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Effect of principal stress rotation in cement-treated sands using triaxial and simple shear tests

A. H. Eghbali[1], K. Fakharian[2,]*

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Abstract

Portland cement can be mixed with sand to improve its mechanical characteristics. Many studies are reported in literature on this topic, but the effect of principal stress rotation has not been investigated yet. Considering the inherent anisotropy of most sands, it is not clear whether the added cement shall contribute to equal increase in strength and stiffness at vertical and horizontal directions or not. Furthermore, it is not well understood how the cement as an additive in non-compacted (loose) sand compared to compacted (dense) sand without cement, contributes to improving the material behavior in undrained condition such as limiting the deformations and the liquefaction potential. In this research, undrained triaxial and simple shear tests under different stress paths are carried out on different mixtures of Portland cement (by adding 1.5, 3 and 5 percent) with clean sand to investigate the effect of principal stress rotations. The triaxial test results revealed that the cement mixture reduces the anisotropy, while it improves the mixture mechanical properties compared to compacted sand without cement. The results of the simple shear tests validated the triaxial test results and further clarified the effect of the α parameter or rotation of principal stresses on the behavior of cemented sand mixtures.

Keywords: *Cemented sand, Portland cement, Stress path, Anisotropy, Rotation of principal stresses, Triaxial, Simple shear.*

1. Introduction

Uniformly distributed fine sands are formed on coastal zones resulted from deposition of rivers, having loose structure especially at shallow depths. Low density saturated fine sands are susceptible to failure even under static loads. When subjected to cyclic loads such as earthquake, the pore water pressure rises followed by shear strength loss resulted in initial liquefaction or substantial settlements. One of the improvement methods of loose liquefiable sands from decades ago is injection or mixture of Portland cement (Dupas and Pecker [1]). Extensive research is available in literature on cement-treated soils and their mechanical properties (Haeri et al. [2]; Hamidi and Haeri [3]) and also the effect of cement type (Haeri et al. [4]), but they are mostly limited to triaxial compression, one-way cyclic and Brazilian tensile tests on cemented soils (e.g. Schnaid et al. [5]; Consoli et al. [6]; Rotta et al. [7]; Hassanlourad et al. [8]).

Few studies have been reported, however, on the stress reversal or alteration of principal stress directions. Malandraki and Toll [9] used a mixture of sand and kaolin (13%) that was fired at $500\,^{\circ}C$ to form weakly bonded sand particles and concluded that: "The behavior of the bonded soil is strongly influenced by clockwise changes in the stress path direction during shearing. The available strength was reduced if the soil was subjected to a complex stress path. Therefore, for complex stress paths that may be present in the field, bonding degradation should be considered if the design is based on a limiting strength. In this case, data from conventional triaxial tests could significantly overestimate the available strength, leading to an unconservative design". This is while in many loading/unloading conditions, the major principal stress may change its direction with respect to vertical and the stress path on the soil element after from, for example, compression to extension, hereafter referred to as "Comp" and "Ext", respectively. Examples are elements of soil on slopes or adjacent to excavations. Considering the anisotropy of most sands, it is not well understood whether the cement addition increases the strength and stiffness in the vertical and horizontal as well as the intermediate directions or not.

No study has been reported in literature on utilization of other devices such as simple shear to study the behavior of cemented sands.

In this study, undrained triaxial tests with one-way monotonic Comp and Ext stress paths on Portland cemented

* *Corresponding author: kfakhari@aut.ac.ir*

1 Ph.D. Candidate, Department of Civil and Environmental Engineering, Amirkabir University of Technology, Tehran 15875, Iran

2 Assistant Professor, Department of Civil and Environmental Engineering, Amirkabir University of Technology, Tehran15875, Iran

sand are carried out to investigate the variations of α and $b = \frac{\sigma_2 - \sigma_3}{\sigma_1 - \sigma_3}$ parameters. In addition, undrained simple shear tests are performed to investigate the compatibility with triaxial tests and also effect of α values between 0 and 90°.

2. Materials

Firoozkuh sand, Type 161 has been used in this study. This artificial crushed Silica sand is well-known and documented in many research studies across the country (e.g. Askari et al. [10], Ghiasian et al. [11]). The physical properties of Firoozkuh sand are presented in Table 1 and the grain size distribution is shown in Fig. 1. Knowing G_s=2.65 from previous studies, e_{max} and e_{min} were determined according to ASTM D4253-4.

Type II Portland cement with different contents of 1.5, 3 and 5 % were added to the sand.

Table 1 Physical properties of Firoozkuh sand (F161)

G_s	e_{max}	e_{min}	$D_{50(mm)}$	C_u	C_c
2.65	0.86	0.58	0.26	1.9	0.88

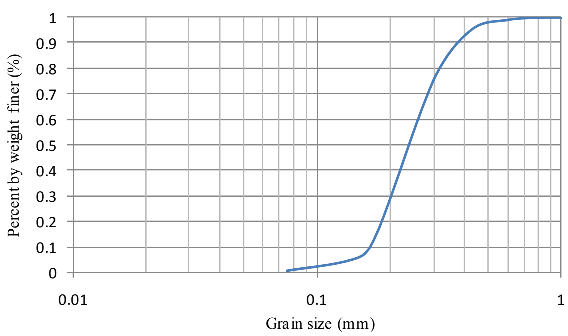

Fig. 1 Grain size distribution for Firoozkuh sand (F161)

3. Apparatus and Sample Preparations

Fully-automated triaxial and simple shear devices developed at the Department of Civil and Environmental Engineering, Amirkabir University of Technology have been utilized in this study. Both apparatuses are capable of stress-path and monotonic/cyclic load- and displacement-controlled testing.

Triaxial samples are 71 mm in diameter with the height to diameter ratio of 2.1 to 2.2. Each sample was compacted in 8*20-mm thick layers. The simple-shear samples are 71 mm in diameter and 20 mm height for the base sand (uncemented) and 26 mm height for the cemented samples. The triaxial tests have been carried out in consolidated undrained condition and saturated, while the simple shear tests were conducted on 5% water-content samples at constant volume, equivalent to undrained condition. The Ladd [12] wet under-compaction method with U_{ni} of 4 and 5, respectively, for uncemented and cemented samples was used in all specimens to achieve the expected density, uniformly distributed along the sample height. These U_{ni} parameters were adopted from some preliminary trial and error tests. Although repeatability of wet tamped specimens are generally less than pluviated ones, but by using the Ladd under-compaction method and following a systematic testing procedure, it is believed to have the best repeatable results. In addition, whenever needed, the specimen preparation and testing was repeated to ensure that at least fair results have been acquired.

In uncemented samples, a water content of 5% was used, while in cemented samples an extra water content equivalent to $\frac{W}{C} = 1$ was added to the previous value to guarantee full hydration of cement particles. In cemented samples, after adding the cement to the base soil, it was hand-mixed in dry condition, required water was added and tamping started.

