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Effect of Stress Release on the Strength of Coastal Soils

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Introduction

Several coastal regions of Bangladesh have recently been affected by severe cyclones and storm surges which produced quite significant damage to livestock, agriculture, power system, telecommunication, housing and other physical infrastructure facilities. Flood protection embankments, retaining structures and cyclone shelters are essentially needed to be constructed in large numbers in these coastal regions. For geotechnical design of these structures requires knowledge of the engineering properties of the foundation soils which are in general silty clays of low to high plasticity. The behaviour of the ground and foundations are predicted on the basis of soil parameters obtained from the laboratory investigation of the sampled soil. However, sampling introduces two major difficulties, both associated with the disturbance that a sample experiences before being transported to the laboratory where it is tested. Firstly, mechanical disturbance is caused due to deep penetration of sampler into the clay. This produces shear distortion and subsequent compression of clay close to the inside wall of the sampler. The second source of disturbance is caused as a result of stress relief due to removal of the sample from the ground to zero total stress state in the laboratory. The first source of disturbance is directly associated with sampler design and can be controlled to certain extent. Disturbance due to stress release, however, is unavoidable even though its effects may be different depending on the depth of sampling and soil properties. In order to understand the effects of stress relief on the undrained shear characteristics of clays, a number of researchers (Skempton and Sowa, 1963; Ladd and Lambe, 1963; Hight et al., 1985) have idealized the process of stress release

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in the laboratory either by undrained release of the total deviator stress to zero from an in situ anisotropic condition, but maintaining an isotropic total stress state. Others, however, simulated stress relief by unloading both the deviator stress and isotropic stresses to zero, i.e., by reducing the total stresses to zero (Noorany and Seed, 1965; Davis and Poulos, 1967; Kirkpatrick and Khan, 1984; Kirkpatrick et al., 1986; Graham and Lau, 1988; Sarker, 1994). A summary of the effects of stress relief on some engineering properties of various clays is presented in Table 1. All of the clays studied were laboratory prepared reconstituted clays apart from those tested by Noorany and Seed (1965). In Table 1 all ratios refer to results from "in situ" samples. It can be

TABLE 1 : Summary of the Effects of Stress Release Disturbance on Some Engineering Properties of Various Clays

Soil	Index Values	OCR	Ratio of Su	Ratio of ep	Ratio of Ei	Reference
Weald Clay	LL = 46	1.0	0.98*	1.29†	-	Skempton and Sowa (1963)
	PI = 24	2.0	1.03	0.88	-	
		14.0	1.08	-	-	
Kaolin	PI = 30	1.0	0.53	2.75	0.48	Kirkpatrick and Khan (1984)
Illite	PI = 40	1.0	0.64	3.50	0.52	
Kaolin	PI = 30	2.0	0.39	1.75	0.32	Kirkpatrick et al. (1986)
Illite	PI = 40	2.7	0.50	2.50	0.27	
Illite	PI = 40	5.0	0.86	1.10	0.94	
North Sea Clay	LL = 32	1.0	0.72	8.00	1.19	High, Gens and Jardine (1985)
	PI = 17	7.4	0.96	1.00	0.47	
Soft Clay	LL = 88 PI = 45	1.0	0.95**	1.05**	0.9**	Noorany and Seed (1965)
Kaolin	LL = 55 PI = 22	1.0	0.81	-	-	Davis and Poulos (1967)
Boston Blue Clay	LL = 33 PI = 15	1.0	0.93	2.5	-	Ladd and Varallyay (1965)
Dhaka Clay	LL = 45 PI = 23	1.0	0.97	1.16	1.67	Sarker (1994)

