Geomechanical aspects of some tropical clay soils from Dhaka, Bangladesh

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Abstract: Laboratory testing has been carried out to evaluate the geomechanical aspects of some tropical clay soils from Dhaka, Bangladesh. The tropical clay soils of Dhaka are mainly composed of illite, kaolinite, chlorite and some non-clay minerals. These soils have random open microfabric of silt and clay.

Undrained triaxial tests have been carried out on seven undisturbed natural samples collected from a borehole in the Curzon Hall area of Dhaka. Samples were consolidated at a range of confining pressures from 50 kPa to 800 kPa, before shearing. The results are discussed in terms of deviator stress versus axial strain curves, stress paths, stress ratio and excess pore water pressure versus axial strain curves. It was observed that the samples consolidated at low effective stress initially showed peak positive values of excess pore water pressures followed by negative values at higher strains due to the sample dilating. No negative pore water pressures were observed at high confining pressures. Some of the samples at low confining pressures approached the critical state at very large strains. High confining pressure samples may not have reached the critical state due to the formation of distinct shear surfaces. An undrained failure surface for these soils is also identified. Difficulties in defining undrained critical state conditions are also discussed. Values of the critical state stress ratio M were estimated to be 0.95-0.96.

Résumé: On a exécuté des tests du laboratoire pour évaluer les aspect géologique et méchanique` de sol argileux et tropical de Dhaka du Bangladesh. Il est certain que le sol de Dhaka est composé principalement d'illite, de chlorite, de kaolinite et en petite quantité des matériaux qui ne sont pas argileux. On a observé que le sol montre une micro-structure aléatoire du limon et de l'argile.

On a exécuté un test triaxiaux de sondage inasséché sur sept échantillons naturels et inchangés ramassés d'un trou de sonde de Carzon Hall à Dhaka. On les a consolidé avec une pression entre 50 k pascal et 800 k pascal avant de cisailler. Les résultats ont exprimé en fonction des relations entre la pression de déviation et la déformation axial, des relations entre la pression interstitielle et la déformation axial, le chemin de contrainte et le taux de pression. On a observé que les échantillons ont montré initialement des plus hautes valeurs pour la pression interstitielle excès et positive qui ont été suivies par les valeurs négatives pour les plus grandes déformations. Il s'est passé parce que les échantillons ont eu une tendance à se dilater. Il n'y avait aucune pression interstitielle négative pour grande pression qui confine. Quelques échantillons étant confinés par les pressions plus bases sont arrivées à un stade critique aven grandes déformations. Les échantillons confinés par grande pression ne sont pas arrivés au stade critique peut-être à cause de la formation distincte de la surface de cisailler. Une surface de rupture non-drainé aussi a été identifiée. On a discuté les conditions non-drainées et critiques. On a estimé le taux de pression M à un stade critique entre 0.95 et 0.96.

Keywords: Clay, stress, pore pressures, failures, triaxial tests, mechanical properties.

INTRODUCTION

Dhaka City is located in the central part of Bangladesh, on the southern tip of a Pleistocene terrace, the Madhupur tract. The climate is generally humid and tropical with short cool winters and long, hot, very wet summers. The city is bounded by the Turag River in the northwest, the Buriganga River in the south and southeast, the Balu River in the east and the Buriganga and the Turag Rivers in the west.

Alam et al. (1990) of the Geological Survey of Bangladesh renamed the Madhupur Clay Formation as a Madhupur Clay Residuum (a residual soil horizon). The reddish brown clay soils of Dhaka are tropical residual soils that have developed in situ (Monsur, 1995). They are mainly composed of illite, kaolinite, chlorite and some non-clay minerals and of intermediate to high plasticity inorganic clay (CI to CH). The clay particles are randomly orientated or clustered and intermixed with larger silt grains (Hossain, 2001). The undisturbed soil has intra and intergranular pore space. The silt grains are coated with clay and iron oxide. These soils contain ferruginous cements, concretions and iron nodules. Monsur (1995) noted that the reddish brown colour of these soils is clearly related to iron compounds. He also mentioned that these are insitu developed soil and do not represent transported or re-deposited soil materials. He also mentioned that these soils have undergone intensive weathering processes that released 'Fe' ions in free state and appeared in the form of nodules or in association with clay.

