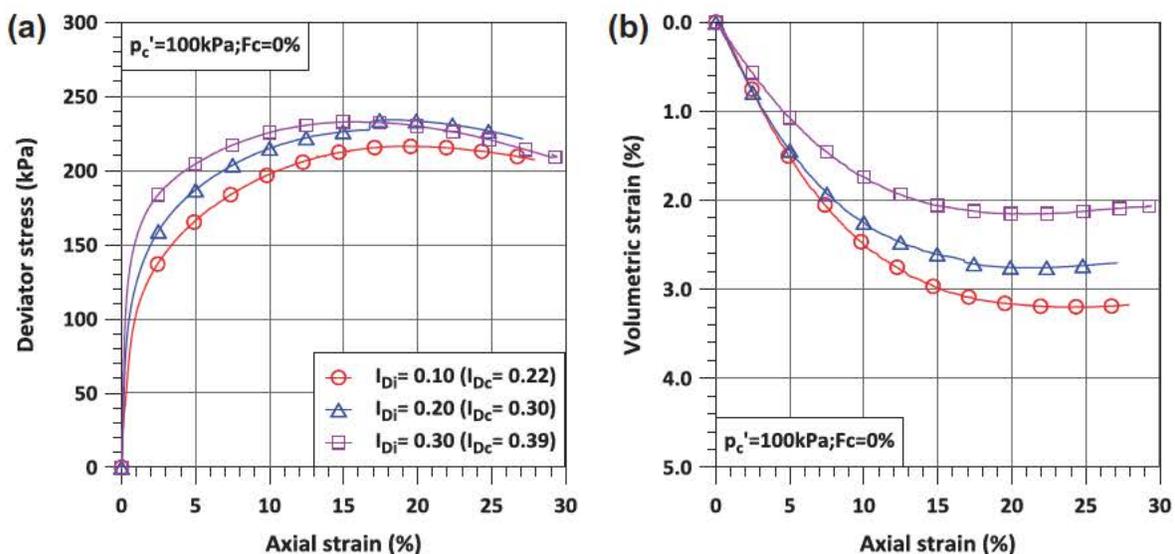


Figure 7. Effect of void ratio on the drained behaviour of clean sand (a) $q - \epsilon_a$; (b) $\epsilon_V - \epsilon_a$.

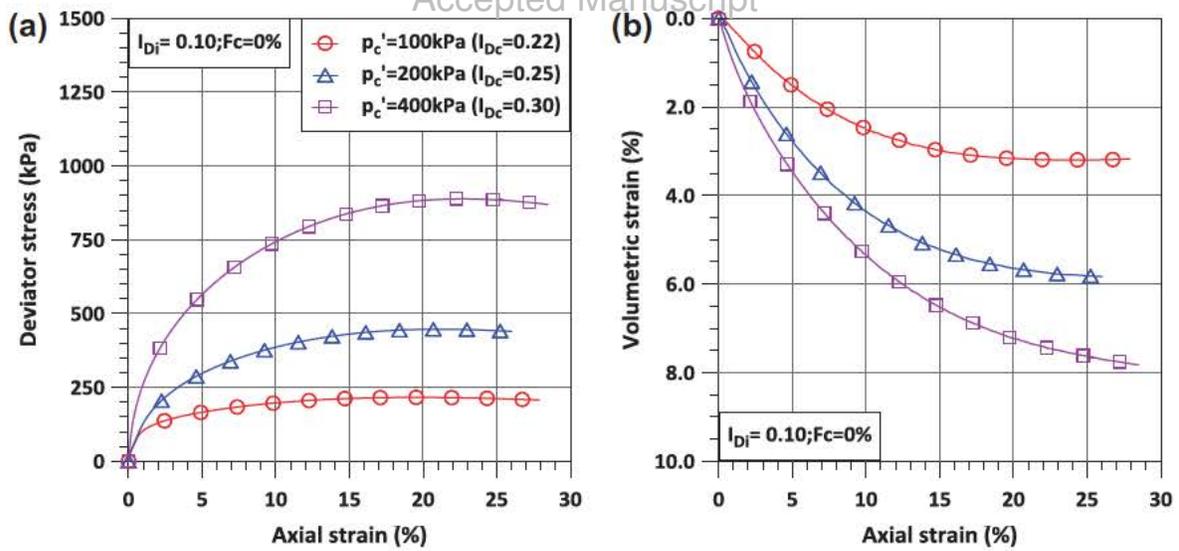


Figure 8. Effect of confining pressure on drained behaviour of clean sand (a) $q - \varepsilon_a$; (b) $\varepsilon_V - \varepsilon_a$.

