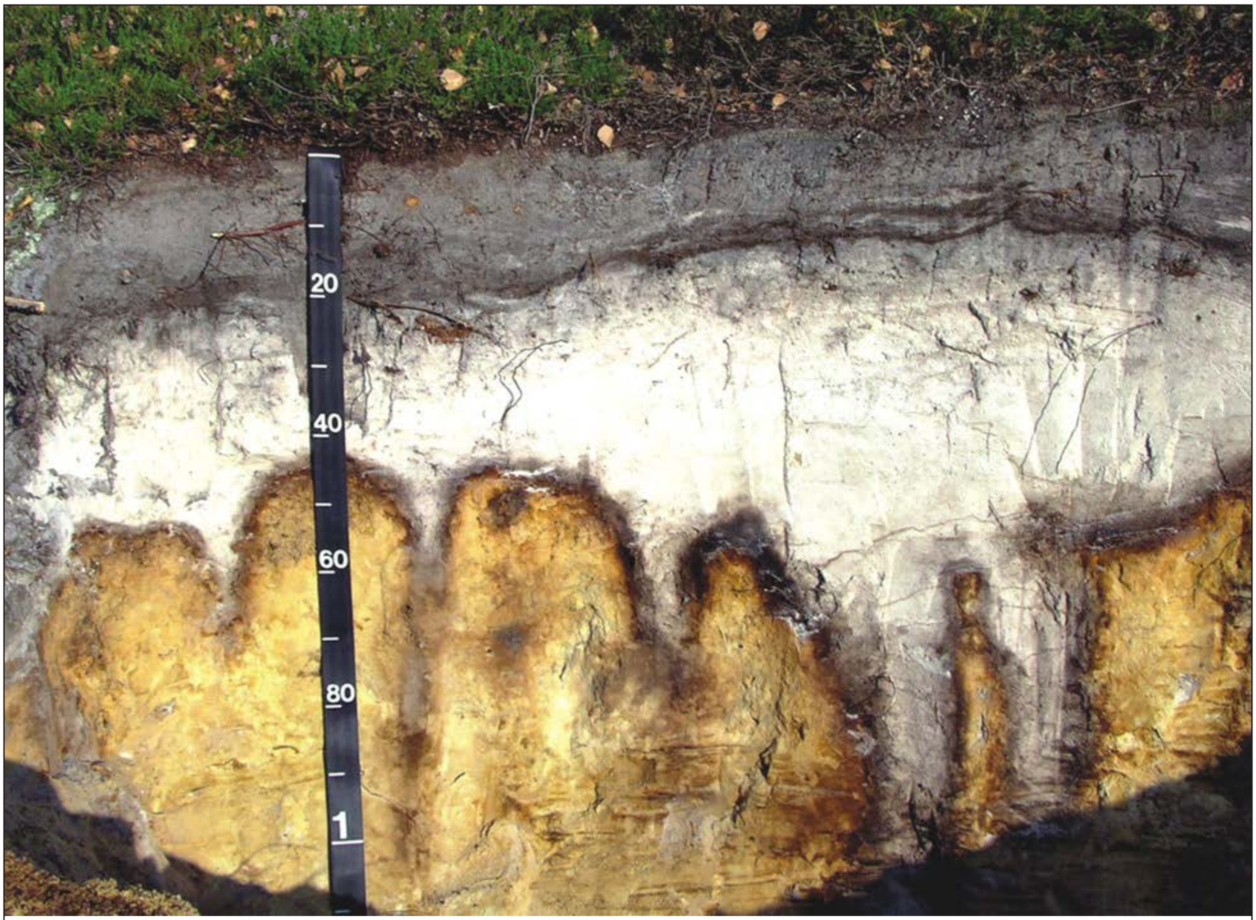

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Evaluation of Factors Affecting Parameter m in Drained Shear Strength of Over Consolidated Soils

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Abstract: This study presents the results of laboratory direct shear tests on two natural over consolidated clay samples and compacted laboratory over consolidated samples of different mixtures of bentonite sand. The intact shear strength of over consolidated soils can be obtained from $\tau_{oc} = \tau_{NC}\,(OCR)^{1-m}$. In that the intact shear strength of over consolidated soils are higher than shear strength of normally consolidated soils of the same constituent by factor of $(OCR)^{1-m}$. Application of this relationship for compacted over consolidated soil near optimum water content is considered. The effect of composition on the parameter m is sought. The effect of structure on m is evaluated by comparison of above test results with natural samples and with artificially cemented samples tested in direct shear test. The results show decrease in m value with increasing plasticity and cementation.

