Proceedings of the 9ICEG
9th International Congress on Environmental Geotechnics
25-28 June, 2023 | Chania, Greece

https://doi.org/10.53243/ICEG2023-276

9ICEG

On Equivalent Intergranular Void Ratio and Monotonic Behaviour of sand with plastic and non-plastic fines

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ABSTRACT

The paper presents results of an investigation into the effects of fines plasticity on the monotonic response of sands through the equivalent intergranular void ratio concept. Monotonic tests on mixtures of sand with 5 % and 15 % non-plastic fines and plastic fines of varying plasticity were performed. At a given fines content, confining effective stress and void ratio, the results show that the undrained shear strength decreases with increasing plasticity index of fines up to a threshold value. Above this threshold value, the undrained shear strength increases with increasing plasticity index of fines. The Critical State theory known as a characteristic state of soil behaviour and the equivalent state concept were used to the interpretation of the laboratory tests results. By estimating parameters b and a -which recognize that silt and clay fines contribute differently to the strength of the sand- and consequently the equivalent intergranular void ratio, $(e_g)_{eq}$, a single Critical State Line, CSL, is determined in the $(e_g)_{eq}$-log(p´) plane, independently of the content and type of fines based on the monotonic tests results. Parameters b, and a, depend on silt content and clay content which reflects the fines plasticity, while $(e_g)_{eq}$ proves to be a suitable parameter for the estimation of the monotonic behavior and undrained critical state strength of granular mixtures irrespectively of the type and the content of their fines. The effectiveness of state parameter, ψ, and equivalent state parameter, $(\psi_g)_{eq}$, in the estimation of the undrained critical state strength of sand-fines mixtures is confirmed.

Keywords: equivalent intergranular void ratio, plasticity, fines, sands, critical state, shear strength

1 INTRODUCTION

Liquefaction of sandy soils under static loading conditions is considered one of the major causes of damage to earth structures and foundations. Up to date, a great research effort has been devoted to improving the knowledge concerning the liquefaction characteristics of natural soil deposits and the ability to predict the nature and the extent of liquefaction phenomenon. Most of the research effort has been focused on the behaviour of clean sands and sands containing non-plastic (NP) fines (Zlatovic & Ishihara, 1995; Thevanayagam et. al 2002; Papadopoulou & Tika, 2008; Porcino et. al 2020) mainly due to the fact that sands with plastic fines are considered to present lower liquefaction potential (Ishihara, 1993).

Numerous case histories, however, concerning failures due to earthquake-induced liquefaction are correlated with the presence of sands containing plastic fines. Kishida (1969), reported that soils with 10 % clay content liquefied during Mino-Owar, Tohankai, and Fukui earthquakes, in Japan. Chang (1987) summarised liquefaction ground failures caused by the 1976 Tangshan Earthquake in China and concluded that sands containing plastic fines are not immune to liquefaction. Youd et al. (1989) reported that silty sands containing as much as 10 % clay liquefied at the Kornbloom site in the Imperial Valley, during the 1981 Imperial Valley earthquake in USA. Miura et al. (1993) also reported the liquefaction of sands with up to 48 % fines and 18 % clay fraction during the 1993 Hokkaido Nansai-Oki earthquake.

Moreover, existing semi-empirical SPT based procedures for the assessment of the undrained residual strength (Seed, 1987; Stark & Mesri, 1992) of sands consider only the presence of NP fines, implying that sands with plastic fines would be unlikely to liquefy, despite the extensive opposite field evidence mentioned above.

Most of the previous laboratory investigations concerning the effect of plastic fines on the monotonic behaviour of sands (Georgiannou et al. 1990; Ovando-Shelley & Perez, 1997; Wood & Kumar, 2000; Bouferra & Shahrour, 2004) have been concerned mainly with artificial clayey sands which were mixtures of clean sands with plastic fines only (kaolin, illite and montmorillonite). There has been limited number of studies on sands in which plastic and NP fines co-exist (Ni et. al 2006), which is the case most encountered in nature.

In laboratory studies, sands containing fines are considered as consisting of two matrices, the sand grains matrix and the fines matrix and their behaviour is analysed in terms of the interaction with each other (Thevanayagam et al. 2000). The nature of the contribution of sand and fines matrices may be expressed in terms of the intergranular and interfine void ratios, respectively. The intergranular void ratio, e_g, expresses the relative contribution of sand fraction on the behaviour of the mixture and is given by the following equation (Mitchell, 1976):

$$e_g = \frac{V_w+V_f}{V_g} = \frac{w\cdot\left(\frac{G_{sf}}{S_r}\right)+f_c}{(1-f_c)\cdot\left(\frac{G_{sf}}{G_{sg}}\right)} \tag{1}$$

where V_w is the volume of voids, V_f is the volume of the fines, V_g is the volume of sand grains, f_c is the fines content in decimal, w is the water content of the specimen, G_{sf} is the specific gravity of the fines and G_{sg} is the specific gravity of sand grains. For saturated specimens (S_r=100 %) and considering that $G_{sf} \approx G_{sg}$, the intergranular void ratio after the consolidation of the specimen is expressed as follows:

$$e_g = \frac{e_c+f_c}{1-f_c} \tag{2}$$

where e_c is the void ratio of the mixture after consolidation. Equations (1) and (2) hold for $f_c \le f_{cth}$.