The main objective of this study was effect of cement on behavior of loose sands, therefore the relative density (D_r) of the base soil was selected to be 10 to 15% on the basis of which the corresponding dry density was calculated. As the addition of cement fine grains contribute to increase the density, on the basis of Ismail et al. [13] and Consoli et al. [14], the cement weight percentage was calculated in such a way to achieve a reasonably constant final dry density for all cement percentages as well as the uncemented base soil. To compare the effect of cement content on loose sands with uncemented dense sands, several samples with 65 ~ 70% relative density were also prepared and tested. The details of sample preparation and testing program for both triaxial and simple shear tests are described in the following sub sections.

3.1. Triaxial tests

The clean uncemented sand samples were percolated with CO_2, de-aired water and finally subjected to 150 ~ 200 kPa back-pressure and saturation, achieving minimum B-values of 0.96. The cemented samples were casted in split U.P.V.C Teflon molds and then placed in water-proof bags and kept submerged in water for 28 days. On the testing date, similar procedure to the clean sand was followed to saturate the sample to reach a minimum B-value of 0.92 (by applying a 200~250 kPa back-pressure). Both clean and cemented sand samples were consolidated at confining pressures of 100 and 300 kPa and then subjected to undrained monotonic loading either in Comp or Ext stress paths, pertained to to to α=b=0 and α=90^{o}, b=1, respectively. Considering the nature of sand samples which were basically SP according to USCS with a D_{50} of 0.26, the membrane compliance effect was assumed to be negligible in all tests; however in cemented samples, wherever needed, a fresh lean Bentonite paste was applied to minimize the compliance effect. The loading was strain-controlled at 0.33% per minute in both Comp and Ext stress paths. A total of 20 experiments was planned, the details of which are presented in Table 2. To minimize the effect of end plate on top and bottom of specimens, a thin layer of liquid Teflon was sprayed on end platens before setting up the specimens. The other required corrections to triaxial test data such as area and membrane effects were applied according to Head [15].

Table 2 Details of monotonic triaxial tests

Comp (α=0 ; b=0)					Ext (α=90 ; b=1)				
Test I.D.	Dr %	γ_d	σ_3	CC %	Test I.D.	Dr %	γ_d	σ_3	CC %
01M15Ucem100c	10~15	0.455	100	0	11M15Ucem100e	10~15	0.455	100	0
02M15Ucem300c	10~15	0.455	300	0	12M15Ucem300e	10~15	0.455	300	0
03M70Ucem100c	65~70	0.591	100	0	13M70Ucem100e	65~70	0.591	100	0
04M70Ucem300c	65~70	0.591	300	0	14M70Ucem300e	65~70	0.591	300	0
05MCem1.5-100c	10~15	0.455	100	1.5	15MCem1.5-100e	10~15	0.455	100	1.5
06MCem1.5-300c	10~15	0.455	300	1.5	16MCem1.5-300e	10~15	0.455	300	1.5
07MCem3--100c	10~15	0.455	100	3	17MCem3--100e	10~15	0.455	100	3
08MCem3--300c	10~15	0.455	300	3	18MCem3--300e	10~15	0.455	300	3
09MCem5--100c	10~15	0.455	100	5	19MCem5--100e	10~15	0.455	100	5
10MCem5--300c	10~15	0.455	300	5	20MCem5--300e	10~15	0.455	300	5

3.2. Simple Shear tests

Constant volume tests were planned to resemble the undrained condition and to avoid the associated difficulties with excess pore water pressure measurements, as reported by Boulanger et al. [16]. The presented method by Vaid and Finn [17] was followed in the constant volume tests. In this method, the vertical strain is maintained constant during shearing (called constant height) and then the vertical stress is varied to compensate for the tendency to compression or dilation. Dyvic et al. [18] performed both undrained tests on saturated samples and constant volume tests on unsaturated samples and concluded that the differences are negligibly small.

In constant volume simple shear tests, the specimen is consolidated under K_0 condition by applying specified vertical stress from top platen and also surrounding horizontal stress from the mold rigid wall. Following that, horizontal shear lateral load is applied while the vertical displacement of loading ram is restricted. In this condition, if the specimen tends to dilate, the rigidity of surrounding rings together with the restriction of vertical upward displacement provides the constant volume condition. A vertical normal force is applied from the vertical actuator and so the vertical effective stress rises up which is equivalent to excess pore water pressure decrease, if the soil was saturated under undrained condition. On the other hand, if the specimen tends to contract, it shows tendency to a downward displacement which is equivalent to decrease in effective vertical stress applied from non-moving vertical load-cell to the specimen and vice versa. This latter case is corresponding to excess pore pressure build up and could even continue until a separation between top platen loading shaft and load-cell occur (equivalent to zero effective vertical stress) which is known as a situation like initial liquefaction (in static or dynamic loading) or r_u=1.

A stack of annular thin aluminum plates, 2 mm thick each, and outside and inside diameters of 92.5 and 72.5 mm, respectively, was used as sample container in simple shear tests. Similar to triaxial tests, wet tamping was used to place the clean sand samples, but all in one layer as the sample height is only 20 mm. The sample is then placed on the bottom pedestal located on the rolling table and then the top platen lowered to apply the vertical pressure and then shearing. Figure 2 shows the simple shear apparatus, mold, base and top platens, and sheared sample in stacks of rings.

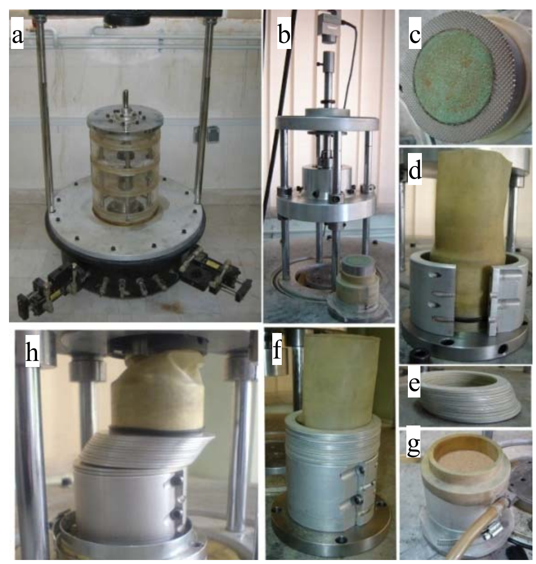

Fig. 2 Photos of several components of simple shear apparatus: (a) cell, main frame and loading ram (b) internal bracing and stiffener (c) serrated surfaces in contact with top and bottom of specimen (d) pedestal, membrane and its supporting wall (e) stacks of confining aluminum plates (f) assembled parts before remolding the sample (g) remolded sample after preparation (h) sample situation at the end of shearing

An ordinary latex membrane is placed inside the stacks of plates to prevent sand particles spreading between the plates, while the plates provide the plane strain condition for the specimen during simple shear deformation. Dry Teflon was spraying on the ring surfaces to minimize the sliding friction. The top and bottom plate surfaces were serrated to penetrate the sand under vertical pressure and assure a non-slip surface during shearing.