Key words: Shear strength, overconsolidation, peak strength, fully softened strength, residual, cemented soil

INTRODUCTION

One of important subject in geotechnical engineering that has been evaluated extensively over long period of time is shear strength of soils (Mesri and Shahien, 2003, 2004; Hamel, 2004; Yudbir, 2004; Duncan, 2001a, b; Christian and Baecher, 2001; Gilbert *et al*., 1998). The application and importance of shear strength of soils in practice have also been challenged in many geotechnical and geoenvironmental projects around the world (Filz *et al*., 2001; Eid *et al*., 2000; Zornberg *et al*., 1998a, b). Of many factors affecting shear strength of soils directly or indirectly, density, effective stress and soil structure are the most important (Terzaghi *et al*., 1996). Obtainable density under geological conditions is related mainly to the size, shape, surface characteristics and strength of particles constituent of soil. The effect of type of mineral (composition) of soil particles and physiochemical environment on shear strength is indirectly through their control of these important particle characteristics (Terzaghi *et al*., 1996).

The relationship between shear strength and effective normal stress for natural clays and clay shales is curved and concave to the effective normal stress axis (Stark and Eid, 1997; Mesri and Shahien, 2003) and there is no shear strength at zero effective normal stress (Terzaghi *et al*., 1996). The intact strength envelope of over consolidated clays and clay shales displays a pronounced curvature because swelling and softening intensify as effective normal stress decreases toward zero (Mesri and Shahien, 2003). The empirical equation by Mesri and Abdelghafar (1993) for intact strength envelope of over consolidated soils is:

$$\tau_{oc} = \sigma'_n \tan\phi' \left(\frac{\sigma'_p}{\sigma'_n}\right)^{[1-m]} \tag{1}$$

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which means the intact strength is higher than normally consolidated strength of the same constituent assuming linear relationship between shear strength effective normal stress for the latter. In Eq. 1, σ'_p is over consolidation pressure, σ'_n is effective normal stress on sliding surface and m is slope of failure line log τ_f Vs log σ'_n. The magnitude of m depends on structure and composition of soil (Mesri and Abdelghafar, 1993). High curvature of failure envelope means lower m and for very low curvature the magnitude of m nears one. Recently, low curvature failure envelopes has been also reported for fully softened and residual states (Stark and Eid, 1997; Mesri and Shahien, 2003). The fully softened strength envelope displays a curvature because, even for a random arrangement of particles, high effective normal stresses promote face to face interaction of plate shaped particles. The residual strength envelope is curved because a higher degree of particle orientation in direction of shearing is possible at high effective normal stresses (Mesri and Shahien, 2003).

In this study, the effect of composition, structure and degree of cementation on parameter m using results of direct shear tests on natural and compacted laboratory over consolidated soil samples is evaluated.

MATERIALS AND METHODS

In this research, both natural over consolidated samples from Khuzestan province and compacted laboratory over consolidated soil samples at optimum water content were tested.

Natural Soil Specimen Preparation

Natural soils tested were Shelby tubes samples of Behbahan clay obtained from depth of 4 m below the surface and core samples of Ahwaz clay from depth of 24 m. Samples of Behbahan clay were obtained from the site of faculty of Engineering of Behbahan and those of Ahwaz clay from site of Ahwaz metro in Golestan area. Consolidation tests were performed on samples to determine over consolidatation pressure of soil. Direct shear tests were performed to determine failure envelope. Consolidation specimens were carefully placed into the ring using surgical blade and after transferring to loading frame were loaded incrementally with load increment ratio $\frac{\Delta\sigma_v}{\sigma_v} = 1$ until the end of primary consolidation to maximum pressure of 1600 kPa.

Direct shear specimens were placed into the shear box and consolidated under normal stress of 24, 48, 192 and 347 kPa before they were sheared at the rate of 1 mm min^{-1}. the lowest rate of the machine. Mesri and Abdelghafar (1993) showed that c' (cohesion intercept) obtained from triaxial and direct shear tests for 25 natural clays compared well. This and the fact that direct shear test would give the result of drained test in a faster time than triaxial test were the primary reasons to choose this type of equipment for our testing program.

In order to obtain normally consolidated state failure envelope of natural soil constituent, samples were prepared following the procedure given by Terzaghi *et al*. (1996). In this procedure samples were dried ball milled and passed from the No. 200 US sieve before they were mixed with water at water content equal to liquid limit. Then the samples were transferred into direct shear box and were sheared under normal stress of 24, 48, 96, 192 and 347 kPa.