* Average of five samples
† Average of four samples
** Average of two samples

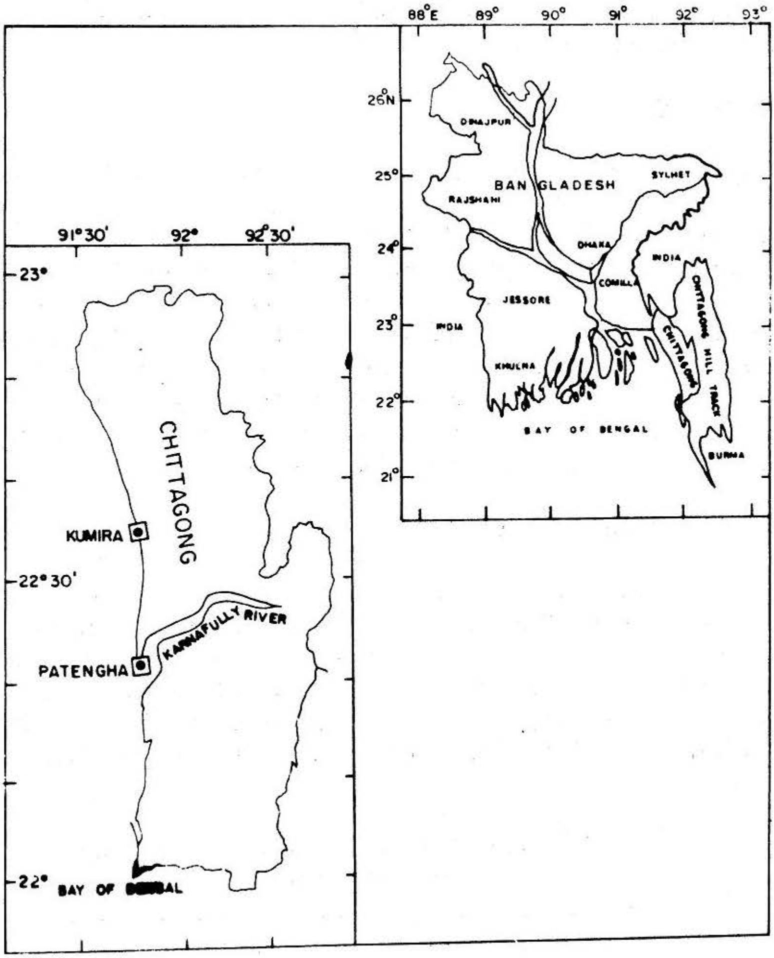

FIGURE 1 : Map of Bangladesh Showing the Location of Coastal Sites for Sample Collection

seen from Table 1 that, in general, the effects include reduction in undrained strength and stiffness, and an increase in strain at peak deviator stress. Most of the previous works have concentrated only on the effects of stress release disturbance on the engineering properties of the clays. Attention must also be directed towards techniques for minimizing the effects of stress release by recovering in the laboratory the strengths and stiffness of the in insitu soil.

This paper presents further investigation into the effects of stress release disturbance on the undrained stress-strain-strength, stiffness and pore pressure characteristics of two clays collected from the Chittagong coastal region. The work has also been aimed to examine different reconsolidation techniques, both isotropic and anisotropic, in order to minimize the effects of stress release disturbance on these clays.

Soils Used

Disturbed soils from two coastal sites, namely Patengha and Kumira, situated along the coastal belt of Chittagong, were collected for the present investigation. The location of these sites are shown in Fig. 1. Sampling was carried out according to standard procedure. For each location, approximately 2 m × 2 m area was excavated to a depth of 2 m to 3 m, using hand shovels. Disturbed samples were collected from the bottom of the borrow pit through excavation by hand shovels. All samples were packed in large polythene bags and were eventually transported to the laboratory. Index properties of the soils were determined in order to characterize the soil. Table 2 shows the index properties and classification of the soils studied.

Geology of the Study Area

The coastal zones of Bangladesh falls in a deepest part of Bengal Basin, known as the Patuakhali trough, which occupies the Hatiya, Barisal, and Faridpur areas and has sediments more than 18,000 m thick. The Chittagong Coastal Plain comprises the generally narrow strip of land between the Chittagong hills and the sea, together with the Halda, the Karnafully and the Sangu floodplains and the offshore islands. This is known

TABLE 2 Index Properties and Classification of the Coastal Soils Studied

Index Properties and Classification	Location	
	Patengha Chittagong	Kumira 'Chittagong
Specific Gravity	2.78	2.75
Liquid Limit	44	57
Plastic Limit	26	24
Plasticity Index	18	33
% Sand	2	0
% Silt	61	58
% Clay	37	42
Group Symbol	CL	CH

to be occupied by gently sloping piedmont alluvial fans with mainly loamy soils. Tidal clay plains occupy most of the offshore island. The fan deposits from the Chittagong hills and deposits of coastal currents are mixed in a complicated manner in the Chittagong coastal area. The geological formations and soil characteristics of this area are very complicated due to the multifold shallow bedrock of the above hills.

