THE MECHANICAL BEHAVIOR OF A GRAVELY SAND CEMENTED WITH GYPSUM

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ABSTRACT
The alluvium in north Tehran can be considered as a cemented coarse-grained gravely sand. Several triaxial compression tests were carried out on this soil to investigate its mechanical behavior. The tests are done on samples with 1.5, 3, 4.5 and 6 percent gypsum content. Drained and undrained triaxial tests were carried out on the cemented soil. The tests are carried out with different confining pressures from 25 kPa to 500 kPa. According to the tests results, the cemented soil showed a brittle behavior. The strength increased to a peak value and after that reduced to the residual strength at large strains. The test results show that the peak shear stress increases with confining pressure and cement content. At failure stage all the samples show dilative behavior with a marked shear zone. The failure envelopes are different for drained and undrained conditions.

RÉSUMÉ
Les alluvions de la partie nord de Téhéran sont constituées des graviers sableux plus ou moins cimentés. Au cours de cette recherche, les essais de compression triaxiale, drainé et non drainé, ont été effectués sur les échantillons de ce sol dont la cimentation a été reconstituée par l'introduction de 1.5, 3, 4.5 et 6 pourcent de gypse. Les essais ont été déroulés sous la pressions latérales allant de 25 à 500 kPa. Les résultats de ces essais démontrent que ces graves cimentées ont un comportement relativement fragile. Les éprouvettes ont une résistance de pic et une résistance résiduelle, la premiére augmente avec l'augmentation du taux de gypse. A la rupture, toutes les éprouvettes ont dilateés ayant une zone de cisaillement bien démarqueé. Les courbes intrinséques sont différentes pour les essais drainés et non drainés.

1. INTRODUCTION

The study of the behavior of cemented soils has received considerable attention in recent years. Many researchers have worked on this category of material which can be considered as an intermediate state between soil and rock. Saxena and Lastrico (1978), Clough et al. (1981), Leroeil and Vaughan (1990), Gens and Nova (1993), Coop and Atkinson (1993), Cuccovillo and Coop (1997), Malandraki and Toll (2001) and Rotta et al. (2003) extended the basic concepts of the behavior of cemented sands.

All of the mentioned studies have focused on the mechanical behavior of the fine sandy soils. In recent years the mechanical behavior of the cemented gravely soils has been introduced as a new branch of the behavior of this materials. Haeri et al. (2002, 2003, 2004), Asghari (2003) and Hamidi et al. (2004) have worked on the mechanical behavior of cemented gravely sands. Different cementing agents like hydrated lime, Portland cement and gypsum was used in these studies.

Haeri et al. (2002) introduced a representative gradation for the North Tehran alluvium which can be named as a gravely sand in the Unified system of soil classification. Based on this definition Haeri et al. (2002) and Yasrebi and Asghari (2002) conducted some large scale direct shear tests on the artificially cemented soil using hydrated lime as the cementing agent. Haeri et al. (2003) and Asghari (2003) conducted some triaxial tests on this soil and showed that the failure envelope is curved for this material. Haeri et al. (2004) continued this research using Portland cement. Finally Hamidi et al. (2004) investigated the mechanical behavior of the cemented gravely soil using gypsum as the cementing agent. Using the results of the latter study, the objective of the present paper is to illustrate some new features of the behavior of the cemented gravely soil with gypsum as the cementing agent.

2. MATERIAL TESTED

The gradation curve of the tested material is shown in Figure 1. According to this figure the soil consists of 49% sand, 45% gravel and about 6% fine content. The largest grain size is limited to 12.5 mm. Also Table 1 shows the mechanical parameters of tested soil. According to this table the soil can be named as SW-SM in the Unified system of soil classification.

3. SAMPLE PREPARATION

The samples are prepared in a mould with a diameter of 100 mm and a height of 200 mm. In this manner the ratio of the sample diameter to the largest grain size is about 8. Each sample is prepared in eight layers. The samples are prepared with 1.5%, 3.0%, 4.5% and 6.0% gypsum plaster. Appropriate amount of soil is weighted for each layer and is mixed with the desired amount of gypsum and about 8.5% of distilled water. After that each layer is placed in the mould and is compacted to reach to the desired height. The specific volume of the soil is set to about 18 KN/m^3. This value corresponds to a relative density of 65%. The three part mould is opened after replacement of all layers and the sample is placed in oven to be cured at a constant temperature of 50°c. Each sample is cured to get to a constant weight.