Preparation of Artificial Compacted Soil Samples

Compacted soil samples tested were mixtures of 90% sand 10% bentonite, 85% sand, 15% bentonite, 80% sand, 20% bentonite and 70% sand, 30% bentonite. In all mixtures sand portion was finer than No.40 US sieve. For mixture of 80% sand, 20% bentonite sand finer than No.10 US sieve was also used. All mixtures were compacted at their optimum water content using the standard Proctor procedure and consolidated to the maximum pressure of about 1600 kPa.

After consolidating to the maximum pressure and before dismantling the test, each specimen was unloaded in two steps. In first step, the specimen was unloaded to the pressure that was equal to the normal effective stress at which it was going to be sheared and allowed to swell until primary swelling. Then the water in the oedometer was emptied and the specimen was unloaded to seating pressure, taken out from the cell and quickly placed into shear box and the pressure at which the samples was unloaded in the first stage was applied on the specimen before it was sheared to failure. The over consolidated state failure envelopes for different mixtures in the range of effective normal stresses of 24, 48, 96, 182 and 350 kPa were determined in this way.

Sample preparation for bentonite-sand mixtures for determining failure envelope of normally consolidated state was the same as for natural soil samples explained earlier and it is not repeated here. However because of long period of time required for performing these tests especially for samples with higher bentonite content they were performed only on mixture of 90% sand, 10% bentonite. Therefore, failure envelope for normally consolidated state of other mixtures were obtained using empirical relationships between ϕ' and I_p (Terzaghi *et al*., 1996).

RESULTS

Index Properties of Samples Tested

Results of liquid and plastic limits, natural water content and hydrometry tests for natural soil samples are shown in Table 1 and for bentonite-sand mixtures are shown in Table 2. In Table 2 optimum water content at which test samples were prepared are also given.

Consolidation Tests Natural Samples

Results of consolidation tests on undisturbed samples of Ahwaz and Behbahan clays in the from of end of primary e-log σ'_v are shown in Fig. 1 and 2. In Table 3, consolidation parameters such as recompression index C_r, compression index C_c, initial void ratio e_0 and over consolidation pressure σ'_p

Table 1: Index properties of natural soils

	Soil	
Properties	Behbahan clay	Ahwaz clay
W_n (%)	19.20	18.00
W_l (%)	27.10	39.70
W_p (%)	18.90	20.00
CF (%)	68.00	75.00
A_C	0.12	0.28

Table 2: Index properties of bentonite-sand mixtures

	Mixture			
Properties	70% sand, 30% bentonite	80% sand, 20% bentonite	85% sand, 15% bentonite	90% sand, 10% bentonite
W_l (%)	88.00	59.40	44.9	32.50
W_p (%)	21.10	15.80	14.9	12.60
$W_{optimum}$ (%)	18.00	16.00	15.0	14.00
CF (%)	30.00	20.00	15.0	10.00
A_C	2.19	2.18	2.0	1.99

Table 3: Consolidation properties of natural soils

	Properties			
Soil	e_0	C_r	C_C	σ'_p (kPa)
Behbahan clay	0.51	0.007	0.067	150
Ahwaz clay	0.45	0.006	0.071	560

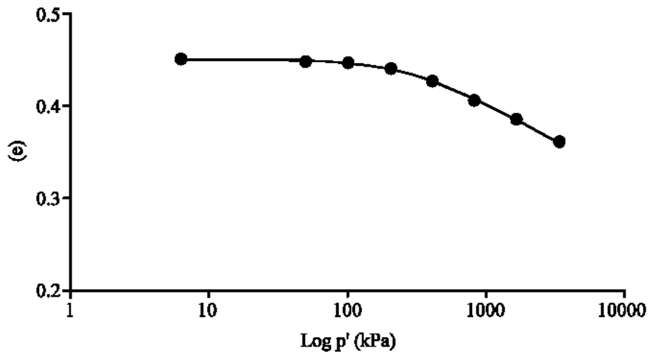

Fig. 1: EOP e-logp' relation for Ahwaz clay

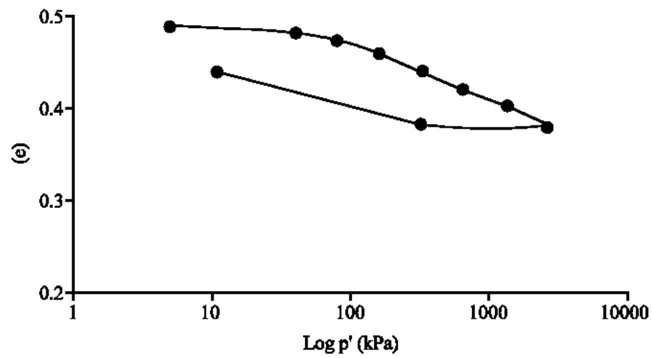

Fig. 2: EOP e-logp' relation for Behbahan clay

for these two clays are given. The values shown in Table 3 are in the range of values reported in literature for over consolidated soils and shales (Terzaghi *et al*., 1996). Over consolidation pressures are obtained using Cassagrandes method. Over consolidation pressure of the Behbahan clay from depth of 4 m is estimated about 150 kPa and that for the Ahwaz clay from depth of 24 m is about 560 kPa.

