Journal of Civil Engineering
The Institution of Engineers, Bangladesh
Vol. CE 31, No. 1, 2003

DIRECTIONAL DEPENDENCY OF STRENGTH IN COMPACTED DHAKA CLAY

Mohammad Shariful Islam[1] and Eqramul Hoque[1]

ABSTRACT: Direct shear (DS) and unconfined compression (UC) tests were performed in the laboratory to characterise anisotropy in the strength of compacted clay. For this purpose, the samples were reconstituted from air-dry grounded 'Dhaka clay' particles passing through #200 sieve. The samples were trimmed from a soil-cake of single mold at different orientations relative to its compaction direction. The orientation angle θ of a test specimen was defined to be zero if the specimen was trimmed from the mold while keeping the sampler axis vertical; that is, the major principal stress direction during shearing was parallel to the compaction direction (in case of UC test). Samples so extracted for $\theta=0^0$, 45^0 and 90^0 from a single mold were tested and the results were compared to each other so as to examine if strength was directionally dependent. Effects of compaction effort, molding moisture content and soaking on anisotropy were also investigated. It was observed that the reconstituted clay was noticeably anisotropic in strength. Compaction effort increased the anisotropy to some extent, but molding water content and soaking had little affects on the degree of anisotropy.

KEYWORDS: Anisotropy, compacted clay, direct shear, reconstitution, strength, unconfined compression

INTRODUCTION

Practical problems such as bearing capacity and slope stability of embankments are usually analysed based on the strength of a vertical specimen (S_1). That is, element test on a vertical specimen (i.e., compaction/pluviation direction is along the sampling direction) is performed and the soil mass is assumed isotropic, having strength S_1 in all directions. If the soil is anisotropic and assumed to be isotropic in the stability analysis (which is usually done), the stability index may increase or decrease relative to the actual value depending on the nature of anisotropy. If the ratio of vertical specimen strength (S_1) to horizontal specimen strength (S_2) is greater than unity (i.e., $S_1/S_2 > 1.0$), the stability number will be decreased. Therefore, the embankment slope will be in the more unsafe side than that indicated by the estimated value based on isotropic strength characteristics, and vice versa (Duncan and Seed, 1966; Toh and Donald, 1979; Su and Liao, 1999). Besides, Lo (1965) computed stability index of slope for a practical range of coefficient of anisotropy (i.e., S_1/S_2) and the slope angle, and concluded that for steep slopes, the effect of anisotropy is small. However, for flatter slopes, the influence of anisotropy on the stability conditions is significant.

---

[1] Department of Civil Engineering, BUET, Dhaka-1000, Bangladesh

Embankments are necessary for roads in marshy and waterlogged areas, for approaches to bridges, for railways and for water retaining structures like earth dams, reservoir bunds, etc. In the construction of such engineering structures loose fills are compacted to increase the soil density, to improve their strength characteristics, and to decrease the permeability. Although compacted clays are extensively used for soil structures, literature survey showed that for simplicity the role of anisotropy is not generally considered in the stability analysis of slopes. Seed and Chan (1961), Rao et al. (1982) and Hoque and Islam (2000) had reported a few works in this regard.

Micro-mechanical studies on particles of both cohesionless and cohesive soils reveal anisotropic distribution of particle-orientation in a soil mass (Oda, 1972; Yoshinaka and Kazama, 1973). X-ray diffraction analysis of soil particles showed that platy and elongated clay minerals tend to align their faces perpendicular to the direction of consolidation, and thereby soils become inherently anisotropic (e.g., Martin, 1962; Kazama, 1996). This anisotropic stage can be further modified during the successive stages of shearing to develop a different anisotropic micro-structure called the induced anisotropy (Kurukulasuriya et al., 1999). Besides, isotropic soils (in the sense of microstructure) respond anisotropically to stress change under an undrained condition (Duncan and Seed, 1966; Ohta and Nishihara, 1985); this anisotropy is called the stress-induced anisotropy. Micro-mechanical studies, therefore, reveal that both cohesive and cohesionless soils may behave anisotropically in strength and/or deformation characteristics due to depositional (compaction or pluviation) and stress-strain histories, as well as current stress-states. This finding has been reported from experimental studies of several works on both types of soils [e.g., Ohta and Nishihara (1985), Hoque and Tatsuoka (1998), Kurukulasuriya et al. (1999)].