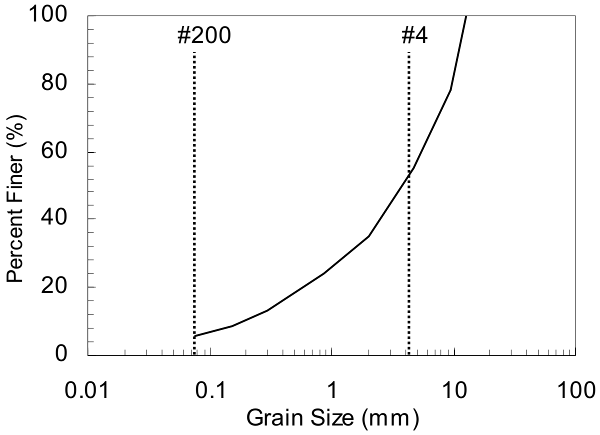

Figure 1. Gradation curve of tested soil

Table 1. Physical properties of tested Soil

Soil property	Value
Soil name	SW-SM
Specific gravity	2.58
Average particle size	4 mm
Effective diameter	0.2 mm
Fine content	6%
Sand content	49%
Gravel content	45%
Minimum unit weight	16 kN/m^3
Maximum unit weight	18.74 kN/m^3

4. TESTING PROGRAM

Triaxial compression tests were performed on the prepared samples. The samples were tested in drained and undrained conditions. The samples with different cement contents were tested under different confining pressures such as 25, 100, 300 and 500 kPa. The outer side of the sample was covered with a thin film of clay-fine sand mixture to minimize membrane penetration effects. Light silicon oil with the similar viscosity as water was used to saturate the samples in order to prevent reduction of gypsum bonds stiffness and strength if water is introduced as the pore fluid. CO_2 gas was flushed first through the samples under a very low pressure of 10 kPa to replace air bubbles between the grains. Then the oil was flushed very slowly from bottom of the sample to minimize any distortion. Finally cell and back pressures were ramped simultaneously to ensure sample saturation and to reach to a Skempton pore pressure coefficient, B value of more than 0.95.

The sample, then was consolidated under the desired confining pressure. The volume change in consolidation stage was recorded. After that, the axial loading was started with a strain rate of 0.2 percent per minute. The cell and back pressure, volume change, pore pressure, axial displacement and axial load were recorded using an electronic data logger system which was calibrated before the tests.

5. TEST RESULTS

5.1. Stress-strain relation

Figure 2 shows the results of the tests in drained and undrained condition. In these figures drained and undrained tests are designated by D and U, respectively, and are followed by a number that shows the amount of confining stress in kPa. In this figure the stress-strain curves are represented for the cemented soils.

According to this figure the peak shear stress increases with confining pressure. The strain associated with the peak shear stress increases for drained tests with increase in the confining pressure. However, that does not follow a clear trend for undrained tests. A comparison between the results of drained and undrained tests shows that the strains associated with the peak shear strengths are more in undrained tests compared to the drained tests. This shows a more brittle behavior in drained condition compared to undrained one. Malandraki and Toll (2001) showed that volume changes occur in drained condition helps to the earlier break down of the cemented bonds between soil grains. But this phenomenon occurs gradually and slow in undrained condition. This can be the reason for the more brittle behavior in drained condition.

Consoli et al. (1998) defined the brittleness index I_B to define the tendency of brittleness in cemented soils as follows:

$$I_B = (\sigma'_1-\sigma'_3)_f / (\sigma'_1-\sigma'_3)_r - 1 \quad [1]$$

Figure 3 shows the variation of brittleness index with confining pressure for different cement contents. It is evident that the brittleness of the cemented soil decreases as the confining pressure increases. Also the brittleness index increases with increase in cement content.