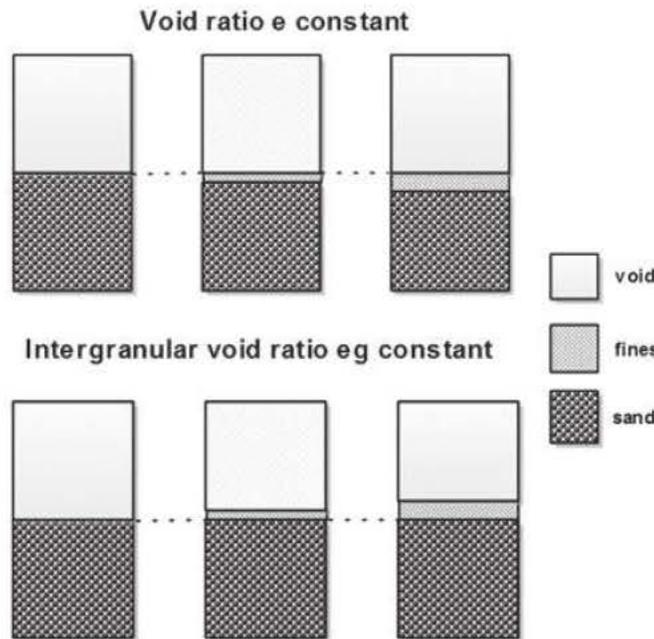


Figure 9. The difference between the constant void ratio and the constant intergranular void ratio.

3.3.1. Effect of fines content on the initiation of instability tests on specimens with a constant initial void ratio e_i

In the literature, the investigation of the influence of the fines has often been conducted on samples reconstituted with the same initial void ratio, which means reducing the amount of sand when increasing the amount of fines. In line with this, we performed two series of tests; in each one, the specimens had the same initial void ratios, 1.05 and 1.00, corresponding to density indexes of .10 and .20, respectively, of the mixture of sand and fines, with the percentage of fines varying between 0, 5, 10, 15 and 20%.

In Figure 10, we report the result of the series of tests performed at the initial void ratio of 1.05. In order to clearly highlight the evolution of the peak shear strength, the latter of the two series is reported in Figure 11 as a function of the fines content. Both series of tests gave similar results. Although it was expected to increase, the peak strength decreased

