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Model Studies on Shear Behavior of Reinforced Reconstituted Clay

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***Abstract*—**In this paper, shear behavior of reconstituted clay reinforced with varying diameter of sand compaction piles with area replacement-ratio (a_s) of 6.25, 10.24, 16, 20.25 and 64% in 100mm diameter and 200mm long clay specimens is modeled using consolidated drained and undrained triaxial tests under different confining pressures ranging from 50kPa to 575kPa. The test results show that the stress-strain behavior of the clay was highly influenced by the presence of SCP. The insertion of SCPs into soft clay has shown to have a positive effect on the load carrying capacity of the clay, resulting in a composite soil mass that has greater shear strength and improved stiffness compared to the unreinforced clay due to increased reinforcement area ratio. In addition, SCP also acts as vertical drain in the clay thus accelerating the dissipation of excess pore water pressures that are generated during loading by shortening the drainage path and activating radial drainage, thereby reducing post-construction settlement. Thus, sand compaction piles currently stand as one of the most viable and practical techniques for improving the mechanical properties of soft clays.

***Keywords*—**Reconstituted clay, SCP, shear strength, stress-strain response, triaxial tests.

I. Introduction

NATURAL soft clayey soil deposits are widely distributed especially along the coastal area, having large potential for settlement with low inherent shear strength. These natural clays in in-situ condition exhibit different structure compared to that of reconstituted clays in the laboratory, which often breaks down under loading wit high degree of uncertainties. Therefore, to examine these issues without uncertainties due to variations in geological history and sample disturbance, clay has been reconstituted in the laboratory under carefully controlled conditions to create reproducible soil specimens in order to develop a structure similar to the structure of natural clay. Many researchers have demonstrated that this response can be described using the framework of critical state soil mechanics [1]. The compressibility and shear strength of remoulded clays can be used as a frame of reference for the behaviour of a natural, undisturbed sample. The properties of such type of soil are referred to as intrinsic, as these parameters were felt to be unique and inherent for a given soil type [2]. Similar work to examine the mechanical properties of reconstituted clay has been carried out recently and in the past [3]-[6]. Laboratory tests are commonly used to determine design parameters of natural clays. Many aspects of the engineering behaviour of cohesive soils have been explained by various authors [7], [8]. Sample disturbance is one of the major factors influencing the accuracy of measured mechanical properties of natural soft clays [9], [10]. Kaolin clay has been widely used both in fundamental studies of soil behaviour and in physical model tests. Since the mechanical behaviour of structured soils can usually be quantitatively assessed based on the mechanical behaviours of the corresponding reconstituted clays, therefore, an attempt has been made to study the shear behaviour of reconstituted kaolin clay specimen reinforced with sand compaction pile (SCP) by using consolidated drained and undrained triaxial tests. Since the soft ground is having large potential for settlement with low inherent shear strength, their improvement has been extensively implemented for the various development projects all over the world due to extremely limited stable construction sites [11]. Although there are a variety of ground improvement techniques under different categories, ground improvement by using column-type techniques such as sand compaction piles (SCPs) are used on an increasing scale for soft ground improvement for improving the engineering characteristics of soft soils. Sand compaction pile (SCP) is a method of constructing large diameter sand columns in the ground. This method of ground improvement has been widely used for rapid improvement of soft ground, and also in near-shore regions for land reclamation works [12]. Thus, the aim of this study is to model the behavior of soft cohesive soil reinforced with different diameter SCPs by observing the change in drained and undrained shear strength of the composite ground. In this study, the shear strength behavior of reconstituted clay specimens of 100mm diameter reinforced with sand compaction piles is investigated with different cell pressures (50-575kPa) using consolidated undrained triaxial tests. The composite specimens were prepared with area replacement-ratio (a_s) in the range of 6.25-64% in different specimens by driving a PVC casing into the sample and then backfilling the cavity with sand column after removing the casing in a custom-designed setup. A custom-designed arrangement was developed for installation of SCPs in kaolin specimen consolidated on the laboratory floor before the triaxial testing (Fig. 1). A pneumatic compactor was used to compact reconstituted clay specimens reinforced with SCPs before subjecting to triaxial compression tests. It has been observed that the stress-strain characteristics of reconstituted clay specimens was greatly improved and load bearing capacity and stiffness of composite soil is higher compared to untreated reconstituted clayey soil. SCPs also accelerate the dissipation

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of excess pore water pressures by shortening the drainage path in radial direction compared to vertical direction in untreated clayey soil under loading. Hence, SCP is one of the most effective technique for reducing post construction settlement and improve the mechanical properties of soft soils.

