CHAPTER 6
EFFECT OF PRELOADING ON SHEAR STRENGTH BEHAVIOUR
OF KUTTANAD CLAY

6.1 Introduction

Preloading which is the temporary application of a load at a construction site is a popular soil improvement technique for soft soils (Leong et al. 2000). As the excess pore water pressure generated under the preload dissipate, the effective stresses in the soil increase and an increase in shear strength is achieved. During this process, the water content, void ratio and coefficient of permeability decrease while the undrained shear strength, modulus of compressibility and penetration resistance increase. The increase in shear strength due to water content reduction in each stage of loading can be determined by performing shear tests. Determination of shear strength is important not only for obtaining the bearing capacity after removal of preload but also for deciding the magnitude of preload to be applied in subsequent stages of loading. The improvement in shear strength will depend greatly on the shear strength characteristics of the particular soil. Hence one of the main objectives of the present study is to characterise the shear strength behaviour of Kuttanad clay.

Natural water content for most of the Kuttanad soils tested is found to be very high and is only slightly less than the corresponding liquid limit values. It is nearly impossible to do triaxial test on undisturbed samples due to difficulty in preparation of test specimens at their natural water contents. The degree of improvement can be assessed based only on the shear strength of soil at its natural water content i.e., prior to application of any ground improvement technique.
Considering all these difficulties, it was intended to propose a simplified procedure for determination of shear strength at any stage during preloading.

Experimental studies were conducted on preloaded samples to determine the improvement in shear strength due to preloading. Fall cone test which was originally developed as a method for estimating the strength of remoulded cohesive soils in Scandinavia (Koumoto 2001) was used for the determination of shear strength. This method of testing could compete in simplicity and cost with any other shear test as it is easier to perform and the results are not so dependent on the design of the apparatus and the manner in which it is used. The undrained shear strength ($c_u$) of soil specimens is determined using the following equation proposed by Hansbo (1957).

$$c_u = k \frac{W}{d^2} \tag{6.1}$$

where $k$ is a constant, $W$ is the weight of the cone and $d$ is the depth of penetration. The value of $k$ depends mainly on the apex angle of the cone (Kumapley and Boakye 1980 cited by Harison 1988). The value of the constant $k$ in Eqn.(6.1) adopted by various workers lies in the range 0.8-1.2 for 30° cones, with 1 being the commonly used value (Wood 1982, Whyte 1982). Hence, in the present investigation, wherein a 30° cone was used, the value of $k$ was adopted as unity for calculation of undrained shear strength. Fall cone tests were done on undisturbed Kuttanad clay samples preloaded under different pressures for assessing the improvement in shear strength quantitatively by a single observation of depth of penetration of the cone.

Preloading is usually done in stages. It is a pre-requisite in the design of any preloading system to have knowledge of the undrained shear strength of subsoil under a given preload before applying subsequent increments of loading. The ratio of the undrained shear strength to the corresponding effective stress is, thus, an important parameter in the design of preloading.
system. The actual value of the undrained shear strength can be determined using a shear test, which may be time consuming. On the other hand, if some correlations can be developed to predict the ratio of the undrained shear strength to the corresponding effective stress based on easily determinable soil parameters, it will be of great use in view of the time and cost involved in laborious experimental investigations. Hence, correlations were tried to predict the above mentioned ratio as a function of index properties of the samples.

During preloading, improvement in shear strength is always accompanied by reduction in water content. The relationship between shear strength and corresponding water content is thoroughly examined to arrive at useful conclusions applicable to practical problems.

In brief, it is attempted in this Chapter to examine the following for undisturbed Kuttanad clay:

- The relationship between the undrained shear strength and effective preload pressure.
- The relationship between the undrained shear strength and corresponding water content.
- The possibility of generating correlations to predict the undrained shear strength under different preload pressures.
- The possibility of developing simplified procedures for assessing the degree of improvement at any stage of preloading.

6.2 Experimental study

In order to determine the improvement in shear strength due to preloading, samples were first subjected to preloading pressures in standard oedometer and fall cone tests were done
Tests were carried out on undisturbed samples of Kuttanad clay collected from the different locations as mentioned in section 3.2 of Chapter 3. The details of procedure adopted for preloading of undisturbed samples and for the conduct of shear tests on preloaded samples are presented in the subsequent sections.

6.2.1 Preloading of undisturbed Kuttanad clay samples

Samples of Kuttanad clay were preloaded in standard fixed-ring consolidometers using brass rings, 60 mm diameter and 20 mm high. The inside of the rings was lubricated with silicon grease to minimize side friction between the ring and the soil specimen. Representative samples for testing were carefully extruded from the sampling tubes, care being taken to ensure that the thickness of the extruded discs of soil is somewhat greater than the height of the consolidation ring. The consolidation ring was gradually inserted into the soil disc by pressing with hands and carefully removing the material and the ring (Photograph 6.1 through 6.5). The soil sample thus obtained was trimmed flush with the top and bottom edges of the ring with special care. The ring and the specimen were placed centrally on the bottom porous stone and upper porous stone, and then the loading cap was placed on top. Both the porous stones used were in damped condition to avoid absorption of water from the sample. Filter papers were positioned on the top and bottom of the soil specimen to prevent finer particles from being forced into the pores of the porous stones. The consolidation cell was mounted and positioned on loading frame and a vertical deformation dial gauge capable of reading to an accuracy of at least 0.01 percent of specimen height was properly fixed in position. The cell was inundated with distilled water and a seating pressure of 6.25 kPa was applied. After reaching equilibrium, conventional oedometer tests were performed on all the undisturbed soil samples. A load
increment ratio of one was adopted and each load increment maintained until near equilibrium was attained. For determining improvement in shear strength due to preloading of different intensities, identical specimens were loaded to five different preload pressures (25, 50, 100, 200 and 400kPa).