Vaughan and Kwan (1984) pointed out that initial structure and porosity are a product of weathering process rather than insitu stress history. Vaughan (1988) and Vaughan, Maccarini and Mokhter (1988) discussed that the residual soils are developed in place without transportation. The particles and their arrangement evolve progressively as a consequence of chemical weathering, with widely varying mineralogy and void ratio. The void ratio of residual soils is very variable and does not vary systematically with soil type, parent rock etc. Weathering has a strong influence on the formation of tropical residual soils.

Table 1. Borehole descriptions of soils (Location: Curzon Hall, Dhaka)

Depth (m)	Lithologic description
0-1.05	Firm to stiff plastic clay with silt. Some carbonaceous matter present.
1.05-1.98	Highly oxidized, reddish grey (2.5YR 4/6), yellowish brown to reddish yellow (7.5YR 6/6), mottled plastic clay with some silt..
1.98-2.59	Firm to stiff silty clay, mostly reddish (2.5YR 4/6), plastic and containing ferruginous concretions.
2.59-3.20	Red (2.5YR 4/6) with reddish yellow (7.5YR 6/6), mottled, highly oxidized silty clay, some ferruginous concretions.
3.20-4.11	Red (2.5YR 4/6) to reddish yellow (7.5YR 6/6), reddish grey mottled silty clay, ferruginous concretions present, oxidized, nodules and some mica present.
4.11-4.50	Reddish yellow (7.5YR 6/6) to moderate reddish brown (10R 4/6) silty clay.
4.50-5.33	Red (2.5YR 4/6) to reddish yellow (7.5 YR 6/6) mottled silty clay, highly oxidized, ferruginous concretions, mica, iron nodules and manganese spots present.
5.33-5.94	Yellowish brown to reddish yellow (7.5 YR 6/6), very stiff clay with some silts, less oxidized, some iron nodules, mica present.
5.94-6.40	Yellowish brown to red (2.5YR 4/6), less oxidized, mottled, stiff clay, some silts and mica.
6.40-7.16	Mainly medium dense to loose sands, silts & clay.

Table 2. Summary of basic geotechnical properties (Hossain, 2001)

Location	**Sample no.**	**Depth (m)**	**Sand (%)**	**Silt (%)**	**Clay (%)**	**Particle density (ρ_s) Mg/m^3**	**LL (%)**	**PL (%)**	**PI (%)**	**Classification**
Curzon Hall	1	4-4.5	8	61	31	2.60	51	19	31	CH
	2	5-5.5	8	59	33	2.62	46	20	26	CI
	3	6-6.5	9	63	28	2.61	48	21	28	CI

This paper presents the results of a series of consolidated undrained triaxial tests with pore water pressure measurement conducted on undisturbed samples of red tropical clay soils collected from a borehole of the same geologic formation in the Curzon Hall area (Latitude:23°43.6′N to 23°43.8′N and longitude: 90°24.5′E to 90°24.8′E) of Dhaka, Bangladesh.

METHODOLOGY

The whole work was carried out under the following heads:

- Drilling and sampling
- Laboratory investigation
- Result interpretation

Drilling and Sampling

Light cable percussion drilling was used to collect the samples. Undisturbed soil samples were collected with thin wall open Shelby tubes (U100) and disturbed samples were collected by using split spoon sampler. Before each test, undisturbed natural samples were trimmed carefully with the help of very sharp knife. Cylindrical test specimens of length 76 mm. and diameter 38 mm. were prepared vertically from the central core part of the U100 tube. The test samples are described in Table 1 and index data of some of the samples are in Table 2.