This paper describes the results of unconfined compression (UC) and direct shear (DS) tests designed to quantify the directional variations, if any, in undrained strength of soil. Attempts have been made to evaluate the effects of moisture content at the time of compaction, the effect of compaction effort and soaking on undrained shear strength.

DHAKA CLAY

Dhaka stands on the southern part of Modhupurgar at an elevation of 7m to 9m above the mean sea level. In general, the top layer of subsoil that extends up to a depth of 7m to 8m is typical Dhaka clay. It is, in fact, a mixture of silt and clay, reddish in color, and is formed by older Pliestocene sediments. The Pliestocene sediments are flood plain deposits of earlier Ganges and Brahmmaputra. The consistency of Dhaka clay is medium to stiff and it is over-consolidated. The optimum moisture content (w_{opt}) and the maximum dry density (by Standard Proctor test) of the clay are 22% and 1.65 gm/cm^3, respectively. Deposits of sand and gravel occur at relatively deeper horizons with a sequence of finer material at the top and coarser materials downward.

SAMPLE PREPARATION

Reconstituted clay was prepared from Dhaka clay in the laboratory. Physical properties of this clay are LL= 46%, PL= 20%, PI= 26% and G_s= 2.60. The clay was collected by 'block sampling' technique from Chamelibag, Dhaka from a depth of 4.5m during the excavation for foundation of the proposed Concord-Grand Tower project. The natural clay was cleaned first from unwanted materials (such as, brick chip, stone, grass etc.), broken into small pieces, air-dried, reduced into powder by grinding mechanically, and then passed through #200 sieve. Materials that passed-through the sieve were mixed with specific amount of water. Uniform-consistency was ascertained by mechanical mixing for at least 30 minutes. The paste in plastic state was poured layer-by-layer into a standard CBR mold. After pouring each layer of finite depth, the cap was inserted into the mold on to the clay. A specific number of blows with a particular hammer were applied. This process was continued until the desired depth of clay was achieved in the predetermined number of layers. The reconstituted mass was then ready for sampling the test specimen.

The groups of samples reconstituted for tests are listed in Table 1, together with the physical variables during reconstitution of compacted mass. The compaction effort was varied by changing weight and height of fall of the hammer and the number of blows applied per layer (Table 1). To verify the reproducibility of the test results, more than one molded-specimen was reconstituted to represent a given group of sample (e.g., CM1, CM2, etc.). For unconfined compression (UC) tests, four molded-specimens were reconstituted for each CM1, CM2 and CM3 groups and two for each MCM0, MCM1 and MCM2 groups. On the other hand, for direct shear tests, two molded-specimens were prepared for each of the CM1, CM2 and CM3 groups. The molding moisture content was mostly the optimum moisture content (w_{opt}) determined by Standard Proctor energy, except for the specimens of MCM1 and MCM2 groups. In the latter cases, moisture content was w_{opt}+4% 'wet of optimum moisture content' and w_{opt}-4% 'dry of optimum moisture content', respectively.

TEST RESULTS

Three test specimens were extracted from cylindrical-cake of each molded specimen at orientation angles θ=0°, 45° and 90° in a vertical plane (inset of Fig. 1a and 1b). Each specimen, 38-mm diameter and 76-mm high for UC test, and 63.5-mm diameter and 25.4-mm high for DS test, was subjected to the respective test. The DS tests were performed at a normal stress (σ_n) of 100 kPa. Figures 1a and b show typical stress-strain relationships, respectively, for a UC and DS tests. Peak stress was measured from the axial stress and axial strain relationship of UC test (e.g., Fig. 1a), and from shear stress and shear strain relationship of DS test (e.g., Fig. 1b). If a definite peak was not observed (as in the case of Fig. 1b), peak stress was defined as the one that occurred at 20% of strain (i.e., axial and shear strains, respectively, in case of UC tests and DS test). The relationships between stresses and strains for θ=0°, 45° and 90° specimens retrieved from a single mold were not similar (Figs. 1a and b). Rather they exhibited consistently dissimilar peak values as well as deformations at various strain levels. The UC and DS test results are described in the following sections.