Figure 4 shows the variation of stress ratio with axial strain for the undrained triaxial tests under 500 kPa confining pressure. According to the figure the peak stress ratio increases with cement content. But the strain corresponds to the maximum stress ratio decreases with increase in cement content. For the drained condition the peak shear strength of the cemented soil is coincided to the peak stress ratio. But according to the Figure 4 the peak shear strength takes place after peak stress ratio in the undrained condition.

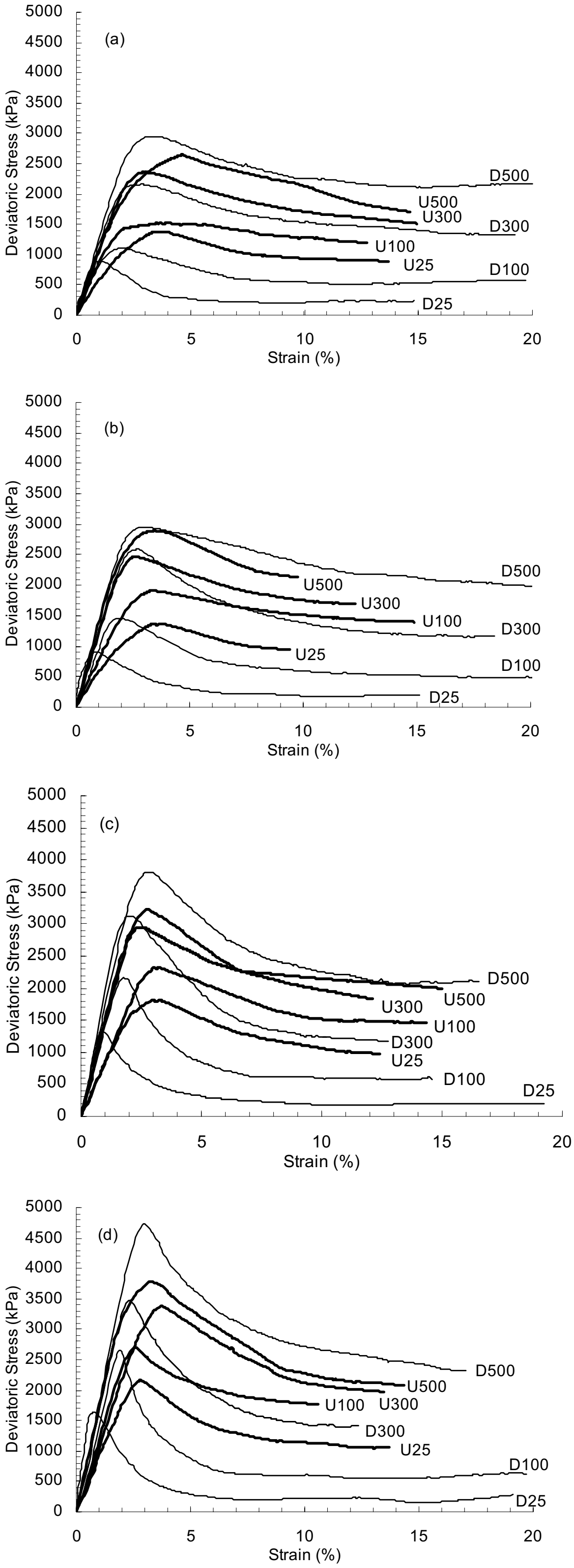

Figure 2. Stress-strain relation
(a)1.5% (b)3.0% (c)4.5% (d)6.0%

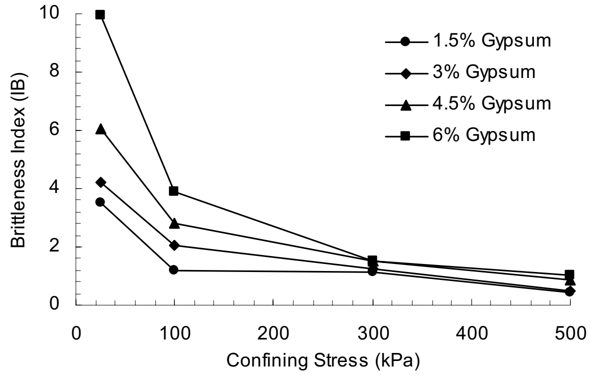

Figure 3. Brittleness index variation with confining pressure for different cement contents