Direct Shear Tests Natural Samples

Plots of horizontal displacement versus vertical displacement for direct shear tests on the natural clays of Behhahan and Ahwaz are shown in Fig. 3 and plots of τ versus δ_h are shown in Fig. 4. These clays during shear at low normal stresses expanded and at high normal stresses compressed, typical behavior of over consolidated clays.

Failure Envelopes for Natural Samples

In order to plot intact failure envelopes of the natural clays of Ahwaz and Behbahan values of τ_{max} are plotted against σ'_n and are shown in Fig. 5. The best curve is fitted to test data points. In Fig. 5 failure envelopes for natural sample composition in normally consolidated state are also shown. The normally consolidated state envelopes for both clays is linear and those for natural samples are curved as expected. It is noted that overconsolidated and normally consolidated states failure envelopes for the Behbahan clay merged at a pressure of 150 kPa equal to overconsolidation pressure obtained in oedometer test. This was only true for the Ahwaz clay when the normally consolidated state failure

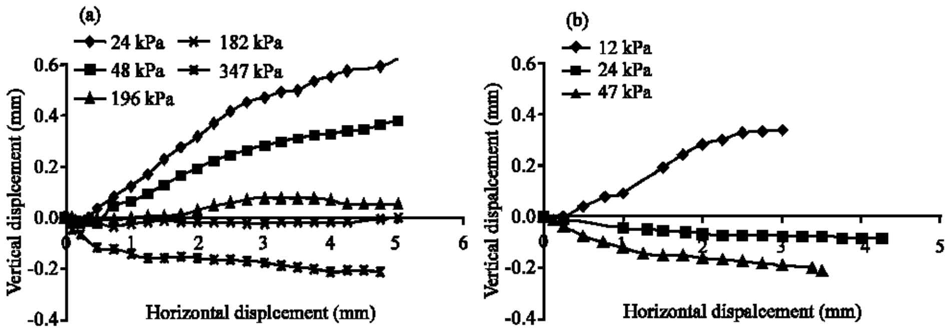

Fig. 3: δ_v-δ_h relation for (a) Ahwaz and (b) Behbahan clay

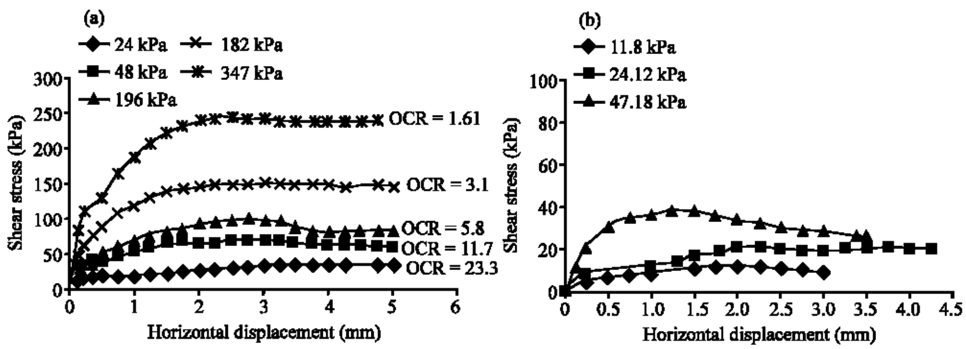

Fig. 4: τ-δ_h relation for (a) Ahwaz and (b) Behbahan clay

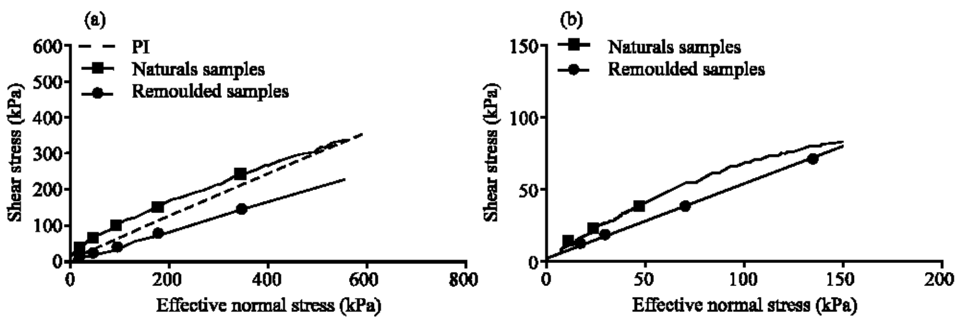

Fig. 5: τ-σ'_n relation for (a) Ahwaz and (b) Behbahan clay

envelope was obtained using the empirical relation of $I_p-\phi'$ by Terzaghi *et al.* (1996). Mesri and Abdelghafar (1993) indicated that overconsolidation pressure obtained in oedometer and that obtained from failure envelope were the same for most clays they reviewed but for some others they were not.

Determination of Parameter m for Natural Samples