Table 1. List of sample groups and test variables

Sample Groups	Molding Moisture Content (%)	Mold Size (mm)	Hammer Weight (N)	No. of Layer	No. of Blows/ Layer	Height of Fall (mm)
CM1	w_{opt}	D=117, H=102	44.4	3	25	305
CM2	w_{opt}	D=127, H=152	44.4	3	25	305
CM3	w_{opt}	D=127, H=152	88.96	5	55	457
MCM0	w_{opt}	D=127, H=152	44.4	3	55	357
MCM1	w_{opt}+4	D=127, H=152	44.4	3	55	357
MCM2	w_{opt}-4	D=127, H=152	44.4	3	55	357

H= Height of the mold; D= Diameter of the mold; w_{opt} = Optimum moisture content

UC TEST RESULTS

Physical properties of the tested specimens in UC tests are listed in Table 2. As mentioned earlier, more than one molded specimens were reconstituted for a particular sample group (e.g., CM1, CM2, etc.). In Table 2, the statistical mean values, together with the standard deviation, of each group, are listed. For example, the CM1 group consisted of four molded samples; three test samples were extracted from each of the molded samples. Thus, the listed values of dry density (γ_d) and water content (w) for CM1 group samples are those obtained by averaging the respective values of 12 (4x3) test specimens. The values of γ_d and w were more-or-less uniform within a sample group; mean value of γ_d varied from ±0.007 to ±0.022 gm/cm^3, while that of w varied from ±0.13% to ±0.58%. In Table 3, the compressive strength q and the coefficient of anisotropy q_{uv}/q_{uh} and q_{uv}/q_{u45} (q_{uv}, q_{uh} and q_{u45} were the q values for θ = 0°, 90^0 and 45^0 specimens, respectively) are listed. In these cases, statistical mean value of the samples in a group was calculated based on the number of data available for a test sample of a given orientation. The mean values of these quantities varied within a reasonable range (Table 3).

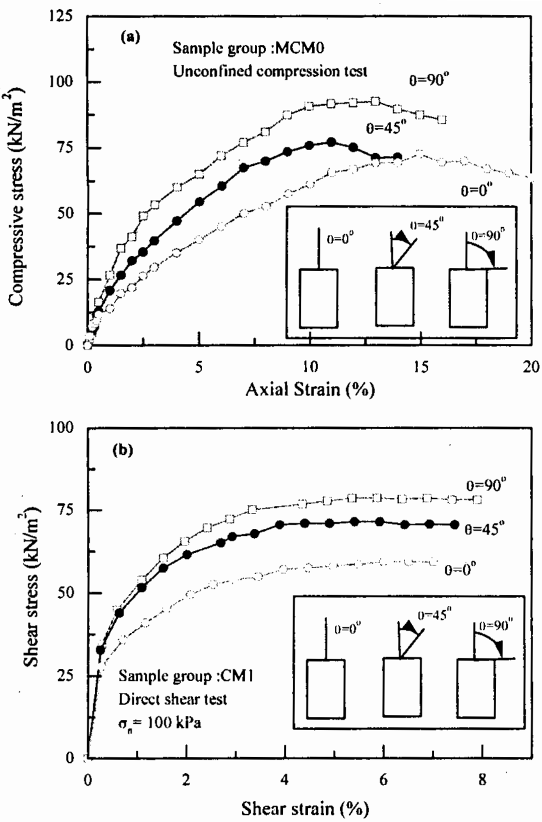

Fig. 1. Typical stress-strain relationships for (a) UC and (b) DC tests

Table 2. Physical properties of tested specimens in UC test

Sample Group	No. of Samples	Dry Density γ_d (gm/cm³)	Water Content w (%)
CM1	4	1.6010 ±0.022	21.95 ±0.58
CM2	4	1.5060 ±0.022	21.93 ±0.50
CM3	4	2.0300 ±0.020	20.88 ±0.27
MCM0	2	1.5300 ±0.009	23.18 ±0.51
MCM1	2	1.3360 ±0.011	27.92 ±0.13
MCM2	2	1.4150 ±0.007	18.16 ±0.22