6.2.2 Shear strength measurement using fall cone penetrometer

After each specimen had undergone primary consolidation under the applied preload pressure, the load was released and the dial gauge readings were noted until the difference between consecutive readings became less than 0.002mm/hr. The consolidated sample in the ring was carefully taken out of the loading frame and the excess water on top of the sample was wiped off. Extreme care was taken to remove filter paper without disturbing the sample in the ring. Finally, the soil specimen together with a glass plate placed at the bottom was transferred to the base stand below the fall-cone for the penetration test (Photographs 6.6 through 6.8). To measure penetration depths with reasonable degree of accuracy, a 30° cone of total weight 1.30N was used for all the tests. The cone was made to touch the top horizontal surface of consolidated specimen in the ring and subsequently allowed to fall under its own weight and the penetration depth after 5 seconds was noted. For each specimen in the ring, fall-cone tests were carried out at centre and at four positions at equal radial distance of 1.5cm and the average value is reported as penetration depth (d). The undrained shear strength corresponding to each depth of penetration was computed using Eqn. (6.1). After each fall-cone test, the corresponding water content of the soil specimen was also measured.
6.3 Results and Discussion

6.3.1 General

The improvement in shear strength due to preloading was observed from the results of fall cone tests made on undisturbed samples of Kuttanad clay preloaded in standard oedometer ring under different preload pressures. The undrained shear strength \( (c_{ur}) \) of the soil specimens after removal of preload from the oedometer, were calculated using Eqn (6.1). As the magnitude of preload pressure increases, water content and depth of penetration decreases resulting in increased value of undrained shear strength. The undrained shear strength values after the removal of pressure were plotted against corresponding applied preload pressure. Figs 6.1(a) through (r) show the variation of the undrained shear strengths with the applied preload pressures for all the eighteen samples used in the present study. It is seen that the undrained shear strength bears a linear relationship with the applied preload pressure. The shear strength improvement under different magnitudes of preload pressure can therefore be expressed as the ratio of the undrained shear strength of the soil specimens after removal of preload to the corresponding preload pressure \( (c_{ur}/p) \). The value of \( c_{ur}/p \) after removal of preload pressure is found to vary from 0.137 to 0.251 for the eighteen samples tested (Table 6.1). This ratio is an important parameter in designing the magnitude of preload pressure to be applied for achieving a specific improvement in shear strength. The possibility of predicting this ratio based on easily determinable soil properties is examined in the subsequent section.

6.3.2 Correlation of ratio of undrained shear strength after removal of preload to preload pressure \( (c_{ur}/p) \) with index properties

The relationship between various index properties and initial state parameters with \( c_{ur}/p \) were examined as part of the present study. It was seen that no correlation exists between \( c_{ur}/p \)
and any of the index properties/ initial state parameters. Fig.6.2 presents the values of $c_{ur}/p$ plotted against corresponding plasticity indices. It is seen that no definite relationship exists between the observed values of $c_{ur}/p$ and the plasticity index. Non-existence of any correlations may, perhaps, be due to the absence of vertical effective stress on the sample at the time of shear strength determination. The improvement in shear strength on account of preloading is mainly due to reduction in void ratio/water content. Therefore, it is worth examining the relationship between undrained shear strength and corresponding water content. This is explored in the next section.

6.3.3 Relationship between undrained shear strength and water content

With increase in magnitude of preload pressures, water content decreases, undrained shear strength increases, and undrained shear strength versus water content relationship is observed to be non-linear (Fig 6.3(a) through (r)). It is seen that the relationship could be expressed in the following form with acceptable values of correlation coefficients:

$$c_{ur} = M \cdot w^{-N}$$  \hspace{1cm} (6.2)

Table (6.2) presents $M$ and $N$ values obtained for all the eighteen samples tested along with the corresponding correlation coefficients. The shape of the curve resembles a rectangular hyperbola for all the soils tested. Since the relationship is that of a rectangular hyperbola, it can be replotted in terms of $c_{ur}$ versus $c_{ur}/w$ such that a linear relationship is obtained. Fig.6.4 (a) through (r) present such relationship for all samples of Kuttanad clay and it is seen that treating the $c_{ur}$ versus $c_{ur}/w$ relationship as linear is quite reasonable. This suggests that the undrained shear strength of the undisturbed highly plastic Kuttanad clay could be represented by the intercept ‘$m$’ and slope ‘$n$’ (of the linear relationship), for any value of water content, in the following form:
\[ c_{ur} = \frac{mw}{w-n} \quad (6.3) \]

The value of ‘m’ varies from 2.517 to 8.343 whereas ‘n’ varies from 24.335 to 81.183 for the samples tested in the present study (Table 6.3). It was found from a careful comparison of Tables (3.1) and (6.3) that though the parameters ‘m’ and ‘n’ generally show an increase with increasing plasticity characteristics, they do not bear any relationship with any of the index properties.