In order to determine parameter m for Ahwaz and Behbahan clays, log τ_{max} are plotted against log τ'_n in Fig. 6. Parameter m for the Ahwaz clay is about 0.61 and for the Behbahan clay is 0.8. These values are in the range of values reported for stiff clays and shales with intact and disturbed structure (Terzaghi *et al.*, 1996). The Behbahan clay sample from depth of 4 m may have acquired disturbed structure in nature or during sampling or laboratory preparation; in fact we had difficulty taking the

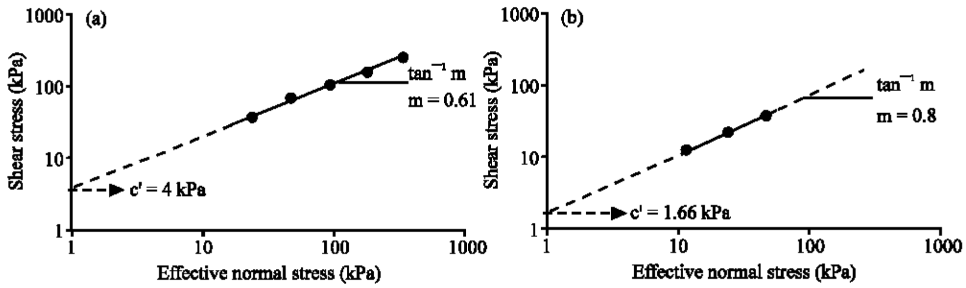

Fig. 6: log τ-log σ'_n relation for (a) Ahwaz and (b) Behbahan clay

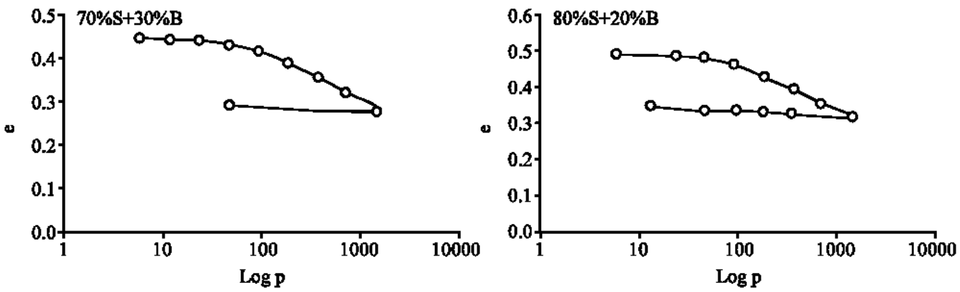

Fig. 7: EOP e-logp' relation for bentonite sand mixtures

sample out from Shelby tube. According to Mesri and Shahien (2003) there can be a wide variation in the intact strength at any effective normal stress because the stiff clay or clay shale may experience different degree of fissuring and softening during its geological history.

Consolidation Test Sand Mixtures

The results of consolidation tests on bentonite-sand mixtures in the from of end of primary e-log σ'_v are shown in Fig. 7.

Direct Shear Tests Sand Mixtures

The plots δ_v versus δ_h and τ vs δ_h for direct shear tests on compacted over consolidated bentonite sand samples are shown in Fig. 8 and 9, respectively. All samples at all effective normal stresses tested first compressed and then expanded during shear.

Over Consolidated and Normally Consolidated Failure Envelopes

The values of τ_{max} versus σ'_n for over consolidated bentonite-sand mixtures are plotted in Fig. 10. The best curve is fitted through the test data. In Fig. 10 also failure envelopes for normally consolidated state of these samples using empirical relationship between Φ' and I_p from Terzaghi *et al.* (1996) are shown. For mixture of 90% sand, 10% bentonite, normally consolidated state failure envelope is obtained from direct shear tests on remolded samples at water content equal to liquid limit. All overconsolidated and normally consolidated failure envelopes merged at overconsolidation pressure of about 1600 kPa at which all overconsolidated samples experienced in oedometer test.