Testing details

Consolidated undrained triaxial tests with pore water pressure measurements were carried out on the undisturbed natural samples collected from several depths from a borehole in the Curzon Hall area of Dhaka. The triaxial test was computer controlled and logged including volume change apparatus, load cell, displacement transducer and, cell and pore pressure transducers. The tests were monitored using the TRIAX programme developed by Toll (1993).

Local axial strain measurements were used up to 2% strain with a pair of electrolevels (Jardine et al., 1984), standard methods were used at higher strain. Pore pressure was measured with a mid height pore water pressure probe (Hight, 1982) in addition to the standard base measurement. All samples were saturated before isotropic consolidation and compression. Most samples attained a saturation B value of 0.98, after being saturated for up to 4-5 days with a backpressure of 300 kPa. Samples were consolidated for up to 24 hours at a range of confining pressures from 50 kPa up to 800 kPa before shearing. The undrained compression stage was carried out at a constant rate of 1.25%/hr up to 30% strain.

Test results

Sample test details are in Table 3. In this research, void ratio is considered as a function of weathering process and is not related to stress history. All samples are named by using letters and numbers. Two letters are used to designate each test. The first letter in each test indicates the undrained compression, and the second letter indicates that the sample is 'natural', that is undisturbed and at natural moisture content. The numbers following indicate the effective consolidation stress. For example, test un100, in Table 3, indicates a test on a sample, tested undrained in a natural state, which had an effective consolidation stress of 100 kPa before shearing.

Table 3. Testing details of natural soils of Curzon Hall, Dhaka

Sample Number	**Depth (m)**	**Initial moisture content (w %)**	**Particle density ρ_s (Mg/m^3)**	**Bulk density ρ (Mg/m^3)**	**Dry density ρ_d (Mg/m^3)**	**Initial void ratio (e_0)**	**Wet weight of Sample (g)**	**Dry weight of sample (g)**
un50	1.10-1.30	17.9	2.59	2.176	1.846	0.403	187.62	159.14
un100	1.32-1.47	17.1	2.62	2.171	1.859	0.410	187.62	160.21
un200	3.50-3.65	14.2	2.64	2.150	1.883	0.402	185.35	162.27
un300	3.68-3.88	15.3	2.62	2.016	1.749	0.498	173.80	150.74
un400	4.13-4.33	14.1	2.60	2.062	1.806	0.440	177.77	155.70
un500	4.60-4.80	14.9	2.59	1.972	1.720	0.505	170.45	148.24
un600	5.45-5.65	16.0	2.63	2.060	1.773	0.482	177.59	153.00
un800	6.09-6.40	17.2	2.61	2.049	1.748	0.493	176.62	150.65

Stress-strain curves

The deviator stress versus axial strain curves for these samples are shown in Figure 1. It can be seen from this figure that the stress strain curves show a maximum stress level in each case. After reaching the maximum stress level, there is a reduction in deviator stress with increasing strain. In many tests, the deviator stress decreases after the maximum point at almost a constant rate at very large strain. The peak values of q for samples consolidated to effective pressures of 600 kPa or more have a more prominent peak than those consolidated to lower stresses. It can also be seen from this figure that the maximum deviator stress increases with increasing effective consolidation pressure.

