

Shear strength and pore-water pressure characteristics of sandy soil mixed with plastic fine

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Abstract Recent earthquake case histories have revealed the liquefaction of mixtures of sand and fine particles during earthquakes. Different from earlier studies which placed an emphasis on characterisation of liquefaction in terms of the induced shear stress required to cause liquefaction, this study adopted a strain approach because excess pore-water pressure generation is controlled mainly by the level of induced shear strains. The current study includes the results of a set of laboratory tests carried out on sand specimens with the same relative densities and variation in the plastic fines (kaolinite or bentonite) contents ranging from 0 to either 30 % and consolidated at mean confining pressure of 100, 200 and 300 kPa using static triaxial test apparatus, in order to study the influence of fine content and other parameters on the undrained shear strength and liquefaction potential of clayey sand specimens; also, pore-water pressures in the specimens are discussed. Results of tests show that the peak strength decreases as the fines (kaolinite or bentonite) content increases up to a threshold content of fines (FC_{th}) after which, increases in plastic fine content lead to improve the peak shear strength of specimens, and also the ultimate steady-state strength has been improved due to the increased in plastic fines content. Also, pore

pressure build-up in clayey sands is generally slower than that observed in pure sand.

Keywords Shear resistance · Clayey sand · Void ratio · Kaolinite · Bentonite · Static triaxial test

Introduction

The term “liquefaction” has taken on different meanings in the literature, and it is therefore important to start by defining it and other key terms used in this report. The terms “sand-like” and “clay-like” are used in this report to describe fine-grained soils whose stress–strain behaviour during monotonic and cyclic undrained shear loading is fundamentally similar to that of sands and clays, respectively. On the other hand, liquefaction of saturated granular soils during earthquakes has been one of the most important and challenging problems in the field of geotechnical earthquake engineering. During earthquakes, the ground shakes, causing cohesionless soils to lose their strength and behave like liquid. This phenomenon is called soil liquefaction and occurs due to an increase in the excess in pore-water pressure and a corresponding decrease in the effective overburden stress in a soil deposit which will then cause settlement of buildings, landslides, failures of earth dams or other hazards.

Soil mixtures such as silty clays, clayey sands and silty sands are more commonly found in nature and in earth structures than pure sands, silts and clays. However, the behaviour of soil mixtures is not well understood since soil mechanics has concentrated mostly on pure soils. In fact, most liquefaction research was carried out on clean sands with the assumption that the behaviour of silty sands is similar to that of clean sands. In the last decade for instance, research studies related to the influence of fines on liquefaction potential accelerated markedly. The purpose of these

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investigations was to quantify the effect of fines on the liquefaction potential of soils containing non-plastic and plastic fines. It has been indicated since the 1960s that the presence of fines will affect a resistance of sands to liquefaction. Nevertheless, a review of studies published in literature shows that no clear conclusions can be drawn regarding how altering the fines fraction affects the liquefaction resistance of sand under monotonic and cyclic loading. Several laboratory studies have been carried out and have shown what appear to be conflicting results. Recent researches made by ASTM D 4767-92 (2002), Zlatovic and Ishihara (1995), Lade and Yamamuro (1997), Thevanayagam et al. (1997), Thevanayagam (1998), Yamamuro and Lade (1998), Amini and Qi (2000), Naeini (2001), Naeini and Baziar (2004), Della et al. (2009) and Belkhatir et al. (2010) indicate that sand deposited with silt content is much more liquefiable than clean sand, decrease the liquefaction resistance of the sand (Shen et al. 1977; Troncoso and Verdugo; 1985; Georginnou et al. 1990; Finn et al. 1994; Vaid 1994; Erten and Maher 1995; Zlatovic and Ishihara 1997) or decrease the liquefaction resistance until some limiting fines content (FC_{th}) is reached and then increase its resistance (Law and Ling 1992; Koester 1994; Bouferra and Shahrou 2004). Moreover, numerous studies have reported that the behaviour of sand mixed with fine grained soils depends mainly on the fines content and the plasticity index (such as Ishihara et al. 1980). Indeed, to a certain limiting value of fines content, fine-grained soils occupy solely the voids and do not influence the behaviour of the mixture. For this reason, the notion of skeleton void ratio was suggested to characterise those kinds of soils (Kenny 1977; Mitchell 1993).