Preparation of Reconstituted Sample

Reconstituted soils are those which are prepared by breaking down natural soils, mixing them as slurry and consolidating them. Reconstituted soil enables a general pattern of behaviour to be established and comparisons with the response of intact samples may be used to identify and special features associated with fabric, stress history or bonding. The major advantages of using data from reconstituted soils are that the ambiguous and substantial effects of sample inhomogeneity can be eliminated, while the essential history and composition of in-situ soils can be represented. Jardine (1985) discussed the difficulties of implementating detailed investigations of general stress- strain and strength properties using intact samples and it was found that the most comprehensive studied invariably employed reconstituted soil. However, the disadvantages of using reconstituted soil are that the important effect of post-depositional process, such as ageing, leaching, etc. and of variations of composition and fabric are not included. So the pattern of behaviour for the reconstituted coastal samples discussed in the following articles will be taken to represent that of a young or unaged soils where no post-depositional processes have operated.

Reconstituted samples of the two coastal soils were prepared in the laboratory by K_0 – consolidation of a uniform slurry of the clay in a cylindrical consolidation cell of 260 mm diameter and 305 mm in height. The slurry had a water content of approximately 1.5 times the liquid limit of the soil. A consolidation pressure of 100 kN/m^2 was used so as to get a clay of soft consistency. Water contents of the consolidated samples from Patengha and Kumira were $34 \pm 1\%$ and $36 \pm 1\%$ respectively and the respective values of bulk density were 18 ± 0.04 kN/m^3 and $18 + 0.02$ kN/m^3.

Equipment and Instrumentation

For the determination of undrained shear properties of the samples a triaxial apparatus together with volume change and pore pressure measuring devices were used. A 38 mm typical Soil test triaxial machine was used for compression test. The cell had the facility of drainage through both top and bottom of the sample. Cell pressure was applied using a standard pressure gauge of operating range of 0 to 1700 kN/m^2. Back pressure was applied using dash pot and control cylinder system. For measuring axial deformation, strain gauge with a resolution of 0.0254 mm was used. A Bell and Howell pore water pressure transducer of operating range of 0 - 1034 kN/m^2 has been used to monitor pore pressure. A burrette system (Bishop and Donald, 1961) was used for measuring volume change during consolidation.

Types of Test Samples

In Situ Sample

The soil cake prepared by K_0 – consolidation was extruded from the consolidation cell. The cake was sliced by the wire knife into small blocks and samples of nominal dimensions of 38 mm diameter by 76 mm high was prepared by trimming a block sample using piano wire, a soil lathe and a split mould. These samples were consolidated under K_0 – conditions (K_0 = 0.49 and 0.52 for the clays from Patengha and Kumira respectively) in the triaxial cell to its in situ vertical effective stress, σ_{vc}' (i.e., 100 kN/m^2). A back pressure of 270 kN/m^2 has been used during K_0 – consolidation of the samples. These samples have been termed as in situ samples. The in situ samples prepared from the soils from Patengha and Kumira have been designated as PI and KI respectively.

"Perfect" Sample

This type of sample was prepared from in situ sample in the triaxial cell. The in situ shear stress, i.e., deviatoric stress of the in situ sample was first released from its in situ anisotropic stress condition. At this stage, the sample was subjected to an allround isotropic stress (i.e., cell pressure). The cell pressure was then reduced to zero and thereby the sample was subjected to zero total stress. This sample has been termed as "perfect" samples prepared from the soils from Patengha and Kumira have been designated as PP and KP respectively.

Testing Programme

The best programme consisted of carrying out the following three types of tests :

(1) Firstly, undrained triaxial compression tests on the two in situ samples were performed in order to determine the reference undisturbed behaviour of the clays. In this tests, after the completion of K_0 – consolidation, each sample was sheared in compression under undrained condition at a deformation rate of 0.025 mm/minute.