Thevanayagam (2000) suggested the introduction of an additional parameter, b (0<b<1), in the expression of e_g in order to take into account the percentage of fines, which actually contribute to the maintenance of the contacts between the grains and consequently to sustaining the stresses, proposing for $f_c \le f_{cth}$, the equivalent intergranular void ratio, $(e_g)_{eq}$:

$$\left(e_g\right)_{eq} = \frac{V_w+(1-b)\cdot V_f}{V_g+b\cdot V_f} = \frac{e+(1-b)\cdot f_c}{1-(1-b)\cdot f_c} \tag{3}$$

The parameter b recognizes that different percentages of fines contribute differently to the strength of the sand. A value of parameter b equal to 0 indicates that there is no contribution of the fines in supporting the coarse grains skeleton at sustaining forces, while a unity value of b indicates that the total of fines, actively participate in sustaining the imposed to the coarse grain skeleton forces.

Ni et al. (2006) noted that the $(e_g)_{eq}$, concept was based on mixtures of sand with only one type of fines (either clay or silt) but in natural soils silt and clay coexist and their particles are characterized by differences in size, shape and mineralogy and hardness. They argued that in order to study the behaviour of such mixtures it is appropriate to consider the contribution of silt and clay separately and assign different contribution factors, b and a, respectively, proposing the following equation:

$$\left(e_g\right)_{eq} = \frac{V_w+(1-b)\cdot V_S+(1-a)\cdot V_C}{V_g+b\cdot V_S+a\cdot V_C} = \frac{e+(1-b)\cdot SC+(1-a)\cdot CC}{1-(1-b)\cdot SC-(1-a)\cdot CC} \tag{4}$$

where V_S is the volume of the silt fines, V_C is the volume of the clay fines, SC is the silt content, CC is the clay content, parameter b accounts for the contribution of silt fines (0<b<1) and parameter a accounts for the contribution of clay fines and may receive negative (a<0) values.

The Critical State theory is widely used as a characteristic state of soil behaviour, according to which there is a unique relationship between the void ratio, e, and the mean effective stress, p', which is expressed by the Critical State Line (CSL), in the e-log(p´) plane. According to the critical state concept, the behaviour of a sand depends not only on density, but also on stress level. The true state of a sand is described by the location of its current state of stress and volume relative to the CSL. When the state of a sand is above the CSL, the sand has the tendency to contract upon shearing, whereas when its state is below the CSL it has the tendency to dilate. Various normalized parameters have been proposed to characterize the difference between the actual state and the CSL. Been & Jefferies (1985) have quantified the distance of the current state from the CSL by means of a state parameter, $\psi=e-e_{cs}$, which is the difference in void ratios between the current state and the CSL at the current mean effective stress, p'_{cs}.

The objective of this study is the estimation of the values of parameters b and a, and consequently $(e_g)_{eq}$, in order to determine a single CSL at the $(e_g)_{eq}$-log(p´), plane, independently of the content of fines and their plasticity, for the tested sand-fines mixtures. The CSLs were obtained from undrained triaxial monotonic tests. The effects of silt and clay content and fines plasticity on the estimated b, and a, values is investigated together with the suitability of $(e_g)_{eq}$ to be used as an estimation parameter of the monotonic behaviour and undrained critical state strength of granular mixtures with various types and contents of fines. Based on the tests' results correlations of b with f_c based on the type of test are derived. The effectiveness of state parameter, ψ, and equivalent state parameter, $(\psi_g)_{eq}$, in the estimation of critical state strength of the tested sand-fines mixtures is investigated.

2 TESTED MATERIALS AND EXPERIMENTAL PROCEDURE

2.1 Tested materials

The materials used in the testing program were a clean natural quartz sand (S) with well-rounded grains, a NP silt (ground product of natural quartz deposits in Assyros, Greece), and Speswhite kaolin. The physical properties and the grain size distributions of the materials used are presented in Table 1 and Figure 1, respectively.

Tests were conducted on sand and on two groups of mixtures of sand with fines at f_c=5 % (SF5) and f_c=15 % (SF15). The physical properties of the tested mixtures are presented in Table 2. The fines fraction was obtained by mixing the silt and the kaolin at various weight proportions to vary the plasticity of fines. As shown in Table 2, for each one of the two tested groups of mixtures with f_c=5 % and 15 %, the change in the plasticity of fines reflects the change of clay content.

Table 1. *Physical properties of materials used.*

Mixture	G_s	e_{max}	e_{min}	D_{50} (mm)	D_{10} (mm)	C_u	5 μm< % <75 μm	% < 5μm	LL (%)	PL (%)	PI (%)
Sand (S)	2.649	0.762	0.544	0.30	0.24	1.3	-	-	-	-	NP[a]
Assyros silt	2.663	0.872	0.496	0.02	0.004	7.5	88	12	-	-	NP[a]
Speswhite kaolin	2.610	1.120	0.648	-	-	-	10	90	65	30	35

[a] NP=Non plastic

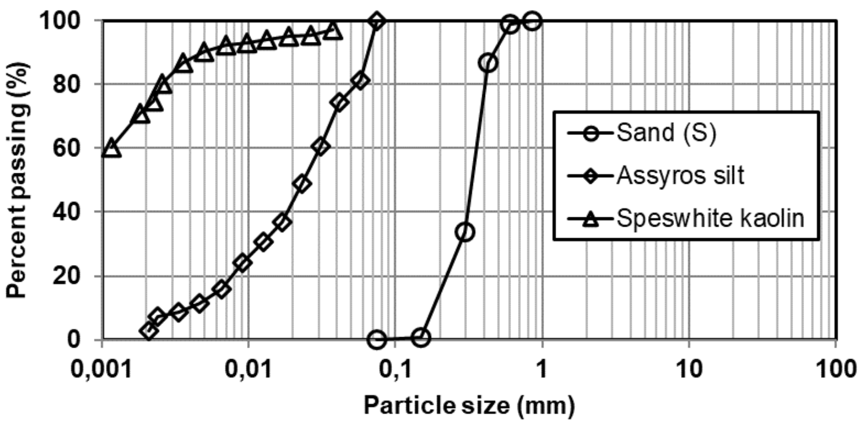


Figure 1. *Grain size distributions of sand (S), Assyros silt, and Speswhite kaolin, used in the tests.*