The above procedure used for clean sand sample preparation would not work properly for cemented sand, mostly because of two problems: (1) slip between the top and bottom platen surfaces and the soil, and (2) cemented sand fitting in the rings and membrane.

The first problem was attributed to the relatively hard surface of the cemented sand samples and therefore, lack of penetration of the serrated platens into soil. This problem would have caused noticeable slip between the contact surfaces. To resolve this problem, the total sample height was increased to 50 mm, such that 12 mm of the top and bottom of the sample penetrated into the rings connected to top cap and pedestal. Therefore, 26 mm of the sample remained as the effective height during shearing. Figure 3 presents the modified top cap and bottom pedestal photos. Slight increase in sample height (26 mm compared to 20 mm in clean sand samples) would have contributed to reduce the errors of fixed conditions at the top and bottom, while the sample aspect ratio of 0.4 still satisfies the ASTM-D6528 requirement.

Fig. 3 Modified pedestal and top cap for cemented samples

More trial and errors were experienced to resolve the second problem. As the cemented sand samples are precast, the diameter has to be precisely same as the inside diameter of the rings and membrane to assume no space is left between the inside wall and the sample and that the confined K_0 condition is satisfied. In the first attempt, the inside diameter of top and bottom rings were cut to the same size of the existing aluminum rings, as shown in Fig. 4a. The sample preparation molds were made of U.P.V.C to a height of 50 mm and inside diameter of rings minus double thickness of the membrane. But this method failed as the U.P.V.C molds were deformable during machining and hundredth of millimeter precision could not be achieved.

Aluminum molds were tried in the second stage. After 28 days of sample curing, chemical reaction had occurred between the aluminum surface and cement and other minerals in the soil. Strong bonding would interfere with intact removal of the cured sample from the split molds, as shown in Fig. 4b.

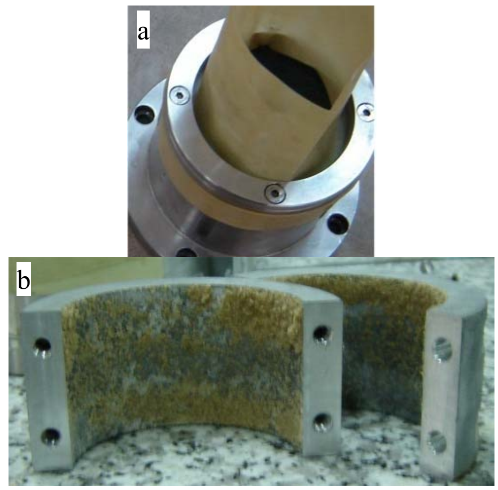

Fig. 4 (a) Modified pedestal with installed membrane in between (b) chemical reaction between sample and aluminum mold prevented intact sample removal.

Two solutions were thought to resolve the problem. The first solution was a thin (40 μm) epoxy paint and the anodization of the inside mold surface. This did not work as, surprisingly, a strong chemical reaction penetrated the paint and bonding was formed between cured sample and mold. Alternatively, a chemically resistant thin plastic layer (10^{th} of mm) was successfully placed inside the molds. Figure 5 shows different steps of precast cemented sand sample preparation.

Total of 14 simple shear tests were carried out and on 8 uncemented clean sand and 6 cemented sand samples, details of which are presented in Table 3. Samples with 10~15, 30, 50 and 70 % relative density were prepared for clean sand samples. Initial vertical pressure of 100 and 200 kPa were used for consolidating the samples and then monotonically sheared under constant volume condition at a rate of 0.5 mm/min.

Fig. 5 Preparation steps after precast for simple shear cemented samples (a) curing in waterproof bags (b) removal from aluminum mold (c) removed sample with thin plastic layer around (d) pedestal with stretched membrane, ready for inserting the sample (e) sample inserted in steel fixity rings (f) sample confined in the membrane (g) aluminum rings and membrane around sample (h) sample between pedestal and top cap under shearing.

Table 3 Details of monotonic simple shear (constant volume) tests

Test I.D.	Dr (%)	γ_d	σ_{V_0}	CC (%)	Test I.D.	Dr (%)	γ_d	σ_{V_0}	CC (%)
21M15Ucem100	10~15	1.455	100	0	28M70Ucem200	70	1.591	200	0
22M30Ucem100	30	1.490	100	0	29MCem1.5-100	10~15	1.455	100	1.5
23M50Ucem100	50	1.539	100	0	30MCem3--100	10~15	1.455	100	3
24M70Ucem100	70	1.591	100	0	31MCem5--100	10~15	1.455	100	5
25M15Ucem200	10~15	1.455	200	0	32MCem1.5-200	10~15	1.455	200	1.5
26M30Ucem200	30	1.490	200	0	33MCem3--200	10~15	1.455	200	3
27M50Ucem200	50	1.539	200	0	34MCem5--200	10~15	1.455	200	5

4. Test Results

All the triaxial and simple shear tests are consolidated / undrained. The stress paths in triaxial tests include axial compression (α=0, b=0), so called Comp, and axial extension (α=90°, b=1), so called Ext.

4.1. Triaxial tests

For all clean and cemented sand samples the tests were carried out monotonically increasing the strain. Achieving 20% of axial strain in Comp tests, 10% axial strain in Ext tests, or sample rupture during the test, whichever occurred first was considered as failure. The failure criteria were specified on the basis of the initial test trends with attention on complete behavior before/after peak strength (if any) to reach steady state condition.

Figures 6 and 7, respectively, show the Comp and Ext test results, on 10 samples each. Each figure consists of q-ε and pp-ε, i.e. deviator stress and pore-water pressure variations with axial strain. Here q has been defined as (σ_1-σ_3). The 10 tests for each Comp and Ext stress paths are comprised of loose and dense clean sand with relative densities of 15 and 70%, loose cemented sand with cement content of 1.5, 3 and 5% and consolidation pressures of 100 and 300 kPa for each set.

Both in Comp and Ext stress paths, the results of Figs. 6 and 7, respectively, show that the loose clean sand samples have exhibited compressive response (+vepwp), but the dense clean sand and all loose cemented samples have shown dilative response (-vepwp). This observation complies with previous studies available in literature (e.g. Ismail [19]) for the Portland cement added to sand. However the aforementioned studies were limited to Comp stress paths only, where as in this study it is observed that the same trend also applies to Ext stress paths.

It is also noticed that the stress-strain response has become more brittle with increase in cement content. The mode of failure in uncemented and 1.5% cement content samples has been in bulge shape, whereas higher cement content samples have clearly exhibited failure planes. Sample failure modes for 1.5, 3 and 5% cement content are shown in Fig.8. The brittleness of the samples with increase in cement content is very obvious in Figs. 8b and 8c, corresponding to 3 and 5% CC, respectively.