These experimental studies have focused on systematically evaluating the effect of plastic fines on liquefaction resistance; however, most of these studies employed cyclic tests. Koester (1994) presented data from cyclic triaxial testing on reconstituted samples of mixtures of sand and fine particles. The results from this study indicated that the cyclic strength generally decreases with increasing fines content up to 20 % fines, after which the cyclic strength increases. Tianqiang and Prakash (1999) synthesised data from various studies on the liquefaction potential of undisturbed and reconstituted silts and silty clays. They concluded that plasticity index (PI) is an important parameter affecting the liquefaction resistance of soils, with the cyclic strength decreasing with increasing PI at low level of plasticity (PI=0–5) but increasing with increasing PI at larger levels of plasticity (PI>about 10).

Pore pressure generation is caused by the rearrangement of soil particles under undrained conditions, and this rearrangement of particles is best characterised by shear strain. Hazirbaba (2005) conducted multiple series of strain-controlled cyclic direct simple shear and cyclic triaxial tests to study the pore pressure generation characteristics of sands

and non-plastic silty sands. The soil specimens were tested at a constant relative density, constant sand–skeleton void ratio and constant overall void ratio. The overall evaluation of these three categories at different fines contents indicated decreasing pore-water pressure, in general, with increasing fines content up to 20 % fines. El Hosri et al. (1984) present data regarding the rate of excess pore pressure development from testing of undisturbed samples of silts and clayey silts. Considering the development of excess pore pressure as a function of the cycle ratio during stress controlled testing, they found that the silts initially displayed a higher rate of pore pressure generation than sands, but that this rate decreased significantly after a pore pressure ratio of about 0.8. For these silts, which had 60–100 % fines, the cyclic strength increased with increasing PI.

Besides, El Mohtar (2008) studied experimental results from resonant column and cyclic and monotonic triaxial tests performed on Ottawa sand specimens prepared with 0, 3 and 5 % bentonite by dry mass of sand. While the bentonite has minimal effects on the static properties of the sand, the cyclic resistance increases significantly. Specifically, for the same skeleton-relative density and cyclic stress ratio, the cyclic tests show a marked increase in the number of cycles required for liquefaction with increased bentonite content.

On the other hand, when relatively loose sand is strained in undrained shearing beyond the point of peak strength, the undrained strength drops to a near constant value over a large strain. This strength is conventionally called the undrained steady-state strength or residual strength. However, if the strength increases after passing through a minimum value, the phenomenon is called limited or quasi-liquefaction. The residual strength S_{us} may be defined as (Ishihara 1993);

$$S_{us} = (q/2) \cos \phi_s = (M/2) \cos \phi_s \left(p'_s \right) \quad (1)$$

$$M = (6 \sin \phi_s) / (3 - \sin \phi_s) \quad (2)$$

Where q , p'_s and ϕ_s indicate the deviator stress ($\sigma'_1 - \sigma'_3$), the effective mean principal stress ($\sigma'_1 + 2\sigma'_3/3$) and the mobilised angle of inter-particle friction at the quasi-steady state, respectively.

Steady state is a state of deformation of soil without effective stress increment or decrement and with no migration of pore water. This means that in the steady state, in a saturated soil element, all the components of effective stress increment are zero whereas at least one component of strain increment is not zero without volume change. This state appears at medium to large deformation during undrained shearing of sand. If the density of the sand is high, almost no drop in effective mean stress can be seen and the behaviour is considerably stiff. When the sand is looser, a point of