***Table 2.** Physical properties of tested mixtures.*

Mixture	f_c (%)	PI (%)	LL (%)	PL (%)	G_s	e_{max}	e_{min}	C_u	SC[a] (%)	CC[b] (%)	5 μm< % <75 μm	% < 5 μm
SF5(NP[c])	5	NP[c]	-	-	2.650	0.762	0.544	1.6	5	0	4.4	0.6
SF5(PI=6)	5	6	28	21	2.649	0.728	0.461	1.6	4.6	0.4	4.1	0.9
SF5(PI=12)	5	12	30	18	2.649	0.797	0.554	1.6	3.85	1.15	3.5	1.5
SF15(NP[c])	15	NP[c]	-	-	2.651	0.750	0.380	8.8	15	0	13.2	1.8
SF15(PI=12)	15	12	30	18	2.649	0.808	0.506	10.0	11.55	3.45	10.5	4.5
SF15(PI=22)	15	22	41	19	2.647	0.872	0.496	18.8	6.9	8.1	6.9	8.1
SF15(PI=30)	15	30	56	26	2.645	1.120	0.648	136.4	2.55	12.45	3.5	11.5

[a] SC=Assyros silt content, [b] CC=Speswhite kaolin content, [c] NP=Non plastic

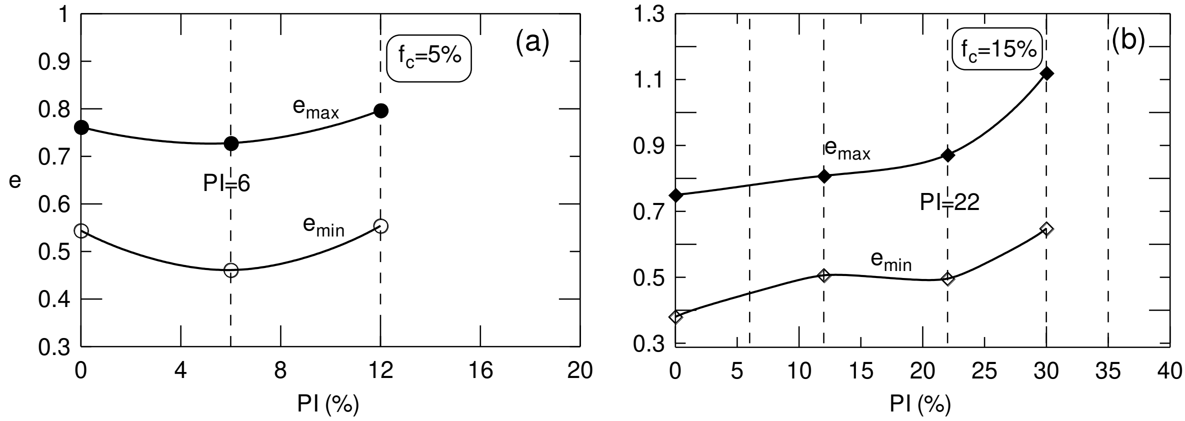


***Figure 2.** Variation of minimum, e_{min}, and maximum, e_{max} void ratios with plasticity index, PI, for the tested mixtures with fines (a) f_c=5 % and (b) f_c=15 %.*

For the determination of minimum and maximum void ratios, e_{min} and e_{max}, of the tested mixtures, the standard ASTM D 4253 and D 4254 methods, respectively, were used. Both these standard methods are applicable to soils that may contain up to 15%, by dry mass, of soil particles passing the No. 200 (75 μm) sieve, provided they have cohesionless characteristics. However, they were also used for the determination of e_{min} and e_{max} of the mixtures of sand with plastic fines to get an approximate estimation of their values to be used as a reference baseline. Figure 2 shows the variation of e_{min} and e_{max} with PI, for the tested mixtures. For the mixtures with f_c=5 %, e_{min} and e_{max} values decrease with increasing PI up to 6 % and increase thereafter with further increasing PI. For the mixtures with f_c=15 %, the rate of increase of e_{min} and e_{max} with PI increases significantly after the PI value of 22 %.

2.2 Experimental procedure

The testing program consisted of undrained monotonic triaxial tests conducted using an MTS closed-loop automatic cyclic triaxial apparatus (Papadopoulou, 2008).

The specimens (H/D≈100 mm/50 mm) were formed by moist tamping at a water content of 5-6 % using the undercompaction method, introduced by Ladd (1978). Moist tamping was preferred to other preparation methods, such as pluviation techniques, to achieve uniform density and homogeneous distribution of fine particles and to enable the formation of loose specimens. Saturation was achieved by the carbon dioxide (CO_2) method. In all the tests, the pore pressure parameter B had values ranging from 0.97 to 1.00. The specimens were isotropically consolidated under an effective isotropic stress, p'_0, ranging from 50 to 200 kPa and subjected to undrained compression at a constant rate of axial displacement of 0.05 mm/min. This rate of axial displacement ensures uniform distribution of excess pore water pressure within the specimen, confirmed also by the readings of the pore water pressure transducers installed at the top and bottom plates of the specimen in the cyclic triaxial apparatus (BS: 1377 Part 8:1990).

Tests were conducted on sand and mixtures of sand with fines at f_c=5 % (PI=0, 6, 12 %) and f_c=15 % (PI=0, 12, 22, 30 %) and at various densities and effective confining stresses, p'_0=50, 100 and 200 kPa.