Determination of Parameter m

Log τ_{max} versus log σ'_n from direct shear testes on bentonite-sand samples are sown in Fig. 11. The value of m falls between 0.62 to 0.8. The value of m is increased with amount of sand.

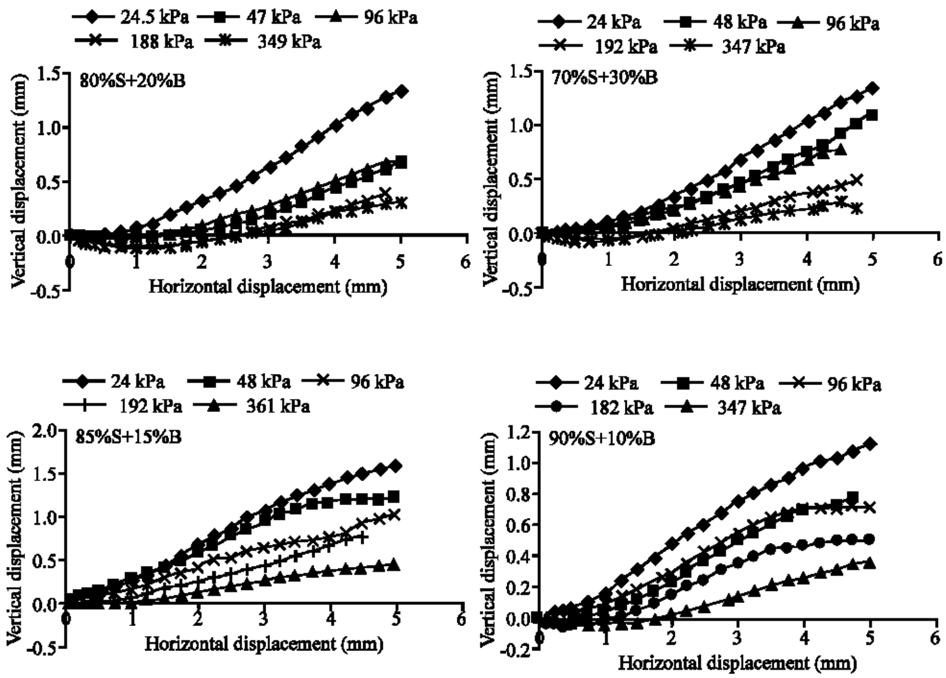

Fig. 8: δ_v-δ_h relation for bentonite sand mixtures

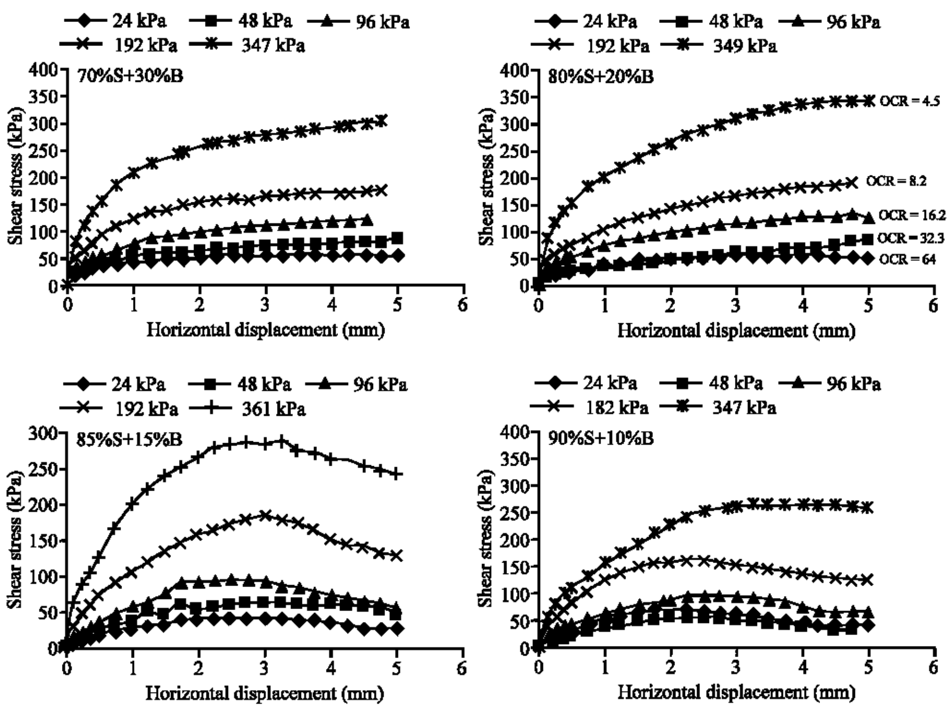

Fig. 9: τ-δ_h relation for bentonite sand mixtures

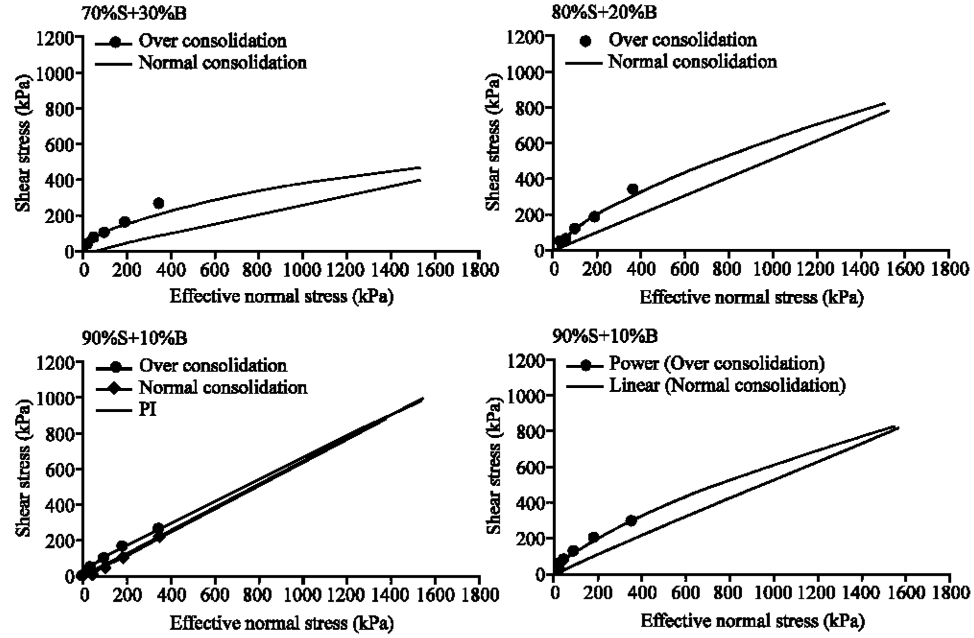