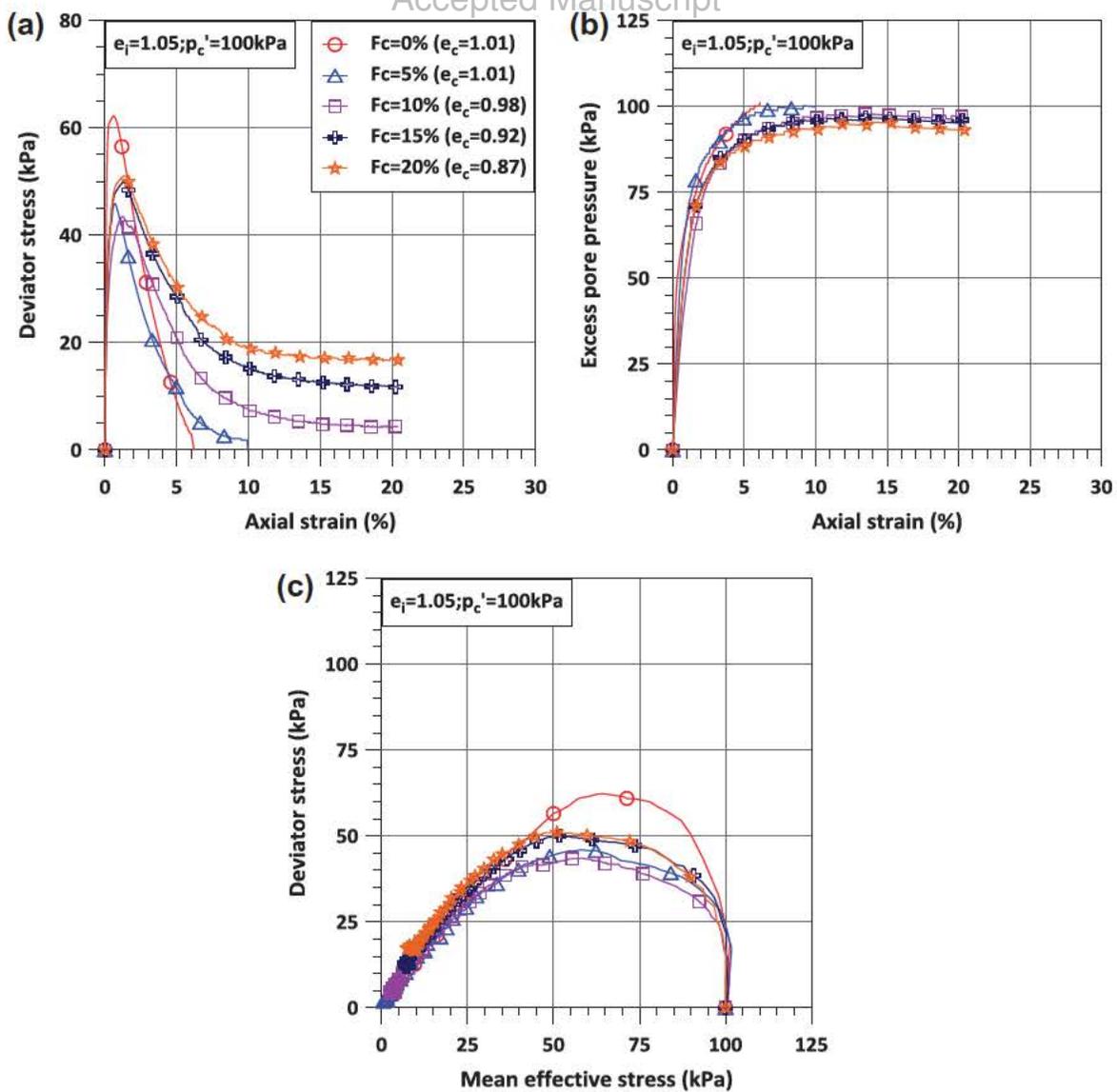


Figure 10. Effect of fines content on undrained behaviour of silty sand – series of specimens with the same initial void ratio (a) $q - \varepsilon_a$; (b) $\Delta u - \varepsilon_a$; (c) $q - p'$.

drastically as the fines content increased up to about 5%, and then slightly up to 10% fines, followed by slight increase with a further increase in fines content. This trend suggests that when the fines content is low, for example, below 5%, the fines particles are almost inactive. They fill the space between the sand particles without contributing to soil strength. The behaviour of the soil is mainly governed by the sand skeleton; its loosening leads to a noticeable drop of peak resistance. With a further increase in fines content up to 20%, some of the fines particles start to participate in the force chain of the structure, so the drop in peak resistance is less violent. Beyond 20% of fines content, the effect of fines content on the resistance may be reversed, as reported in the literature, since the structure is supposed to be dominated by the fines particles.

3.3.2. Effect of fines content on the initiation of instability tests on specimens with a constant initial intergranular void ratio e_i^g

In this part of the study, the specimens were prepared at a constant initial intergranular void ratio, by keeping the same amount of sand and adding the fines particles

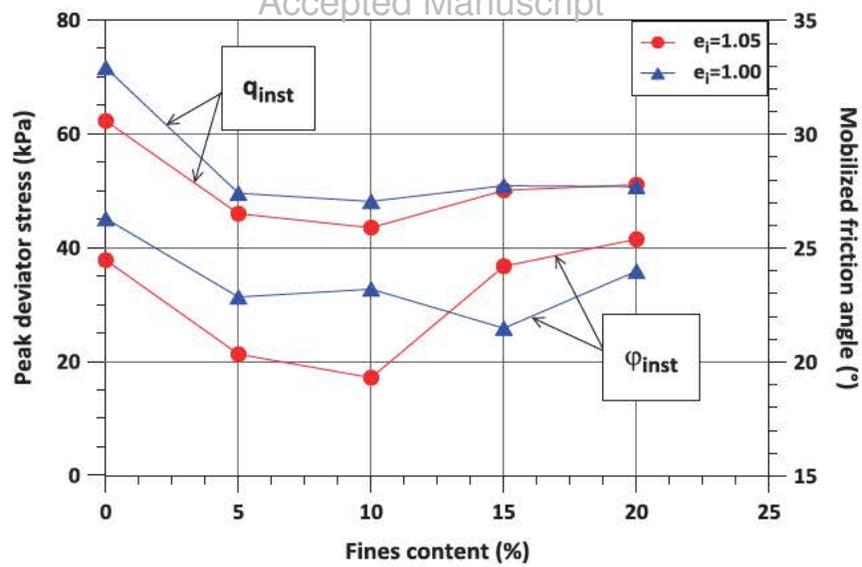


Figure 11. Effect of fines content on the onset of instability – two series of specimens with the same initial void ratio.