(2) Secondly, unconsolidated undrained triaxial compression tests were carried out on the two "perfect" samples. In these tests, soon after

simulation of the release of the total in situ stresses, each sample was subjected to a total isotropic stress (i.e., allround cell pressure) equal to in situ effective vertical stress under undrained condition. When the pore water pressure became steady, each sample was then sheared in compression under undrained condition at a deformation rate of 0.025 mm/min.

(3) Finally, undrained triaxial compression tests were run on six reconsolidated "perfect" samples, three tests on sample from each location. In these tests, after the completion of reconsolidation, the samples were sheared at a deformation rate of 0.025 mm/minute in compression under undrained condition. The following two reconsolidation methods were used.

(i) Isotropic reconsolidation: The hydrostatic consolidation stress is equal to initial mean effective stress, Po of the in situ sample. A back pressure of 270 kN/m^2 has been used during isotropic consolidation of the sample.

(ii) Anisotropic reconsolidation using SHANSEP (Ladd and Foott, 1974) procedure : samples were reconsolidated anisotropically using SHANSEP (Ladd and Foott, 1974) procedure to vertical effective stresses equal to 1.5 and 2.5 times the effective vertical stress (svc) of the in situ sample. A back pressure of 270 kN/m^2 has been used during K_o – consolidation of the samples.

Results and Discussion

Influence of Stress Release Disturbance

Changes in Effective Stress Paths

A comparison of the effective stress paths, in $s'-t'$ space for the in situ and "perfect" samples (which simulated total stress release) are presented in Fig. 2. It can be seen from Fig. 2 that for the in situ sample initially s' decreases with the increase in t' and then it increases with further increase in t', as failure approaches. For the "perfect" sample, however, s' increases with the increase in t' throughout whole stage of undrained shearing. Complete release of total stress, therefore, produced appreciably different effective stress paths. The effective stress paths for the in situ samples are typically similar to those of normally consolidated clays. However, although the "perfect" samples have been prepared from the normally consolidated in situ samples they adopt stress paths similar to those for overconsolidated samples. Marked difference in the effective stress paths between the in situ

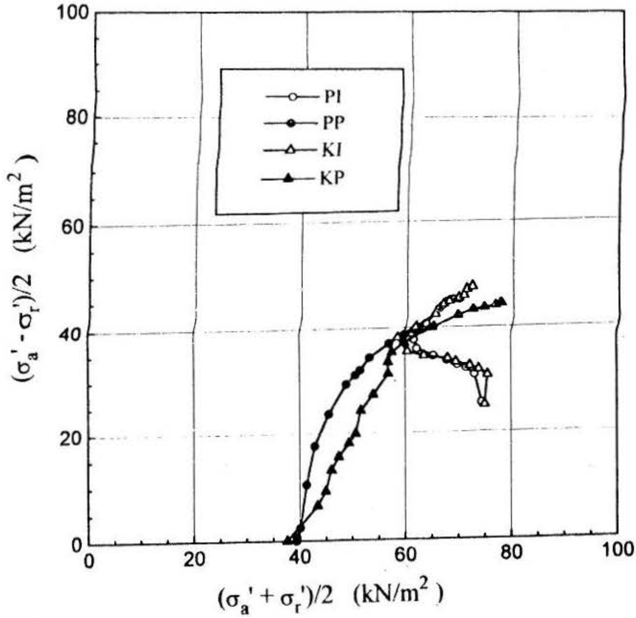

FIGURE 2 : Comparison of Effective Stress Paths of In Situ and "Perfect" Samples

and "perfect" samples has also been reported by several investigators (Skempton and Sowa, 1963; Ladd and Lambe, 1963; Atkinson and Kubba, 1981; Hight et al., 1985).

Changes in Strength, Deformation and Stiffness Properties

Figure 3 shows the deviator stress versus axial strain plots for the in situ and "perfect" samples. From the stress–strain data the undrained strength (s_u), initial tangent modulus (E_i) and axial strain at peak deviator stress (ε_p) have been determined for both the in situ and "perfect" samples. A comparison of the undrained soil parameters of the in situ and "perfect" samples is presented in Table 3. It can be seen from Table 3 that because of the relief of total stress–undrained strength of the samples from Patengha and Kumira (samples PP and KP respectively) decreased by 13% and 7% respectively. Values of axial strain at peak deavitor stress (ε_p), however, increased by 32% and 24% for the samples PP and KP respectively. Decreased in undrained strength due to stress release has been found for other normally consolidated clays by a number of researchers (Skempton and Sowa, 1963; Noorany and Seed, 1965; Davis and Poulos, 1967; Ladd and Varallyay, 1965; Atkinson and Kubba, 1981; Kirkpatrick and Khan, 1984; Hight et al., 1985; Sarkar, 1994). Ladd and Varallyay (1965), Kirkpatrick