The fall cone test for shear strength determination can be conducted only after the removal of applied pressures. But knowledge of the shear strength of soil at any stage of loading, (i.e., prior to removal of the load), will be of great use in deciding the magnitude of loading in the subsequent stage. The nature of relationship between the undrained shear strength and the water contents for the soils tested suggests that the undrained shear strength corresponding to any water content can be easily determined using the constants ‘m’ and ‘n’ for a given Kuttanad clay (Eqn. 6.3). Thus it is possible to determine the undrained shear strength at water contents corresponding to the equilibrium void ratio at different preload pressures. This is attempted in the subsequent section.

6.3.4 Determination of undrained shear strength under preload pressure

The unloading portion of the e-log p relationship for all the samples of undisturbed Kuttanad clay used in this study (Figs.4.1 (a) through (i) of Chapter 4) indicates that the difference in void ratio and hence the water content before and after removal of applied pressure is only marginal. The undrained shear strength under preload pressure (i.e., prior to removal of preload) for each of the samples was found out from the established relationship between shear strength and the water content for that soil. The undrained shear strength thus computed were plotted against corresponding preloading pressure (Fig.6.5(a) through (r)) and it was seen that a
linear relationship with satisfactory value of correlation coefficient exist only up to preloading pressure of 200kPa and thereafter the curve becomes non-linear. This may be due to the fact that at lower water contents (corresponding to preload pressure of 400kPa), the curve $c_{uf}$ versus water content asymptotes the shear strength axis and shows an abrupt increase in the shear strength even for small changes in water content. Therefore the undrained shear strength predicted using water content in this lower range (i.e., at preload pressures higher than 200kPa) may lead to overestimation of undrained shear strength. This possibly implies that the ratio ($c_{up}/p$) can be assumed as constant for each of the soil samples for preload pressures less than or equal to 200kPa. Hence in the present study, the prediction of shear strength from the relationship between undrained shear strength and water content is done only up to preload pressures of 200kPa. The values of $c_{up}/p$ ratio for all soil samples tested are also shown in Table 6.1.

The improvement in shear strength can be easily estimated if the ratio ($c_{up}/p$) could be predicted based on some easily determinable index properties or initial state parameters. It is reported that the ratio between the undrained shear strength and effective overburden pressure is a characteristic property of clay and is dependent on the plasticity of the clay (Skempton, 1948) and if $c_{u}/p$ ratio for different clays is plotted against the corresponding plasticity index values, the points will fall on a single curve. The following empirical relation was also proposed (Skempton, 1957) for predicting the shear strength of normally consolidated clays.

$$\frac{c_u}{\sigma_v} = 0.11 + 0.37 I_p$$ \hspace{1cm} (6.4)

where $c_u$ is the undrained shear strength, $\sigma_v$ is the effective overburden pressure and $I_p$ is the plasticity index represented as a decimal. The above correlation has been developed based on values obtained from field vane shear tests. However, Bjerrum (1974) reported that as plasticity
of soils increases, \( c_u \) obtained from vane shear tests may give results that are unsafe for foundation design and suggested the following correction for \( c_u \).

\[
\begin{align*}
\text{Cu}_{\text{design}} &= \lambda \cdot c_u \text{(vane shear)} \\
\text{where } &\lambda = 1.7 - 0.54 \log I_p
\end{align*}
\]

(6.5) 

(6.6)

The correction factor \( \lambda \) decreases with increase in plasticity index and the use of Skempton's equation will lead to overestimation of the undrained shear strength especially for highly plastic clays. The ratio \( (c_{up}/p) \) calculated in the present study were plotted against plasticity index (Fig.6.6). No correlation is observed to exist between \( c_{up}/p \) and plasticity index for the highly plastic Kuttanad clay studied in the present investigation. The original equation proposed by Skempton (1957) and the corrected equation modified by Bjerrum (1974) are also shown in Fig.6.6 for the sake of comparison. It is seen that most of the values obtained in this study are relatively closer to Bjerrum's line. This may be due to relatively high values of plasticity index of the soils tested. Based on triaxial test results on 10 different soils with plasticity index ranging from 16% to 104 %, (which included two samples of Kuttanad clay), Narain and Ramanathan (1970) have reported that correlations of the type proposed by Skempton (1957) and Bjerrum (1974) cannot be considered as valid for clays where changes in structure are predominant. It is also reported that the highly plastic Kuttanad clay has significant difference in soil structure though the exact causes for the change are not known. The data reported by Narain and Ramanathan (1970) for the two Kuttanad clay samples are also shown in Fig.6.6. From these results and those obtained in the present study, it appears that Skempton’s/ Bjerrum’s correlation is not applicable to the highly plastic Kuttanad clay studied in the present investigation. There have been several studies (Bjerrum, 1954; Leonards, 1962; Bishop and Henkel, 1962) in the past to examine the validity of Skempton’s relation. Although a large number of data agrees with this
relation, an equally large volume of data contradicts it (Nagaraj et al., 1994). Most of the above examinations have been only experimental. The present study suggests that even though no relationship exist between the $c_{up}/p$ ratio and the corresponding plasticity index, it may be possible to predict the undrained shear strength of the preloaded highly plastic clay by knowing the water content as described in section 6.3.3.