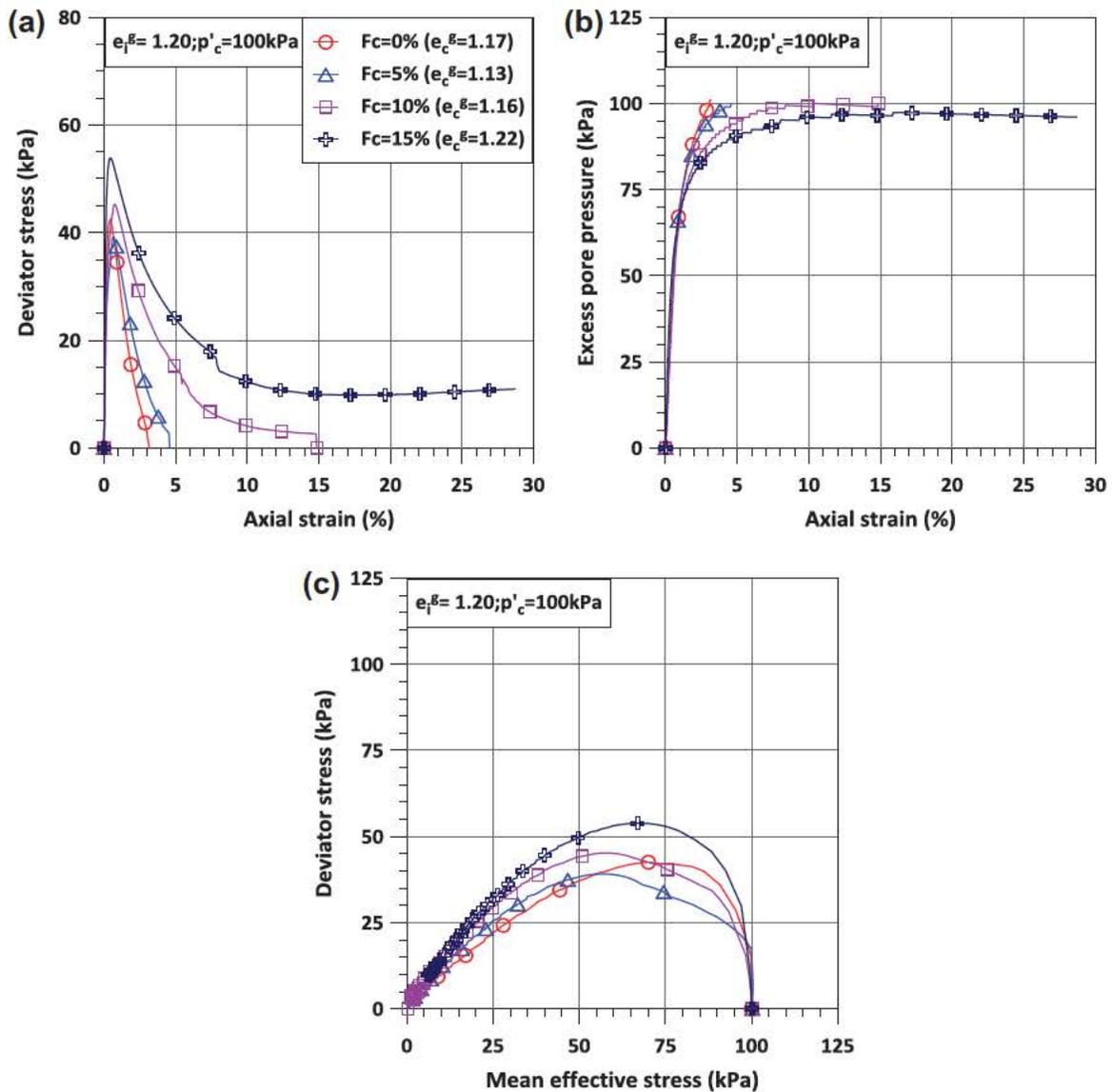


Figure 12. Effect of fines content on the undrained behaviour of silty sand – series of specimens with the same initial intergranular void ratio (a) $q - \varepsilon_a$; (b) $\Delta u - \varepsilon_a$; (c) $q - p'$.