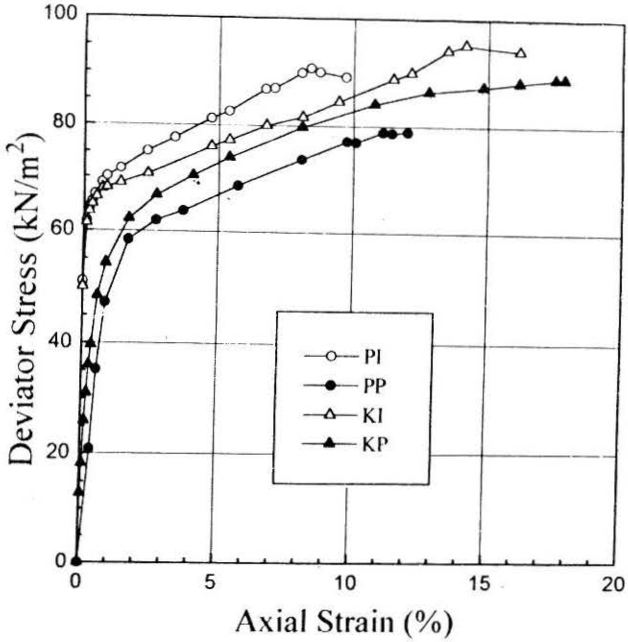

FIGURE 3 : Deviator Stress versus Axial Strain Plots of In Situ and "Perfect" Samples

TABLE 3 Comparison of Undrained Shear Properties of In Situ and "Perfect" Samples of the Two Coastal Soils

Sample	Su (kN/m^2)	ep (%)	Ei (kN/m^2)
PI	45.4	8.4	18030
PP	39.4	11.1	25190
KI	47.7	14.1	22120
KP	44.4	17.5	32580

and Khan (1984), and Sarkar (1994) also observed considerable increase in ep for normally consolidated Boston blue clay (LL = 33, PI = 15), Kaolin (PI = 30) and Illite (PI = 40), and reconstituted Dhaka clay (LL = 45, PI = 23) respectively. A marked difference in the behaviour of these coastal clays is that the values of ε_p for these samples (both in situ and "perfect") samples are considerably higher than those reported by several researchers for other intact and reconstituted clays.

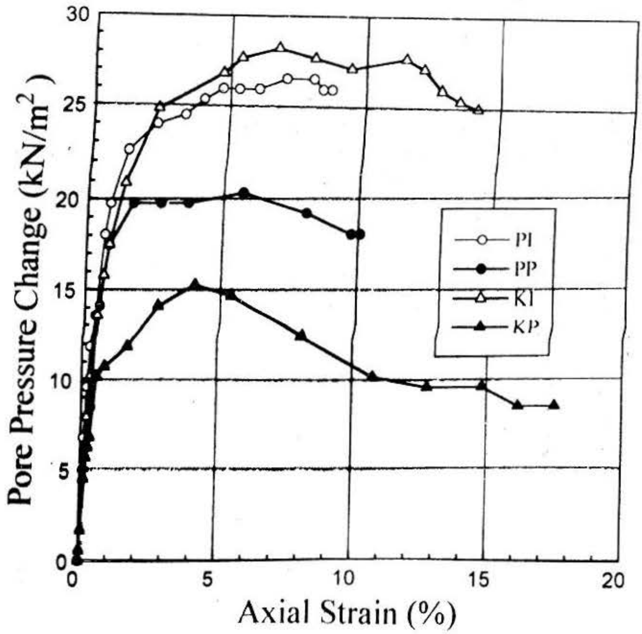

FIGURE 4 : Comparison of Pore Pressure Response of In Situ and "Perfect" Samples

Comparing the effect of stress release on the two clays studied, it is also evident that the extent of reduction in undrained shear strength is higher in the less plastic clay from Patengha (PI = 18) than in the more plastic clay from Kumira (PI = 33). Kirkpatrick and Khan (1984) also reported larger reduction in undrained strength in less plastic kaolin (PI = 30) than in more plastic illite (PI = 40) due to stress relief.