Fig. 10: τ-σ'_n relation for bentonite sand mixtures in overconsolidated and normally consolidated state

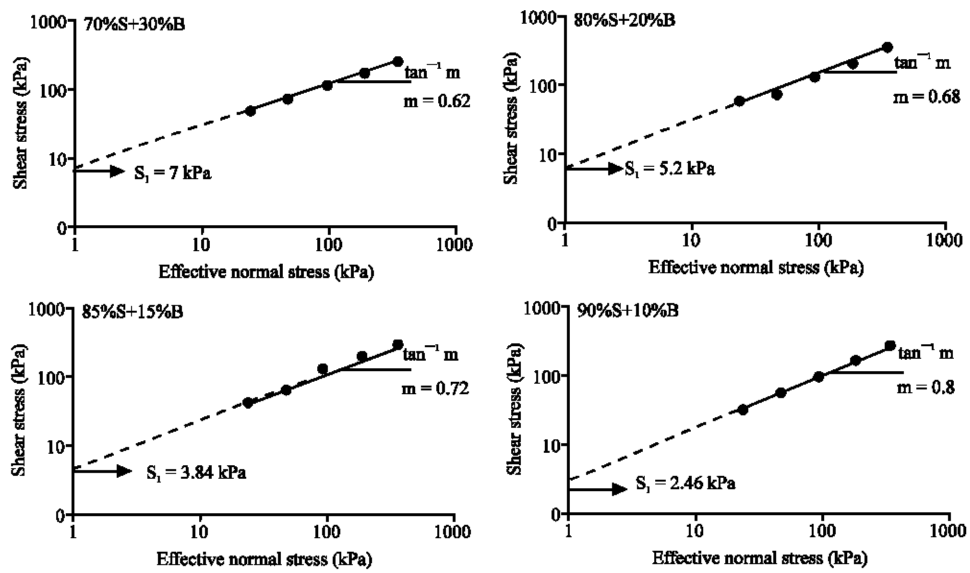

Fig. 11: log τ-log σ'_n relation for bentonite sand mixtures in overconsolidated state

DISCUSSION

The Effect of I_p on m

In Fig. 12 values of m obtained for natural and compacted samples are plotted against I_p. As indicated also by Mesri and Abdelghafar (1993) value of m is decreased with Ip. The values of m for