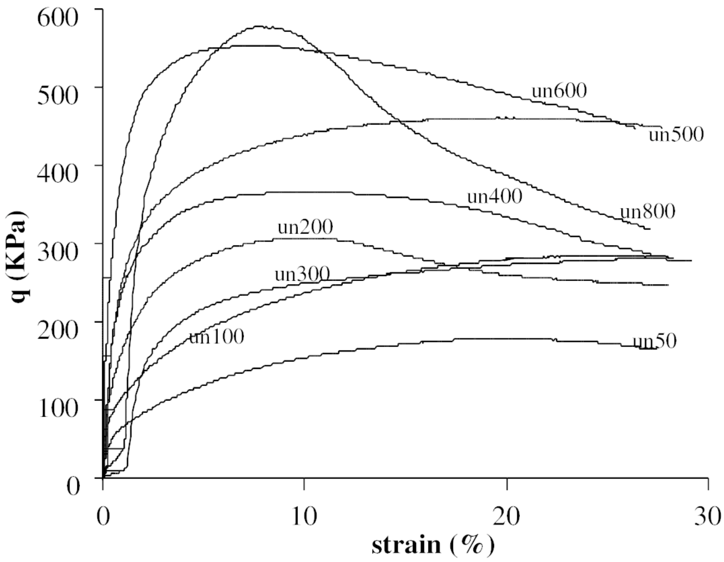

Figure 1. Deviator stress versus axial strain curves of natural soils

The axial strains to attain maximum deviator stress for these samples range from 8% to 26%. The maximum deviator stress and the corresponding values of axial strain, mean effective stress and the value of excess pore water pressure in each case are summarized in Table 4.

Table 4. Summary of stress and strain parameters at maximum deviator stress

Sample number	Maximum deviator stress (q) kPa	Excess p.w.p. at maximum q (u) kPa	Axial strain at maximum q (ε_a%)	Mean effective stress (p') at maximum q kPa
un50	179	-63	20.52	167
un100	284	-128	25.45	308
un200	307	-19	10.13	314
un300	280	85	26.35	296
un400	366	171	9.26	345
un500	461	113	20.34	529
un600	553	143	7.67	636
un800	577	223	7.94	714

Excess pore water pressure versus strain curves

The excess pore water pressures (p.w.p.) versus strain curves for all samples are shown in Figure 2. The excess pore water pressure increased initially and reaches a peak and then decreases with increasing strain. Tests at low confining pressures (50 to 200 kPa) initially showed low values of positive pore water pressures, followed by negative pore pressures at higher axial strains. No negative pore pressure was observed at higher confining pressures. The generation of negative excess p.w.p. values at low confining pressures (50-200 kPa) is due to a tendency to dilate during shearing. This dilation caused a decrease in pore water pressure and ultimately to negative values (Atkinson & Bransby, 1978).

The excess pore water pressure versus axial strain graph (Figure 2) shows that there is a general trend between excess p.w.p. and confining pressure. The maximum positive excess pore water pressure values for all samples lie between approximately 15 kPa to 250 kPa. The highest value of positive excess p.w.p. is for sample un800, whereas the lowest value was in sample un50. The excess p.w.p. values at maximum deviator stress in each case are listed in

Table 3. These values lie in the range of -128 kPa to 223 kPa. In all samples excess pore water pressures have reached their maximum values at lower axial strains than that of the maximum deviator stress.

By comparing the excess p.w.p. and strain curves, it is evident that samples at low effective consolidation pressures, 50 to 200 kPa, showed a continuous change of p.w.p. up to the end of the test and therefore, these samples have not reached a steady state at the end of shearing. If it has been possible to continue shearing at more than 30% strain, it might be possible that these samples might be reached the steady state as they did not form distinct shear surfaces.