3 MONOTONIC TESTS RESULTS

Figures 3a and 4a show the CSLs of the mixtures with f_c=5 % and 15 % respectively on the e_{cs}-p'_{cs}, plane. In the above figures the CSL of the sand is also presented for comparison. For each tested group of mixtures, at p'_{cs} below 300 kPa approximately, the CSLs are nearly parallel and have a small inclination. With increasing p'_{cs}, however, they steepen and start to converge at stresses above 1000 kPa. At f_c= 5%, the CSL on the e_{cs}-p'_{cs} plane of the NP samples is slightly below the CSL of the sand. When increasing PI to 6 %, the CSL of the mixture moves downwards and with further increasing PI to 12 % the CSL moves upwards. Similarly, at f_c=15 %, the CSLs of the mixtures on the e_{cs}-p'_{cs} plane move downwards with increasing PI up to 22 %; with further increasing PI to 30 %, the CSL moves slightly upwards.

In Figures 3b and 4b, the CSLs of the mixtures with f_c=5 % and 15 % respectively are presented on the $(e_g)_{eq,cs}$-p'_{cs}, plane. The solid line depicts the CSL of the sand while the dashed one the CSL of all the mixtures tested, including the sand, to show their coincidence. This was the criterion - the maximum possible coincidence of the CSL of each mixture with that of the sand - based on which the values of parameters a and b, were determined to calculate the $(e_g)_{eq}$ and determine a single CSL, for the sand and the sand-fines mixtures tested.

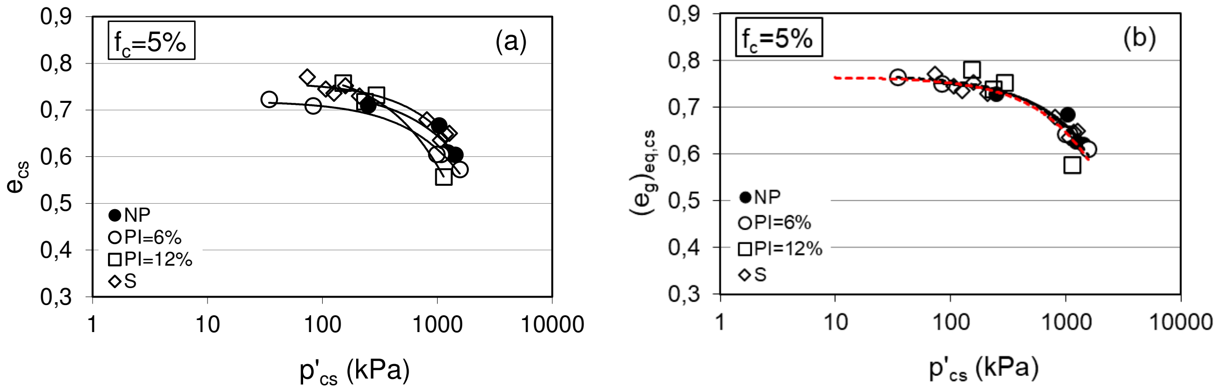


Figure 3. *Critical state lines of the sand and the mixtures with f_c=5% on the (a) e versus p'_{cs}, and (b) $(e_g)_{eq}$ versus p'_{cs}, planes.*

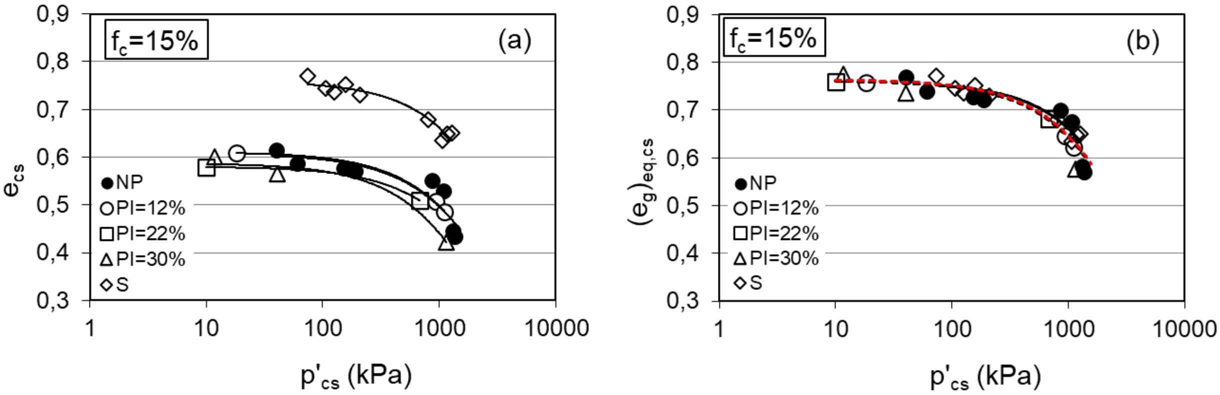


Figure 4. *Critical state lines of the sand and the mixtures with f_c=15% on the (a) e versus p'_{cs}, and (b) $(e_g)_{eq}$ versus p'_{cs}, planes.*

The variations of e_g and $(e_g)_{eq}$ with void ratio, e, - within the range of the maximum, e_{max}, and minimum, e_{min}, void ratio values-for the materials tested are presented in Figures 5a and 5b. As it can be seen in Figure 5a, e_g considers only the content of fines and not their type -silt or clay- and their plasticity, as fines are considered as voids. For a given f_c and e value the plasticity of fines in the mixtures may be attributed only by the variations of their index, e_{max} and e_{min} void ratios. For a given e and f_c, $(e_g)_{eq}$ considers the variation in fines plasticity by defining different $(e_g)_{eq}$-e relations, as shown in Figure 5b. In the latter it can be noted that for all the sand-fines mixtures tested and for a given e value, the corresponding $(e_g)_{eq}$ values are significantly lower than the corresponding e_g ones. The $(e_g)_{eq}$ concept considers the contribution of each type of fines separately, incorporating the effects of mineralogy, shape, size and plasticity besides their content.