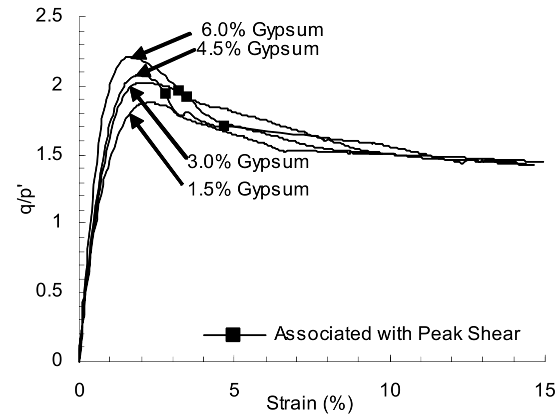

Figure 4. Variation of Stress ratio for undrained tests under confining pressure of 300 kPa

5.2. Volumetric strains and pore pressure hanges

Figure 5 shows the volumetric strain changes with axial strain for two sets of specimens considered in Figure 2. This figure shows that there is a little contraction in small strains followed by a large amount of dilation in the higher strains. According to this figure the final amount of dilation decreases as the confining pressure increases.
Contrary the soil contraction in small strains increases as the confining pressure increases.
Figure 6 shows the variation of pore pressure with mean effective stress for the two sets of samples. The figure shows that there is a positive pore pressure induced in small mean effective stresses. This corresponds to contractive part of behavior.