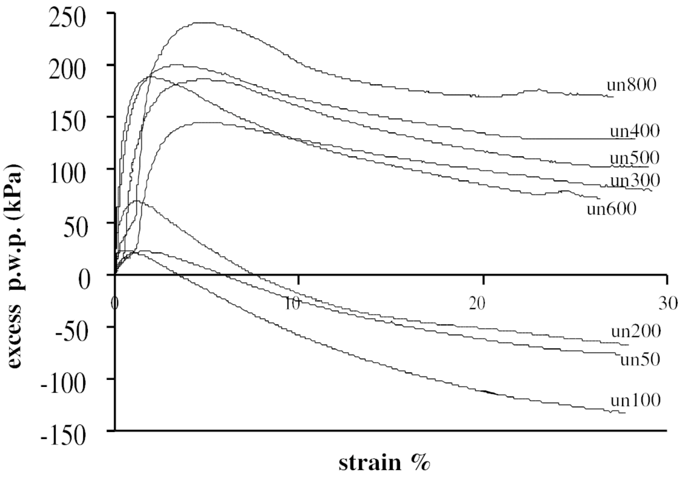

Figure 2. Excess p.w.p. vs. strain percent curves of natural soils

It can be seen from figure 2 that the excess p.w.p. for tests un400 and un500 tended towards constant values at very large strains (approximately in excess of 26% strains), which indicates that they are approaching the critical state. But these two samples formed distinct shear surfaces. It is therefore difficult to justify that these samples have truly reached the critical state. Other samples at high effective consolidation pressures (un300, un600 and un800) seem to be decreasing continuously until the end of the test and they also formed distinct shear surfaces. Therefore, these samples have not reached the critical state at the end of shearing.

Stress ratio versus strain curves

Head (1998) pointed out that the maximum stress ratio (q/p′) value does not necessarily occur at the same strain as the peak deviator stress. The maximum stress ratio criterion is preferable to the peak stress in some ways because it can provide a better correlation of shear strength with other parameters, or between different types of tests. It is particularly useful for clays in which the deviator stress continues to increase at larger strains. Atkinson (1993) mentioned that soils are frictional materials and their strength increases with normal stress and so the stress ratio is more important than the shear stress alone.

The q/p′ ratio versus strain graphs is shown in Figure 3. The maximum q/p′ ratios occurred at strains between 2.53% to 7.94%.

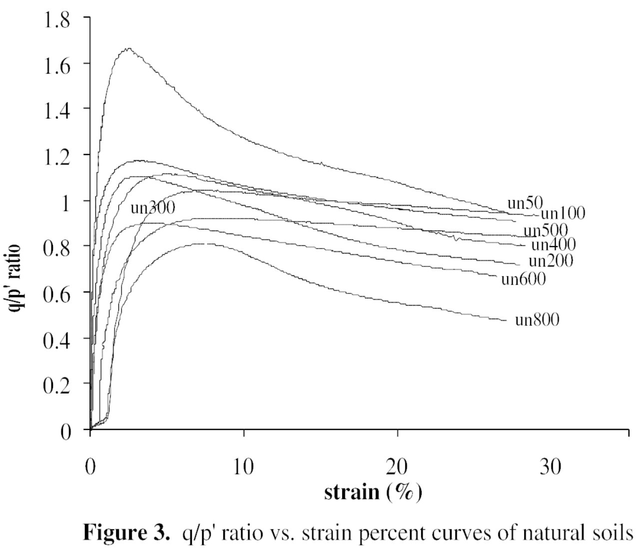

Figure 3. q/p' ratio vs. strain percent curves of natural soils

The maximum q/p′ values for these samples lie in between 0.81 to 1.66. The highest and lowest maximum stress ratio values for these samples are observed in test un50 and un800 respectively. After reaching the maximum q/p′ values at low strains, some of the samples reached a constant stress ratio state at strains greater than 22% (Figure 3). A variation in the maximum q/p′ ratio value is observed with the increase of effective confining pressures for these samples. In general, low effective consolidation pressure samples showed higher values of maximum q/p′ ratio value than the high effective consolidation pressure samples.

Effective Stress paths

The effective stress paths in q-p′ space for a series of consolidated undrained triaxial compression tests on natural state samples are shown in Figure 4. The stress paths at low effective consolidation pressures, 50 to 200 kPa, (Figure 4) initially rapidly and then show a tendency to move towards the right. At higher effective consolidation pressures, 300 to 800 kPa, each stress path at the beginning of each test, shows a tendency to move towards the left with an increase of deviator stress. The clear change of behaviour can also be observed in the excess p.w.p. versus strain curves as shown in Figure 2.

With the increase of mean effective stress and deviator stress, each stress path shows a tendency to move towards the right as there is a reduction in the rate of increasing pore water pressure when they are approaching the failure zone. In the failure zone, the stress paths stabilize for a while and show a tendency to move along the failure envelope. However, the stress paths then curve sharply down to approach an ultimate state (possibly the critical state line, CSL). The values of mean effective stress at maximum deviator stress and the corresponding maximum deviator stress for these stress paths are mentioned in Table 4. It can be seen from Figure 4 that the deviator stress at failure in each case increases with increasing mean effective stress except for test un300. The mean effective stress values at failure also increase with the increase of effective consolidation pressure.

