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The Influence of Preconsolidation Pressure on Undrained Shear Strength Characteristics of Peaty Clay in Sri Lanka

M.W.S. Priyamali and N.H. Priyankara

Abstract: This research study investigated the effect of preconsolidation pressure on the shear strength parameters of very soft peaty clay based on a series of laboratory tests. The study included 15 Consolidated Undrained (CU) triaxial tests on peaty clay extracted from the Nilwala river basin, with preconsolidation pressures ranging from 0 to 100 kPa in 20 kPa intervals and cell pressures of 50, 100 and 150 kPa. The results indicated that the peak deviator stress of peaty clay decreased with increasing preconsolidation pressure, indicating decrease in its shear strength parameters with preconsolidation pressure. It was observed that peaty clay is highly sensitive to disturbance and requires sufficient time to regain its shear strength once its natural microstructure is altered. The study has significant implications for the construction of embankments on soft soils, particularly peaty clays. It highlights the need for accurate estimation of the undrained shear strength parameters of peaty clay, considering soil composition, structure and type of loading. It also emphasizes the importance of minimizing disturbance during construction activities to avoid compromising the stability of embankments.

Keywords: Peaty clay, Preconsolidation pressure, Triaxial CU test, Undrained shear strength

1. Introduction

Peat is a soil type formed from the partial decomposition and disintegration of plants that grow in marshes or other wetlands under anaerobic conditions (lack of oxygen) [1][2][3]. Despite being characterized by high levels of fibrous organic matter, the composition of peat can vary depending on the location, temperature, and degree of humification [4][5]. For better understanding of peat for geotechnical purposes, it can be classified into three categories, namely, fibric peat (least decomposed), hemic peat (intermediate decomposed), and sapric peat (most decomposed) ([6]-[11]). In Sri Lanka, the peaty soil typically has a lower organic content than in other domains, with a unique mixture of peat and clay ([12]-[16]). Peat can be macroscopically divided into three groups, namely, amorphous granular, coarse fibrous and fine fibrous, based on the degree of humification [17]. Amorphous granular is the type of peat with the greatest level of humification and has a high colloidal fraction. Most of the water is held in an absorbed state in amorphous peat. According to Karunawardena and Toki [15], Sri Lankan peaty clay is generally identified as the amorphous type with lower organic content.

Peaty clay is a type of soft soil which has high compressibility, high void ratio, very low shear strength and high water content [12] [13][14]. According to Karunawardena et al. [16], peaty clay found in Outer Circular Highway in Sri Lanka has water content of 75-237%, void ratio of 3.79-5.69, unit weight of 11.3-12.5 kN/m^3 and organic content of 25%. Engineering properties of the peaty clay was reported as poor with compression index of 1.95 and undrained shear strength of 7.2-19 kPa. Moreover, peaty clay was identified as lightly overconsolidated with preconsolidation pressure of 17-32 kPa. Based on the peaty clay specimens collected from Colombo-Katunayake Expressway project, Fernando and Kulathilaka [12] reported that peaty clay possesses significantly high preconsolidation pressure with higher moisture content of more than 200%. Madhusanka and Kulathilaka [13] presented that peaty clay has a

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natural moisture content of 300%, organic content of 22%, pH value of 4.2, specific gravity of 1.46 and a void ratio of 5.56 for the specimens collected from the Southern Expressway. Further, they illustrated that peaty clay has a compression index of 1.51, recompression index of 0.117, preconsolidation pressure of 0 kPa and undrained shear strength of 0.99 kPa. Huat et al. [17] reported that undrained friction angle of peat in West Malaysia is in the range of $3 - 25°$, while O'Kelly and Orr [18] postulated that the cohesion of fibrous peat is higher than zero. As such, it can be noted that peaty soils do not provide favourable conditions for construction on them.

According to Huat et al. [17], highly decomposed soil has higher strength than less decomposed soil. However, Polous [19] stated that soil composition, structure, initial density and type of loading also influenced the strength of organic soil. Although preconsolidation pressure plays a crucial role in enhancing the stiffness and strength of peaty clay by reducing void ratio and water content, only limited research has been conducted [20][21] to investigate the effect of preconsolidation pressure on shear strength parameters of peaty clay. As such, in this research study, a series of Consolidated Undrained (CU) triaxial tests were conducted to study the variation of shear strength parameters of Sri Lankan peaty clay over preconsolidation pressure.

2. Methodology

The methodology adopted to investigate the effect of preconsolidation pressure on shear strength parameters of peaty clay is described in this section.

2.1 Sample Collection

For this research study, dark, blackish ash colour peaty clay samples were collected from the Nilwala flood plain in Matara region, which is located in the southern part of Sri Lanka. The disturbed peaty clay samples were extracted at a depth of 1-3 m from the ground surface. As shown in Figure 1, the collected peaty clays were stored in containers for laboratory studies in such a way to minimize loss of moisture.

2.2 Properties of Peaty Clay

The physical and engineering properties of the peaty clay were determined according to the ASTM standards. Bulk density, specific gravity and saturated moisture content were determined as physical properties while organic content, Loss on Ignition (LOI) and pH value were determined as the chemical properties. Compressibility characteristics of peaty clay were determined using conventional oedometer tests according to ASTM-D2435 standard.

Figure 1 - Photo of Collected Peaty Clay

2.3 Specimen Preparation

Before starting the laboratory experiments to determine the shear strength characteristics for a particular preconsolidation pressure, it is crucial to ensure that the sample is sufficient for reconstituting sampling. Large particles were removed and the rest were converted into small sizes by using a sharpener, in order to prepare a homogeneous sample. The peat was mixed with water for about 15 minutes using a mechanical mixer and then air bubbles were carefully removed to obtain a homogeneous, stiff slurry. The mixture was then transferred into moulds with sizes of 50 mm in diameter and 100 mm in height. The top of the soil sample was left open for 3 days for air drying before subjecting to the preconsolidation test. Since prepared peaty clay was in a slurry form, after transferring the samples to steel moulds, they were allowed to air dry to make them stiffer. Figure 2 provides a clear illustration of the sample preparation process.