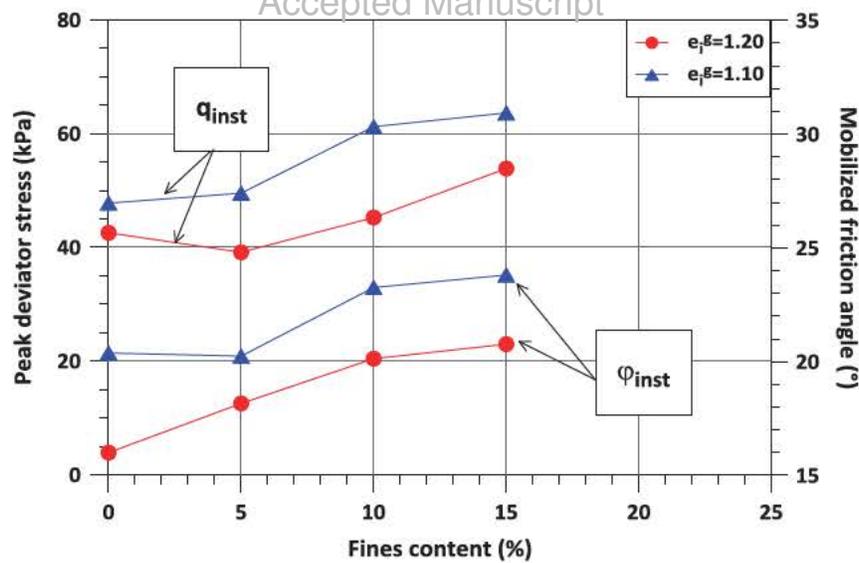


Figure 13. Effect of fines content on the onset of instability – two series of specimens with the same initial intergranular void ratio.

progressively. Two test series were carried out with two initial intergranular void ratios of about 1.20 and 1.10, which correspond to host sand density indexes of -0.20 and 0 , respectively. The fines content was varied by 0; 5; 10 and 15%. It is worth noting that using the intergranular void ratio is pertinent when the percentage of fines is below the threshold of the fines content.

Figure 12 represents the result of the test series performed at an initial intergranular void ratio of 1.20. As above, for more clarity, the peak shear strength versus the percentage of fines of the two series is represented in Figure 13. For the fines content of 5%, the peak shear strength decreases with the addition of fines. Since density increases with fines content, this should lead to higher peak shear strength, although this result runs counter to what one might expect. This could be explained by the fact that below 5%, the fines are completely inactive, which supports the assumption mentioned above. From 5 to 15% fines, peak strength starts to increase steadily with as fines content increases. As mentioned previously, the sand skeleton remains unchanged in this case, so the modification of soil resistance is mainly due to the fines particles. In fact, it can be assumed that all the fines contribute to the overall chain forces and control the behaviour of the soil.

3.4. Drained behaviour of sand with fines

Figure 14(a) presents a comparison of three drained tests carried out on specimens prepared at the same initial intergranular void ratio e_g^i but with different percentages of fines: 5, 10 and 15%.

Although there is no clear trend of the influence of fines on shear strength, as was seen in the case of the undrained tests, a slight increase in shear strength can be seen with the increase in fines. Nevertheless, the volumetric strains seem to be very sensitive to the percentage of fines. Indeed, the mixture of sand and fines is even less contractive as fines content increases. This trend is quite obvious since the specimens have the same amount of sand, and the addition of fines leads to denser and hence less contractive specimens.