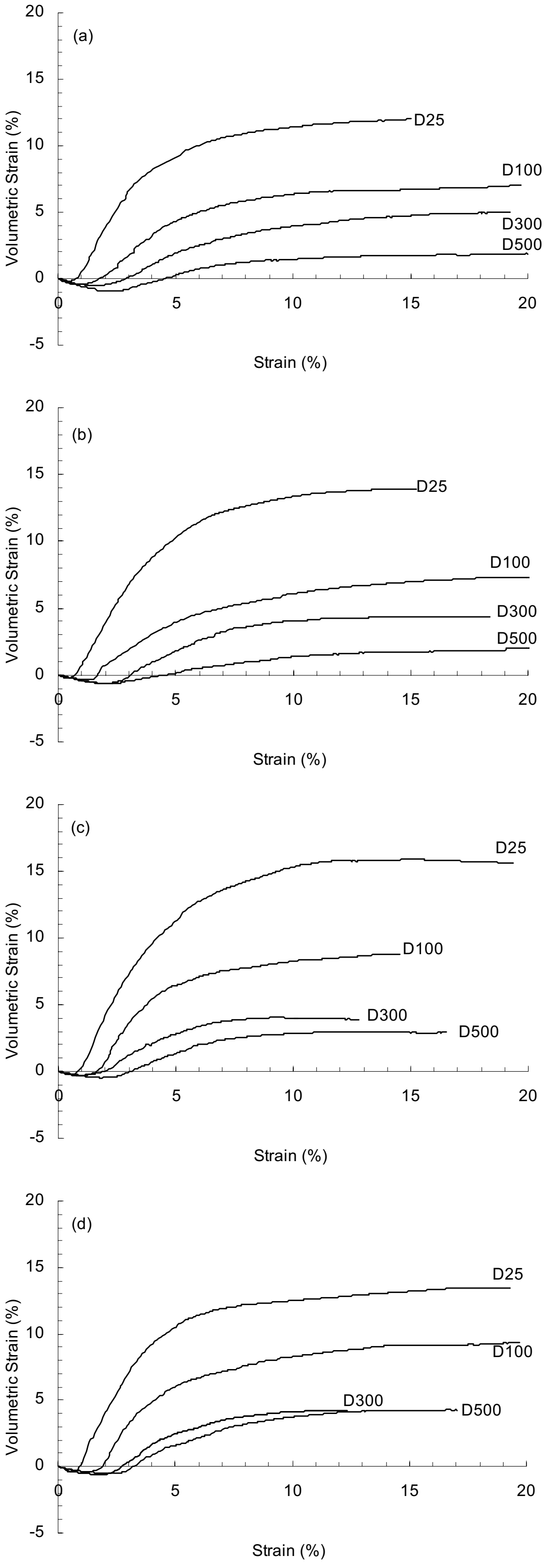

Figure 5. Volumetric strain-strain relation
(a)1.5% (b)3.0% (c)4.5% (d)6.0%

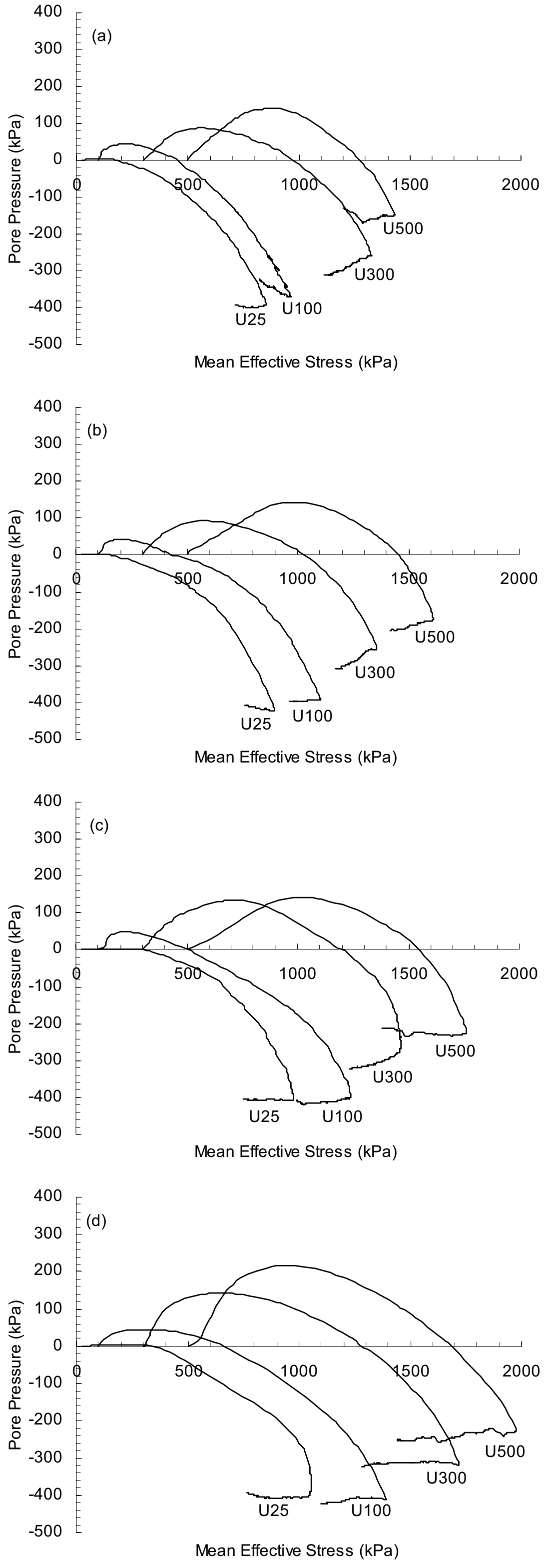

Figure 6. Pore pressure-mean effective stress relation
(a)1.5% (b)3.0% (c)4.5% (d)6.0%

After that the pore pressure reduces to induce soil suction and negative pore pressure in larger mean effective stresses. This includes the dilative part of soil behavior. According to Figure 6 the maximum positive pore pressure increases with confining stress. Also the final negative pore pressure reduces as the confining stress increases.

All the mentioned results confirm decrease in brittleness, dilation and negative pore pressure of cemented soil with confining pressure.

Skempton (1954) introduced pore pressure coefficients A and B to illustrate pore pressure generation in soils. Figure 7 shows the variation of Skempton's pore pressure coefficient at failure, A_f, for cemented and uncemented samples defined by the following equation:

$$A_f=\Delta u_f/\Delta\sigma_f \qquad [2]$$

According to this figure the pore pressure is positive for the uncemented soil in failure stage. However, it is negative in cemented soil. This negative value shows a similarity of the behavior of cemented soil to very dense granular soils that usually show negative pore pressure at failure.