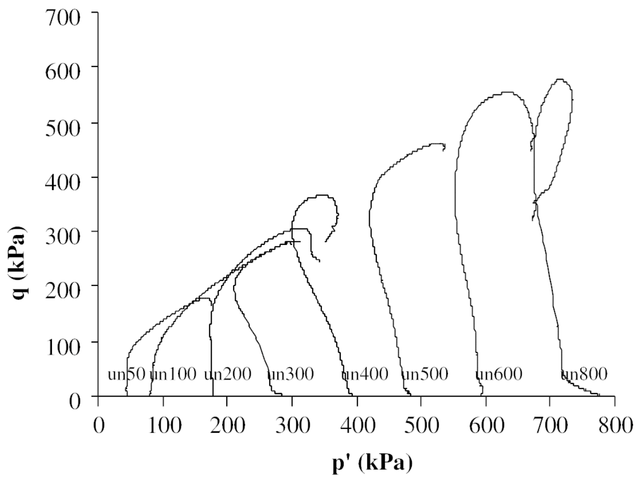

Figure 4. Effective Stress paths for natural soils

Failure surface

A failure envelope in terms of effective stress for a set of tests is plotted in the q-p′ space, as shown in Figure 5. It is clear that it shows curved failure envelope. All the samples failed with the formation of shear planes. At low effective consolidation pressures (50-200 kPa), the shear planes were not distinct. Samples un100 and un200 failed with a number of indistinct shear planes. However, at high effective consolidation pressures, samples un300 to un800, shear planes were more prominent and distinct and the samples failed along a distinct single shear plane.

The failure surface for the whole range of tests for these soils showed a curvature up to approximately p′=400 kPa. The slope of the curve reduces steadily as p′ increases and samples reached a maximum q/p′ ratio before failure followed by a decrease with increasing strain to the end of the test.

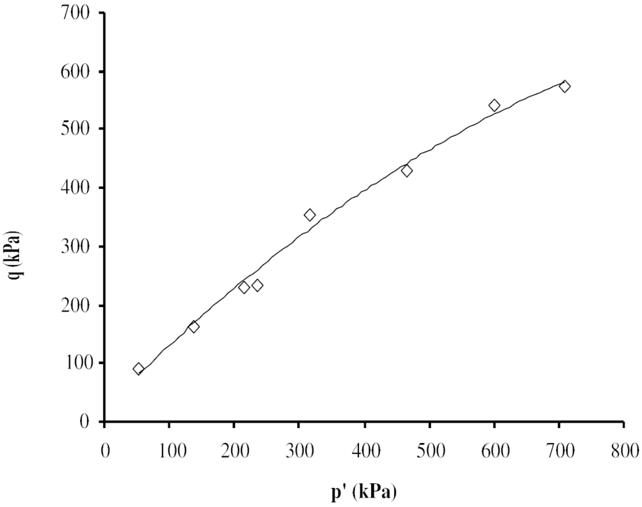

Figure 5. Failure surface for natural soils

Critical state behaviour

In this section undrained critical state behaviour of the tropical clay soils of Dhaka are discussed. Finally, the critical state parameters for these soils were estimated.

From the q/p′ ratio versus strain graphs (Figure 3) it was difficult to identify a single common ultimate stress ratio value for these soils. The values of maximum q/p′ ratio showed a range of variations. The maximum q/p′ values obtained lie in between 0.81 to 1.66. It is interesting to see that three natural samples at low effective consolidation pressures (50, 100 and 300 kPa) have a similar ultimate stress value around 0.96. It is important to note here that these low confining pressure samples did not form distinct shear surfaces; therefore it is possible that the ultimate stress ratio for these tests might represent the critical state. A true critical state was not achieved for these low stress tests, as pore water pressure continued to change even at large strains when the tests were terminated. However, most of the low stress tests showed that the rate of decrease of pore water pressure reducing towards the end of the test and they did not form distinct shear planes indicating that the critical state was being approached. It is also interesting to note that samples at low effective consolidation pressures, 50 to 200 kPa, developed negative p.w.p. at large strain due to a tendency to dilate.