2.4 Preconsolidation Process

Properly prepared peaty clay samples were subjected to different preconsolidation pressures of 0, 20, 40, 60 and 100 kPa. The sample with 0 kPa preconsolidation pressure indicates the peaty clay without being subjected to preconsolidation pressure and those samples were directly used to determine the shear strength parameters using Consolidated

Undrained (CU)triaxial test after air drying process.

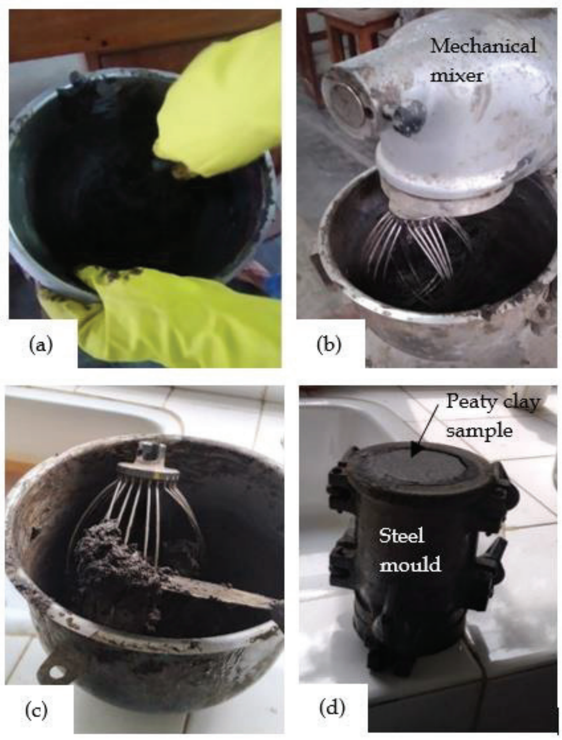


Figure 2 - Sample Preparation Process (a) Removing Large Particles Manually(b) Mixing the Sample (c) Checking the Peaty Clay Slurry (d) Prepared Peaty Clay Sample in the Mould

The specimens were preconsolidated using the conventional oedometer apparatus by modifying the loading frame as shown in Figure 3. The loading arms of the apparatus were extended in order to facilitate to apply load on the 100 mm high specimens. During preconsolidation, drainage was allowed from both top and bottom of the specimen by keeping porous plates. In addition to porous plates, filter papers were placed to accelerate the rate of primary consolidation. Once the oedometer setup has been prepared, prescribed loading (preconsolidation) was applied using the lever arm system. The test was conducted until the axial deformation (settlement) became constant with time, then it can be concluded that primary consolidation is completed. Since, consolidation process was conducted while the specimen was in the mould and, as there was no lateral deformation, this can be considered as 1-D consolidation. Generally, specimens were preconsolidated for 1-2 days in the steel moulds before being extracted for triaxial testing. The preconsolidation process is illustrated in Figure 3.

2.5 Testing Program

The testing program included 15 Consolidated Undrained (CU) triaxial tests, with different preconsolidation pressures, varying from 0 to 100 kPa. For each preconsolidation pressure, three samples were tested under three different cell pressures of 50, 100, and 150 kPa. The triaxial CU tests were conducted under undrained conditions, with a shearing rate (ε_v) of 0.3 mm/min. Table 1 outlines the laboratory testing program used to examine the effect of preconsolidation on shear strength parameters of peaty clay. The automated triaxial apparatus used for this research study is shown in Figure 4.

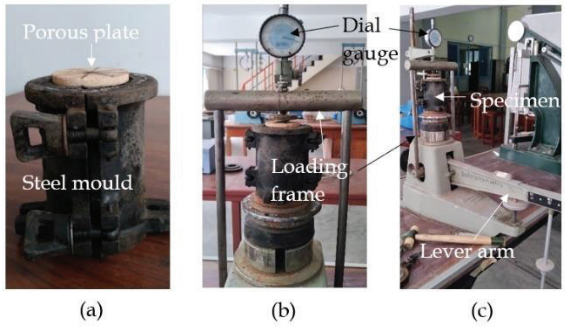


Figure 3 – Preconsolidation Process (a) Specimen (b) Loading Arrangement (c) Complete Preconsolidation Setup

Table 1 - Laboratory Testing Program

Preconsolidation pressure (kPa)	Drainage condition	Confining pressure (kPa)	No. of tests
0 20 40 60 100	Undrained (ε_v = 0.3 mm/min)	50 100 150	15

Figure 4 – Automated Triaxial setup

2.6 Sample Saturation

In the Triaxial CU test, first step is the saturation of the soil specimen. This is achieved by applying a vacuum pressure to the top of the sample while de-aired water is supplied from the bottom. The vacuum causes the water to drain through the sample from bottom to top, filling the voids with water. This process was carried out for 24 hours under slight lateral pressure (cell pressure) of 10 kPa. Then, back pressure was applied to the sample to dissolve air and complete the saturation process. It can be noted that cell pressure is always greater than the back pressure.

In order to verify the degree of saturation of the soil specimen, back pressure was gradually increased by 10 kPa (then cell pressure was also increased by 10 kPa) and kept for 1-2 hours, and change of pore water pressure was recorded. Then, again, back pressure was increased by 10 kPa and the increment of pore water pressure was checked. The degree of saturation was estimated using the "B value" as defined in Equation 1.

$$\text{B value} = \frac{\text{Change of Pore Water Pressure}}{\text{Change of Cell Pressure}} = \frac{\Delta u}{\Delta \sigma} \quad \text{.... (1)}$$

If the B value was less than 0.95, the above process was repeated many times until B value became 0.95. When B value is greater than 0.95, it can be assumed that soil is fully saturated and sample is ready for shearing.