5.3 Failure envelope and shear strength

Figure 8 shows the failure envelopes for the drained and undrained tests. According to this figure the failure envelopes other in drained and undrained conditions are different. The failure envelopes are nearly curved and move up with increase in cement content.

Figure 9 shows the variation of effective principal stress ratio at failure with confining pressure for cemented soil under consolidated drained triaxial tests. According to this figure the ratio of effective principal stresses at failure decreases as the confining pressure increases.

The soil cemented with higher cement content has a larger effective principal stress ratio at failure. Increase in confining pressure decreases the difference between effective principal stress ratios for different cement contents. For a confining pressure of 500 kPa the ratio of effective principal stresses are nearly identical for different cement content. Figures 10-a and 10-b show the variation of shear strength and shear strength ratio with confining pressure for uncemented and cemented soil with different cement contents. It is evident from Figure 10-a that the effect of cementation on the shear strength of cemented soil decreases when the confining pressure increases. In order to show the decreasing effect of cementation with increase in confining pressure, the ratio of shear strength of cemented soil to the shear strength of uncemented soil is plotted for different confining pressures. This is shown in Figure 10-b. According to this figure the shear strength of cemented soil approaches to that of uncemented with increase in confining pressure. The shear strength of cemented soil will approach to the uncemented soil shear strength in high confining pressures.

Figure 11 shows the changes in friction angle and cohesion intercept with cement content. Drained and undrained tests are considered for determination of shear strength parameters. Although the failure envelopes are found to have some curvature, a linear interpolation is used in order to define the failure envelope of all tests and to determine the shear strength parameters. There is a slight increase in friction angle with increase in cement content. This shows the little influence of cement content on friction angle. Contrary to the friction angle the cohesion intercept increases rapidly for cemented soil. According to this figure the important effect of cementation is on the soil cohesion.

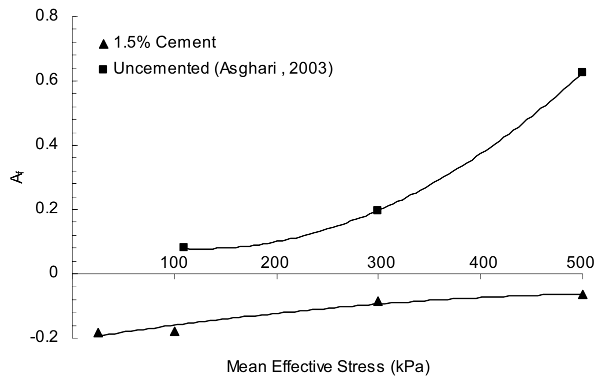

Figure 7. Skempton's A parameter at failure

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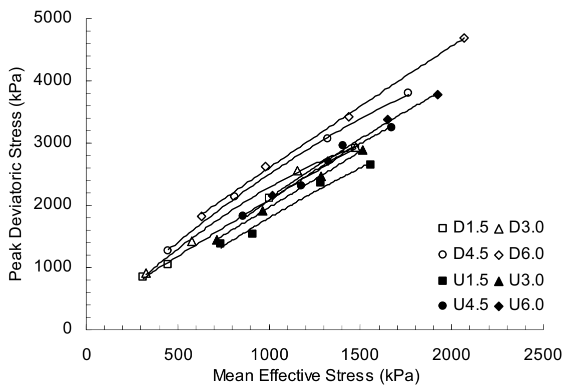