On the other hand, samples at high confining pressures fail along distinct shear in a narrow band and the ‘overall’ stress ratio and volume change is no longer representative of the sample as a whole. It is difficult to justify that these samples have truly reached the critical state. This makes it difficult to construct a single unique CSL for these soils. Allman and Atkinson (1992) mentioned a value of M = 1.38 for the Bothkennar clay and noted that few intact samples of Bothkennar soil reached a reasonably well-defined constant ratio states at very large strains.

All the undrained results of natural soils are also presented in the v (specific volume) versus p′ space to see the change of specific volume with increasing mean effective stress. These results are also evaluated in terms of critical state. The v versus p′ curves for these soils is shown in Figure 6. By considering the starting and end points of each test it is difficult to construct a single critical state line for these samples. However, if greatest consideration is given to the low stress tests, up to 300 kPa, where it was observed that failure did not involve a single distinct shear plane, a trend in terms of changes in mean effective stress (p′) can be seen. For tests at 50, 100 and 200 kPa, p′ increases suggesting these samples start from a state that is denser than the critical state, which is due to the pseudo-overconsolidation nature of the soil. The test at 300 kPa shows p′ decreasing, indicating a state looser than the critical state which is due to the fabric, mineralogy, cementing and degrees of weathering. The pseudo-overconsolidation and bonded nature of these soils are discussed by Hossain (2001). This suggests that the CSL would fall between the end points for these four tests. Again, from Figure 6, it can also be seen that these soils at low effective consolidation pressures (50 to 200 kPa) also moved from left to right side with almost a constant specific volume. It was found difficult to establish a single critical state line for these samples as they do not end on a unique line due to the variations of the amount of cement, grading, packing & mineralogy of the samples. However, the constant stress ratio value (0.96) of samples from 1 m to 3.9 m, consolidated between 50 to 300 kPa, might represent the critical state as they do not form, distinct shear surfaces. These are the difficulties in defining critical state for these tropical residual soils.

On the other hand, the high effective consolidation pressure, samples 400 to 800 kPa, initially moved towards the assumed critical state line due to a tendency to contract that produces positive pore water pressures. That is they moved towards the assumed critical state line with a lower specific volume. The tests at 400-800 kPa (Figure 6) show an overall movement of an increase in p′, even though they are above the apparent CSL. This is probably due to the formation of distinct shear planes in these tests. Initially they do show a decrease in p′ consistent with the sketched CSL. The change in direction may represent the initiation of a distinct shear surface when the pore water pressures measured no longer indicate what is happening within the failure surface itself. However, as these soils at high confining pressures also formed distinct shear surfaces, it is difficult to establish with confidence that these samples reached the critical state at very large strains. It is therefore logical to consider only low stress tests (50-300 kPa) in defining critical state line.