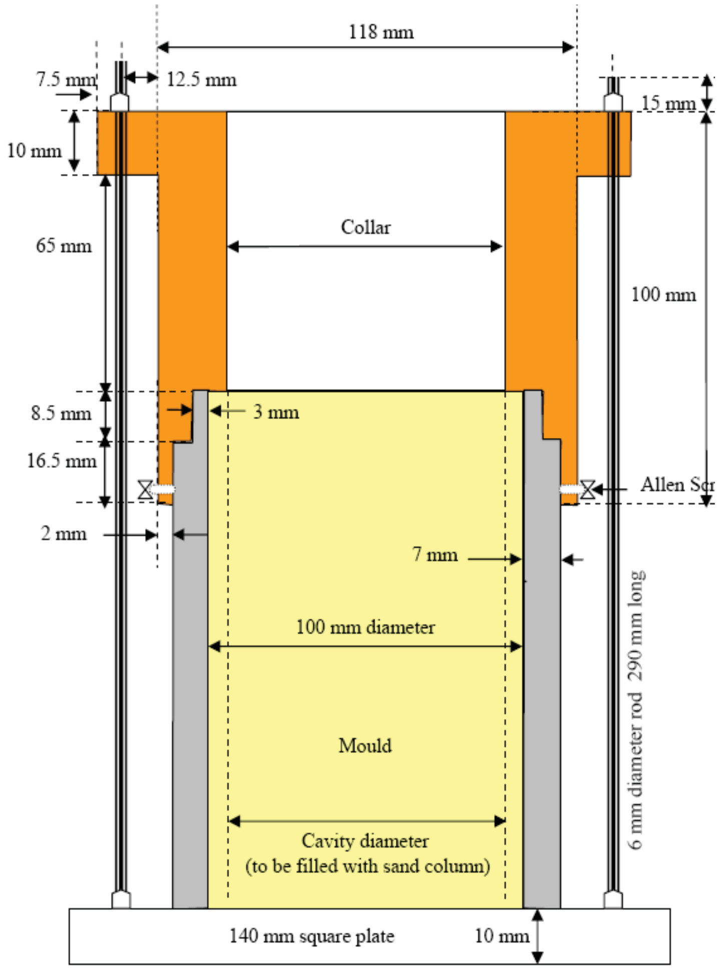


Fig. 1 Custom designed set-up for installation of 80 mm SCP

II. Experimental Program

Isotropic consolidated drained and undrained triaxial shear (CIU) tests were conducted using kaolin clay. Table I summarises the index properties of the clay.

TABLE I
PROPERTIES OF KAOLIN CLAY

Clay (%)	Silt (%)	Liquid limit (%)	Plastic limit (%)	Shrinkage limit (%)	G_s
75	25	49	23	16	2.64

The experimental program consisted of 17 tests (Table II) on clay specimen reinforced with SCP as per relevant codal procedures [13], [14]. The samples were prepared from slurry in 250mm diameter and 450mm long stainless steel cylindrical mould. The de-aired clay slurry was consolidated on the laboratory floor, first under its own self-weight and later under a surcharge which varied between 194 and 352 kN/m^2 applied in steps using a custom designed pneumatic load frame. Fig. 2 shows the consolidation setup and trimming process for reconstituted clay specimen during slurry consolidation.

Preconsolidation pressure (p_c') in 1-D consolidation test was calculated using:

$$p_c' = \sigma_V' \left(1 - 0.67 \sin\phi'\right) \tag{1}$$

where ϕ' is the effective angle of friction obtained from post consolidated undrained shear tests. Likewise, p_c' on the unloading-reloading line (URL) was calculated using:

$$p_c' = \sigma_v' \begin{bmatrix} 0.33 + 0.67(1 - \sin\phi') \\ \exp\{(0.93 - 0.85(1 - \sin\phi'))\ln(OCR)\} \end{bmatrix} \tag{2}$$

where; OCR is the over consolidation ratio (p_c'/p') and p' is the mean effective stress. The soil specimen was then isotropically consolidated under mean effective stress, p' given as:

$$p' = \frac{\sigma_1' + 2\sigma_3'}{3} \quad (3)$$

where σ_1' is the effective axial stress and σ_3' is the effective radial stress. p' was varied between 30- and 575kN/m^2 in different tests. Since the OCR was not 1 in all the specimens, it has often been argued that undrained shear strength of soil deposited due to pure volumetric compression often does not exactly follow the behavior that is prescribed by K_o-consolidated soil [15]. Many researchers have demonstrated that remolded soils had a significant effect on the stress-strain and pore water pressure behaviour under undrained condition [16], [17]. Furthermore, results indicate that the behavior of the failure zone which has reached critical state is essentially the same irrespective how the specimen has been prepared, as expected. The main difference between the predictions of the two methods of preparation lay in the response of the yielded zone that is which has not reached the critical state. However, the soil was remoulded and reconsolidated and there is unlikely to be any significant development of the fabric.

Fig. 2 Consolidation setup on the laboratory floor

After the completion of one-dimensional (1D) consolidation, the block of clay was extruded and trimmed into 100mm diameter and 200mm long cylindrical samples. For preparation of SCP specimens, additional 2 steps were undertaken to prepare the composite samples. In the first step, a cylindrical hole was cored through the centre of the sample using a thin smooth casing (Fig. 3). The diameter of the casing varied between 25mm and 80mm in different specimens, which corresponds to an area replacement ratio (a_s) that ranges between 6.25%, and 64%. This replacement ratio is lower than that often used in current practice [18]. After preparing the composite specimen with SCP, two saturated porous stones were placed at the two ends. The entire assembly was then mounted on the triaxial frame and the chamber filled with water. Care was taken to deair the cell base and drainage line before placing the sample. A check on pore pressure coefficient [19] showed that B-value was more than 0.94 in many samples. Notwithstanding this, a back pressure of 150 to 200kN/m^2 was applied during the tests. In this procedure, the cell pressure and back pressure were increased in steps using two independent air-bladder and water-interface cylinders. The cell pressure was made to lead the back pressure by not more than 10 kN/m^2. The soil samples were then isotropically consolidated under mean effective stress, *p'* which varied between 149 and 575 kN/m^2. Some samples were allowed to swell back to a lower mean effective stress.

TABLE II
EXPERIMENTAL PROGRAM FOR COMPOSITE CLAY SPECIMENS

Test No.	Dia. of casing/ SCP (mm)	Mean effective stress p′ (kN/m²): At the end of 1-D loading	Mean effective stress p′ (kN/m²): At the end of consolidation/ shearing, (p_o')	Preconsol. pressure p_c' (kN/m²)	OCR (p_c' /p_o')
S-1	25 (no smear)	285	100	285	2.85
S-2			150	285	1.87
S-3			300	300	1
S-4	25 (with smear)	285	100	285	2.85
S-5			150	285	1.87
S-6			300	300	1
S-7	30 (no smear)	149	450	450	1
S-9			50	149	2.98
S-10	30 (with smear)	149	450	450	1
S-12			50	149	2.98
S-13	40 (no smear)	149	575	575	1
S-14			375	375	1
S-15			75	149	1.99
S-16	40 (with smear)	149	575	575	1
S-17			375	375	1
S-18			75	149	1.99
S-21	80 (no smear)	149	150	150	1
S-22	80 (with smear)		150	150	1

Fig. 3 Preparation of SCP in reconstituted clay specimen

In the second step, air-dry sand (d_{50} = 0.3mm) was poured into the hole and compacted in layers at 90% relative density using a pneumatic compactor (Fig. 4).

The final diameter of the SCP was equal to the diameter of the hole. In some samples, the surface of the casing was made gritty by painting a paste of coarse sand mixed with araldite. This helped to create a smear zone around the SCP. Thickness of the smear zone was taken equal to the thickness of the paste. The effect of smear beyond this zone was ignored.