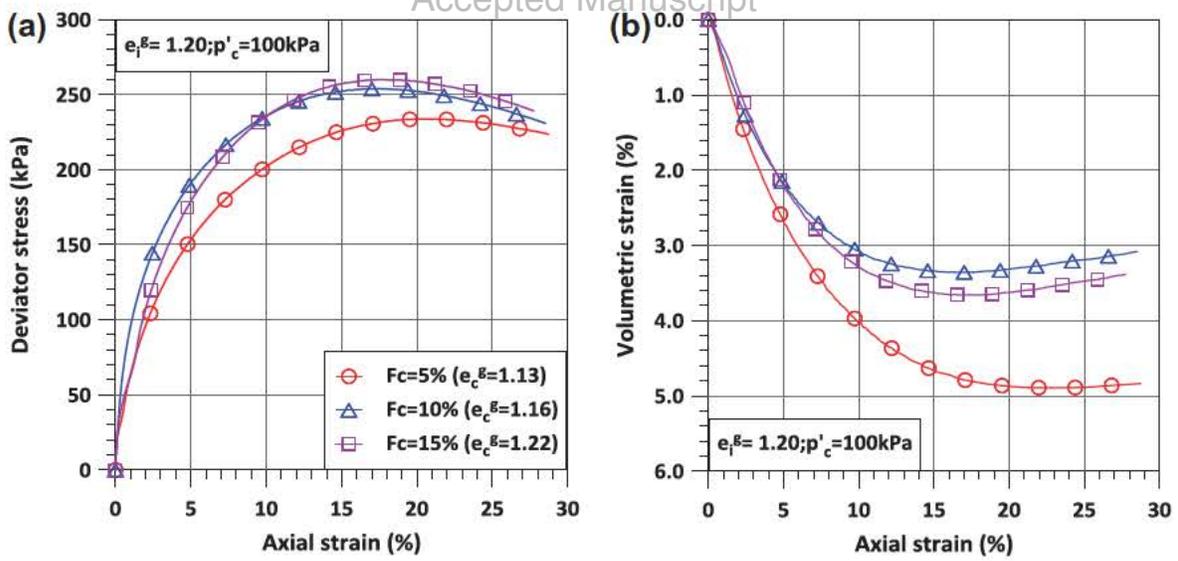


Figure 14. Effect of fines content on the drained behaviour of silty sand (a) $q - \varepsilon_a$; (b) $\varepsilon_v - \varepsilon_a$.

3.4.1. Effect of fines on critical state

All the tests were conducted up to 20 to 30% of axial strain and almost of them resulted in a stable state in terms of either deviator stress or volumetric strain. Figure 15 illustrates the critical state points obtained for different mixtures as well as those obtained on clean sand and clean silt in $q - p'$ space. All the critical state points seem to be aligned on the same line, irrespectively of fines content. However, if we look into more details of the obtained values of the critical friction angle φ_{cs} , we notice that the latter increases very slightly as the fines content increases ($31.3^\circ - 32.8^\circ$). This result confirms those presented by Carraro, Murthy, Loukidis, Salgado, and Prezzi (2007).

Afterwards, the critical state lines were plotted in classical space $e - \log p'$, as presented in Figure 16. It is interesting to note that each mixture has a different critical state line. Hence, the addition of fines particles creates different materials. The critical state line moved downward to a lower void ratio as the fines content increased up to 15%. Similar results were reported by other authors in literature (Huang et al., 2004;

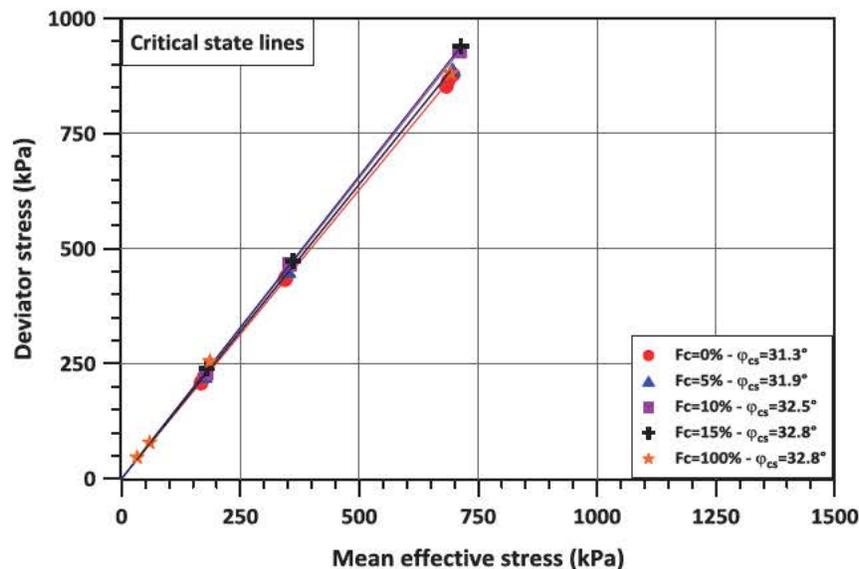


Figure 15. Critical state line in the $q - p'$ space.