6.3.5 An alternative approach for shear strength prediction

In soil engineering, it is common that generalization of a particular behaviour of similar soils can be achieved through normalization by a standard or reference state common to all materials. The water content at liquid limit can be regarded as one such parameter since the shearing resistance at liquid limit of soils with wide variation in liquid limit fall within a narrow range. In the present study, when the shear strength versus water content relationship for all the highly plastic soils tested were plotted together, it was found that the order of the curves bears some association with the liquid limit water contents (Fig.6.7). The unique relationship between the undrained shear strength and water content for the eighteen soils when shown in a single figure, the curves exhibit significant overlapping and the points become clustered. Hence only five curves are shown in Fig.6.7. As the value of liquid limit increases, the curve shifts its position from left to right (along the water content axis) as could be seen from the figure. The minimum value of liquid limit is for soil designation S5 and the maximum value is for soil designation S14. All other curves are positioned in between these two curves. An attempt was, therefore, made to develop a predictive model for undrained shear strength of all the preloaded Kuttanad clays as a function of the water content after preloading as well as the liquid limit of the soil. It is logical to believe that as the water content decreases from the reference state (liquid
limit), some unique pattern of shear strength increase may exist for any particular soil type. A study in this direction resulted in establishing a relationship as presented in Fig.6.8. It is seen that $c_{up}$ versus $c_{up}/(w/w_L)$ relationship can be approximated by the following equation ($c_{up}$ obtained in kPa) with satisfactory value of correlation coefficient ($r = 0.993$) acceptable in engineering applications.

$$c_{up} = 1.668(w_L/w)^{4.3022}$$  \hspace{1cm} (6.7)

The use of above equation eliminates the need of establishing any relationship between undrained shear strength and water content of the preloaded samples through laboratory oedometer and fall cone tests. It can be inferred from the above equation that the undrained shear strength of Kuttanad clay at liquid limit water content is 1.668kPa. This is in good agreement with the value 1.7kPa reported by Wroth and Wood (1978).

6.3.6 Relationship between depth of penetration and water content.

The undrained shear strength was determined using Eqn .6.1 by conducting fall cone penetration test and by observing the corresponding depth of penetration. The relationship between depth of penetration ($d$) and water content ($w$) was also explored in detail as part of present study (Figs.6.9 (a) through(r)). It is found that a linear relationship with very good correlation coefficient exist between water content and logarithm of depth of penetration which can be expressed in the following form:

$$w = P \ln (d) + Q$$  \hspace{1cm} (6.8)

where $P$ and $Q$ are respectively the slope and intercept of the linear relationship. This unique relationship can possibly be utilized for a quick estimation of water content in the field. The procedure to be adopted is briefly described as given below. At least three identical samples of
soil collected from a particular location are required for establishing the relationship between depth of penetration and corresponding water content. Preload pressures are to be applied to each of the sample and the samples are allowed to consolidate under the applied pressures in standard oedometer ring. The three identical samples may be subjected to three different preload pressures (say 50, 100 and 200kPa). Dial gauge readings are to be observed at hourly intervals and the load may be released when the difference in consecutive readings becomes less than 0.002mm/hr. After the loads are released, fall cone penetration test may be conducted on each of the preloaded samples. The depth of penetration and corresponding water content are observed. Variation of water content (in natural scale) is plotted against respective depth of penetration (in logarithmic scale). A linear relationship with very good correlation coefficient is expected to be obtained. Once such a relationship is established, the same can be used for field application. The water content of the sample at any instant during preloading can be obtained from the established relationship by noting the depth of penetration (using a cone of same weight and apex angle as the one which is used for establishing the relationship) at that instant.

6.4 Concluding remarks

Preloading technique is known to increase the undrained shear strength of weak subsoils. But when this technique is adopted, it is important to assess the degree of improvement in shear strength in order to decide the pattern of load application. Based on experimental investigation on eighteen samples of a highly plastic, Kuttanad clay and subsequent analysis, the following conclusions are arrived at.

Preloading is an effective technique of improving the shear strength of Kuttanad clay. The undrained shear strength before and after removal of preload in consolidation tests can be
accurately determined using fall cone penetration test. The fall cone method proves to be a simple, reliable and quick method of shear strength determination, which is also less operator sensitive, compared to other shear tests.

The undrained shear strength increases drastically with the reduction in water content brought about by preloading. The relationship between undrained shear strength and water content of Kuttanad clay is in the form of a rectangular hyperbola. The relationship could be expressed in the following form with acceptable values of correlation coefficients:

\[ c_{uf} = M w^N \]  
(Eqn. 6.2)

This suggests that the undrained shear strength of the undisturbed highly plastic Kuttanad clay could be represented by the intercept ‘m’ and slope ‘n’ (of the linear relationship between undrained shear strength and (undrained shear strength to water content) ratio), for any value of water content, in the following form:

\[ c_{uf} = mw/(w-n) \]  
(Eqn.6.3)

This method of representing the undrained shear strength enables quick and accurate estimation of the same at any stage of preloading by measuring only the water content at that stage.

The relationship between undrained shear strength after removal of preload pressure \( (c_{uf}) \) and the corresponding water content is used to predict the undrained shear strength before removing the preload pressure \( (c_{up}) \). The void ratio and hence the water content prior to removal of the preload pressure was obtained from the e-log p relationship of the soil. It was found that the ratio of undrained shear strength \( (c_{up}) \) to applied pressure \( (p) \) can be taken as a constant in range of pressures less than or equal to 200kPa. Further increase of pressure causes significant increase in the undrained shear strength and the variation of undrained shear strength with
applied pressure is seen to be non-linear at higher pressures. Even in the range of pressures where the $c_{up}/p$ ratio can be assumed as constant, it is found that the values do not obey either Skempton(1957) or Bjerrum (1974) correlation with plasticity index. However, the method of prediction of undrained shear strength at any stage of loading suggested by the present study requires determination of water content at that stage only.