Table 3 also shows that because of disturbance due to stress release, the initial tangent modulus (E_i) increased. Compared with the in situ sample, the values of Ei of the samples PP and KP increased by 40% and 47% respectively. These results, contrast with those reported by Kirkpatrick and Khan (1984), and Atkinson and Kubba (1981) who found reduction in stiffness because of "perfect" sampling. Hight, Gens and Jardine (1985) and Sarker (1994), however, found increase in initial stiffness in normally consolidated low plastic North Sea clays (LL = 32, PI = 17) and Dhaka clay (LL = 45, PI = 23) respectively.

Changes in Pore Pressure Response

Figure 4, shows a comparison of the changes in pore pressure during shearing between the in situ and "perfect" samples. It can be seen from

Fig. 4 that, compared with the in situ sample the chanes in pore pressure for the "perfect" samples are considerably less. From Fig. 4, it appears that for the "perfect" samples at low strains the pore pressure increases rapidly with the increase in deviator stress. The pore pressure, however, typically start to decrease before the peak deviator stress has been reached resulting in considerably lower values of Skempton's pore pressure parameter A at peak deviator stress (A_p) than those for the in situ sample. The "perfect" samples despite being prepared from normally consolidated in situ sample thus showed a pore pressure response which is more typical of overconsolidated clay. The values of A_p for the "perfect" and in situ samples are as follows :

Sample PI	Sample PP	Sample KI	Sample KP
0.65	0.21	0.59	0.10

It can also be seen that the values of A_p reduced considerably (about 68% and 83% for samples PP and KP respectively) because of disturbance due to stress relief. Significant reduction in A_p has also been reported by other investigators (Seed et al., 1964; Noorany and Seed; 1965, Ladd and Varallyay; 1965; Kirkpatrick and Khan, 1984; Sarker, 1994).

Influence of Reconsolidation of "Perfect" Sample

Both the "perfect" samples were reconsolidated isotropically and anisotropically before being sheared in compression under undrained condition in order to examine the suitability of different reconsolidation procedures to restore the in situ behaviour.

The normalized deviator stress versus axial strain plots and pore pressure change versus axial strain plots for the reconsolidated "perfect" sample are presented in Figs. 5 and 6 respectively. In Figs. 5 and 6, the corresponding plots for the in situ sample are also shown for comparison with the reconsolidated samples. Undrained shear strength, s_u, initial tangent stiffness, E_i, pore pressure parameter at peak deavitor stress, A_p have been determined from the stress-strain and pore pressure data. A comparison of normalized undrained soil parameters are presented in Table 4. it can be seen from Table 4 that isotropic reconsolidation, using an effective consolidation pressure equal to p_o' (which are 66 kN/m^2 and 68 kN/m^2 for samples PI and KI respectively) of the in situ sample, has the effect of grossly overestimation of in situ strength, initial tangent modulus and ε_p for both the clays. Increase in undrained strength ratio (s_u/σ_{vc}') and ε_p due to isotropic reconsolidation of "perfect" samples has also been reported by Kirkpatrick and Khan (1984), Graham et al. (1987), Graham and Lau (1988) and Sarker (1994). Compared with the in situ sample, the values of A_p of the reconsolidated "perfect"