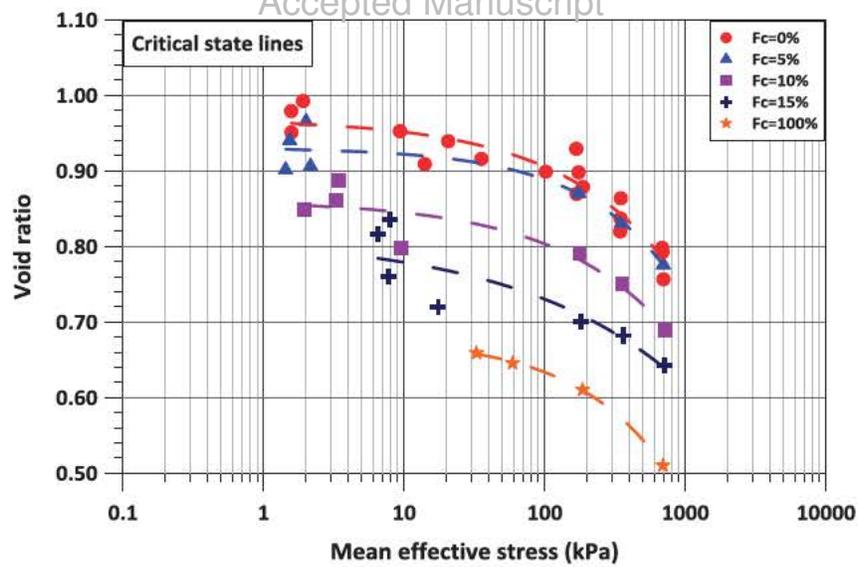


Figure 16. Critical state line in the $e - \log p'$ space.

Naeini & Baziar, 2004; Papadopoulou & Tika, 2008; Rahman et al., 2008; Stamatopoulos, 2010; Thevanayagam et al., 2002; Yang et al., 2006b; Zlatovic & Ishihara, 1995). In addition, since the internal friction angle at critical state remained nearly unchanged, this movement means that at a given void ratio, the shear resistance at the critical state decreased as the fines content was increased up to 15%.

For non-cohesive soils such as sand, it has been recognized that the critical state line is generally not a straight line, but a curve (Riemer & Seed, 1997; Verdugo & Ishihara, 1996). Li and Wang (1998) suggested that the critical state line of sand can be represented by a power function with the following form:

$$e_{cs} = \Gamma - \lambda(p'/p_a)^\zeta, \quad (4)$$

where Γ , λ and ζ are the three fitting parameters and p_a is atmospheric pressure.

Carraro et al. (2007); Stamatopoulos (2010) confirmed that the Equation (4) fits well with their experimental results on the critical state line. These parameters can be obtained by least-squares regression, as summarized in Table 2. A decrease in Γ represents the downward translation of the critical state line as fines content increases. In addition, this parameter is considered as the threshold void ratio value, which differentiates if a soil-flow behaviour is with shear residual strength or not. In fact, the soil whose void ratio is greater than this value does not exhibit any residual strength and may show a total collapse (Ishihara, 1993).

The two parameters λ and ζ represent, respectively, the slope and the curvature of the CSL. It seems that there is no relationship between these two parameters with the

Table 2. Parameters of critical state lines.

Fc	Γ	λ	ζ	M_{CS}	φ_{CS} (°)
0	.98	.07	.57	1.26	31.3
5	.93	.05	.64	1.28	31.9
10	.87	.07	.44	1.31	32.5
15	.82	.09	.37	1.32	32.8
100	.67	.04	.83	1.32	32.8

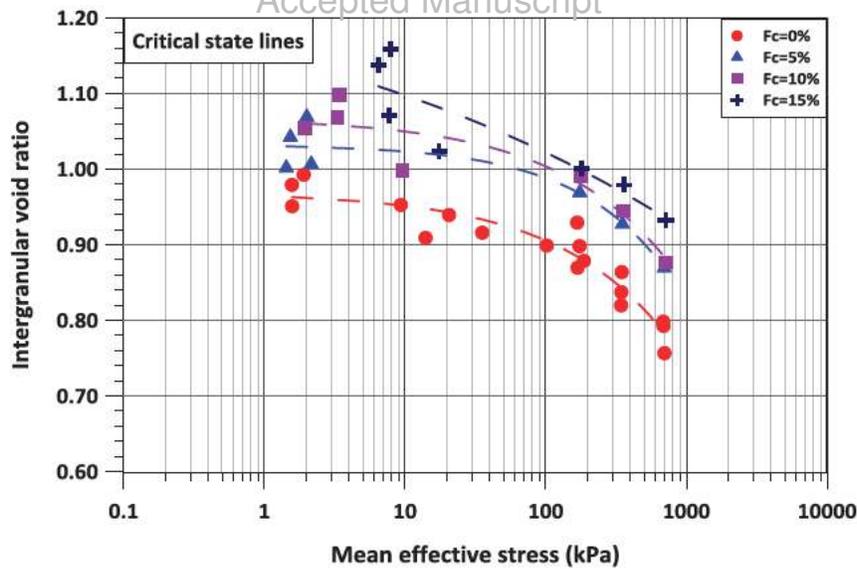


Figure 17. Critical state line in the $e^g - \log p'$ space.

increase in the fines content. This means that the CSL does not rotate with the variation of the fines content as observed by Been and Jefferies (1985) and Bouckovalas, Andrianopoulos, and Papadimitriou (2003).