The back pressure saturation method is commonly adopted in triaxial tests to increase the degree of saturation in samples. Back pressure is considered as a vital factor in undrained shear strength properties and hence Huang et al. [22] recommended to keep the back pressure as a constant value throughout the triaxial test. But, a critical value for back pressure has not been defined for the soft clay and therefore, in this research study, back pressure of 40 kPa was maintained, as it was lower than the minimum confining pressure of 50 kPa.

2.7 Consolidation of Specimen

After saturation, sample was consolidated by applying the cell pressure upto the corresponding confining pressure of the shearing stage, and the drainage valve was opened allowing soil specimen to consolidate until the vertical displacement of the sample becomes constant. Since drainage valve is opened, pore water pressure of the specimen is equal to zero, and it took about 3 days to consolidate the specimen.

2.8 Shearing of Specimen

After completion of the consolidation, the soil specimens underwent Undrained Compression tests at cell pressures of 50, 100 and 150 kPa, with the drainage valve kept closed. Shearing rate was maintained at 0.3 mm/min during undrained conditions. Data, including pore water pressure, axial displacement, and deviator load, were recorded using an automatic data acquisition system.

3. Results and Discussion

3.1 Properties of Peaty Clay

Physical and chemical properties of the peaty clay are summarized in Table 2. It can be noted that Sri Lankan peaty clay has a lower organic content of 20.53% while, due to presence of higher fiber content, it has higher water absorption capacity. The specific gravity of peaty clay in this research study is about 1.75 which is in the upper range of the values (1.38-1.80) reported in literature ([12]-[16]). Peaty clay is acidic with a pH value less than 7 and attributed to the presence of organic acids. This particular property of peaty clay has already been confirmed by several studies [23][24]. Further, it has been demonstrated that the pyrite content in peat, derived from marine sediments, can affect the pH value of the soil. The type and amount of minerals present in the layer below the peat can affect natural fertility and contribute to the reduction in pH value.

Table 2 – Physical and Chemical Properties of Peaty Clay

Property	Value
Bulk unit weight(γ)	14.0 kN/m^3
Specific gravity(G_s)	1.75
Saturated moisture content	275%
Organic content	20.53%
Loss on Ignition (LOI)	31.51%
pH value	4.12

The compressibility characteristics of the peaty clay are presented in Table 3 and void ratio versus effective stress (e vs$\log\sigma'$) curve is depicted in Figure 5. Based on the results, it can be noted that peaty clay is highly compressible.

3.2 Effect of Preconsolidation on the Shear Strength Characteristics of Peaty Clay

3.2.1 Preconsolidation Process

Results of the preconsolidation process on peaty clay specimens are presented in this section. Samples were prepared in such a way that keeping the bulk unit weight and initial moisture content as 12.0-13.7 kN/m^3 and 75-97%, respectively, as shown in Table 4.

A shown in Table 4, it can be noted that time required for the preconsolidation process (until settlement becomes constant) increases with the increase of preconsolidation pressure. It took 24 hours (one day) for the process under 20 kPa preconsolidation pressure while 2 days were spent for the preconsolidation pressure of 40, 60 and 100 kPa. On the other hand, settlement also increased with the increase of preconsolidation pressure.

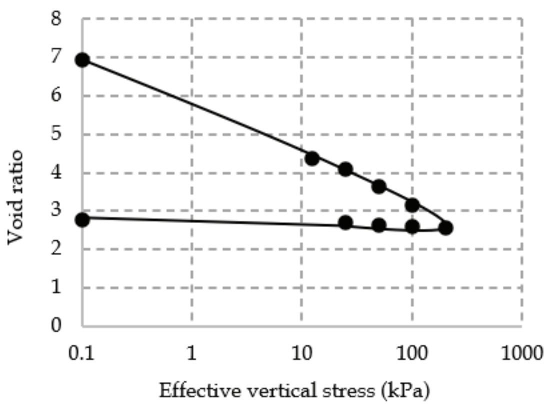


Figure 5 – Variation of Void Ratio versus Effective Stress

Table 3 – Compressibility Characteristics of Peaty Clay

Property	Value
Initial void ratio (e_0)	6.93
Compression index (C_c)	1.88
Modified compression index ($C'_c = \frac{C_c}{1+e_0}$)	0.24
Swelling index (C_s)	0.023
Modified swelling index ($C'_s = \frac{C_s}{1+e_0}$)	0.003
Preconsolidation pressure (P'_c)	15.85kN/m^2
Coefficient of consolidation C_v(for the stress range of 25–200 kPa)	0.91 – 1.24 m^2/year

Table 4 - Properties of Preconsolidated Peaty Clay

Preconsolidation pressure (kPa)	Initial moisture content (%)	Initial sample density (kg/m^3)	Settlement (mm)	Time spent for preconsolidation (days)
0	97.4	1395	0	-
20	96.0	1230	0.27	1
40	82.0	1255	0.58	2
60	79.3	1310	0.76	2
100	75.0	1385	1.28	2

It was noted that density increment does not seem to be significant with the variation of preconsolidation pressure and hence the stiffness of peaty clay cannot be remarkably improved by increasing the preconsolidation pressure.

3.2.2 Saturation Process

The variation of pore water pressure, back pressure and cell pressure during saturation process is illustrated in Figure 6 and it can be noted that cell pressure is always 10 kPa greater than back pressure.