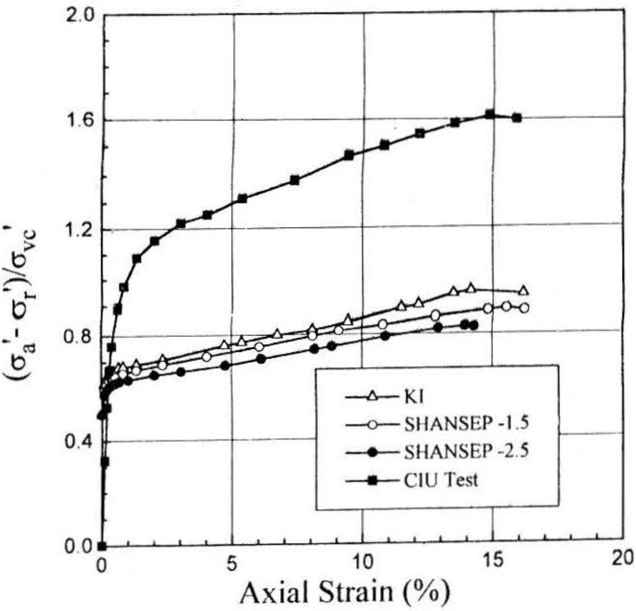

FIGURE 5 : Normalised Deviator Stress Vs. Axial Strain Plots of In Situ and Reconsolidated "Perfect" Samples for Kumira Site

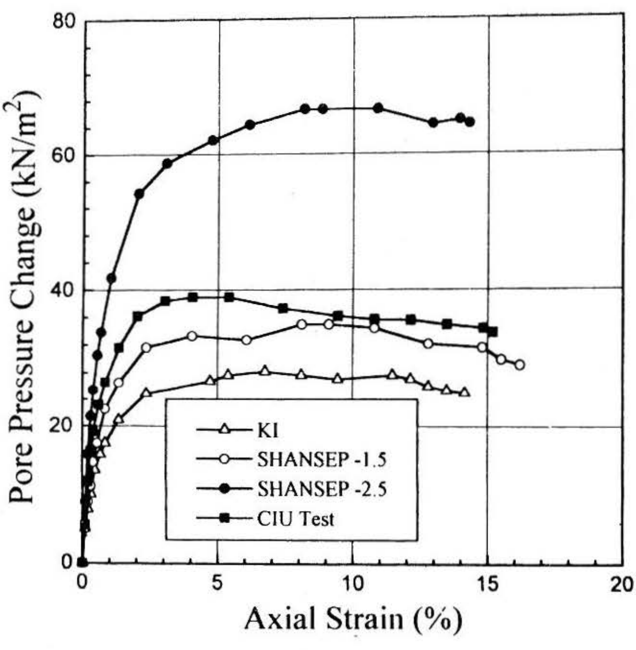

FIGURE 6 : Pore Pressure Change Vs. Axial Strain Plots of In Situ and Reconsolidated "Perfect" Samples for Kumira Site

samples, however, reduced. This finding contrast with those reported by Kirkpatrick and Khan (1984) and Graham et al. (1987) who found an increase in A_p due isotropic reconsolidation. Sarker (1994), however, reported a reduction in A_p for Dhaka clay due to isotropic reconsolidation.

In contrast to isotropic reconsolidation, it can be seen from Table 4 that the strength ratio and stiffness ratio of the "perfect" samples reconsolidated using SHANSEP-1.5 and SHANSEP-2.5 are less than those for the in situ sample. The strength ratio (s_u/σ_{vc}') for the samples (collected from Patengha) reconsolidated using SHANSEP-1.5 and SHANSEP-2.5 procedure decreased by 13% and 20% respectively. The respective reductions for the samples from Kumira are 6% and 13%. The stiffness ratio (E_i/σ_{vc}') reduced by 3.5% and 19% for samples from Patengha reconsolidated using SHANSEP-1.5 and SHANSEP-2.5 procedure respectively. For the Kumira soil, stiffness E_i/σ_{vc}' reduced by 20% and 38% for samples reconsolidated using SHANSEP-1.5 and SHANSEP-2.5 procedure respectively. From the above comparisons, it is evident that despite both the SHANSEP-1.5 and SHANSEP-2.5 reconsolidation procedure provided a lower bound strength and stiffness, the strength and stiffness ratios of the samples reconsolidated using SHANSEP-1.5 compared more closely with the in situ samples than the samples reconsolidated using SHANSEP-2.5 method. The values of A_p of

TABLE 4 Comparison of Undrained Shear Characteristics of In Situ and Reconsolidated "Perfect" Samples of the Two Coastal Soils