minimum effective mean stress appears where the dilatancy behaviour changes from contractive to dilative. This point was named phase transformation by Ishihara et al. (1975). If the void ratio of the sand is larger than about 0.88, we can see a steady state at phase transformation, following the drop of shear stress, where both the shear stress and the effective mean stress reach a minimum. This transient steady state has been called the quasi-steady state (Alarcon-Guzman et al. 1988). It may be noticed that when the sand becomes looser or the initial consolidation stress becomes larger, the phase transformation lines tend to converge on the ultimate steady-state line. If the phase transformation or quasi-steady state coincides with the ultimate steady state, no hardening will be observed after the minimum, post-peak stress condition. This condition will be referred to as the critical steady state. The characteristics of undrained shear behaviour of sand, classified as no excess pore-water pressure, phase transformation, quasi-steady state and critical steady state are schematically illustrated in Fig. 1 (Yoshimine and Ishihara 1998).

Materials tested

Mixtures of sand and kaolinite or bentonite were used in this study. The gradations of these three soils are shown in Fig. 2 and mixtures were tested at 0, 5, 10, 15, 20 and 30 % fines by weight. The used sand has a coefficient of uniformity (C_u)=0.38, coefficient of curvature (C_c)=0.25 and specific gravity (G_s)=2.66. Also, some of characteristics of used kaolinite and bentonite are presented in Table 1.

Experiments procedures

Sample preparation technique

Numerous studies have reported that behaviour of sand can be greatly influenced by specimen reconstitution. The effect

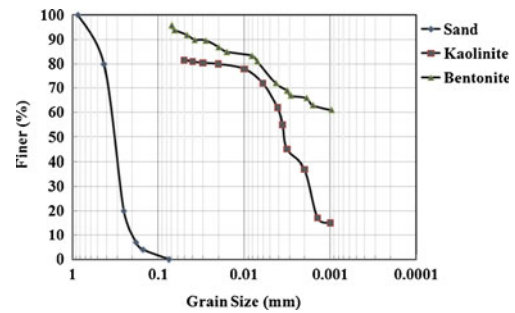


Fig. 2 Grain-size distribution curves of used sand, kaolinite and bentonite

of the preparation method used for the samples has been subjected to many controversial researches and several studies have reported that the resistance to liquefaction is more elevated for samples prepared by the method of sedimentation than for samples prepared by other methods, such as the dry funnel pluviation and the wet deposition (Zlatovic and Ishihara 1997); others find that the resistance to the liquefaction of the samples prepared by wet deposition more elevated than by dry funnel pluviation (Mulilis et al. 1977; Yamamuro and Wood 2004). Benahmed et al. (2004) as well as Canou (1989) and Ishihara (1993) presented results showing that the tests prepared by dry funnel pluviation are more resistant than those prepared by wet deposition. Vaid et al. (1999) confirmed this result while showing that wet deposition encourages the initiation of the liquefaction in relation to a setting up by pluviation under water. Yamamuro et al. (2008) showed that the method of dry pluviation supports the instability of the samples contrary to the method of sedimentation. Wood et al. (2008) found on their side that the effect of the method of deposition on the undrained behaviour decreases, when the density increases. They also found that this influence decreases with the increase of the fines content, particularly with the lower densities.

Since there were different possible modes of formation of the natural sandy solid masses, the use of the two modes of deposition of the sand, allows one to approach to the reality