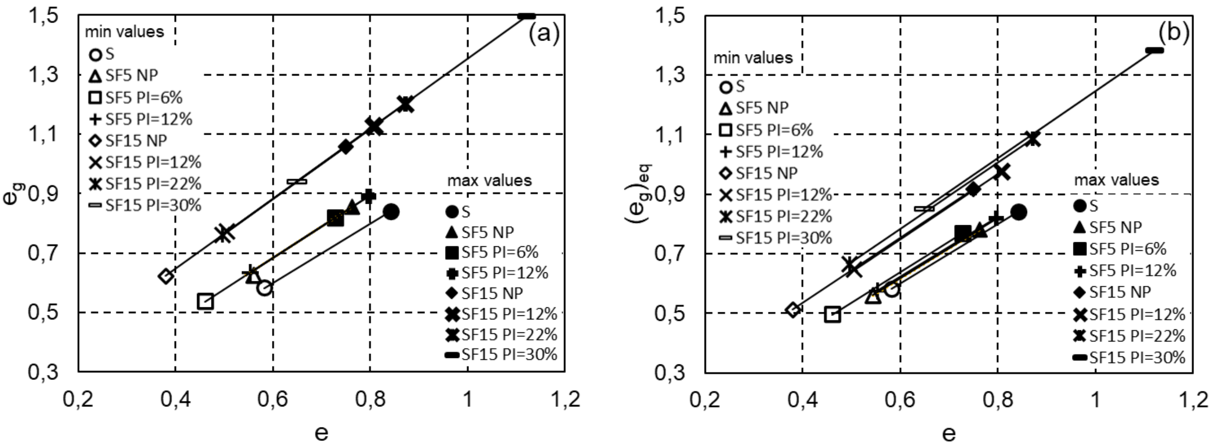


Figure 5. *Variation of void ratio, e, with (a) intergranular void ratio, e_g, and (b) equivalent intergranular void ratio, $(e_g)_{eq}$, for the tested materials, based on their e_{max} and e_{min} values.*

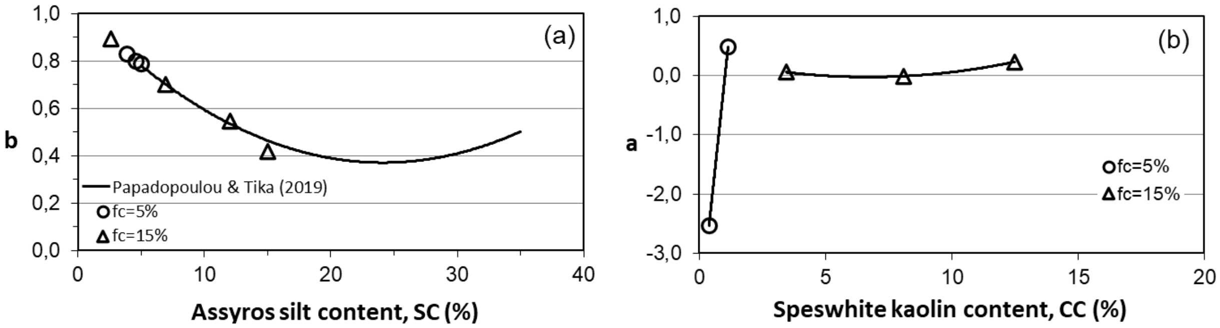


Figure 6. *Variation of $(e_g)_{eq}$ parameters (a) b with Assyros silt content, SC (%) and (b) a with Speswhite kaolin content, CC (%), for the tested mixtures.*