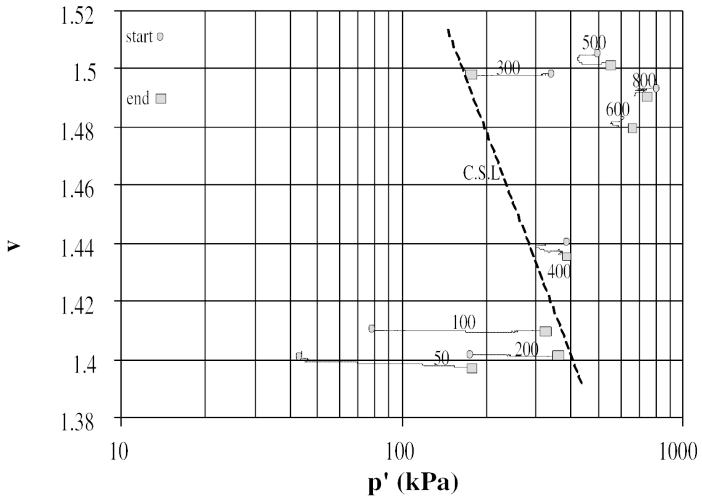

Figure 6. Specific volume vs. p' for natural soils

Atkinson (1993) pointed out that the critical state parameters for a particular soil are generally considered to be constant. The variation seen in the stress ratio values and also difficulties in defining the CSL from v versus p′ curves made it difficult to obtain typical critical state parameters for these soils. However, based on the all observed results on natural soils a rough estimation was made to obtain the critical state parameters for the tropical clay soils of Dhaka. The intrinsic critical state parameters critical state friction constant, M, isotropic normal compression line,λ, and ordinate critical state line, Γ, are listed in Table 5 and compared with the other values as quoted by Atkinson (1993) and Allman & Atkinson (1992).

Table 5. Comparison of obtained critical state parameters of tropical clay soils of Dhaka with some typical soils

Soil	λ	Γ	M
London clay*	0.16	2.45	0.89
Kaolin clay*	0.19	3.14	1.00
Glacial till*	0.09	1.81	1.18
Bothkennar clay+	0.18	2.78	1.38
Tropical clay, Dhaka	0.06	1.83	0.95-0.96

* Atkinson (1993)
\+ Allman and Atkinson (1992)

Atkinson (1993) mentioned that the intrinsic critical state parameters (M, λ and Γ) depend principally on the nature of the soil and might vary due to differences in grading and mineralogy from sample to sample. The critical state values (λ, Γ) obtained for the tropical clay soils of Dhaka are lower than the quoted values for some typical sedimentary clays. The tropical clay soils of Dhaka are of different nature from the other sedimentary and glacial clays as they are not formed by sedimentary processes but by weathering. The tropical clay soils of Dhaka are oxidized, coated with ferruginous cement and contain calcareous and iron nodules, and, therefore, it might be expected that the value of λ, which is related to the compressibility of the soil, would be lower than the other soils listed in Table 5.

CONCLUSIONS

Based on all the experimental results, the following conclusions may be summarized:

Undrained triaxial tests were carried out on samples collected from a borehole in the Curzon Hall area of Dhaka. In undrained shearing at low confining pressures, samples initially showed peak positive values of excess p.w.p. followed by negative values at higher strains due to the tendency to dilate of the samples. No negative pore pressures were observed for those samples consolidated at higher effective stress. An undrained failure surface for these soils is also identified. Effective stress paths for these soils are also derived. In the failure zone, the stress paths stabilize for a while and showed a tendency to move along the failure envelope.

Samples at low confining pressures did not involve a single distinct shear plane and they showed a common ultimate stress ratio value (0.96). These samples showed that the rate of decrease of p.w.p. reducing towards the end of the test indicating that the critical state was being approached at large strains by these samples. It was found difficult to establish with confidence a single critical state line for these tropical residual soils, as they do not end on a unique line due to the variations of grading, mineralogy, fabric and cementing materials. The variation seen in stress ratio values and also difficulties in defining critical state line from v versus p′ curves made it difficult to obtain typical critical state parameters for these tropical residual soils. The samples showed a wide range of ultimate stress ratio values. Few samples showed a common stress ratio value for different tests. Nevertheless value of the critical state stress ratio M was estimated to be 0.95-0.96. A critical state value of $\lambda = 0.06$ were estimated for these samples, and a value of $\Gamma = 1.83$ was estimated under undrained shearing. However, as there are difficulties in defining one critical state condition for these soils, these values are probably only representative of the shallower samples.

Acknowledgements: The work presented in this paper was carried out with funding from the Commonwealth Scholarship Commission, U.K. Thanks are due to the Commonwealth Scholarship Commission, U.K. for funding this research work.

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