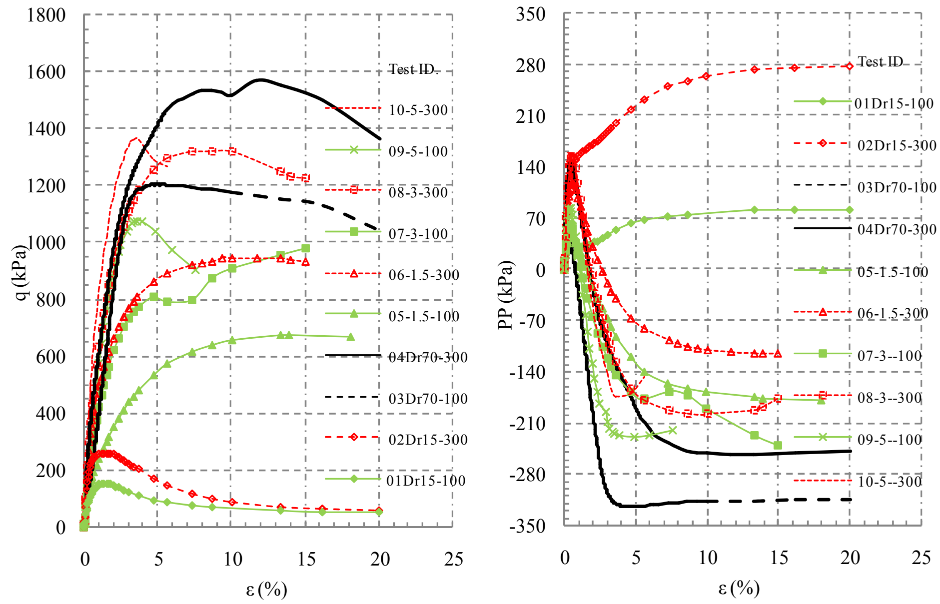

Fig. 6 Undrained triaxial compression (Comp) tests results

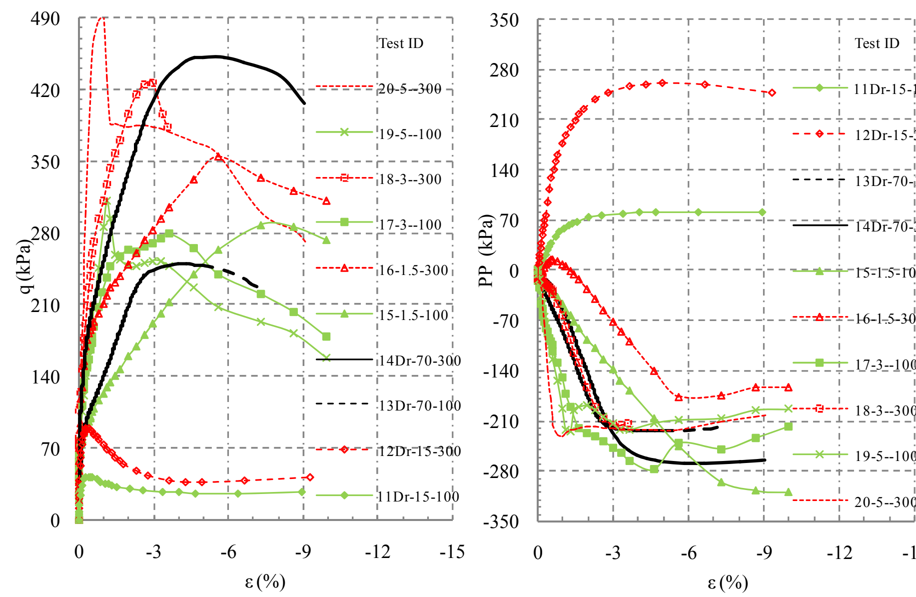

Fig. 7 Undrained triaxial extension (Ext) tests results

Fig. 8 Demonstration of cemented samples condition at failure in Comp tests (a) 1.5% cemented (b) 3% cemented (c) 5% cemented.

Comparing Comp and Ext results, the 70% uncemented samples for each of the confining pressures in Comp tests exhibit higher strength and lower pore-water pressure than the cemented sands, even the 5% cement content. However, Ext tests show strength and negative pore-water pressure for cemented loose sands within the limits of the dense uncemented sand. The practical implication of this observation is that in Ext stress paths, the cement mixture has been more effectively contributing to increase in strength and probably reducing the liquefaction potential.

The graphs also represent that the two different loading conditions have resulted in different strength in the base soil, in such a way that Comp tests show higher peak stresses than the Ext tests. This is attributed to the anisotropy of soil. Despite the anisotropic response, it should be noticed that the rate of increase in strength and initial stiffness (tangent modulus) resulted from cement mixtures in Ext tests is greater than Comp tests. In other words, increase in cement content tends to reduce the anisotropy. As an accepted experiment, in lightly to medium cemented soils a significant percentage of increased modulus, degrade at low shear strains; hence it was decided in this research to use tangent modulus (like as Dupas and Pecker [1], Malandraki and Toll [9]) as the comparison parameter to capture full capacity and improvement of cemented soils at service load condition in which corresponding strains are far below the threshold of bonds degradation.

To further clarify this point, the normalized peak strength and stiffness ratios with respect to the base soil are presented in Table 4. The I_{SNB} and I_{ENB} ratios are, respectively, the percentage of increase in "peak strength" and "tangent modulus" normalized with base soil as shown in Eqs. 1 and 2.

$$I_{SNB} = \frac{q_{\max(sample)} - q_{\max(base\,soil)}}{q_{\max(base\,soil)}} \times 100\,; \qquad (1)$$

$$I_{ENB} = \frac{E_{(sample)} - E_{(base\,soil)}}{E_{(base\,soil)}} \times 100 \qquad (2)$$

Table 4 Comparison of I_{SNB} and I_{ENB} for different samples in triaxial tests.

Ext (α=90; b=1)						Comp (α=0; b=0)					
5	3	1.5	0	CC %		5	3	1.5	0	CC %	
1.455	1.455	1.455	1.591	γ gr/cm³		1.455	1.455	1.455	1.591	γ gr/cm³	
10~15	10~15	10~15	65~70	% Dr		10~15	10~15	10~15	65~70	% Dr	
650	570	590	510	σ_c =100 kPa	I_{SNB}	610	550	340	700	σ_c =100 kPa	I_{SNB}
450	375	300	400	σ_c =300 kPa		425	400	255	500	σ_c =300 kPa	
315	305	135	160	σ_c =100 kPa	I_{ENB}	155	130	40	60	σ_c =100 kPa	I_{ENB}
375	180	110	130	σ_c =300 kPa		140	70	50	70	σ_c =300 kPa	

The results in the table indicate that in compacted uncemented sand, the change in stress path has resulted in lower strength increase ratio (I_{SNB}), as opposed to loose cemented sands. Also the stiffness increase ratio (I_{ENB}), comparing to cemented loose samples (excepting low-cement ones) has resulted in lower values for compacted sand.