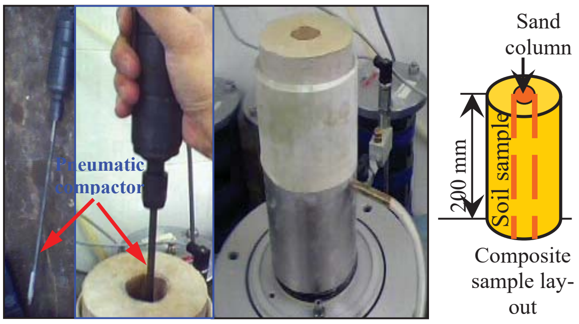


Fig. 4 Compaction of SCP in reconstituted clay specimen

III. Results and Discussion

A. Undrained Shear Behaviour of Reconstituted Clay Specimens with and Without SCP

Figs. 5 (a), (b) show results of deviator stress, q plotted against axial strain, ε_a. As can be seen, all samples reached peak deviator stress (q_{max}) at 6 to 10% axial strain. Figs. 5 (a) and (b) also show that the ultimate strength exhibit transient peaks in some tests. This was expected since these soil samples were overconsolidated prior to the shearing. In few tests on normally consolidated clays, q decreased after passing q_{max} because of instability of the failed samples at high confining pressure.

It is seen that for low stress level tests (100kPa), the ultimate strength for both types of samples is same. However, with increase in stress level SCP diameter and smear effect, the strength behavior is quite different. It is also seen that in case of increasing SCP diameter and confining stress, the stress-strain curves exhibit a hardening behavior. This is understandable that in case of higher replacement ratio for 80mm dia SCP, the stiffness is controlled by SCP rather than clay alone. On the other hand, for 40mm SCP, the stiffness under high stress level (575kPa) is equally contributed by SCP and clay. For normally consolidated samples, the stress-strain curves do not show any significant change in q after passing peak stress. This will help in framing the design charts for field engineers and designers to select a proper SCP size and stress level for improvement of soft ground.

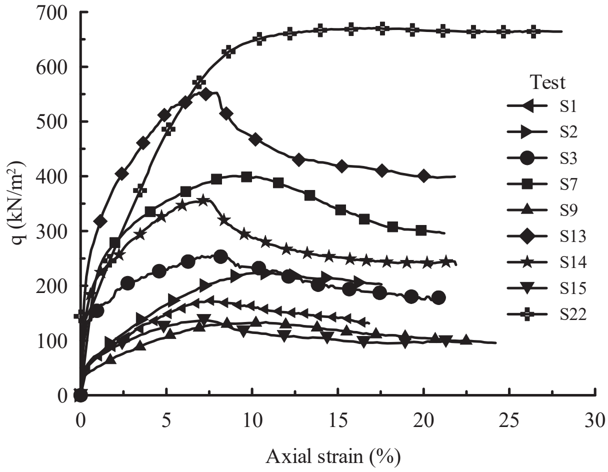
Figs. 6 (a), (b) show results of pore pressure parameter, A [19], plotted against axial strain, ε_a. As can be seen, A_f of these samples varies between 0.7 and 1.1 which is typical for normally consolidated clays.

Pore pressure parameter A is usually found to vary with the stress ratio, q/p' vide the relation:

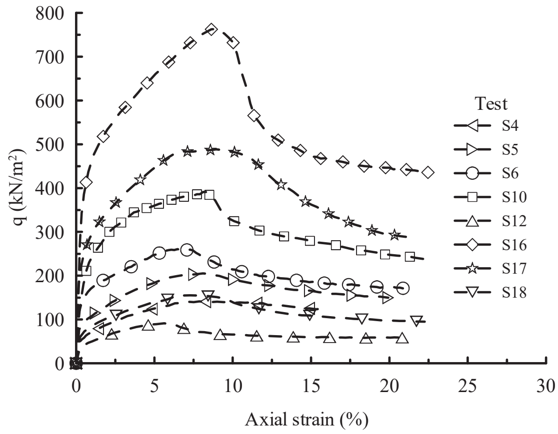
$$A = 0.33 + m\frac{q}{p'} \tag{4}$$

where m is the fitted parameter which relates the shear induced pore pressure to parameter A. m is equal to zero within the elastic zone. Also, pore water pressure in terms of stress invariants can be written as:

$$du = dp + \left(A - \tfrac{1}{3}\right)dq \tag{5}$$

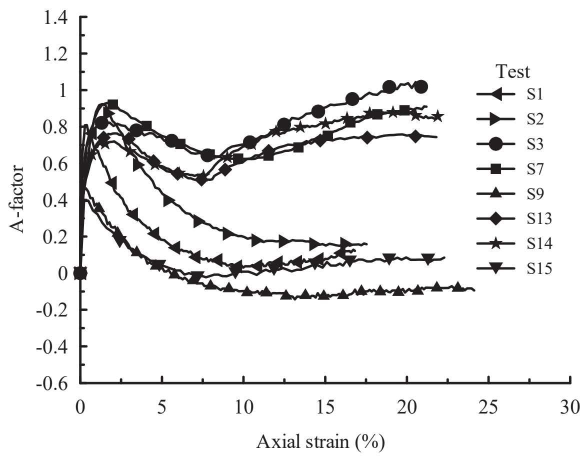


(a)