The relationships between q and θ obtained from UC tests are shown in Figs. 2a and b. Figure 2a shows q~θ variations of sample groups CM1, CM2, CM3 and MCM0. Compaction effort varied among these sample groups (Table 1). Note that mold size of CM1 group was smaller. From the analysis of test results, a number of trends of behaviour could be observed. The maximum strength occurred mostly in a horizontal specimen (i.e., a θ=90⁰ specimen) except in case of CM1 group where the peak occurred in a θ=45⁰ specimen. The minimum q occurred mostly in vertical specimens (i.e., θ=0⁰), except in CM3 group, where the minimum occurred in a θ=45⁰ specimen. The coefficients of anisotropy q_{uv}/q_{uh} and q_{uv}/q_{u45} were 0.79 and 0.74, 0.75 and 0.87, 0.79 and 0.94, and 0.84 and 1.23, respectively, for specimen groups CM1, CM2, MCM0 and CM3. However, compaction effort (i.e., energy imparted) during reconstitution of samples were in ascending order of CM1, CM2, MCM0 and CM3 (Table 1). It seems, therefore, that compaction effort increased the strength anisotropy of clay. However, the effect was not significant. However, it is clear that the stability analysis of a slope that is made of compacted clay will be conservative if the UC results for the vertical specimens are used without considering the anisotropy.

Table 3. Results of unconfined compression tests

Sample Group	Peak Strength, q (kN/m²)			Coefficient of anisotropy	
	θ = 0⁰	θ = 45⁰	θ = 90⁰	q_{uv}/q_{uh}	q_{uv}/q_{u45}
CM1	78.9 ±1.68	107 ±4.6	100 ±4.3	0.790 ±0.02	0.740 ±0.04
CM2	72.0 ±2.65	82.6 ±5.5	95.7 ±7.32	0.750 ±0.07	0.870 ±0.10
CM3	109 ±2.85	88.0 ±1.0	130 ±2.55	0.840 ±0.05	1.230 ±0.05
MCM0	70.8 ±1.82	75.0 ±1.53	90 ±3.1	0.790 ±0.05	0.940 ±0.05
MCM1	30.6 ±2.10	30.0 ±1.7	34.4 ±1.65	0.890 ±0.02	1.000 ±0.03
MCM2	154 ±1.61	121 ±4.0	159 ±5.75	0.970 ±0.07	1.280 ±0.06

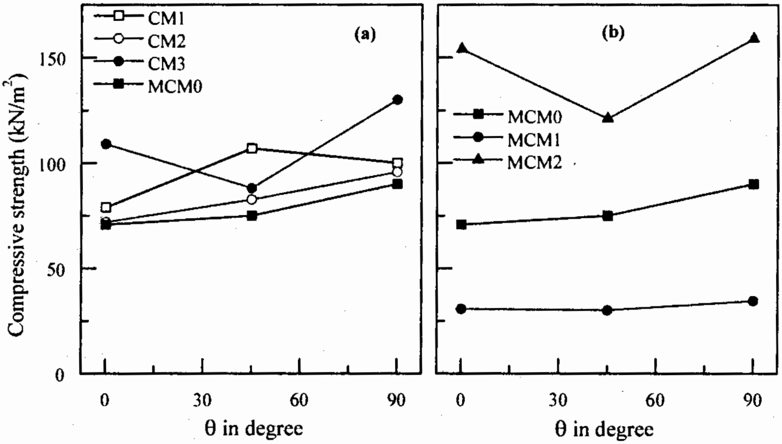

Fig. 2. q~θ *relationships for (a) different compaction efforts, and (b) different molding moisture content*