A possible alternative approach for the prediction of undrained shear strength of preloaded Kuttanad clay in terms of water content after preloading and liquid limit of the clay has also been arrived at. It is seen that $c_{up}$ versus $c_{up}' (w/w_L)$ relationship can be approximated by the following equation ($c_{up}$ obtained in kPa) with satisfactory value of correlation coefficient:

$$c_{up} = 1.668 \left(\frac{w}{w_L}\right)^{4.3022}$$  \hspace{1cm} (Eqn.6.7)

It is found that a linear relationship with very good correlation coefficient exist between water content and logarithm of depth of penetration which can be expressed in the following form:

$$w = P \ln (d) + Q$$  \hspace{1cm} (Eqn. 6.8)

In brief, the main contributions of this Chapter are the development of:

- The relationship between the undrained shear strength and effective preload pressure.
- Unique relationship between the undrained shear strength and corresponding water content.
- Correlations to predict the undrained shear strength prior to removal of preload pressures.
• Simplified procedures which do not require laborious and costly field monitoring equipments and techniques, for assessing the degree of improvement at any stage of preloading.

• Quick and accurate method to determine the in situ water content using cone penetrometer.

6.5 Recommendations

The results of the present study can be applied to field problems as discussed below.

Once undisturbed samples are collected from the site where preloading is planned, one dimensional consolidation tests are to be carried out on identical samples under three different pressures. This is to be followed by the conduct of fall cone test and determination of water content of all the samples. With these results, the $c_{uf}$ versus $c_{uf}/w$ relationship for the field soil can be established. Once the slope ‘n’ and the intercept ‘m’ for that particular soil are obtained from the linear relationship between $c_{uf}$ and $c_{uf}/w$, the undrained shear strength corresponding to any water content can be calculated using Eqn (6.3). Degree of shear strength improvement at any stage of preloading can, now, be assessed easily by observing the water content alone. The depth of penetration corresponding to the three water contents can be plotted on a semi logarithmic plot with the water content on natural scale and the penetration depth on logarithmic scale. From the linear relationship thus established, the water content at any instant can be rapidly determined by taking a sample and observing the depth of penetration of the same cone which is used for developing the correlation. The proposed method thus provides a simple and quick measure of the water content and the undrained shear strength during preloading. The field instrumentation as well as shear tests during preloading can be eliminated by adopting the
proposed method. The shear strength determination is further simplified if the liquid limit of the soil is known. One can use Eqn (6.7) to evaluate the undrained shear strength at any water content.

![Graph showing variation of undrained shear strength](image)

**Fig.6.1 (a)**

**Fig.6.1 (b)**

**Fig.6.1** Variation of undrained shear strength after removal of preload ($c_{ufr}$) with preload pressure
Fig. 6.1 Variation of undrained shear strength after removal of preload ($c_{uf}$) with preload pressure.
Fig. 6.1 Variation of undrained shear strength after removal of preload ($c_{uf}$) with preload pressure
Fig. 6.1 (k) Variation of undrained shear strength after removal of preload ($c_{uf}$) with preload pressure.
Fig. 6.1 Variation of undrained shear strength after removal of preload ($c_{ur}$) with preload pressure
Fig. 6.2 Variation of $c_{ur}/p$ with plasticity index

Fig. 6.3 (a) Variation of undrained shear strength after removal of preload ($c_{ur}$) with water content

Location: Mithrakary
Nature of sample: UD
Depth: 2m
Sample designation: S 1

$c_{ur} = 3E+16w^{-7.7333}$
$r = 0.946$

Fig. 6.3 (b) Variation of undrained shear strength after removal of preload ($c_{ur}$) with water content

Location: Mithrakary
Nature of sample: UD
Depth: 3m
Sample designation: S 2

$c_{ur} = 1E+16w^{-7.5901}$
$r = 0.949$
*Fig. 6.3* Variation of undrained shear strength after removal of preload ($c_{uf}$) with water content.
Fig. 6.3 Variation of undrained shear strength after removal of preload ($c_{uf}$) with water content.
Fig. 6.3

Variation of undrained shear strength after removal of preload ($c_{uf}$) with water content

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Location: Paruthikalam
Nature of sample: UD
Depth: 2m
Sample designation: S 11

\[ c_{uf} = 1 \times 10^w \cdot 4.5312 \]
\[ r = 0.983 \]

Fig. 6.3 (k)

Location: Paruthikalam
Nature of sample: UD
Depth: 3m
Sample designation: S 12

\[ c_{uf} = 1 \times 10^w \cdot 4.0196 \]
\[ r = 0.994 \]

Fig. 6.3 (l)

Location: Pallathuruthi
Nature of sample: UD
Depth: 2m
Sample designation: S 13

\[ c_{uf} = 2 \times 10^w \cdot 3.8429 \]
\[ r = 0.990 \]

Fig. 6.3 (m)

Location: Pallathuruthi
Nature of sample: UD
Depth: 3m
Sample designation: S 14

\[ c_{uf} = 2 \times 10^w \cdot 3.3514 \]
\[ r = 0.987 \]

Fig. 6.3 (n)
Fig. 6.3 Variation of undrained shear strength after removal of preload ($c_{uf}$) with water content.
Fig. 6.4 Variation of $c_{uf}$ with $c_{uf}/w$
Location: Chenankary  
Nature of sample: UD  
Depth: 2m  
Sample designation: S 5

\[ c_{uf} = 24.335 \frac{c_{uf}}{W} + 6.6616 \]
\[ r = 0.997 \]