In real-life load conditions, there exist many situations during which the direction of major principal stress varies continuously throughout loading. A well-known case is upward propagation of shear wave (S-wave) resulted from earthquake inducing shear stress in the horizontal layers. The application of shear stress to an at-rest element in equilibrium results in gradual rotation of the major principal stress. Simple-shear test is capable of simulating this kind of loading path. To further investigate the effect of α variation on anisotropy of cement-treated sands, it was decided to perform simple-shear tests, as explained in the next sub-section.

4.2. Simple Shear Tests

4.2.1. Parameters and relations

The α value is zero before shear stress application to the specimen, referred to as initial K_0-condition. With increase in shear stress, the α eventually increases. Figure 9 shows the Mohr circles at K_0 and shear application in constant volume condition.

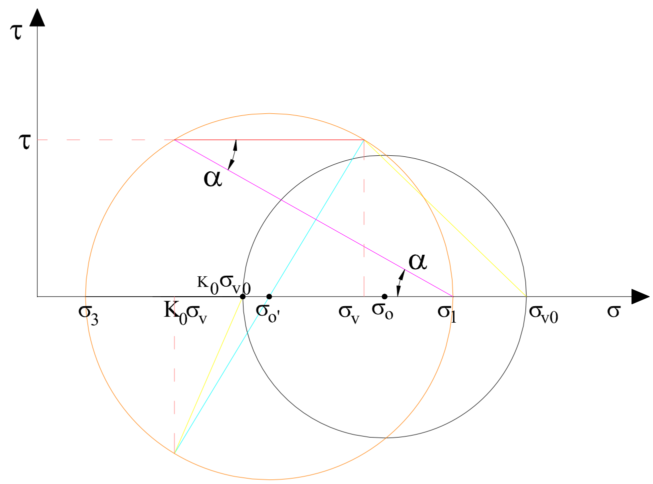

Fig. 9 Mohr circle diagrams at initial-state and shearing states of simple shear tests in constant volume condition.

The circle on right represents the soil element at initial state subjected to principal stresses $\sigma_1=\sigma_{V0}$ and $\sigma_3=\sigma_{H0}=K_0\sigma_{V0}$, in which K_0, σ_{H0} and σ_{V0} are respectively, at-rest lateral pressure coefficient, normal stress in horizontal and normal stress in vertical directions. When shear stress, τ, is acting on the elements, the normal stress acting on the horizontal plane decreases or increases, depending on the compressive or dilative response of the specimen to shear loading. The vertical normal stress is varied during shearing to maintain the specimen height (volume) constant. The Mohr circle during constant volume shearing is depicted on left in Fig. 9. The σ_v and σ_h acting on vertical and horizontal planes, respectively, are no longer the principal stresses due to shear stress component on those planes. Therefore, some derivations are required to calculate the principal stress components (σ_1 and σ_3) as well as the rotation angle, α. The following equations can be written from the figure:

$$\tan\alpha_{CV} = \frac{\tau}{\sigma_1 - K_0\sigma_V} \tag{3}$$

in which α_{CV} is the rotation of principal stresses with respect to vertical in constant volume (C.V.) state. The center, σ_0, and radius, R', of the circle can be obtained as Eqs. 4 and 5:

$$\sigma_{O'} = \frac{K_0+1}{2}\sigma_V \ ; \ R' = \sqrt{\tau^2 + (\sigma_V - \sigma_{O'})^2} \qquad (4), (5)$$

The magnitudes of principal stresses would become (implementing σ_0 and R' of Eqs. 4 and 5):

$$\sigma_3 = \sigma_{O'} - R' \ ; \ \sigma_1 = \sigma_{O'} + R' = \left(\frac{K_0+1}{2}\right)\sigma_V + \sqrt{\tau^2 + \left(\frac{1-K_0}{2}\right)^2 \sigma_V^2} \tag{6}$$

and hence:

$$\tan\alpha_{CV} = \frac{\tau}{\left(\frac{1-K_0}{2}\right)\sigma_V + \sqrt{\tau^2 + \left(\frac{1-K_0}{2}\right)^2 \sigma_V^2}} \tag{7}$$

It is evident from the above equations that the α parameter in C.V. simple shear test cannot be maintained constant during the test, but it is still a meaningful value in soil behavior interpretation. For example, increase in α parameter results in decrease of the vertical effective stress and therefore, increasing the liquefaction potential. This has been evaluated and discussed in the next subsection.

As reduction in vertical stress is important in test result interpretations, another parameter is defined as the vertical stress reduction ratio, r_r:

$$r_r = \frac{\sigma_{V_0} - \sigma_V}{\sigma_{V_0}} \tag{8}$$

In all equations defined, K_0 has been considered to be equivalent to Jaky [20] as $K_0 = 1 - \sin\phi'$ in which ϕ' is the effective angle of internal friction. On the basis of Atkinson's et al. [21] suggestion, some previous triaxial test results have been used to calculate ϕ' for the base clean sand as summarized in Table 5.

Table 5 ϕ' (drained) and K_0 values for Firoozkuh sand (F161)

DR %	15	30	50	70
ϕ^{o}	27	29	33	36
K_0	0.546	0.515	0.455	0.412

For cemented sands, however no specific correlation has been proposed for variation of ϕ' with cement content and contradictive results have been reported. Jaky [20] equation is also not applicable to cemented sands. Therefore, direct measurement of K_0 in cemented sand could be more helpful. In order to obtain these measurements, two different methods were taken into account. As the first method, a series of triaxial tests were conducted on different cemented specimens throughout which the Poisson's ratio was directly measured and implemented in $K_0 = \frac{1-\upsilon}{\upsilon}$. Some approximation is involved due to using it for not fully-isotropic elastic cemented specimens. K_0 value was in this way determined for different cemented specimens, results of which are presented in table 7. It should be noted that using the equation $K_0 = \frac{1-\upsilon}{\upsilon}$ is true, if and only if, the material could be assumed an elastic isotropic homogenous material in which case E_x=E_y=E_z and ν_x=ν_y=ν_z in the stress-strain relationship matrices of continuum mechanics. But according to the former triaxial test results having shown that adding Portland cement decreases anisotropy of the sand and increases the stiffness (E modulus) and hence the elasticity, the introduced method could be applied in K_0 measurements with tolerable approximations.

As the second method, K_0 direct measurement from previous researchers was searched in the literature. The results reported by Zhu et al. [22] are compatible with the sand physical properties, sample preparation method and cement type used in this study. They used a modified Oedometer capable of measuring lateral and vertical pressures at rest for Ottawa sand (round) and Marine sand (angular), with variable cement contents between zero to 8 weight percent of Portland cement. The Marine sand properties are presented in Table 6, showing similar parameters with F161 sand.