Figure 8. Failure envelopes of cemented soil

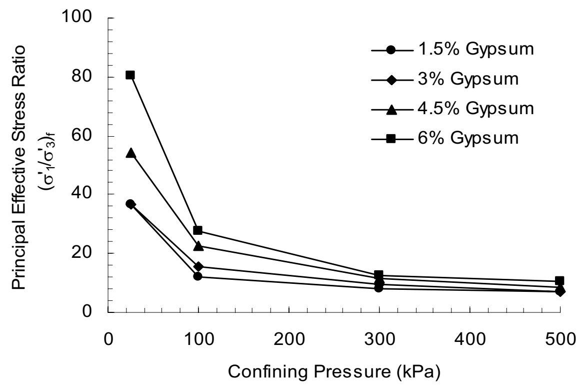

Figure 9. Variation of effective principal stress ratio

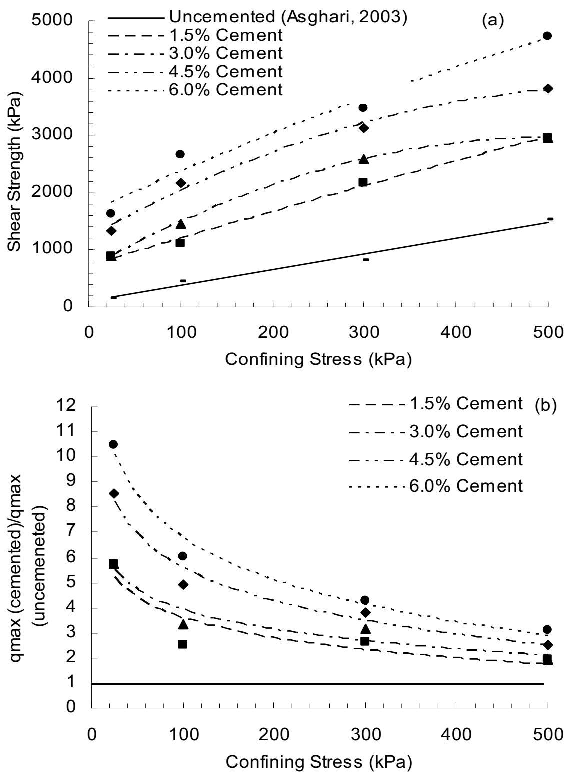

Figure 10. Variation of shear strength ratio

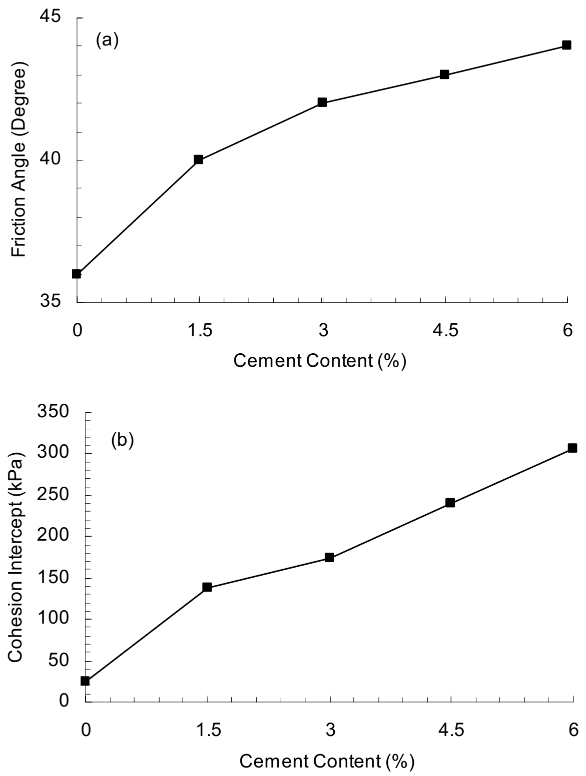

Figure 11. Friction angle and cohesion of cemented soil

6. SUMMARY

The influence of gypsum cementation on the shear behavior of a cemented gravely sand was evaluated and some new features of the behavior of cemented soils are shown and discussed. The results revealed that the cemented gravely soils with gypsum behave brittle and their stress-strain curves have apparent peak points. The behavior of soil is dilative at failure with a large amount of dilation in drained condition or a large negative pore pressure in undrained case. The failure curves are nearly curved and move up with increase in cement content. The friction angle increases slightly with cement content but the cohesion intercept increases more rapidly with increase in cement content. It is interpolated that the ratio of effective principal stresses decreases with increase in confining pressure. The shear strength of cemented soil approaches to the uncemented one in high confining pressures. This is an indicator of the decreasing effect of cementation in high confining pressures.

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