Fig.6.4 (e)

Location: Chenankary  
Nature of sample: UD  
Depth: 3m  
Sample designation: S 6

\[ c_{uf} = 51.935 \frac{c_{uf}}{W} + 2.517 \]
\[ r = 0.999 \]

Fig.6.4 (f)

Location: Mithrakary  
Nature of sample: UD  
Depth: 2m  
Sample designation: S 7

\[ c_{uf} = 81.272 \frac{c_{uf}}{W} + 4.4075 \]
\[ r = 0.996 \]

Fig.6.4 (g)

Location: Mithrakary  
Nature of sample: UD  
Depth: 2m  
Sample designation: S 8

\[ c_{uf} = 68.208 \frac{c_{uf}}{W} + 2.7469 \]
\[ r = 0.994 \]

Fig.6.4 (h)

Fig.6.4 Variation of \( c_{uf} \) with \( \frac{c_{uf}}{W} \)
Fig. 6.4 Variation of $c_{uf}$ with $c_{uf}/w$
120 Location: Pallathuruthi
Nature of sample: UD
Depth: 2m
Sample designation: S 13

\[ c_{uf} = 71.977c_{uf/w} + 7.3198 \\
\[ r = 0.995 \]

**Fig.6.4 (m)**

120 Location: Pallathuruthi
Nature of sample: UD
Depth: 3m
Sample designation: S 14

\[ c_{uf} = 66.661c_{uf/w} + 8.3431 \\
\[ r = 0.993 \]

**Fig.6.4 (n)**

120 Location: Champakkulam
Nature of sample: UD
Depth: 2m
Sample designation: S 15

\[ c_{uf} = 62.895c_{uf/w} + 4.2111 \\
\[ r = 0.997 \]

**Fig.6.4 (o)**

120 Location: Champakkulam
Nature of sample: UD
Depth: 3m
Sample designation: S 16

\[ c_{uf} = 43.883c_{uf/w} + 4.1734 \\
\[ r = 0.997 \]

**Fig.6.4 (p)**

**Fig.6.4 Variation of c_{uf} with c_{uf/w}**
Location: Thakazhi  
Nature of sample: UD  
Depth: 2m  
Sample designation: S 17

\[
c_{uf} = 57.614c_{uf}/w + 3.0083 \\
r = 0.999
\]

Fig. 6.4 (q)

Location: Thakazhi  
Nature of sample: UD  
Depth: 3m  
Sample designation: S 18

\[
c_{uf} = 49.366c_{uf}/w + 2.9112 \\
r = 0.998
\]

Fig. 6.4 (r)

Fig. 6.4 Variation of \( c_{uf} \) with \( c_{uf}/w \)

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Location: Mithrakary  
Nature of sample: UD  
Depth: 2m  
Sample designation: S 1

Fig. 6.5 (a)

Location: Mithrakary  
Nature of sample: UD  
Depth: 3m  
Sample designation: S 2

Fig. 6.5 (b)

Fig. 6.5 Variation of undrained shear strength prior to removal of preload (\( c_{ud} \)) with preload pressure
Fig. 6.5 Variation of undrained shear strength prior to removal of preload ($c_{ur}$) with preload pressure
Fig. 6.5 Variation of undrained shear strength prior to removal of preload ($c_{ur}$) with preload pressure
Fig. 6.5 Variation of undrained shear strength prior to removal of preload ($c_{uu}$) with preload pressure
Fig. 6.5 Variation of undrained shear strength prior to removal of preload ($c_{ur}$) with preload pressure.
Fig. 6.6 Variation of \( \sigma_{up}/p \) with plasticity index

Fig. 6.7 Variation of undrained shear strength after removal of preload (\( \sigma_{ud} \)) with water content for five samples (S3, S4, S5, S10, S14) in the order of their liquid limits
Fig. 6.8 Variation of $c_{up}$ with $c_{up}/(w/w_L)$

$c_{up} = 1.668(w_L/w)^{4.3022}$
$r = 0.993$
$n = 90$
Fig. 6.9 Variation of water content with depth of penetration of cone penetrometer

\[ w = 20.079 \ln(d) + 50.064 \]
\[ r = 0.952 \]

\[ w = 20.275 \ln(d) + 46.197 \]
\[ r = 0.953 \]
Fig. 6.9 (c) and (d) show the variation of water content with depth of penetration of a cone penetrometer for samples at different locations and depths.

Location: Mankombu
Nature of sample: UD
Depth: 2m
Sample designation: S3

Location: Mankombu
Nature of sample: UD
Depth: 3m
Sample designation: S4

The equations for water content, $w$, with respect to depth of penetration, $d$, are:

- For sample S3: $w = 24.754 \ln(d) + 10.436$, with $r = 0.997$.
- For sample S4: $w = 32.745 \ln(d) + 3.2418$, with $r = 0.988$.