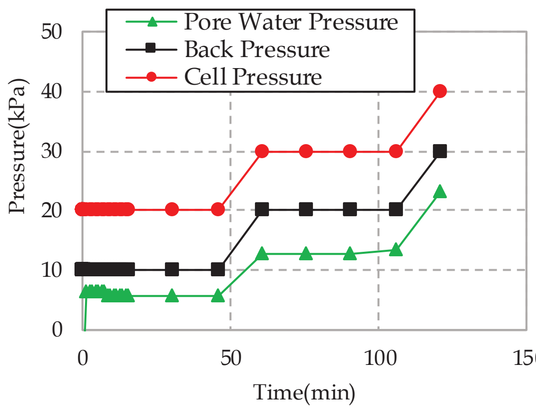


Figure 6 - Variation of Pore Water Pressure, Back Pressure and Cell Pressure during Saturation Process

3.2.3 Consolidation Process

The typical variation of settlement over time during consolidation process is depicted in Figure 7. Based on the results of consolidation process, coefficient of consolidation (C_v) was calculated for each consolidation pressure and each cell pressure and presented in Table 5 and Figure 8.

The results presented in Figure 8 clearly illustrate that C_v values of preconsolidated peaty clay are always greater than that of non-preconsolidated samples. Even though, there is no any clear relationship between C_v and preconsolidation pressure, it can be concluded that C_v values increase with the increase of preconsolidation pressure irrespective of the cell pressure.

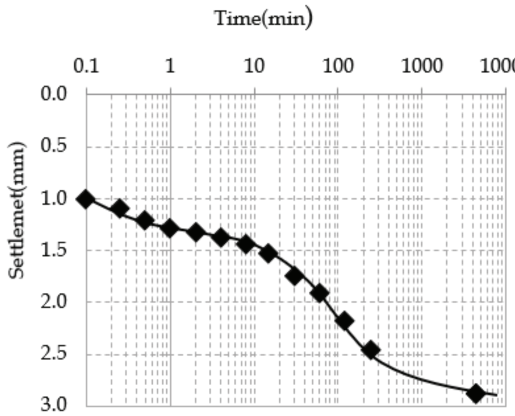


Figure 7 – Variation of Typical Settlement over time during Consolidation Process (Cell Pressure =150 kPa and Preconsolidation Pressure = 40 kPa)

Table 5 - Summary of Consolidation Test Results

Preconsolidation pressure (kPa)	Coefficient of Consolidation (C_v)(m²/year)		
	Cell pressure (kPa)		
	50	100	150
0	0.26	0.21	0.24
20	1.08	0.52	0.89
40	0.37	0.92	1.36
60	1.29	0.52	0.65
100	1.23	0.26	0.43

3.2.4 Stress-Strain Relationship

The typical variation of deviator stress and pore water pressure over axial strain under different cell pressures at 20 kPa preconsolidation pressure are shown in Figure 9 and Figure 10, respectively.

Results of the triaxial tests indicate that, regardless of the preconsolidation pressure, the axial strain at the yield point (maximum deviator stress) decreased with increasing confining pressure. Consequently, all the samples failed within the same range of axial strain irrespective of the preconsolidation pressure. However, it is a challenge to establish a precise relationship between the axial strain at

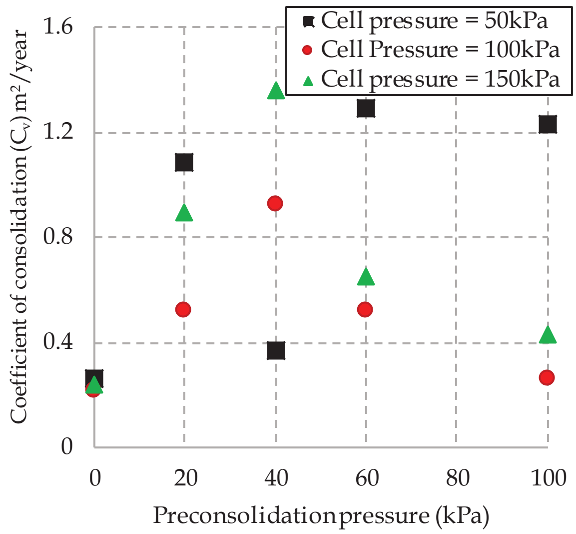


Figure 8 – Variation of Coefficient of Consolidation (C_v) over Preconsolidation Pressure