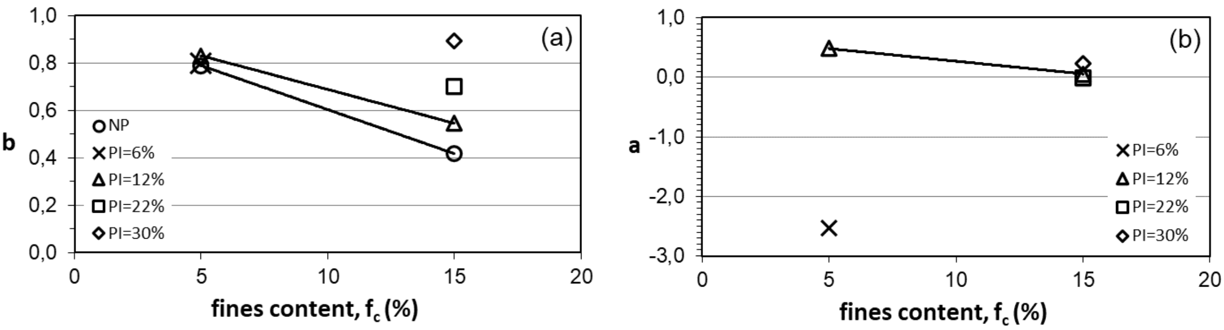
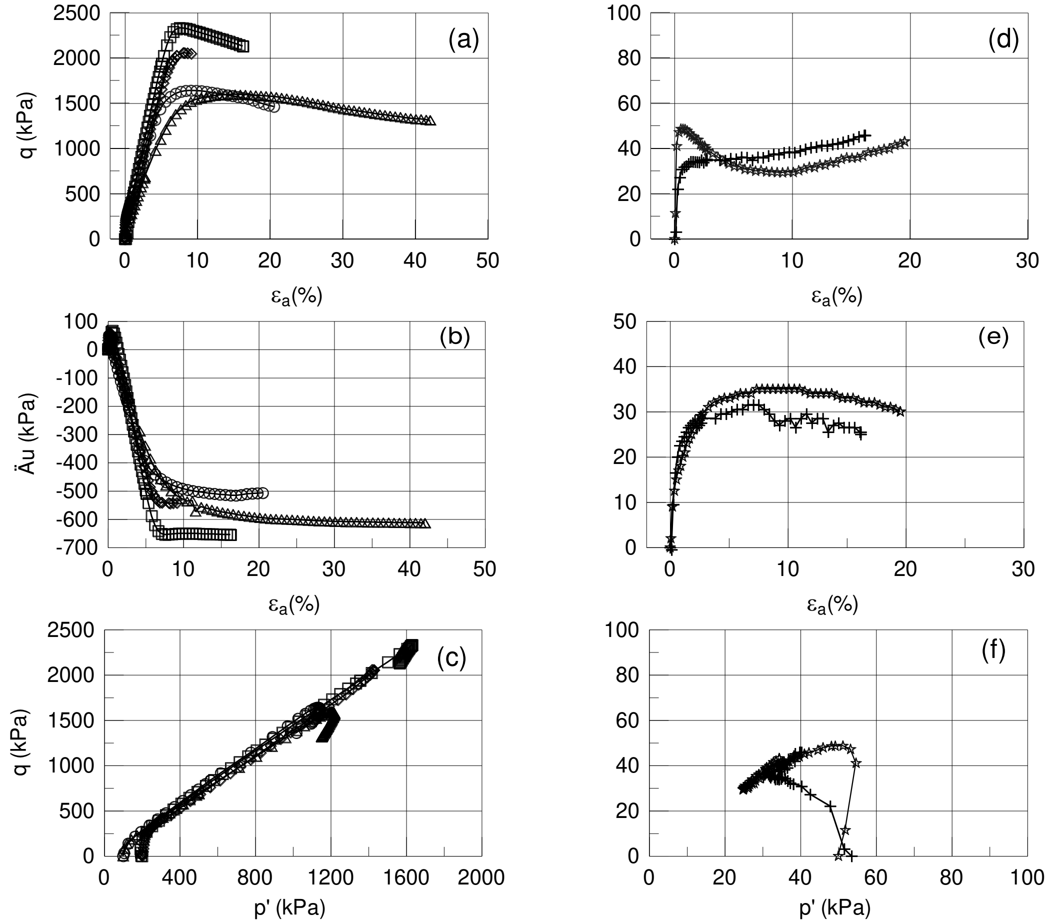


Figure 7. *Variation of $(e_g)_{eq}$ parameters (a) b and (b) a with fines content, f_c (%), for the tested mixtures.*

The variation of parameter b, with Assyros silt content, SC, for the tested mixtures, is given in Figure 6a. Papadopoulou & Tika (2019) estimated the values of parameter b for each one of the mixtures of sand (S) with 5, 10, 15, 25 and 35 % Assyros silt, in order to determine a single CSL, independently of the content of their silt fines. They found that the b-SC relation can be expressed satisfactorily by a quadratic polynomial equation, while the minimum value of b corresponds to a silt content of 25 %, lower than the threshold value of 35 %. With parameter b it is recognized that different silt contents contribute differently to sand's strength. The content of silt that practically contributes to sustain the imposed stresses, in sand-silt mixtures, depends on the particles' arrangement and changes as their percentage in the mixture increases, making the use of a constant value of b parameter for all silt contents not suitable. The determined from Papadopoulou & Tika (2019), b-SC relation is also presented in Figure 6a and was used for the estimation of the separate contribution of the silt particles through the b parameter for the various silt contents of the tested sand-fines mixtures. To account for the content of silt into the mixtures the content of the added Assyros silt, SC, was used instead of the percentage of particles sized between 5 μm and 75 μm to consider the differences in mineralogy between the silt sized particles of Assyros silt and Speswhite kaolin.

The part of the speswhite kaolin particles contribution into the sand-fines mixtures ability to sustain



Symbol	Tested material	f_c (%)	PI (%)	p'_0 (kPa)	e	e_g	D_r (%)	$(e_g)_{eq}$
○	SF5 PI12%	5	12	100	0.557	0.639	98.8	0.577
△	SF15 PI30%	15	30	100	0.422	0.673	147.9	0.577
□	SF5 PI6%	5	6	200	0.573	0.656	58.1	0.611
◇	SF5 NP	5	0	200	0.604	0.688	72.5	0.621
☆	SF15 NP	15	0	50	0.614	0.899	36.8	0.769
+	SF5 PI6%	5	6	50	0.723	0.814	1.9	0.764

Figure 8. *Monotonic behaviour of tested mixtures with similar $(e_g)_{eq}$ values, in the (a), (d) q-ε_a, (b), (e) Δu-ε_a, and (c), (f) q-p', planes.*