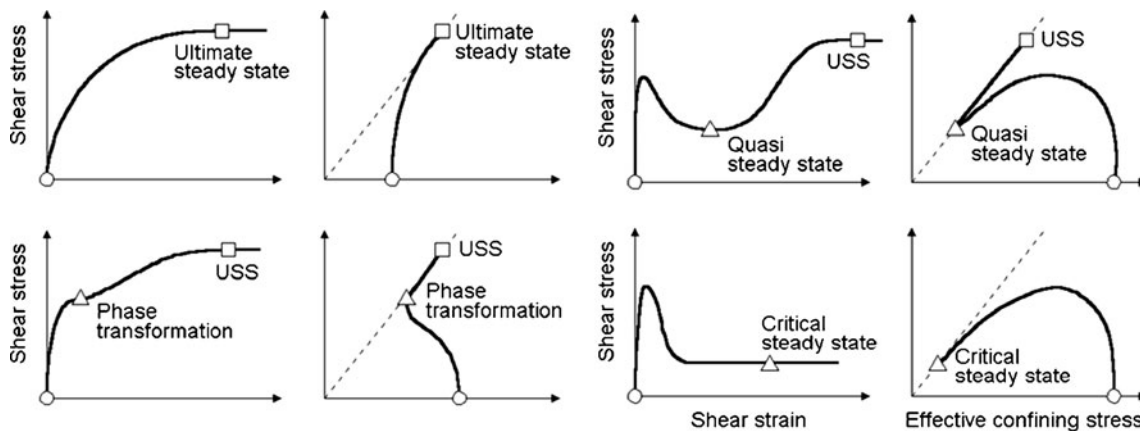


Fig. 1 Quasi-steady state and critical steady-state concepts

Table 1 Characteristics of used clayey fines

Type of clayey fine	Specific gravity	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)
Kaolinite	2.59	49	31	18
Bentonite	2.81	341	38	303

of the area, the final purpose being the characterisation of the behaviour of this sand to liquefaction, especially since the region is known for its high seismicity and soil liquefaction. On the other hand, these works indicate that the choice of a proper specimen preparation technique is important in determining the liquefaction potential of sands. Current field sampling techniques are not readily able to produce high-quality undisturbed granular soil specimens for laboratory testing at an affordable cost. Accordingly, numerous sample reconstitution methods have been developed for use in the laboratory. Among these methods, wet compaction technique has the advantage that it is relatively easy to control the global specimen density achieved, even for loose specimens (Frost and Park 2003). In this work, wet compaction technique has been used for preparation of specimens. All the specimens have been constructed with the diameter of 70 mm and the height of 140 mm, using seven layers with a constant thickness of 20 mm for each layer, so height to diameter ratio of 2 is kept constant. This length to diameter ratio of 2 selected in order to minimise the effects due to end platens of the apparatus and to reduce the likelihood of buckling during testing. The specimens are prepared with the help of a mould constituted two semi-cylindrical shells. The two shells can easily be joined or embossed one with the other with the help of a hose clamp. In order to maintain the cuff made of latex along the partitions of the mould, four aspiration ducts are pierced in the conducted shells. These ducts communicate with the inside of the mould by rows of small holes. They are joined to flexible hoses that are assembled in a single tube. This last one can be connected to a void pump.

Initial degree of saturation

For sample preparation, dry sand and fine (kaolinite or bentonite) have been mixed with respect to the considered different weight ratios. To prevent the grains separation, while soil is being poured in the mould and to achieve the desired density easily, for all other samples, the initial moisture percentage is kept constant equal to 8 %. The required amount of dry soil mass and water for each layer of samples have been determined exactly.

Final degree of saturation

The saturation is an important stage in the experimental procedure because the response of the sample under

undrained loading depends on its quality. To get a good degree of saturation, the technique of carbon dioxide elaborated by Lade and Duncan (1973) was used. This technique consists of making the carbon dioxide circulate through the circuits of drainage and the sample to weak debit during a certain time, in order to occupy all the voids to chase the air contained in the sample. Then, we make the deaerated and demineralised water circulate to chase the interstitial gas and to occupy its place. In spite of the passage of water, some voids remain occupied by the carbon dioxide. As the solubility of the gas is raised, water can dissolve what remains of the carbon dioxide after its passage, it generally permits us to ensure a good saturation of the sample. In this study, after doing necessary measurements, the specimens have been first subjected by CO₂ at least for 30 min and then saturated them by de-aired water. The control of saturation degree is done by means of Skempton's pore pressure parameter *B*. According to JGS 0523 (2000) and ASTM D 4767-92 (2002), specimens have been considered to be fully saturated if *B* is at least equal to or greater than 0.95. In this study, backpressure of 300 kPa has been applied during the tests to achieve the saturation state.