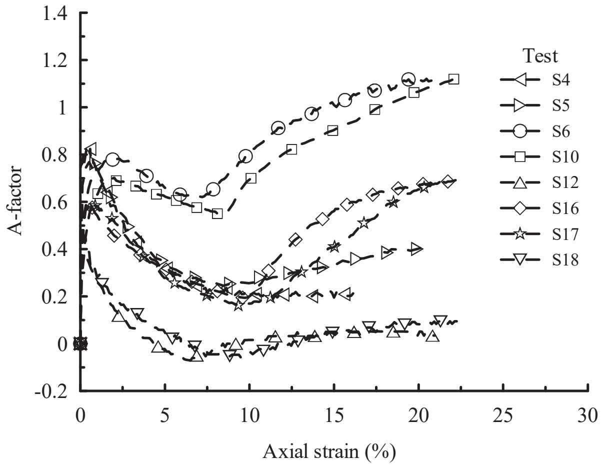


(b)

Fig. 5 q versus ε_a for: (a) Without smear zone; and (b) With smear zone



(a)



(b)

Fig. 6 Variation of A with ε_a for: (b) Without smear zone; and (c) With smear zone

Substituting (5) into (4) and integrating it leads to:

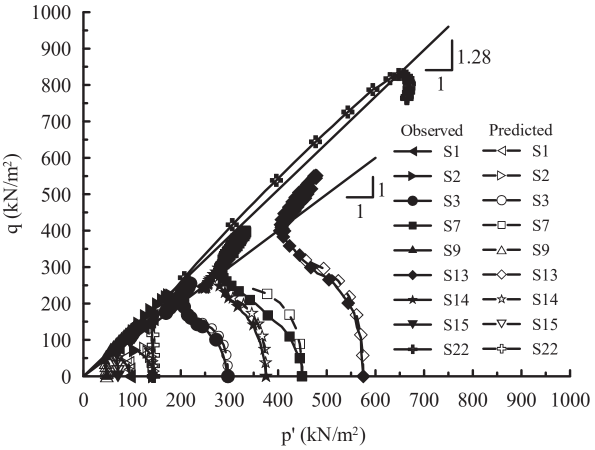
$$p_0'^2 - p'^2 - mq^2 = 0 \tag{6}$$

The effective stress paths inferred by curve fitting the data points to (6) are shown in Figs. 7 (a), (b) along with the observed data. As can be seen, the observed effective stress paths are reasonably well predicted using the fitted parameter. Fig. 8 shows variation of the fitted parameter, m in composite samples with OCR. A trend of decreasing m with the increase in OCR is quite evident, which was to be expected. From Fig. 8, it is seen that m further reduced when the samples had the smear zone around the SCP. In other words, the measured shear induced pore pressures were less in the case of samples prepared using the smear zone. It appears that the smear zone did not permit the excess pore water in the clay to dissipate quickly towards the SCP as the result the pore pressure transducer located below the sand column recorded less pore pressure changes. This is also obvious from Figs. 9 (a) and (b) (selected specimens), which show water content at different locations measured after the completion of the tests. The figure shows that water content was not uniform throughout the sample. It is interesting to note that the water content was higher in the samples with the smear zone which supports the supposition that the smear zone does not permit the complete dissipation of the pore pressure.

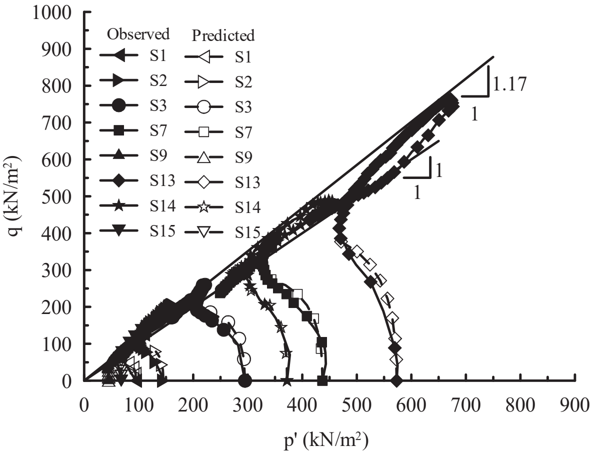
This is also evident from Scanning Electron Microscope (SEM) images (Figs. 10 (a)-(d)) taken on post compression tests of specimens with and without smear.