The effects of moisture content on anisotropy were investigated by performing similar tests on specimen groups MCM0, MCM1 and MCM2; moisture content was w_{opt}, w_{opt}+4% and w_{opt}-4%, respectively. The coefficients of anisotropy of q_{uv}/q_{uh} and q_{uv}/q_{u45} were 0.97 and 1.28, 0.79 and 0.94 and 0.89 and 1.0 for the groups MCM2, MCM0 and MCM1, respectively. Figure 2b shows the corresponding q~θ relationship. The strength (Fig. 2b) and the coefficients of anisotropy (Table 3) were higher for the lower moisture content though all the samples were prepared with same energy in an identical mold. The peak stress was attained in MCM2 specimens at strains lower than 5%. The failure was brittle type as the stress-strain curves suffered sharp decrease after attaining the peak stress (Islam, 1999). At 'dry of optimum' moisture content (i.e., w_{opt}-4%), the compacted clays became flocculated-structure in fabric and hence exhibited higher strength. However, the structure pattern might not be similar in all direction, resulting in more anisotropy. Seed and Chan (1961) also reported similar results. On the other hand, the MCM1 specimens exhibited much lower strength as well as the degree of anisotropy. The lower strength could be due to the fact that at 'wet of optimum' moisture content (i.e., w_{opt}+4%), clay possessed more disperse-structured fabric, and compaction may not have brought about substantial changes in fabric of clays having higher moisture content than w_{opt} (Baracos, 1977).

DS TEST RESULTS

Results of unconsolidated undrained direct shear (DS) tests are listed in Tables 4 and 5. Table 4 describes the physical properties of the tested specimens in DS tests. The test results of DS tests are presented in the Table 5. Similar DS tests were also conducted on soaked-samples. Soaked samples were prepared to simulate the submerged condition (during flood) of compacted clay. For this purpose, a specimen enclosed within a mold was immersed into water under 25.4-mm water-head for 96 hours (4 days). During soaking, the surcharge axial load was 10-lb. Test specimens were retrieved in a manner similar to that used for the UC test samples, for both soaked and unsoaked conditions. The values of γ_d and w were more-or-less similar within a sample group, such as CM1, CM2 and CM3 (Table 4).

The relationships between the shear strength τ_m and θ are shown in Figs. 3a and b for unsoaked and soaked samples, respectively. Figure 3a shows that the horizontally oriented specimens (CM1 type) exhibited the maximum strength, whereas the vertically oriented specimens exhibited the lowest. The specimens trimmed at 45° showed intermediate strength. Different trend was observed in the results of CM2 type specimens; the horizontal and the vertical specimens showed the similar shear strengths, which were the maximum, and the specimen trimmed at 45° showed the minimum shear strength. In case of CM3 type specimens, the horizontally oriented specimens exhibited the minimum strength, the θ=45° oriented specimens exhibited the maximum strength and the vertical specimens showed strength close to those of θ=90° specimens. Strength characteristics of soaked samples were rather consistent to each other. The undrained strength was the minimum for vertical specimens, while the maximum values were recorded for horizontal specimens.

Table 4. Physical properties of tested specimens in DS test

Sample Group	Pre-test Condition	Dry Density γ_d (gm/cm^3)	Water Content w (%)
CM1	Un-soaked	1.589 ±0.0.016	21.40 ±0.26
CM2		1.607 ±0.013	22.17 ±0.28
CM3		1.763 ±0.023	20.95 ±0.43
CM1	Soaked	1.596 ±0.020	21.45 ±0.24
CM2		1.591 ±0.016	21.98 ±0.13
CM3		1.787 ±0.021	20.55 ±0.26