Table 6 Physical properties of Marine sand used by Zhu et al. [20]

e_{max}	e_{min}	D_{10} (mm)	D_{50} (mm)	C_u
0.88	0.55	0.20	0.33	1.7

Table 7 shows summarized magnitudes of the related parameters from Zhu et al. [22] measurements, in which a K_0 reduction factor, F_r, is shown for cement contents of 1.5, 3 and 5 % and initial vertical stresses of 100 and 200 kPa. The reduction factor, F_r, states the reduction of K_0 with respect to the clean base soil in similar condition. But the specimens in their study have had a relative density of 50 % and in another test series of that study, 25 % increase in K_0 has been reported for every 20 % reduction in relative density for 2 % cement content (CC) samples. Also, Consoli et al. [6] have shown that linear relationship exists between strength parameters and CC at different relative densities. Therefore, it can be concluded that if the reduction factor, F_r, is increased 50 %, the output shall be equivalent to 10 ~ 15 % relative density, as shown in Table 7. Also this table represents the back-calculated K_0 values for cemented specimens with ϕ of 27° and 15 % relative density, on the basis of modified F_r.

Authors believe that using either of the back-calculated or measured K_0 values in this study does not significantly affect the results. Also, because in simple shear tests the condition is more similar to the Oedometer, hence an average value between the back-calculated and the measured K_0 were decided to apply in α calculations as presented in table 7.

Table 7 Reduction factor for K_0 adopted from Zhu et al. [20], back-computed, experimentally measured and averaged K_0 values for different cemented samples.

Cement Content %	σ_{V_0} (kPa)	F_R	Modified F_R (50 % increased)	Back-calculated K_0	K_0 values from experimental measurements	Averaged K_0 to apply in α calculations
1.5	100	0.370	0.555	0.303	0.318	0.311
	200	0.400	0.600	0.328	0.322	0.325
3	100	0.190	0.285	0.156	0.189	0.173
	200	0.230	0.345	0.188	0.285	0.237
5	100	0.130	0.195	0.106	0.138	0.122
	200	0.140	0.210	0.115	0.156	0.136

4.2.2. Test results

Having the Triaxial test results, it was planned to shear the specimens in simple shear tests up to 25 % of shear strain or rupture shear failure, whichever occurred first. The shear stress-strain graphs for different densities on clean sand (no cement) are compared to different cement contents (1.5, 3 and 5%) at low density, to evaluate the improvement effects of cementation compared to increase in compaction energy. Then the variations of α and r_r, defined in previous section, are evaluated in different conditions and discussed.

Figure 10 presents the τ-γ variations of all cemented and non-cemented samples of different densities (15 to 70 % for clean sand in Fig. 10a), different cement contents (1.5, 3 and 5 % in Fig. 10b), and initial normal stresses of 100 and 200 kPa.

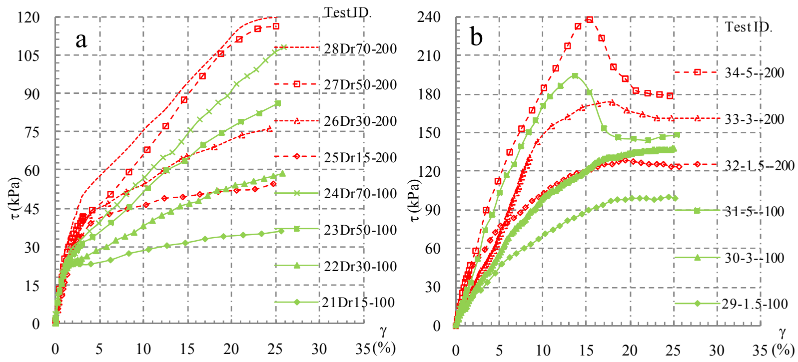

Fig. 10 Comparison of τ–γ for all simple shear samples (a) uncemented (b) cemented

Similar to triaxial test results, the results of Fig. 10 for simple shear tests indicate that adding even 1.5 % of Portland cement to the loosest sand (about 15 % D_r) has resulted in substantial increase in strength. The results also show that the strain at peak strength has decreased with increase in CC which is very well pronounced in 5 % CC, after which a substantial strain softening is observed. A more practical expression for this observation is that the material response has become more brittle with increase in CC. The results comply quite well with those presented in triaxial tests.

To further clarify the brittleness of the cemented samples at higher CC, a photograph of a 5 % CC sample at shear failure is depicted in Fig. 11.

The shear failure line having an approximate angle of 40° with horizon within the fixed top and bottom boundaries represents the brittleness of the cemented sand. The cement content beyond 3 % has resulted in higher strength, but also more brittleness which may not be a suitable material when subjected to repeatedly loading such as those in seismic excitations.

Fig. 11 Failure line and slip plane demonstration for 5 % cemented sample.

Figure 12 represents the mobilized shear stress at 5, 10 and 25 % shear strains and at peak for both uncemented and cemented samples. It is noticed (as expected) that the mobilized shear stress has increased with relative density for clean sand and with CC for cemented sands. The role of increase is substantially higher, however, for loose (D_r of about 15 %) cemented sands. This is again clearly showing the effect of cementation on loose saturated sand improvement as compared to increasing the compaction efforts.

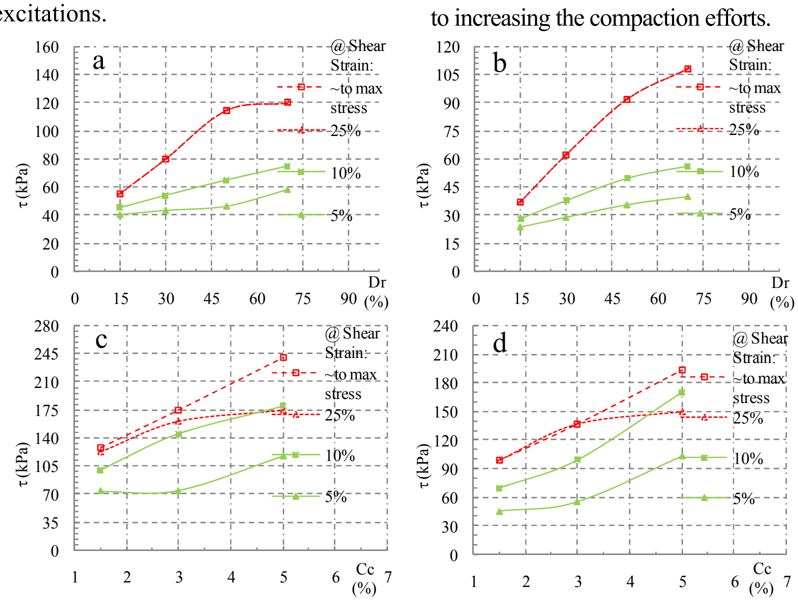

Fig. 12 Simple shear strength comparison at various strain levels (a) uncemented, $\sigma_{V_0} = 200kPa$ (b) uncemented, $\sigma_{V_0} = 100kPa$ (c) cemented, $\sigma_{V_0} = 200kPa$ (d) cemented, $\sigma_{V_0} = 100kPa$

To more clearly indicate the effect of principal stress rotation during shearing, variations of α with respect to γ and also τ and r_r with respect to α are plotted in Fig. 13 for an uncemented sample with D_r of 50 % (Fig .13a) and a cemented sample with CC of 5 % (Fig .13b). It is noticed that α increases rapidly with increase in γ (and τ) up to the phase transformation point of r_r, shown hereafter as α_{phase}. Thereafter, α experiences a peak and then approaches a residual value, denoted by α_{max} and α_{res}, respectively.