7.5mm x 7.5mm x 7.5mm air dried samples were prepared at room temperature for SEM images. The images of samples with and without the smear zone show differences in the microstructure. The clay minerals in the smear zone appear to be closely packed with reduced pore space. As such, permeability of the composite samples with the smear zone is reduced with reduced pore space in this zone, and the pore water developed was not uniform throughout the sample [20].

The results seem to suggest that radial drainage can give rise to significant non-uniformities during consolidation of soil specimens. Similar results have also been reported by [21], [22]. The properties within the smear zone are also shown to vary with the overburden pressure [23]. However, the focus of this study is to note the difference in the soil behavior when a well-defined smear zone is formed surrounding the sand column.



(a)



(b)

Fig. 7 Effective stress path: (a) Samples without smear zone; and (b) Samples with smear zone

Fig. 11 shows the variation of $\frac{s_u}{p_0'}$ in composite samples. Although, the decrease in $\frac{s_u}{p_0'}$ with the increase in OCR is apparent, there still remains substantial scatter amongst the data points. Much of this scatter can be reduced by normalising $\frac{s_u}{p_0'}$ to the ratio of the diameters of the SCP and the sample, $\frac{d}{D}$. These results are shown in Fig. 12. Comparison of Fig. 12 with Fig. 11 shows much tighter banding of the results on composite samples. Equation of the curve fit to the data without the smear zone is of the form:

$$\frac{s_u}{p_0'} = 1.25\frac{d}{D}[OCR]^{-0.35} \tag{7}$$

A curve fit to the data with the smear zone can be expressed in the form:

$$\frac{s_u}{p_0'} = \frac{d}{D}[OCR]^{-0.35} \tag{8}$$

It seems clear that the presence of smear zone has reduced the ultimate undrained shear strength by 25%.

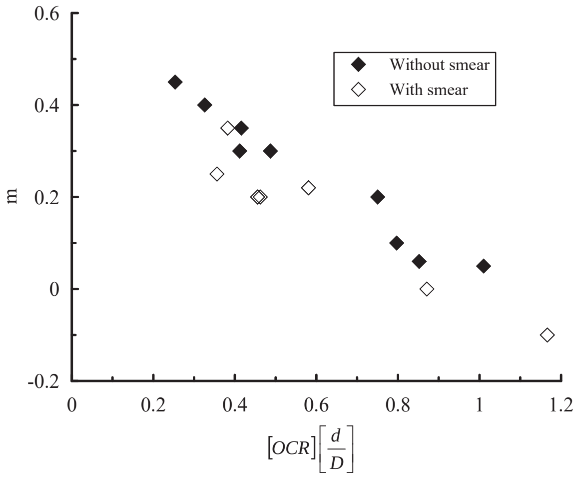
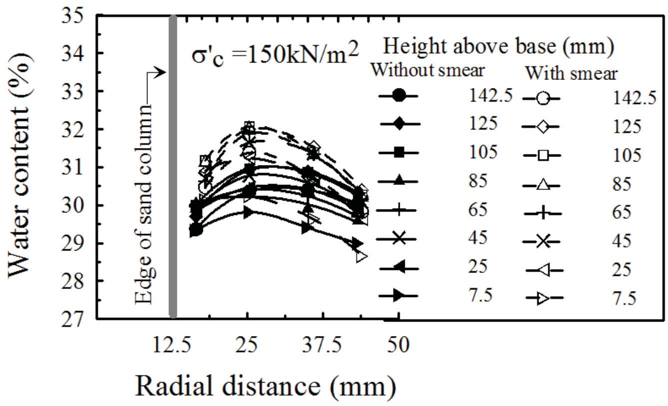
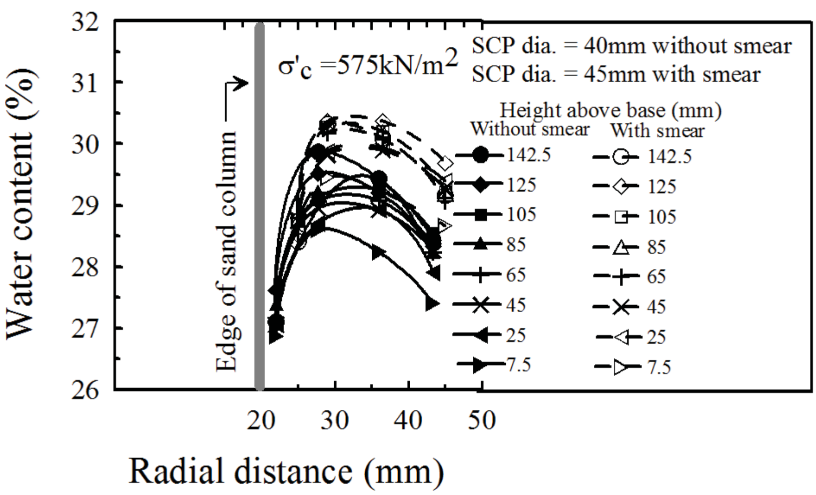