Consolidation and loading

After the saturation process, the specimens were subjected to the confining stress for consolidation. During consolidation, the difference between confining pressure and backpressure has been arranged such that for each specimen the effective consolidation pressure was fixed equal to 100, 200 or 300 kPa. There are numerous factors affecting the undrained behaviour of silty sand, such as sand grading characteristics, fine content, initial density and confining stress. In the present work, the effects of different factors were studied for a given initial density. The choice of 100, 200 and 300 kPa for confining stress as a mean value in geotechnical practice purposes was based on various research works existing in literature, such as Ishihara's (1993). After consolidation step and calculation of e_0 (e.g. void ratio after consolidation), axial load has been applied on specimen in constant strain rate manner.

Results of tests

Figures 3, 4 and 5 show the stress–strain curves and changes of pore-water pressure during shearing stage of sand

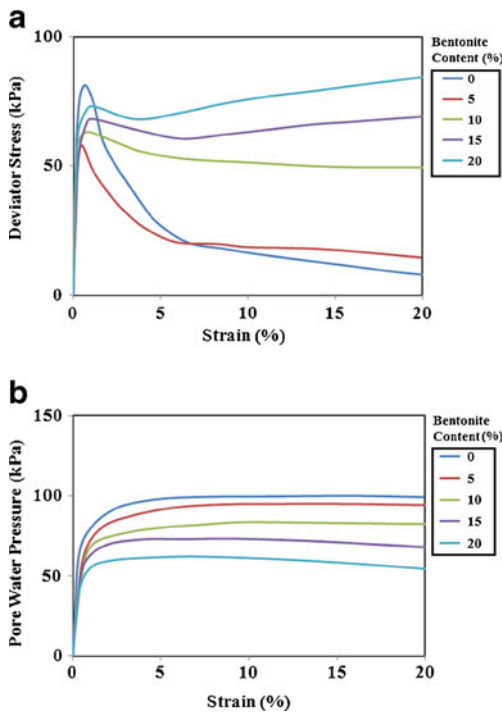


Fig. 3 Mechanical behaviour of specimens of sand–bentonite mixture at 100 kPa confining pressure. **a** Stress–strain curves. **b** Pore-water pressure–strain curves

specimens with various bentonite contents (i.e. 0, 5, 10, 15 and 20 %) at net confining pressures of 100, 200 and

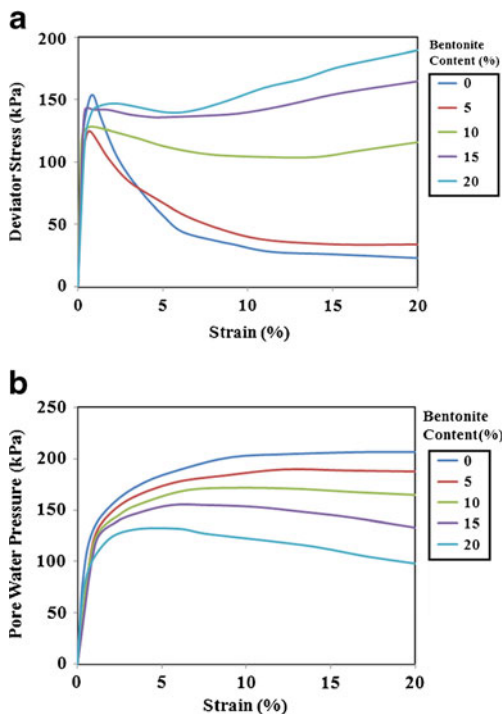


Fig. 4 Mechanical behaviour of specimens of sand–bentonite mixture at 200 kPa confining pressure. **a** Stress–strain curves. **b** Pore-water pressure–strain curves