Fig. 6.9 Variation of water content with depth of penetration of cone penetrometer.
Fig. 6.9 Variation of water content with depth of penetration of cone penetrometer
Fig. 6.9 (g)  
**Location:** Kainakary  
**Nature of sample:** UD  
**Depth:** 2m  
**Sample designation:** S 7

\[ w = 47.561 \ln(d) + 19.913 \]

\[ r = 0.986 \]

---

Fig. 6.9 (h)  
**Location:** Kainakary  
**Nature of sample:** UD  
**Depth:** 3m  
**Sample designation:** S 8

\[ w = 34.949 \ln(d) + 16.739 \]

\[ r = 0.982 \]

---

**Fig. 6.9** Variation of water content with depth of penetration of cone penetrometer
Fig. 6.9 Variation of water content with depth of penetration of cone penetrometer
Fig. 6.9 (k) and (l)

Location: Paruthikalam
Nature of sample: UD
Depth: 2m
Sample designation: S 11

\[ w = 43.592 \ln(d) - 0.7505 \]
\[ r = 0.997 \]

Location: Paruthikalam
Nature of sample: UD
Depth: 3m
Sample designation: S 12

\[ w = 44.294 \ln(d) - 2.9916 \]
\[ r = 0.992 \]

Fig.6.9 Variation of water content with depth of penetration of cone penetrometer
Location: Pallathuruthi  
Nature of sample: UD  
Depth: 2m  
Sample designation: S 13

$w = 57.354 \ln(d) + 2.9723$  
r = 0.997

Fig. 6.9 (m)

Location: Pallathuruthi  
Nature of sample: UD  
Depth: 3m  
Sample designation: S 14

$w = 64.791 \ln(d) - 11.349$  
r = 0.996

Fig. 6.9 (n)

Fig. 6.9 Variation of water content with depth of penetration of cone penetrometer
Fig. 6.9 (a) and (b) show the variation of water content with depth of penetration of cone penetrometer at two different locations: Champakkulam, with samples S15 and S16.

For Sample S15 (Location: Champakkulam, Nature of sample: UD, Depth: 2m, Sample designation: S15), the water content, $w$, is given by the equation $w = 31.035 \ln(d) + 22.418$ with a correlation coefficient $r = 0.996$.

For Sample S16 (Location: Champakkulam, Nature of sample: UD, Depth: 3m, Sample designation: S16), the water content, $w$, is given by the equation $w = 25.607 \ln(d) + 8.8078$ with a correlation coefficient $r = 0.999$. 

Fig. 6.9 Variation of water content with depth of penetration of cone penetrometer.
Fig. 6.9 Variation of water content with depth of penetration of cone penetrometer

**Fig. 6.9 (q)**

Location: Thakazhi  
Nature of sample: UD  
Depth: 2m  
Sample designation: S 17  

\[ w = 24.707 \ln(d) + 22.37 \]

\[ r = 0.997 \]

**Fig. 6.9 (r)**

Location: Thakazhi  
Nature of sample: UD  
Depth: 3m  
Sample designation: S 18  

\[ w = 23.033 \ln(d) + 15.455 \]

\[ r = 0.998 \]
Table 6.1 $c_{up}/p$ values before and after removal of preload pressures

(Maximum preload pressure = 200kPa).

<table>
<thead>
<tr>
<th>Soil designation</th>
<th>$c_{up}/p$ After removal of preload ($c_{up}/p$)</th>
<th>$c_{up}/p$ Under preload</th>
</tr>
</thead>
<tbody>
<tr>
<td>S 1</td>
<td>0.229</td>
<td>0.243</td>
</tr>
<tr>
<td>S 2</td>
<td>0.167</td>
<td>0.203</td>
</tr>
<tr>
<td>S 3</td>
<td>0.181</td>
<td>0.211</td>
</tr>
<tr>
<td>S 4</td>
<td>0.168</td>
<td>0.223</td>
</tr>
<tr>
<td>S 5</td>
<td>0.159</td>
<td>0.237</td>
</tr>
<tr>
<td>S 6</td>
<td>0.137</td>
<td>0.217</td>
</tr>
<tr>
<td>S 7</td>
<td>0.213</td>
<td>0.287</td>
</tr>
<tr>
<td>S 8</td>
<td>0.143</td>
<td>0.221</td>
</tr>
<tr>
<td>S 9</td>
<td>0.176</td>
<td>0.330</td>
</tr>
<tr>
<td>S 10</td>
<td>0.183</td>
<td>0.302</td>
</tr>
<tr>
<td>S 11</td>
<td>0.144</td>
<td>0.294</td>
</tr>
<tr>
<td>S 12</td>
<td>0.162</td>
<td>0.246</td>
</tr>
<tr>
<td>S 13</td>
<td>0.251</td>
<td>0.405</td>
</tr>
<tr>
<td>S 14</td>
<td>0.251</td>
<td>0.403</td>
</tr>
<tr>
<td>S 15</td>
<td>0.203</td>
<td>0.374</td>
</tr>
<tr>
<td>S 16</td>
<td>0.184</td>
<td>0.313</td>
</tr>
<tr>
<td>S 17</td>
<td>0.162</td>
<td>0.240</td>
</tr>
<tr>
<td>S 18</td>
<td>0.147</td>
<td>0.275</td>
</tr>
</tbody>
</table>
Table 6.2 Parameters in the $c_{mf}$ versus water content relationship