Fig. 8 Variation of m with OCR for Composite samples

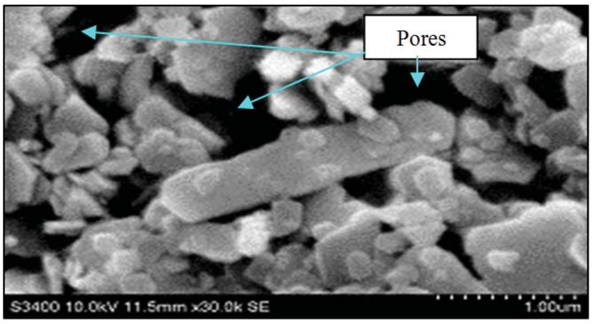


(a) Mean eff. stress -150kPa

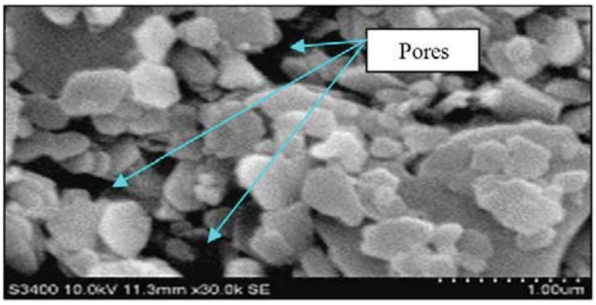


(b) Mean eff. stress -575kPa

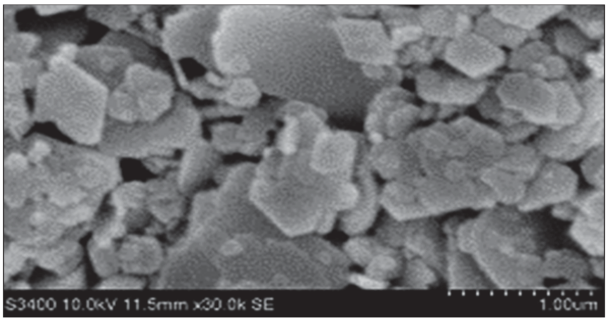
Fig. 9 Water content at different locations measured after the completion of the tests for composite samples



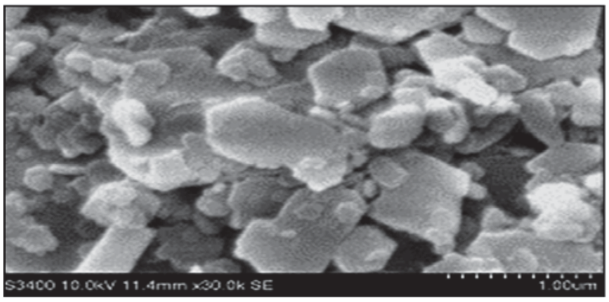
(a) Mean eff. stress -50kPa



(b) Mean eff. stress -450kPa



(c). Mean eff. Stress-50kPa



(d). Mean eff. Stress-450kPa

Fig. 10. SEM images: (a-b) Composite samples without smear zone, (c-d) Composite samples with smear zone