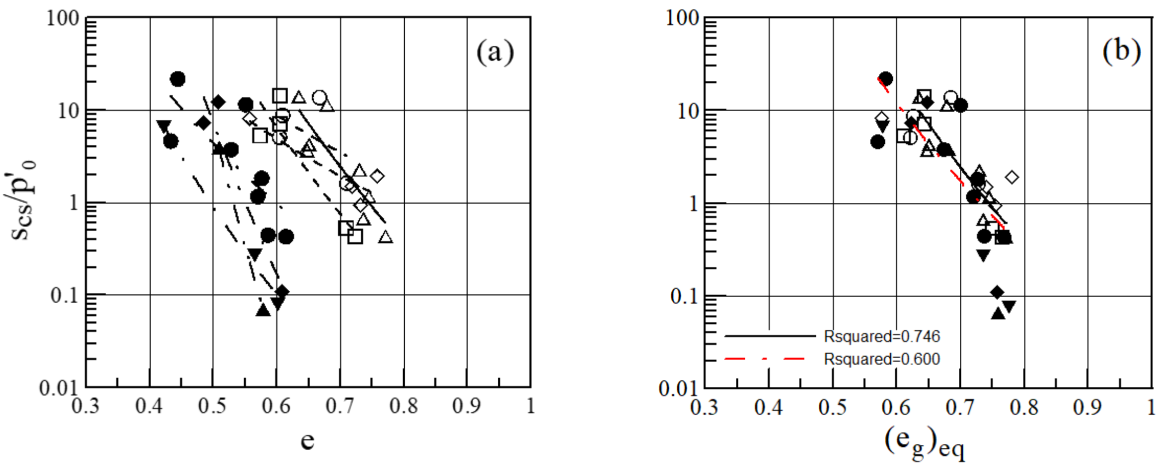
stresses should be accounted for separately since these particles are different from the silt particles in shape, size, plasticity and mineralogy, Ni et al. (2006). The variation of parameter a, with Speswhite kaolin clay content, CC, for the tested mixtures, is given in Figure 6b. The content of Speswhite kaolin added to the mixtures was considered as the CC, and not the percentage of particles sized <5 μm, to consider the different mineralogy of kaolin. Given the assumption that the contribution of the silt content in the sand - fines mixtures is the same as if silt was the only type of fines into the mixtures there is no sensitivity in the estimation of the values of parameter a. For f_c=5 %, as CC increases and SC decreases the contribution of kaolin from extremely negative (a=-2.53) becomes positive (a=0.48). For f_c=15 %, the a-CC relation shows minor variations. With an increase in CC reflecting the increase in fines plasticity, the contribution of clay is slightly positive (a=0.056) at PI=12 % and becomes negative (a=-0.011) with increasing PI to 22 % while it changes to again positive with further increasing PI to 30 %. For f_c=5 % and 15 % the contribution of clay content in the mixtures ability to sustain stresses is negative in the mixtures with fines plasticity 6 % and 22 %, respectively, and becomes positive with further increasing PI.

Figures 7a and 7b present the variations of parameters b and a, respectively with fines content where fines are both silt and clay in different proportions to achieve fines PI values of 0, 6, 12, 22 and 30 %. Both b and a decrease with increasing f_c for a given fines PI value, for the tested mixtures. For f_c=5 %

and 15 % the values of b increase with increasing PI as this increase corresponds to a decrease in the content of silt. In the mixtures with 5 % fines this increase is minor since the contents of silt are small (SC=3.85-5.0 %). As shown in Figure 7b, for f_c=5 %, although the Speswhite kaolin content is only 0,4 % (%<5μm=0.9 %) its contribution in the mixture's stability is extremely negative and can be explained by considering its microstructure while it becomes positive when CC=1.15 % (%<5μm=1.5 %). For f_c=15 % the variations of a with fines PI are within a narrow range of -0.011 to 0.232.

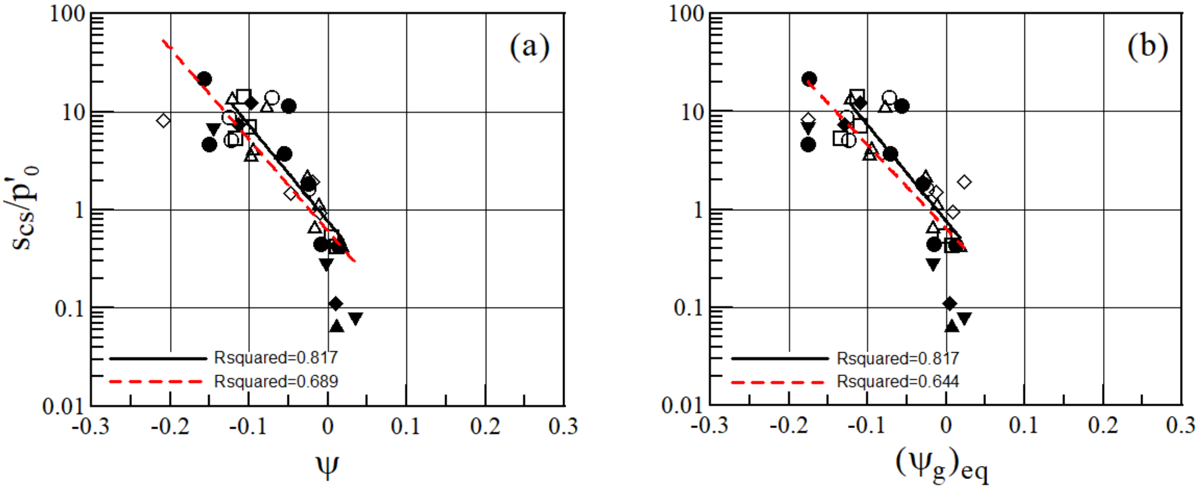
A crucial question is whether the sand-fines mixtures will have similar monotonic behaviour with that of the sand for a given $(e_g)_{eq}$. In Figures 8a-f the stress-strain diagrams, q-ε_a, the pore water pressure to strain diagrams, Δu-ε_a, and the deviatoric-mean effective stresses q-p', diagrams of three (3) pairs of sand-fines mixtures with similar $(e_g)_{eq}$ values, (0.577, 0.611-0.621 and 0.769-0.764), are presented. In Figures 8a-c dilative types of behaviour may be distinguished while in Figures 8d-f the behaviour is contractive. For similar values of $(e_g)_{eq}$, the developed monotonic behaviour displays a satisfactory approximation. If the comparison was made based on void ratio, e, or intergranular void ratio, e_g, or even relative density, D_r, the variations would be significant. Therefore, parameter $(e_g)_{eq}$, proves to be sufficient for the estimation of the monotonic behaviour of sand-fines mixtures based on the behaviour of their coarse grain fraction, given the correct determination of the values of parameters b and a, to separately consider the different contributions of silt and clay. This is important as soils containing both silt and clay in varying proportions are predominant in nature and are usually found in geotechnical works.