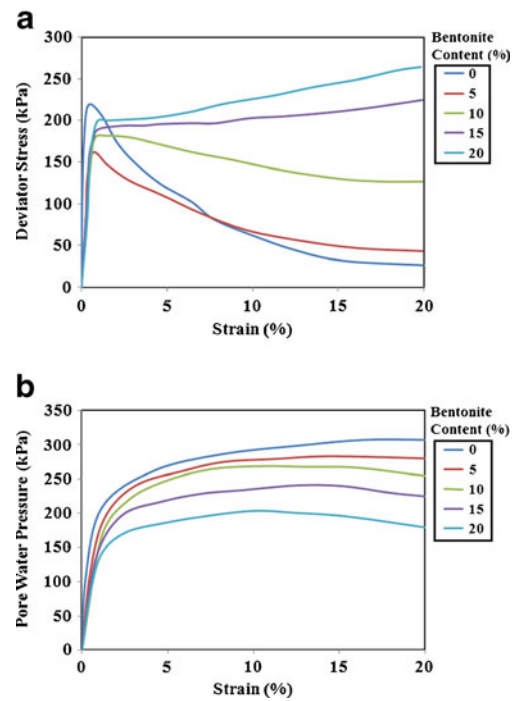


Fig. 5 Mechanical behavior of specimens of sand–bentonite mixture at 300 kPa confining pressure. **a** Stress–strain curves. **b** Pore-water pressure–strain curves

300 kPa. Also, the test results of the sand specimens with various kaolinite contents (i.e. 0, 5, 10, 20 and 30 %) are presented in Figs. 6, 7 and 8. As expected, the undrained

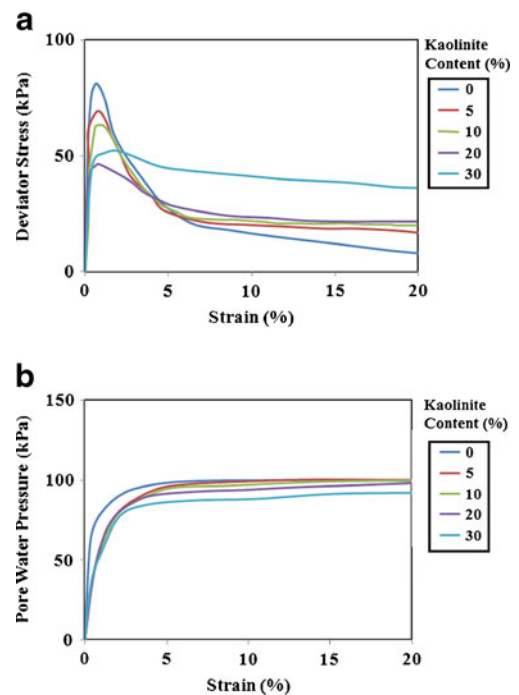


Fig. 6 Mechanical behavior of specimens of sand–kaolinite mixture at 100 kPa confining pressure. **a** Stress–strain curves. **b** Pore-water pressure–strain curves

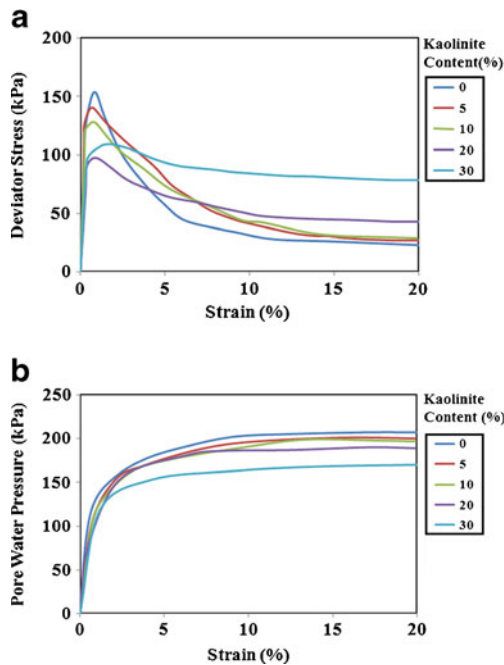


Fig. 7 Mechanical behaviour of specimens of sand–kaolinite mixture at 200 kPa confining pressure. **a** Stress–strain curves. **b** Pore-water pressure–strain curves

resistance of specimens increased as the confining pressures increased for a given kaolinite or bentonite content. Also, it was observed that fine content has considerable effects on stress–strain curves for the same net confining pressure.