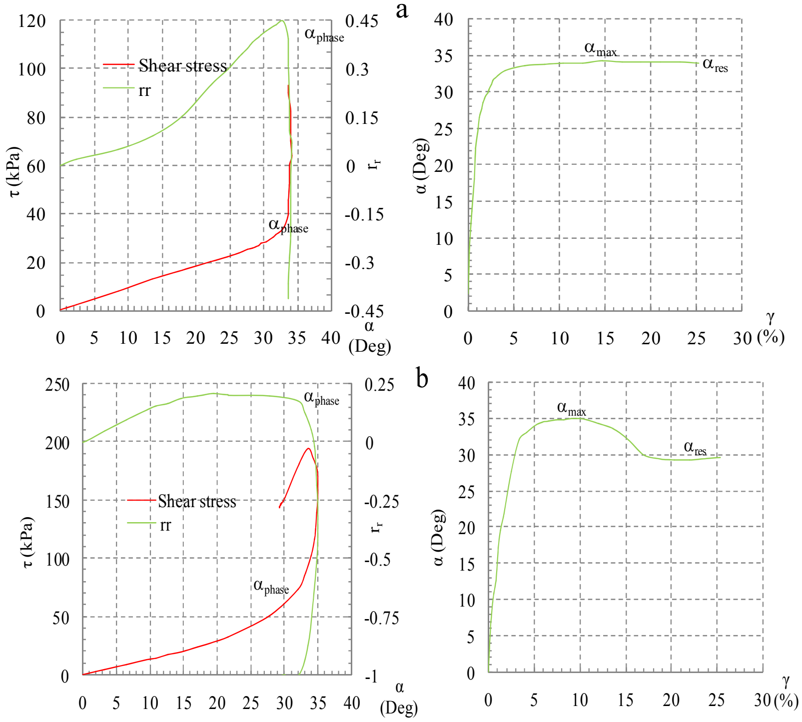

Fig. 13 Typical graphs for α, r_r with respect to τ and α-γ (a) sample with I.D. No. 23 (b) sample with I.D. No. 31

The shear stress, τ, has exhibited a relatively linear relationship with α up to α_{phase}, after which α has shown little variations with further increase in the shear stress. To explain the relatively little variations of α after the phase transformation of r_r, or in fact when the dilation of the sample starts, Eq. 10 is rewritten in the format of Eq. 9, indicating that α can remain constant only when $\frac{\tau}{\sigma_V}$ is constant.

$$\tan\alpha_{CV} = \frac{\frac{\tau}{\sigma_V}}{\left(\frac{1-K_0}{2}\right)+\sqrt{\left(\frac{\tau}{\sigma_V}\right)^2+\left(\frac{1-K_0}{2}\right)^2}} \tag{9}$$

This implies that τ and σ_V need to vary proportionally till α remains constant. Proportional increase in σ_V with τ after the phase transformation point, α_{phase}, is indication of dilative response of the sample. This is well-understood and expected for dense dilative responses in uncemented samples such as Fig. 13a. Loose cemented samples (such as Fig. 13b) have also shown similar trend. This dilative response of loose cemented samples is in accordance with observations of other studies (e.g. Ismail et al. [19]).

In cemented samples, α reduces after peak stress and approaches a residual value, α_{res}. This is corresponding to the residual strength of the cemented sample at which the $\frac{\tau}{\sigma_V}$ ratio is constant (Eq. 9) because no more dilation/compression occurs and shearing continues with no further variation in normal stress (or in fact pwp, or effective normal stress). The α reduction between peak strength to α_{res} in cemented sands is attributed to cement bond failure during post peak softening. Lambe [23] and Clough et al. [24] have shown that some bonding remains even when the residual strength has reached.

Figures 14 and 15 present the variations of α and r_r for all test specimens at phase transformation, α_{phase}, at peak, α_{max} and at residual strength,α_{res}. It is noticed in Figs. 14a and b that all α values have decreased with increase in density. In cemented sands shown in Figs. 14c and d, the α_{phase} and α_{max} increase with CC while α_{res} generally has decreased slightly. It should be noted, however, that α values are several degrees lower than the uncemented sand specimens. In cemented samples, α_{res} is considerably lower than the max and phase values for higher CC. This is attributed to more pronounced post-peak softening in higher CC samples as a result of debonding between sand particles.

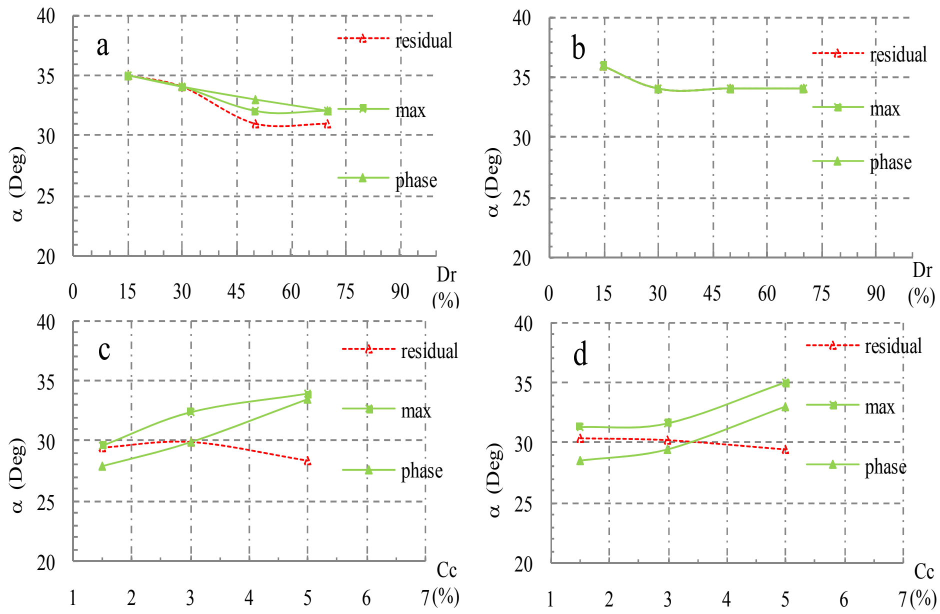

Fig.14 Comparison of α for all samples (a) uncemented, $\sigma_{V_0} = 200\,kPa$ (b) uncemented, $\sigma_{V_0} = 100\,kPa$ (c) cemented, $\sigma_{V_0} = 200\,kPa$ (d) cemented, $\sigma_{V_0} = 100\,kPa$