Table 5. Results of direct shear tests

Sample Group	Pre-test Condition	Shear Strength, τ_m (kN/m^2)			Coefficient of Anisotropy	
		$\theta = 0^0$	$\theta = 45^0$	$\theta = 90^0$	τ_{mv}/τ_{mh}	τ_{mv}/τ_{m45}
CM1	Un-soaked	61.55 ±2.25	72.4 ±0.9	78.2 ±0.4	0.79 ±0.03	0.85 ±0.02
CM2		86.95 ±0.65	73.5 ±1.5	85.8 ±0.4	1.02 ±0.005	1.19 ±0.04
CM3		93.05 ±1.35	100.85 ±1.55	75.55 ±1.45	1.24 ±0.015	0.92 ±0.02
CM1	Soaked	37.05 ±0.65	41.9 ±0.8	62.45 ±0.95	0.59 ±0.05	0.88 ±0.01
CM2		43.75 ±0.25	41.85 ±0.85	48.5 ±0.5	0.90 ±0.02	1.05 ±0.02
CM3		51.85 ±0.15	69.75 ±0.75	80.9 ±0.9	0.64 ±0.01	0.745 ±0.005

The coefficients of strength anisotropy in direct shear specimens, defined as τ_{mv}/τ_{mh} and τ_{mv}/τ_{m45}, for each molded specimens were calculated from the respective undrained strengths, such as $\tau_{mv} = (\tau_m)_{\theta=0^\circ}$, $\tau_{mh} = (\tau_m)_{\theta=90^\circ}$, and $\tau_{m45} = (\tau_m)_{\theta=45^\circ}$, and the quantities are listed in Table 5. For unsoaked samples, the coefficients τ_{mv}/τ_{mh} and τ_{mv}/τ_{m45} were 0.79 and 0.85, 1.02 and 1.19, and 1.24 and 0.92 for sample groups CM1, CM2 and CM3, respectively. On the other hand, for soaked-samples, the coefficient values were comparatively scattered in nature. In both soaked and unsoaked cases, the CM3 group specimens exhibited more anisotropic strength, which might be due to high compaction energy imparted during the reconstitution of clay. This fact was also clarified in UC test.

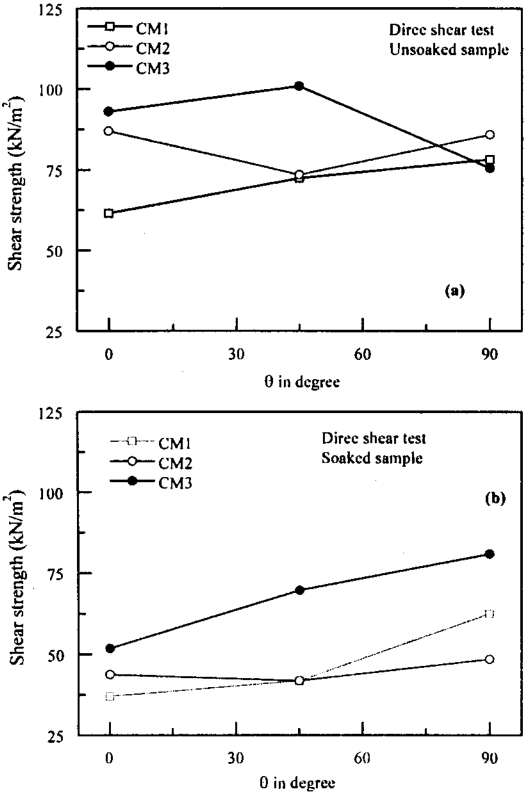

Fig. 3. τ_m-θ *relationships from DS tests for: (a) unsoaked specimens, and (b) soaked specimens*

COMPARISON BETWEEN UC AND DS TEST RESULTS

The deformation characteristics, i.e., the stress-strain relationships of specimens for θ= 0°, 45° and 90° (retrieved from a single mold) show significant directional dependency (Figs. 1a and 1b). The trend was clearly observed in all the molded specimen groups. However, the coefficients of anisotropy for a particular direction in case of UC (Table 3) and DS (Table 5) tests were not the same. This may be attributed to the fact that the definition of the angle θ is same for both the tests but the failure plane is not the same for the tests. This fact should be considered in applying these test results in the analysis of structures that are made of compacted clay. It may be noted that the directional dependency of strength and deformations does not appear to be due to the changes in the moisture content or due to the change in dry densities within a single mold.