<table>
<thead>
<tr>
<th>Soil designation</th>
<th>M</th>
<th>N</th>
<th>Correlation coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>S 1</td>
<td>$3E+16$</td>
<td>7.733</td>
<td>0.946</td>
</tr>
<tr>
<td>S 2</td>
<td>$1E+16$</td>
<td>7.590</td>
<td>0.949</td>
</tr>
<tr>
<td>S 3</td>
<td>$7E+09$</td>
<td>4.799</td>
<td>0.993</td>
</tr>
<tr>
<td>S 4</td>
<td>$1E+09$</td>
<td>4.214</td>
<td>0.992</td>
</tr>
<tr>
<td>S 5</td>
<td>$3E+05$</td>
<td>2.567</td>
<td>0.998</td>
</tr>
<tr>
<td>S 6</td>
<td>$1E+12$</td>
<td>6.001</td>
<td>0.985</td>
</tr>
<tr>
<td>S 7</td>
<td>$1E+11$</td>
<td>4.735</td>
<td>0.985</td>
</tr>
<tr>
<td>S 8</td>
<td>$1E+12$</td>
<td>5.553</td>
<td>0.974</td>
</tr>
<tr>
<td>S 9</td>
<td>$6E+10$</td>
<td>4.794</td>
<td>0.989</td>
</tr>
<tr>
<td>S 10</td>
<td>$3E+08$</td>
<td>3.606</td>
<td>0.997</td>
</tr>
<tr>
<td>S 11</td>
<td>$1E+10$</td>
<td>4.531</td>
<td>0.983</td>
</tr>
<tr>
<td>S 12</td>
<td>$1E+09$</td>
<td>4.020</td>
<td>0.994</td>
</tr>
<tr>
<td>S 13</td>
<td>$2E+09$</td>
<td>3.843</td>
<td>0.990</td>
</tr>
<tr>
<td>S 14</td>
<td>$2E+08$</td>
<td>3.351</td>
<td>0.987</td>
</tr>
<tr>
<td>S 15</td>
<td>$7E+11$</td>
<td>5.457</td>
<td>0.994</td>
</tr>
<tr>
<td>S 16</td>
<td>$8E+09$</td>
<td>4.827</td>
<td>0.997</td>
</tr>
<tr>
<td>S 17</td>
<td>$4E+12$</td>
<td>6.074</td>
<td>0.996</td>
</tr>
<tr>
<td>S 18</td>
<td>$4E+11$</td>
<td>5.712</td>
<td>0.996</td>
</tr>
</tbody>
</table>
### Table 6.3  Parameters in the $c_{ul}$ versus $c_{ul}/w$ relationship

<table>
<thead>
<tr>
<th>Soil designation</th>
<th>m</th>
<th>n</th>
<th>Correlation coefficient</th>
<th>r</th>
</tr>
</thead>
<tbody>
<tr>
<td>S 1</td>
<td>4.0702</td>
<td>71.580</td>
<td>0.998</td>
<td></td>
</tr>
<tr>
<td>S 2</td>
<td>3.1802</td>
<td>71.407</td>
<td>0.999</td>
<td></td>
</tr>
<tr>
<td>S 3</td>
<td>4.6503</td>
<td>43.971</td>
<td>0.998</td>
<td></td>
</tr>
<tr>
<td>S 4</td>
<td>4.6118</td>
<td>49.808</td>
<td>0.998</td>
<td></td>
</tr>
<tr>
<td>S 5</td>
<td>6.6616</td>
<td>24.335</td>
<td>0.997</td>
<td></td>
</tr>
<tr>
<td>S 6</td>
<td>2.5170</td>
<td>51.935</td>
<td>0.999</td>
<td></td>
</tr>
<tr>
<td>S 7</td>
<td>4.4075</td>
<td>81.272</td>
<td>0.996</td>
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</tr>
<tr>
<td>S 8</td>
<td>2.7469</td>
<td>68.208</td>
<td>0.994</td>
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</tr>
<tr>
<td>S 9</td>
<td>4.0772</td>
<td>69.369</td>
<td>0.996</td>
<td></td>
</tr>
<tr>
<td>S 10</td>
<td>5.4095</td>
<td>63.231</td>
<td>0.997</td>
<td></td>
</tr>
<tr>
<td>S 11</td>
<td>3.8234</td>
<td>64.231</td>
<td>0.994</td>
<td></td>
</tr>
<tr>
<td>S 12</td>
<td>4.2893</td>
<td>61.268</td>
<td>0.996</td>
<td></td>
</tr>
<tr>
<td>S 13</td>
<td>7.3198</td>
<td>71.977</td>
<td>0.995</td>
<td></td>
</tr>
<tr>
<td>S 14</td>
<td>8.3431</td>
<td>66.661</td>
<td>0.993</td>
<td></td>
</tr>
<tr>
<td>S 15</td>
<td>4.2111</td>
<td>62.895</td>
<td>0.997</td>
<td></td>
</tr>
<tr>
<td>S 16</td>
<td>4.1734</td>
<td>43.883</td>
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<tr>
<td>S 17</td>
<td>3.0083</td>
<td>57.614</td>
<td>0.999</td>
<td></td>
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<tr>
<td>S 18</td>
<td>2.9112</td>
<td>49.366</td>
<td>0.998</td>
<td></td>
</tr>
</tbody>
</table>
Photograph 6.1. Undisturbed sample inside the sampling tube placed over a rigid cylindrical rod
Photograph 6.2. Undisturbed sample being gently pushed out of the sampling tube

Photograph 6.3  Oedometer ring being slowly inserted into the sample
Photograph 6.4 Oedometer ring filled with the sample

Photograph 6.5 Oedometer ring with the sample being cut off from the sampling tube
Photograph 6.6 Oedometer ring after trimming excess sample and leveling both sides

Photograph 6.7 Oedometer ring with the sample after consolidation
Photograph 6.8 Cone penetration test on consolidated sample in the oedometer ring.