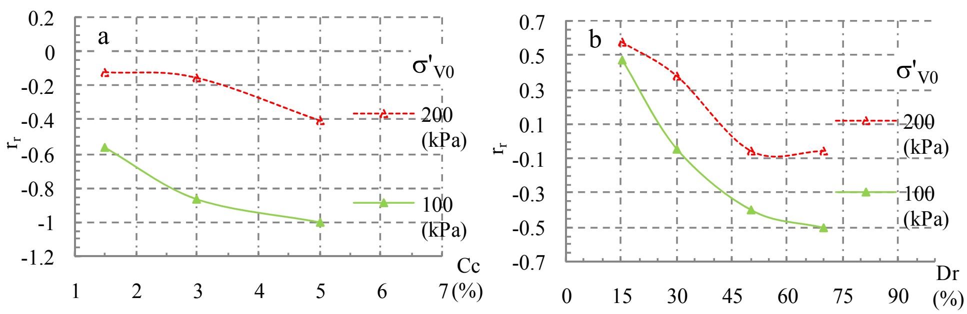

Fig. 15 Comparison of r_r at peak shear strength for all samples (a) cemented (b) uncemented

Comparing the r_r variations at max shear stress of Fig. 15 for uncemented and cemented samples indicate that r_r decreases both with D_r and CC or in fact samples undergo more dilative response. While the range of r_r variations is greater for uncemented sands with D_r, compared to variations with CC for cemented sands, but cemented samples have exhibited more dilative behavior.

Similar to triaxial test results, the peak strength increase ratios with respect to the base soil, I_{SNB}, for simple shear test results are presented in Table 8. As the α values for simple shear tests at peak strength, α_{max}, are between 30 to 35^{o}, the stress paths may be categorized a condition between Comp and Ext of triaxial tests. The quantitative comparison between results of Tables 4 and 8 may not be easily possible as the simple shear samples are subjected to initial anisotropy as opposed to isotropically consolidated triaxial samples. Moreover, the α values are not constant throughout each test in simple shear. Still I_{SNB} values in Table 8 show similar trend to triaxial results, in that the CC has had considerably higher impact on strength increase compared to the compaction efforts.

Table 8 Comparison of increased peak strength normalized to base soil value for different samples.

σ_c =200 kPa					σ_c =100 kPa				
5	3	1.5	0	CC %	5	3	1.5	0	CC %
1.455	1.455	1.455	1.591	γ gr/cm^3	1.455	1.455	1.455	1.591	γ gr/cm^3
10~15	10~15	10~15	65~70	% Dr	10~15	10~15	10~15	65~70	% Dr
340	220	140	120	I_{SNB}	440	275	180	200	I_{SNB}

5. Discussion

All the triaxial and simple shear test results have indicated that the shear strength increases in loose cemented sands compared to dense uncemented clean sands. The rate of increase is considerably higher when the direction of major principal stress deviates from the direction of sand specimen deposition. In other words, in Ext compared to Comp triaxial tests, and in simple shear tests with continuously increasing α, the strength increase ratio, I_{SNB}, is significantly higher for cemented sands.

Simple shear test results show that within a certain strength level, for example in 70 % compacted clean samples and 1.5 % cemented loose samples, lower α has been measured in cemented samples. As in simple shear tests, α variation is shown to be compression/dilation dependent (defined by r_r), the sample resistance against the rotation of principal stresses is an indication of higher resistance to preserve the sand grain structure which is in fact interpreted as higher shear strength in soil mechanics sense.

As the cemented sands strength increase rate, even in loose state, under stress paths with $\alpha \neq 0$ is considerably higher than the $\alpha = 0$, it can be stated that adding Portland cement contributes to reduction in strength anisotropy of sand.

For more clarifications, comparison between Comp, Ext and simple shear stress paths for different samples at 100 kPa consolidation pressure is illustrated in Fig. 16. Because q was defined as (σ_1-σ_3) in triaxial tests, so q=2×τ was adopted from simple shear tests as the comparing parameter.

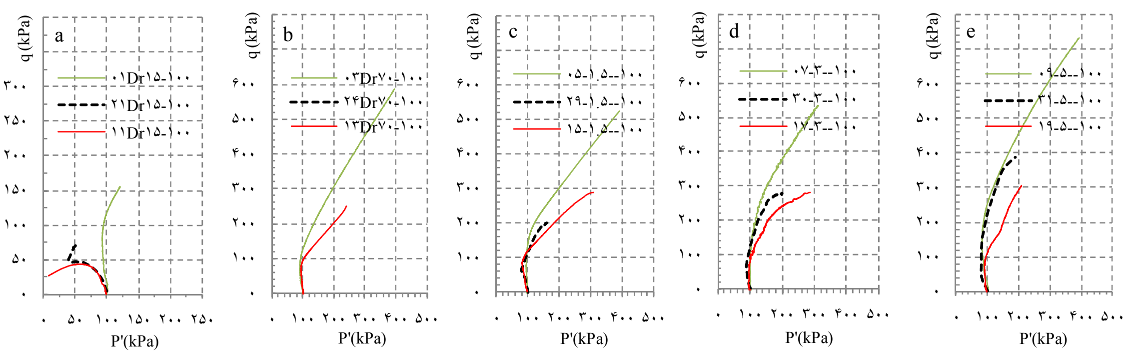

Fig. 16 Comparison between Comp, Ext and simple shear stress paths for different samples (a) uncemented, Dr=15 (b) uncemented, Dr=70 (c) 1.5 % cemented (d) 3 % cemented (e) 5 % cemented.

Although consolidation condition in simple shear and triaxial tests are different (K_0 versus isotropic), it is obvious from this figure that adding Portland cement and increasing cement content have pushed the stress path from Ext towards Comp which is also well in conformity with lower residual α values.

6. Conclusions

Monotonic triaxial and simple shear tests under different stress paths on clean and cemented Firuzkooh silica sand have been carried out to investigate the effect of variation of principal stress direction on the strength and anisotropy effects of cementation. Modifications were made in the simple shear apparatus specimen preparation and placement of cemented samples. The most important findings and contributions of the study are summarized below:

1. Portland cement contributes to reduction of strength anisotropy of silica sand.
2. Increase in strength and stiffness resulted from cement content is higher in Ext than Comp stress paths.
3. Because of anisotropic nature of sands, effect of compaction and cement addition improvement methods is different under various stress paths that has to be noticed in soil improvement practical applications.
4. Cement addition increases the brittle state of the samples.
5. The inter-relations between α and variations of normal stress (expressed by parameter α_{CV}) in constant volume testing is helpful in interpretation of simple shear tests and compressive/dilative response of cemented/uncemented sands.
6. The α_{phase} is a useful parameter that reduces with increase in sample strength and can be considered as a resistance parameter against further loading.

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