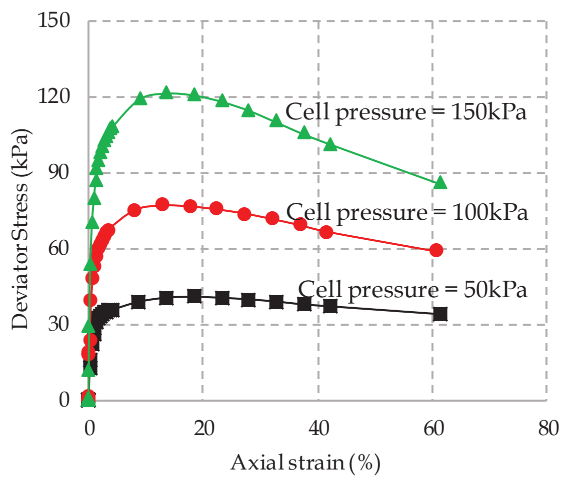


Figure 9 – Stress - Strain Relationship under 20 kPa Preconsolidation Pressure

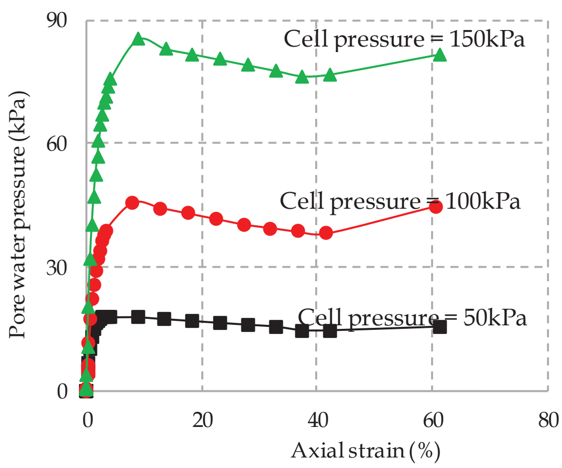
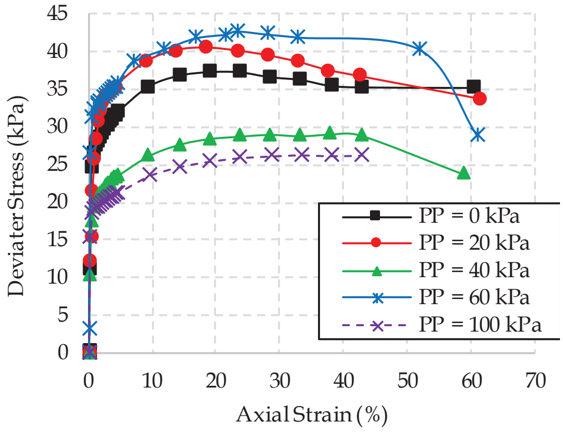
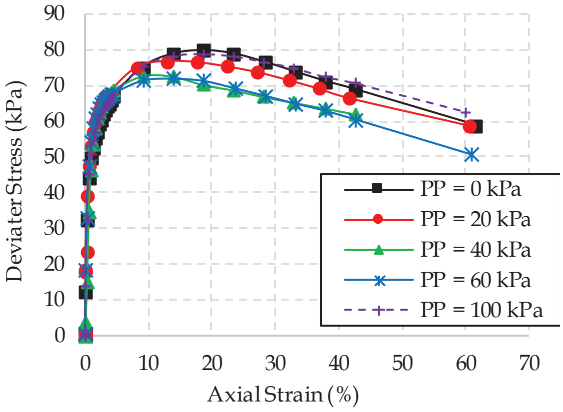


Figure 10 - Variation of Pore Water Pressure over Axial Strain under 0 kPa Preconsolidation Pressure

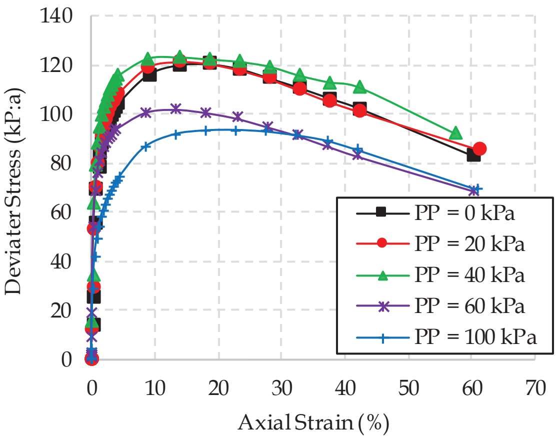
The yield point and the preconsolidation pressure. To gain a better understanding of the effect of preconsolidation pressure on deviator stress, the relationship between deviator stress and axial strain was analyzed for different preconsolidation pressures (PP) under the same cell pressure as depicted in Figure 11.



(a) **Cell Pressure = 50kPa**



(b) **Cell Pressure = 100 kPa**



(c) **Cell Pressure = 150kPa**

Figure 11 - Relationship between Deviator Stress and Axial Strain under Different Cell Pressures

It can be seen that, even though all specimens illustrated the same behaviour with respect to deviator stress -axial strain, peak deviator stress slightly decreased with the increase of preconsolidation pressure irrespective of the cell pressure. This behaviour is clearly illustrated in Figure 12. Moreover, it can be noticed that peak deviator stress increases with the cell pressure for a particular preconsolidation pressure. However, when the deviator stress is normalized with respect to cell pressure (CP) as illustrated in Figure 13, it can be clearly seen that there is no significant effect of cell pressure on load carrying capacity of the peaty clay at low preconsolidation pressures (PP), i.e.when the preconsolidation pressure is less than 20 kPa. This is obvious from their behaviour during triaxial tests as peaty clay is normally consolidated when the preconsolidation pressure is less than 50 kPa. On the other hand, cell pressure has a considerable effect on the deviator stress at higher preconsolidation.

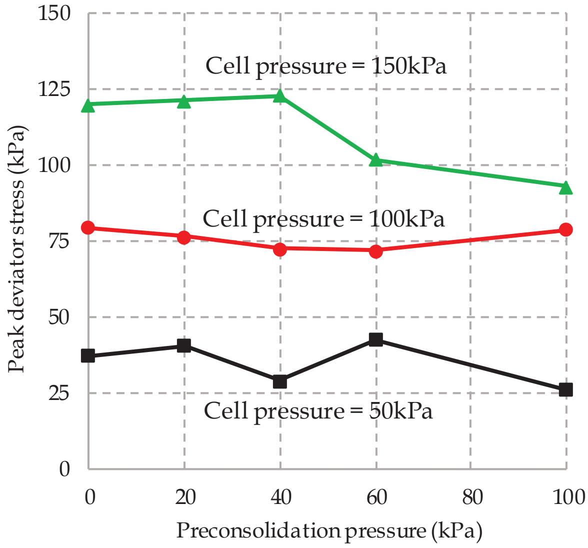


Figure 12 - Variation of Peak Deviator Stress Over Preconsolidation Pressure

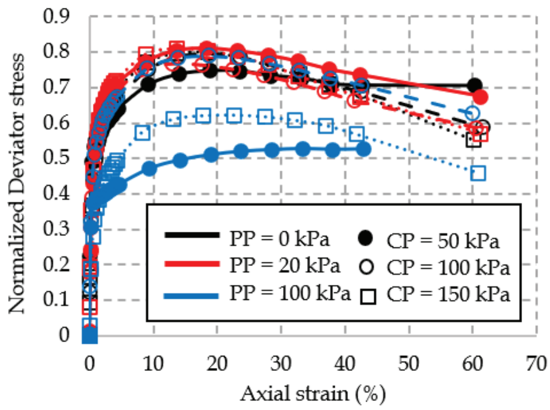


Figure 13 - Variation of Normalized Deviator Stress Over Axial Strain

pressures, i.e. when the cell pressure is less than the preconsolidation pressure in which peaty clay is overconsolidated.