IV. Conclusions

The study presents data on pore pressure measurement and the ultimate undrained shear strength of composite sample with sand column tested to failure. The effect of smear has been studied using 17 consolidated undrained triaxial tests on composite samples. It is concluded that SCP method is a viable method of improvement of soft ground. However, the presence of smear zone around SCP reduces pore water pressure dissipation and hence reduced permeability by about 20%, which was manifested by the SEM images of closely packed clay crystals with reduced pore space in this zone. SEM images taken on post shear tests indicate a distinct division between smaller voids interpreted as constituting the intra-aggregate pore spaces and the larger inter-aggregate voids under no smear condition whereas specimen with smear appears more homogeneous and closely packed particles. The intra-aggregate voids are typically between 0.1 µm and 2µm

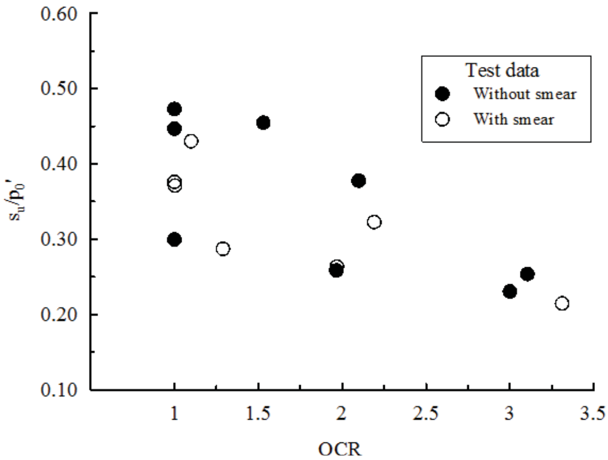


Fig. 11 Variation of shear strength of composite samples with OCR

The stress-strain behavior of the clay was also influenced by the presence of smear zone and undrained shear strength is reduced by about 25%. However, with increase in area replacement ratio and SCP size, the effect of smear is not much prominent. Also, in present study, a hollow open ended casing was used to remove the soil and form a cavity for installing sand columns, which may not yield realistic pore pressures in the clay during installation. Therefore, the effective stress under uniform cell pressure is not uniform throughout the specimen. It was also difficult to predict the relationship between the stress concentration factor and the area replacement ratio because of the wide scatter in the data. One reason for this variation could be because of the variation observed in the water content measured across the samples. These changes affect the effective horizontal stress in the clay and hence the load carried by the individual sand columns.

Furthermore, it is seen that amongst the various techniques for improving in-situ ground conditions, columnar inclusions such as sand compaction pile (SCP) is one of the most effective ground improvement techniques as well as good drainage systems.

V. Future Scope of Work

Since the effective stress manifested due to the all-round uniform cell pressure is not uniform throughout the specimen.

The non-uniformity of consolidated ground and its consequence on subsequent construction of structures needs further study. In order to simulate the better field conditions, installation of driving-in of closed ended casing may be used.

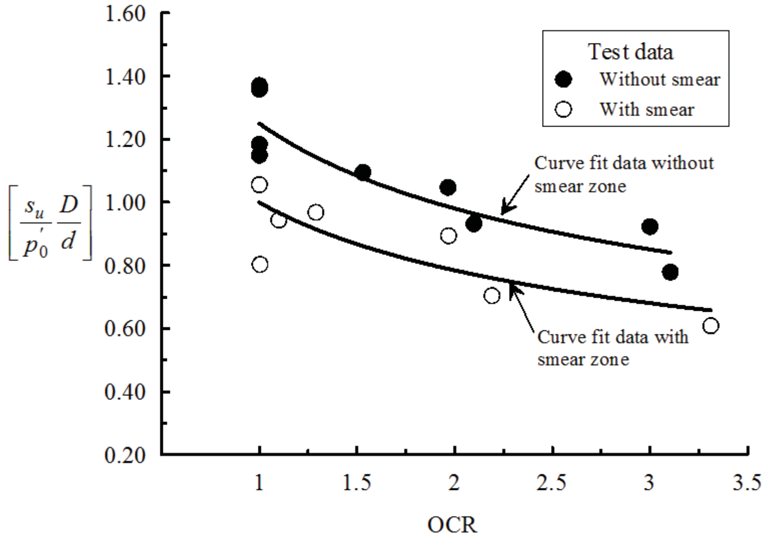


Fig. 12 Relationship between $\left[\frac{s_u}{p_0'}\frac{D}{d}\right]$ and OCR

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