CONCLUSIONS

Strength of reconstituted Dhaka clay was directionally dependent (i.e., anisotropic). The existence of anisotropy was clearly noticeable in compacted clays irrespective of molding moisture content and compaction effort. The coefficient of anisotropy S_1/S_2 (≈ strength of vertical specimen /strength of horizontal specimen) varied from 0.74 to 1.28 in unconfined compression and direct shear (unsoaked) tests, while it was between 0.59 to 1.05 in case of direct shear tests on soaked samples. Therefore, it indicates that the coefficient of anisotropy depends on the condition of the material whether it is soaked or unsoaked.

The clay was observed to become more anisotropic with the increase of compaction effort during reconstitution. However, the rate of increase with compaction effort was not significant. Molding moisture content also affected anisotropic behaviour. The test results indicate that the effect of anisotropy should be considered in the stability analysis of structures that are made of compacted clay for safe and economic design.

REFERENCES

Baracos, A. (1977), "Compositional and structural anisotropy of Winnipeg soils— a study based on scanning electronic microcsopy and X-ray diffraction analysis," Canadian Geotechnical Journal, Vol. 14, No. 1, pp. 125-143.

Duncan, J.M. and Seed, H.B. (1966), "Anisotropy and stress reorientation in clays," Journal of SMFE Div., ASCE, Vol. 92, No. SM5, pp. 21-50.

Hoque, E. and Islam, M.S. [2000], "Anisotropy in strength of Dhaka-clay," International Conference on Geotechnical and Geological Engineering, GeoEng 2000, Australia.

Hoque, E. and Tatsuoka, F. (1998), "Anisotropy in elastic deformation of granular materials," Soils and Foundations, Vol. 38, No. 1, pp. 163-179.

Islam, M.S. (1999), "Strength and deformation anisotropy of clays," M.Sc. Engg. thesis, Bangladesh University of Engineering and Technology, Dhaka.

Kazama, H. (1996), "Study on microstructure of clayey soils and their effects on their consolidation and shear behaviours," Doctoral thesis, Saitama University, Japan.

Kurukulasuriya, L.C., Oda, M. and Kazama, H. (1999), "Anisotropy of undrained shear strength of an over-consolidated soil by triaxial and plain strain tests," Soils and Foundations, Vol. 39-1, pp. 21-29.

Lo, K.Y. (1965), "Stability of slopes in anisotropic soils," Proc. ASCE., Vol. 91, No. 3, SM4, pp. 85-106.

Martin, T. R. (1962), "Research on the physical properties of marine soils, August 1961-July 1962," Research report R62-42, Soil Engrg Div.

Publication No. 127, Massachusetts Inst. of Tech., Cambridge.

Oda, M. (1972), “Initial fabrics and their relations to mechanical properties of granular materials,” Soils and Foundations, Vol. 12-1, pp.17-36.

Ohta, H. and Nishihara, A. (1985), “Anisotropy of undrained shear strength of clays under axi-symmetric loading condition,” Soils and Foundations, Vol. 25-2, pp. 73-86.

Rao, E.S., John, K.A. and Bhagwan, J. (1982), "Influence of anisotropy in compacted clays as applicable to the stability analysis of slopes," Highway Research Bulletin, No. 17, pp. 1-43.

Seed, H.B. and Chan, C.K. (1961), "Structure and strength characteristics of compacted clays," Trans. ASCE 126, pp. 1344-1385.

Su, S.F. and Liao, H.J. (1999), "Effect of strength anisotropy on undrained slope stability in clay," Geotechnique, Vol. 49, No. 2, pp. 215-230.

Toh, C.T. and Donald, I.B. (1979), "Anisotropy of an alluvial clay," Proc. 6th Asian Regional Conference on Soil Mechanics and Foundation Engineering, Vol. I, pp. 99-106.

Yoshinaka, R. and Kazama, H. (1973), “Micro-structure of compacted kaolin clay,” Soils and Foundations, Vol. 13-2, pp. 19-33.