The variation of axial strain at yield point over preconsolidation pressure is presented in Figure 14. Even though, there is no clear relationship between axial strain at yield point over preconsolidation pressure, for a particular cell pressure, it can be observed that axial strain at yield point decreases with the increase of preconsolidation pressure upto 40 kPa and then axial strain at yield point increases with the preconsolidation pressure.

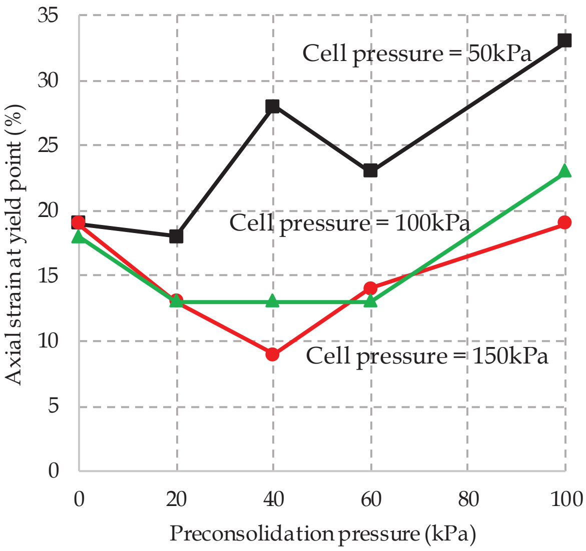


Figure 14 - Variation of Axial Strain at Yield Point OverPreconsolidation pressure

Moreover, higher cell pressure leads to a lower axial strain at the yield point for a particular preconsolidation pressure. This behaviour is attributed to the failure of soil microstructure during preconsolidation, as remoulded peaty clay is loaded before achieving sufficient shear strength.

As shown in Figure 10, it can be seen that maximum pore water pressure was not developed at the yield point, and it was developed at the larger axial strain than that of yield axial strain. In order to clearly understand the behaviour of soft peaty clay under undrained condition, stress paths were plotted for different cell pressures. Stress path is a relationship between deviator stress (q) and mean effective stress (p') as defined in Equation 2 and Equation 3, where σ_1' is the effective major principal stress and σ_3' is the effective minor principal stress.

$$q = \sigma_1' - \sigma_3' \quad \text{.... (2)}$$

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

The typical variation of q over p' under 60 kPa preconsolidation pressures is illustrated in Figure 15. The results clearly depict that cell pressure has a significant effect on the maximum load that can be carried by the preconsolidated peaty clay, where higher the cell pressure, higher the load carrying capacity for a particular preconsolidation pressure.

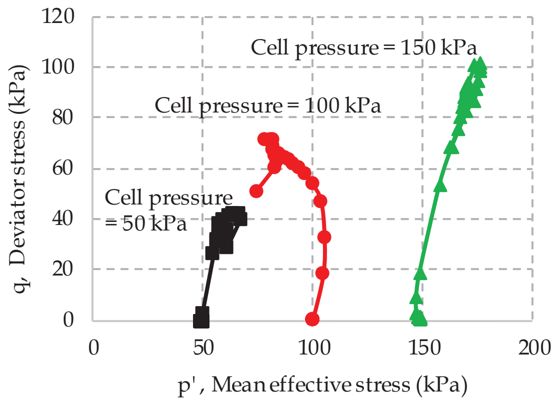


Figure 15 – Effective Stress Path for 60 kPa Preconsolidation Pressure

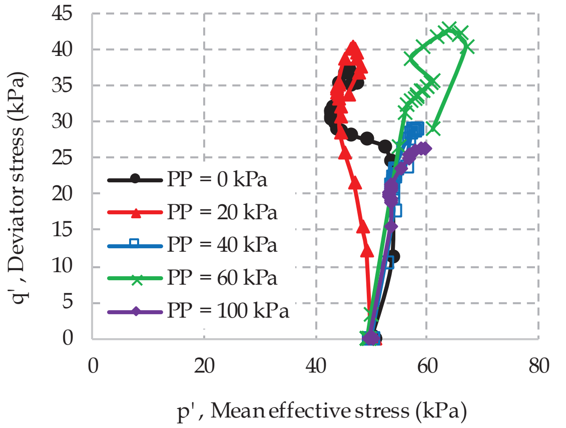
In order to further study the effect of preconsolidation pressure on load carrying capacity of the soft soil, effective stress paths were plotted for different cell pressures. The results presented in Figure 16 (a) to Figure 16(c) depict that preconsolidation pressure has a significant effect on the maximum load that can be carried by the peaty clay. It can be seen that, when the preconsolidation pressure is greater than 20 kPa, lower peak deviator stress can be observed irrespective of cell pressure. Further, it can be observed that pore water pressure has been reduced (ultimately it became negative) under the higher preconsoldation pressures as shown in Figure 17. Once the peaty clay is preconsolidated under higher stress and applied cell pressure in the triaxial test is low, soil becomes over consolidated with higher Over Consolidation Ratio (OCR), resulting in negative pore water pressure during shearing. The typical failure mode of the aforementioned samples under undrained condition is illustrated in Figure 18.