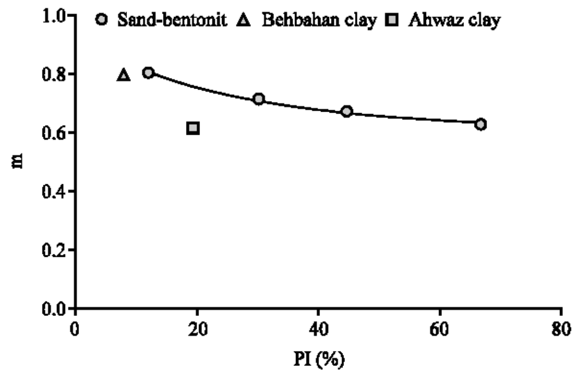

Fig. 12: Coefficient m-plasticity index relation for natural clays and compacted overconsolidated bentonite sand mixtures

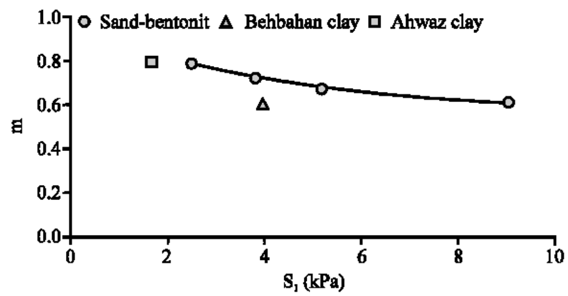

Fig. 13: Coefficient m-S_1 relation for natural and compacted overconsolidated bentonite sand mixtures

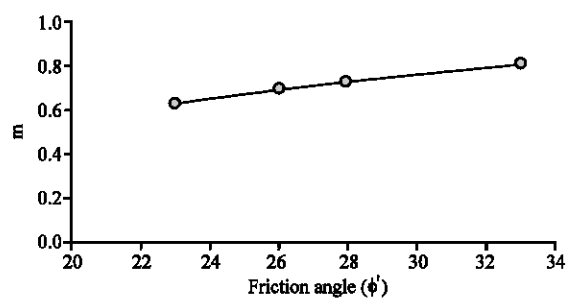

Fig. 14: Coefficient m-Friction angle, ϕ' for bentonite sand mixtures

natural samples falls below the trend for compacted samples. Compacted samples lack bondings due to cementation, diagenetic and etc. that exist for natural samples.

Relation of m with s_1

In Fig. 13, the coefficient of m is plotted against s_1, the intercept of log $\tau -$ logσ'_n for natural clays and bentonite sand mixtures. A decreasing trend in m is observed with increasing s_1.

Relation of m with ϕ'

In Fig. 14 the coefficient of m is plotted against angle of friction ö' obtained from I_p- ϕ' empirical relation from Terzaghi *et al*. (1996) for bentonite sand mixtures. An increasing trend of m with ϕ' is observed.

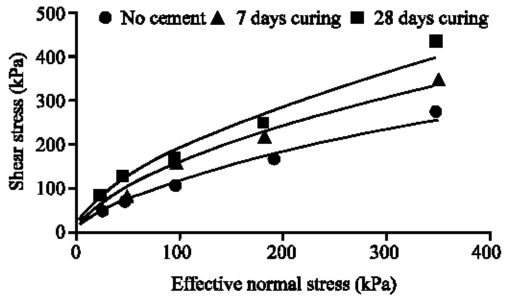

Fig. 15: τ-σ'_n relation for cemented compacted and overconsolidated bentonite samples

Determination of Effect of Cementation on m

In order to quantify the influence of cementation on parameter m, mixtures of 70% sand, 30% bentonite was mixed with 5% of Portland cement and compacted at optimum water content and over consolidated to maximum pressure of 1600 kPa. Two series of samples were prepared, one series was cured for a period of 7 days and second series was cured for 28 days. After curing period, samples were tested in direct shear. Failure envelope for these samples and samples without cement are compared in Fig. 15. Values of m for these samples were 0.59 and 0.57 for 7 and 28 days curing times respectively as compared to 0.62 of untreated samples. As it is shown cementation causes a decrease in value of m.

CONCLUSIONS

Natural over consolidated soil samples and compacted over consolidated bentonite- sand mixtures at optimum water content were tested in direct shear test in order to determine the parameter m. The range of values of m obtained for natural samples in this research was in the range that has been reported by others for stiff clays and shales. It is also indicated that as the plasticity of soil increases the value of m decreases, another word as the amount of sand in soil decreases, the value of m decreases. The results of test on cemented samples showed that cementation also decreases the value of m.

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