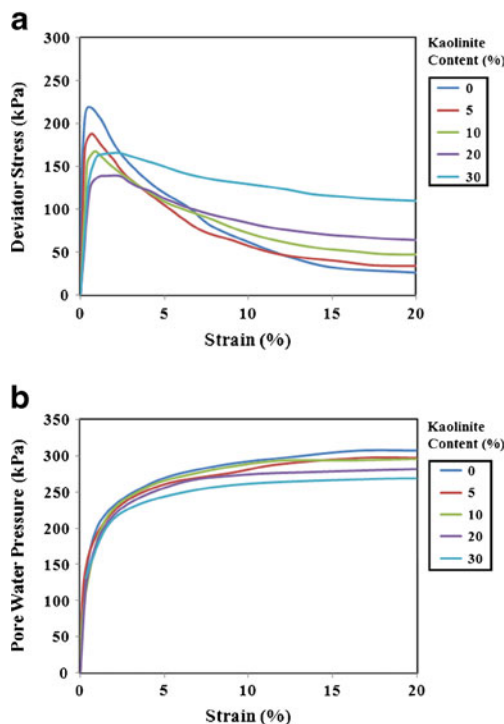


Fig. 8 Mechanical behaviour of specimens of sand–kaolinite mixture at 300 kPa confining pressure. **a** Stress–strain curves. **b** Pore-water pressure–strain curves

As shown in of Figs. 3a, 4a and 5a, for a given net confining pressure, the specimen containing 5 % bentonite (threshold value) has the lowest initial peak strength while the specimen containing 0 % bentonite has the highest peak strength (also shown in Fig. 9). On the other hand, the pure sand specimen has the lowest ultimate steady-state strength while the specimen containing maximum value of bentonite (20 %) has the highest steady-state strength for a given net confining pressure.

For the specimens of sand–kaolinite mixture, increasing of kaolinite content up to 20 % results in a decrease in the peak shear strength of the specimens, after which increasing of kaolinite content leads to an increase in the peak shear strength of the specimens. On the other hand, the specimen containing 20 % kaolinite content (threshold value) has the lowest peak strength for a given net confining pressure (see Figs. 6, 7, 8, 9). Besides, the specimen containing 0 % kaolinite content exhibits the highest peak shear strength and positive excess pore-water pressure than the other specimens containing kaolinite at a given net confining pressure.

One possible explanation for this behaviour is that at low amounts of plastic fine (less than threshold value), fine particles occupy locations near the contact points between the sand grains and can hold the sand grains slightly apart. These fines result in breaking the load bearing chain in the specimen and this will lead to a decrease in the peak shear strength. This happened while the ultimate steady-state strength slightly increased due to the increase in plastic fine content. The reason of this phenomenon is that slips of sand grains are less than that during the beginning of the loading stage. This leads to the increase of soil compressibility and the decrease of soil undrained resistance.

By increasing plastic fine content (more than threshold value), some of the fines are located in the voids between sand grains. The results of the tests show that plastic fines have the main role in determining the behaviour of soil at the state of $FC > FC_{th}$. In this case, plastic fines are close to each other and sand grains break away. This means that silt fines play the main role in soil-bearing skeleton. In other words, PI of plastic fine has an important role at this state.

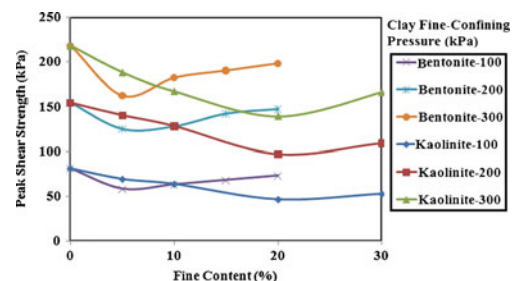


Fig. 9 Results of tests, plastic fine content plotted against peak strength

However, if the strength increases after passing through a minimum value (i.e. threshold value), the phenomenon is called limited or quasi-liquefaction. Hardening is observed following quasi-steady state in our laboratory testing. The authors stated that the hardening is mainly due to the end restraint of the specimen and not a true behaviour of sand.