3.2.5 Shear Strength Parameters

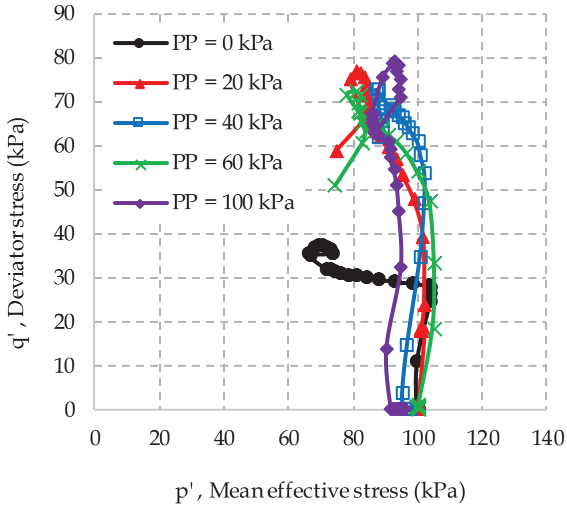
Based on the triaxial test results, Mohr-Coulomb failure envelopes were developed to determine the shear strength parameters. Figure 19 and Figure 20 show the typical Mohr-Coulomb failure envelopes under total stress and effective stress analyses, respectively, for

20 kPa preconsolidation pressure. The shear strength parameters obtained in terms of total and effective stress are presented in Table 6.

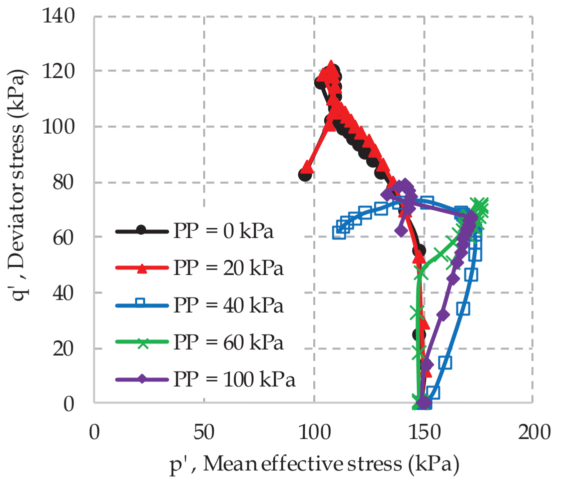
The variation of total shear strength parameters over preconsolidation pressure is shown in Figure 21. It can be seen that undrained cohesion remains around zero irrespective of the preconsolidation pressure while undrained friction angle slightly decreases with preconsolidation pressure.



(a) Cell Pressure = 50 kPa



(b) Cell Pressure = 100kPa



(c) Cell pressure = 150kPa

Figure 16 – Effective Stress Path for Different Cell Pressures

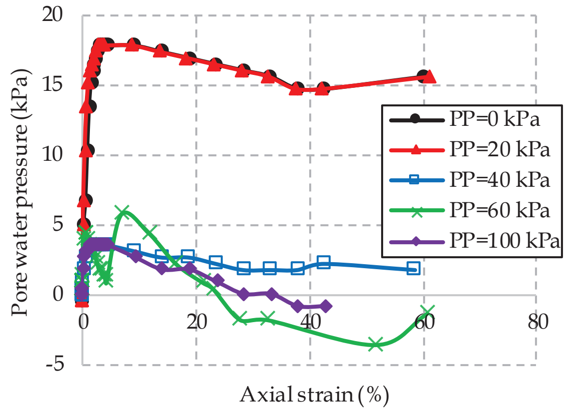


Figure 17 – Variation of Pore Water Pressure Under Different Preconsolidation Pressures for Cell Pressure of 50 kPa

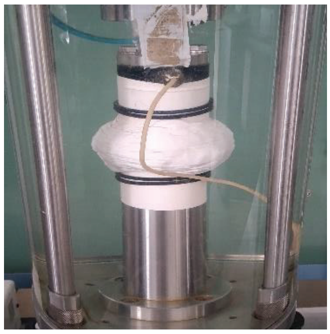


Figure 18 – Typical Failure Pattern of the Samples Under CU Triaxial Test

Cohesionless nature of the material indicates that peaty clay is normally consolidated. Similar behaviour can be seen in the effective shear strength parameters as illustrated in Figure 22.

Even though, there is no clear relationship between friction angle and preconsolidation pressure, generally, friction angle decreases with the increase of preconsolidation pressure. However, at low preconsolidation pressure, i.e. 20 kPa, a considerable shear strength gain can be observed. With an increase in preconsolidation pressure, the microstructure of the peaty clay fails as soil is highly sensitive, resulting in lower shear strength parameters. This behaviour clearly illustrates the importance of stage loading in very soft soil under ground improvement.

Table 6 - Peak Shear Strength Parameters

Preconsolidation pressure (kPa)	Total stress analysis		Effective stress analysis	
	Cohesion $C_u(kPa)$	Friction angle $\varphi_u(°)$	Cohesion $c'(kPa)$	Friction angle $\varphi'(°)$
0	0	16.67	0	21.80
20	0	16.67	0	23.96
40	0	15.64	0	20.80
60	6	14.57	4	15.10
100	0	13.50	0	16.69

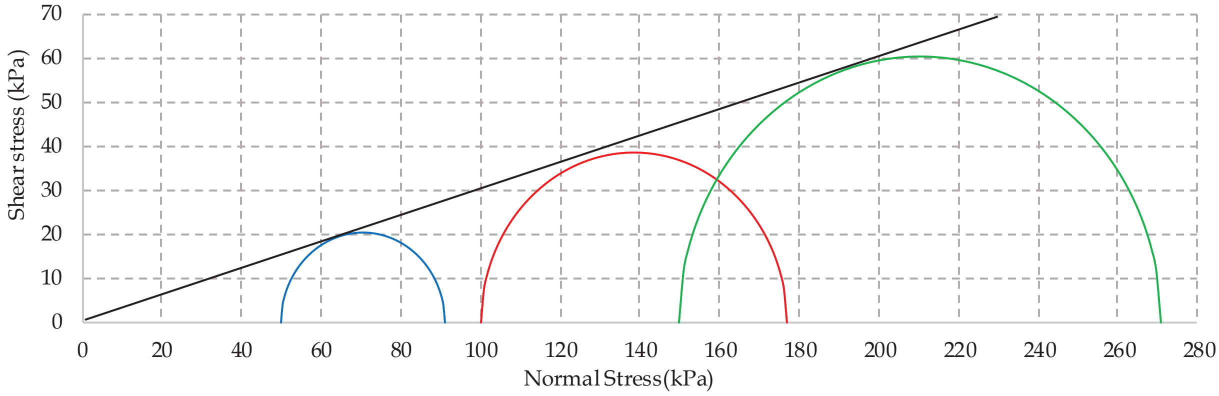


Figure 19 - Mohr-Coulomb Failure Envelops under Total Stress Analysis at 20 kPa Preconsolidation Pressure