Figures 9a and 9b, present the variation of the normalized to the effective stress, p'_0, undrained critical state shear strength, $S_{cs}=q_{cs}/2$, defined at the ultimate state, with e and $(e_g)_{eq}$, respectively, for the tested sand-silt mixtures. A decrease of strength with increasing e and $(e_g)_{eq}$, is observed for all tested soils. For the range of e values tested the lowest values of S_{cs}/p'_0 are of the order of 0.110, 0.067 and 0.077 for f_c=15 %, p'_0=100 kPa and PI=12, 22 and 30 %, respectively. At the same e, the results indicate that the presence of (plastic or NP) fines reduce S_{cs} of the mixtures. As shown in Figure 9a, at a relatively narrow range of e, S_{cs}/p'_0 decreases with increasing PI up to 6 % and 22 % for f_c=5 % and 15 %, respectively and increases thereafter with further increasing PI. This may be attributed to the fact that at a constant e value, as f_c increases, e_g -considering fines as voids- and $(e_g)_{eq}$, -considering fines that do not participate in stress sustaining as voids -increase resulting in a looser structure of the sand grains into the mixture. In the case of fines with plasticity, $(e_g)_{eq}$, considers separately the effect of the particles of clay participating in the soil skeleton, which may be negative, resulting in an even looser structure of the sand grains into the mixture. In Figure 9b, the solid line represents the sand while the dashed one all the mixtures including the sand, to show their coincidence and the values of their coefficients of determination, $R_{squared}$, are also given. It is shown that given the density of a mixture through the void ratio, e, its fines content and by estimating the values of parameters b and a, $(e_g)_{eq}$, may be calculated



	PI (%)				
f_c (%)	NP	6	12	22	30
0	△				
5	○	□	◇		
15	●		◆	▲	▼

Figure 9. *Variation of the normalized undrained critical state strength, S_{cs}/p'_0, with (a) void ratio, e, and (b) intergranular void ratio, $(e_g)_{eq}$, for the sand and the mixtures with plastic and non-plastic fines.*



	PI (%)				
fc (%)	NP	6	12	22	30
0	△				
5	○	□	◇		
15	●		◆	▲	▼

Figure 10. *Variation of the normalized undrained critical state strength, S_{cs}/p'_0, with (a) state parameter, ψ, (b) equivalent intergranular state parameter, $(\psi_g)_{eq}$, for the sand and the mixtures with plastic and non-plastic fines.*

and by using the S_{cs}/p'_0 - $(e_g)_{eq}$, variation for the clean sand the critical state strength of the sand's mixtures at the various tested f_c and PI values with may be estimated with a good approximation.

Figures 10a and 10b show the variation of S_{cs}/p'_0 with ψ and $(\psi_g)_{eq}$, for the tested mixtures, respectively. A decrease of S_{cs} with increasing both ψ and $(\psi_g)_{eq}$, is observed, due to increasing contractiveness of the soils. The solid lines represent the sand while the dashed ones all the mixtures including the sand, to show their coincidence. Equally, the state and the equivalent state parameters, ψ and $(\psi_g)_{eq}$, prove to be suitable and effective in the estimation of the undrained critical state strength. These correlations should be used with engineering judgement in the cases where $\psi \geq 0$.

4 CONCLUSIONS

From the results of the present study the following conclusions can be drawn:

The behaviour of the mixtures deteriorates initially with increasing PI up to a threshold value, PI_{th}, and improves thereafter with further increasing PI. For the tested mixtures, the PI_{th} value is of the order of 6 % and 22 % at f_c=5 % and 15 %, respectively.

At a given value of e and f_c, the undrained critical strength of the mixture decreases with increasing PI of fines up to PI_{th}, and increases thereafter with further increasing PI. Similarly, the CSLs of the mixtures with f_c=5 % and 15 % move downwards relative to the CSL of the sand on the e-p'_{cs} plane up to PI_{th} and then upwards with further increasing PI.

The equivalent intergranular void ratio, $(e_g)_{eq}$, is an appropriate density parameter for the expression of the monotonic behaviour through a single CSL of granular mixtures irrespectively of fines PI and content. In mixtures of sand with silt and clay as fines, the contribution of each type of sand should be considered separately since the silt fines exhibit a positive contribution while the contribution of the clay fines may be extremely negative. The parameters b, and a, depend on the content of fines (silt and clay) on the mixture and their plasticity.

The state parameter, ψ, and the equivalent state parameter, $(\psi_g)_{eq}$, prove to be sufficient parameters for the estimation of the monotonic behaviour and undrained critical state strength, S_{cs}, for granular mixtures. The correlations should be used with engineering judgement in the cases of contractive behaviour, $\psi > 0$.

In existing semi-empirical procedures for the estimation of undrained residual strength, the type of fines (silt or clay) and their plasticity should be accounted for besides their content, to avoid overestimations.

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