As shown in the results of tests, the specimen containing 0 % fine (bentonite or kaolinite) behaves in a more contractive manner as indicated by the increase in positive excess pore-water pressure. This excess pore-water pressure decreased due to increasing amount of fine content.

Figure 9 shows that for the specimens containing 10 % of kaolinite or bentonite, values of peak shear strengths are close together. Also, it was observed that bentonite content has considerable effects on the mechanical behaviour of sand than kaolinite. In other words, before and after this point, decreasing or increasing of shear strength is

considerable in the sand–bentonite specimens. This different effect is due to the higher plastic index of bentonite (Table 2 presents the summary of the physical parameters of specimens). Our results are in good agreement with previous research findings (such as Tianqiang and Prakash 1999; Seed et al. 2003; Ghahremani and Ghalandarzadeh 2006; Ghahremani et al. 2006; Gratchev et al. 2007; El Mohtar 2008; Bayat and Bayat 2012).

Conclusions

A series of consolidated undrained static triaxial tests were carried out on sand with low plastic fines (kaolinite and bentonite) contents at mean confining pressures of 100, 200 and 300 kPa. The effects of variation in the fines content and PI were studied. In the light of the experimental evidence, the following conclusions can be drawn:

1. The undrained behaviour of compaction specimens for different confining pressures is not substantially different. As the confining pressure is increased, the liquefaction potential of silty sands is decreased for both mixed specimens with kaolinite and bentonite.
2. The results from this study reveal that PI significantly affects the pore pressure generation response of sands. As the positive excess pore-water pressure decreases for sand mixed with both clay fines (kaolinite and bentonite) and this decrease is greater for the specimens reconstructed with bentonite than specimens reconstructed with kaolinite. On the other hand, bentonite has more effects on mechanical behaviour of sand than kaolinite. In this case, increasing of bentonite content was done to create a slightly more dilative response. This shows that PI is affected factor on the shear strength and excess pore-water pressure of sand-mixed plastic fine.
3. The shear behaviour of specimens reconstructed with low kaolinite content is same of behaviour of loose sand. While the shear behaviour of specimens reconstructed with bentonite is completely different with clean sand. The peak strength of specimens decreased as the fines content increases up to a threshold value of fines content (20 % for kaolinite and 5 % for bentonite) after which it increases, and also steady-state shear strength increased due to the increasing plastic fine (kaolinite or bentonite) content.
4. Results of the current study suggest for bentonite or kaolinite to be successful in increasing liquefaction resistance over the threshold values (FC_{th}) is necessary to be added to a sand specimen. Also it is found that, by increasing plasticity of fine, FC_{th} decreased. In other words, the threshold fine content depends on the characteristics of base sand and fine particles.

Table 2 Summary of the physical parameters of specimens

Fine	Fine content (%)	Confining pressure (kPa)	Void ratio	Plasticity index
Kaolinite	0	100	0.84	N.PI
		200	0.83	N.PI
		300	0.85	N.PI
	5	100	0.86	N.PI
		200	0.84	N.PI
		300	0.86	N.PI
	10	100	0.85	N.PI
		200	0.84	N.PI
		300	0.84	N.PI
	20	100	0.86	N.PI
		200	0.85	N.PI
		300	0.84	N.PI
30	100	0.83	N.PI	
	200	0.86	N.PI	
	300	0.85	N.PI	
Bentonite	0	100	0.84	N.PI
		200	0.83	N.PI
		300	0.85	N.PI
	5	100	0.86	N.PI
		200	0.83	N.PI
		300	0.85	N.PI
	10	100	0.86	15
		200	0.84	15
		300	0.84	15
	15	100	0.86	24
		200	0.83	24
		300	0.85	24
20	100	0.83	43	
	200	0.86	43	
	300	0.85	43	

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