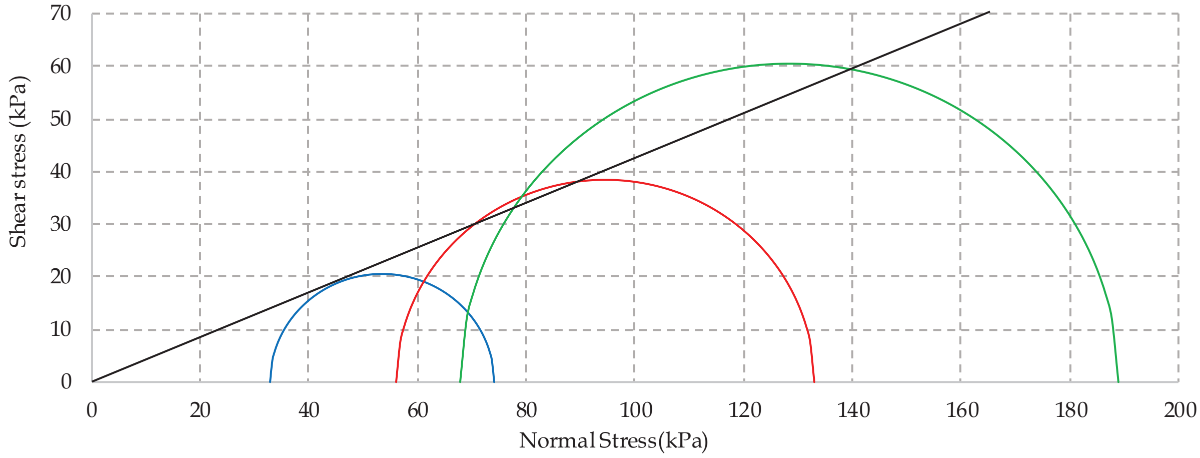


Figure 20 - Mohr-Coulomb Failure Envelops Under Effective Stress Analysis at 20 kPa Preconsolidation Pressure

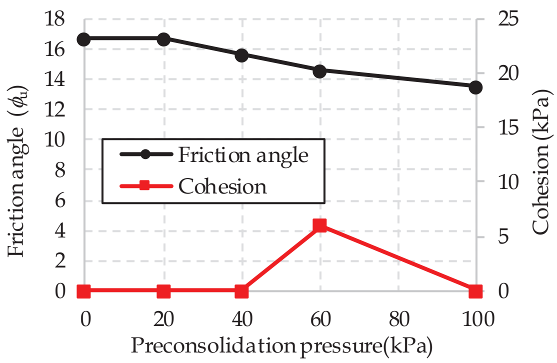


Figure 21 - Variation of Total Shear Strength Parameters Over Preconsolidation Pressure

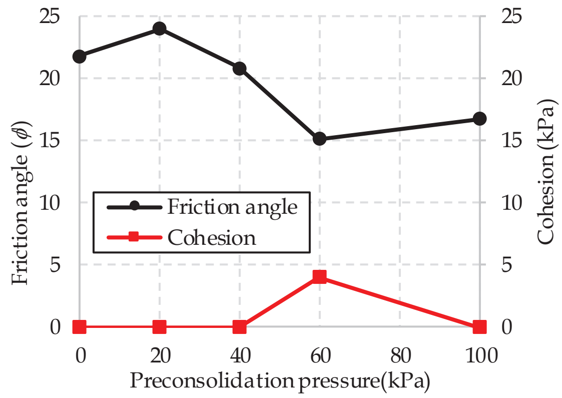


Figure 22 - Variation of Effective Shear Strength Parameters Over Preconsolidation Pressure

4. Conclusions

The conclusions of this research study can be summarized as follows:

1. Laboratory CU Triaxial test results indicate that, irrespective of the preconsolidation pressure, axial strain at the yield point decreased with increasing confining pressure and all the samples failed at a similar range.
2. Peak deviator stress slightly decreased with the increase of preconsolidation pressure, irrespective of the cell pressure. Moreover, peak deviator stress increases with the cell pressure for a particular preconsolidation pressure.
3. When the peaty clay is normally consolidated, there is no significant effect of cell pressure on the load carrying capacity. On the other hand, there is a considerable effect of cell pressure on the load carrying capacity of peaty clay when the soil is overconsolidated.
4. The higher cell pressure leads to a lower axial strain at the yield point for a particular preconsolidation pressure due to failure of the soil microstructure during preconsolidation. This behaviour clearly indicates that peaty clay is highly sensitive to disturbance and requires sufficient time to regain its shear strength once its natural microstructure is altered.
5. Behaviour of $p' - q$ graph clearly illustrates that peaty clay is a highly sensitive soil, in which pore water pressure during shearing becomes negative when the soil is oveconsolidated.
6. Based on a series of laboratory CU triaxial tests, it can be concluded that once the highly sensitive peaty clay is disturbed due to preconsolidation (suddenly change the initial micro structure), there is no considerable shear strength gain as depicted in Figure 21 and Figure 22.
7. The findings of this research study clearly illustrates that, once the initial microstructure of the highly sensitive peaty clay has been significantly altered (failed), it requires considerable time to regain the required shear strength.

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