In Figure 17, we plotted in $e^g - \log p'$ space all the critical state lines, apart from the one of $F_c = 100\%$ since the intergranular void ratio concept is not applicable in case of clean silt. It could be noted that the critical state line moves upward to lower density as fines content is increased. Therefore, at a given intergranular void ratio, the shear resistance at the critical state increases as fines content is increased up to 15%. It should be pointed out that this is linked only to the parameter used for interpretation (void ratio or intergranular void ratio). Similar results were reported by Papadopoulou and Tika (2008).

Figure 18 presents the critical state lines in the $e^* - \log p'$ space. The value of equivalent intergranular void ratio at the critical state was calculated using the Equations

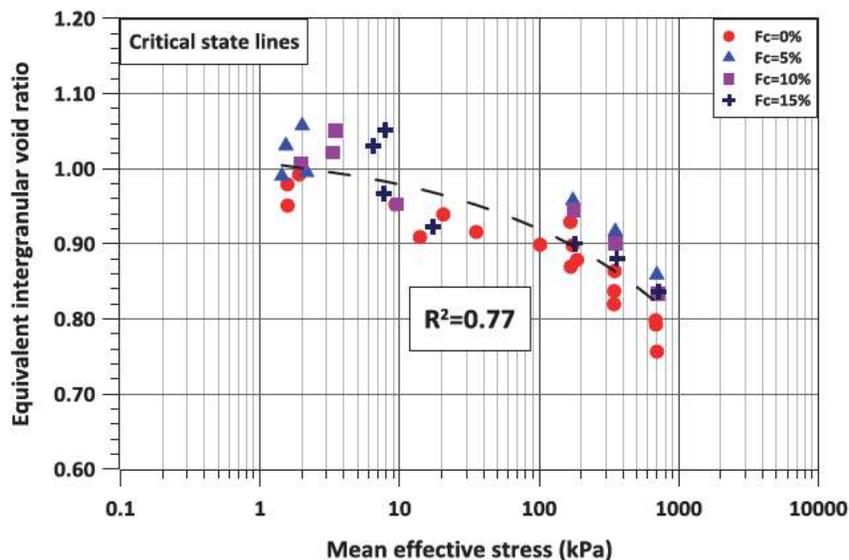


Figure 18. Critical state line in the $e^* - \log p'$ space.

(2) and (3) with the value of threshold fines content $F_{c_{th}}$ determined experimentally of about 20% ($f_{thre} = 0.2$). Similarly as in Figure 17, we did not report the critical state line of clean silt in $e^* - \log p'$ space since the equivalent intergranular void ratio concept is not applicable for fines-dominated structure.

It could be seen that all the critical state lines in this space can be merged into a narrow band. This means that the equivalent intergranular void ratio present a good correlation with the resistance at the critical state.

4. Conclusion

An experimental study was conducted to determine the mechanical behaviour of silty sand using monotonic triaxial compression tests under drained and undrained conditions. The effect of fines content on mechanical response was investigated by using different sand/fines mixtures with different percentages of fines. Different density parameters such as void ratio, intergranular void ratio and equivalent intergranular void ratio were used for interpretation.

The results of undrained tests showed that when the fines are below a certain fines content called the threshold value, about 20% in this study, the sand skeleton plays the dominant role in the mechanical behaviour of soil in general and on shear resistance in particular. However, the fines particles also play a role, although minor, especially when the fines percentage ranges from 5% to the threshold value.

The analysis of the drained behaviour showed that each mixture of sand and fines has a different critical state line, meaning that the addition of fines particles creates a new material. The results also showed that the effect of fines on the critical state strongly depends on the density parameter used for the interpretation, i.e. the void ratio, the intergranular void ratio or the equivalent intergranular void ratio. When using the void ratio, the critical state line moved to higher density as the fines content increased, implying that the fines play a negative role in soil strength. Conversely, when using the intergranular void ratio, the critical state line moved upwards to lower density and the fines content improved the shear strength of soil. Concerning the equivalent intergranular void ratio, it appears to be a suitable variable in critical state analysis of silty sand.

The results also revealed that the internal friction angle of soil is slightly affected by the presence of fines up to 15% in this study. We suppose that this effect can be noticeable or not depending on the internal friction angle of the used fines.

For design and risk analysis of earth-fill structures, it is important to take into account the removal of fines particles from their constitutive material, caused by an erosion process such as suffusion, for example, since this phenomenon leads to the weakening of the strength properties of silty sand, hence to the probability of failure by instability.

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