Location	Test Type	Su/svc or su/sc	ep (%)	Ei/svc or Ei/sc	Ap
Patengha	CIU Test Using sc = po	0.67	13.1	255.9	0.43
	CK0U Test Using SHANSEP-1.5 svc	0.39	16.5	173.9	0.82
	SK0U Test Using SHANSEP-2.5 svc	0.38	13.9	145.8	1.05
	In Situ Sample	0.45	8.4	180.3	0.65
Kumira	CIU Test Using sc = po	0.80	14.8	306.3	0.40
	CK0U Test Using SHANSEP-1.5 svc	0.44	15.5	176.7	0.51
	SK0U Test Using SHANSEP-2.5 svc	0.41	13.9	136.6	0.79
	In Situ Sample	0.47	14.1	221.2	0.59

the samples reconsolidated using SHANSEP-1.5 method also compared better with the in situ samples than the samples reconsolidated using SHANSEP-2.5 method. It therefore appears that K_o – reconsolidation of the "perfect" sample to 1.5 times svc produced the best overall estimate of the *in situ strength, stiffness to 1.5 times σ_{vc}' produced the best overall estimate* of the in situ strength, stiffness and pore pressure response. This is also evident from the comparison of the plots in Figs. 5 and 6 for the Kumira soil in which the relations for sample reconsolidated using SHANSEP-1.5 procedure agrees best with the in situ sample. Graham et al. (1987) and Graham and Lau (1988) also found that K_o – reconsolidation of the "perfect" sample to σ_{vc}' produced the best agreement *between the "perfect" and in situ* sample in terms of the strength, stiffness and A_p. It also appears from the results presented in Table 4 that K_o – consolidation of the samples using SHANSEP-2.5 method could not improve the quality of the "perfect" samples, rather it the strength and stiffness ratio markedly reduced compared with the in situ sample. This may be attributed to progressive destructing of the clay due to large volumetric straining occurred during consolidation. Destructing has also been observed in natural and reconstituted clays during reconsolidation using SHANSEP procedure (Delage and Lafebvre, 1984; Leroueil and Vaughan, 1990; Burland, 1990; Clayton, Hight and Hopper,1992; Sarker, 1994). Compared with the in situ sample, the values of ε_p of the anisotropically reconsolidated "perfect" samples significantly increased. Similar observations were also reported by Graham et al. (1987) for illite samples (LL = 58, PI = 32).

Conclusion

Effects of stress release disturbance on undrained stress- strain-strength, stiffness and pore pressure characteristics of two coastal clays have been investigated. Experimental results indicate that s_u, E_i, A_p were reduced due to disturbance caused by the complete release of total stress. Compared with the in situ samples, the values of ε_p of the "perfect" samples, however, increased significantly. For the two clays studied, the values of s_u, E_i, A_p up to 13%, 47% and 83% respectively. The values of ε_p increased up to 32%. The effective paths of the "perfect" samples have been found to be markedly different from those of the in situ samples. In general, the behaviour of the "perfect" samples are more typical of overconsolidated clays. The findings of the present investigation also compared favourably with those reported by previous researchers. It deserves mentioning that the structure and fabric of natural soils have a marked influence on the behaviour of soils. Recent studies (Ansary, 1993; Safiullah, 1994) on undrained strength of a number of soils from different coastal regions of Bangladesh show. that the undrained strength of undisturbed natural block samples are considerably higher than the reconsituted samples prepared from the same soil. It has also been found that the difference in strength between the natural and reconstituted samples increases with increasing overconsolidation ratio of samples. It worth mentioning that if in situ and "perfect" samples would have been prepared from intact natural samples, then the stress release disturbance effects on the engineering properties between the "in situ" and "perfect" samples might have been more prominent due their special features associated with fabric, bonding and ageing.

The influence of reconsolidation of "perfect" sample to reduce stress disturbance effects was also investigated. Both isotropic and anisotropic reconsolidation using SHANSEP procedures were adopted. It has been found that K_0 – reconsolidation of "perfect" sample to in 1.5 times σ_{vc}' produced the best agreement between the "perfect" and in situ samples in terms of their undrained strength ratio, stiffness ratio and A_p-values.

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Notation

A	=	Skempton's pore pressure parameter
A_p	=	Skempton's pore pressure parameter A at peak deviator stress
c_u	=	undrained shear strength
E_i	=	initial tangent modulus
K_o	=	coefficient of earth pressure at rest
LL	=	liquid limit
PI	=	plasticity index
p_o'	=	initial mean effective stress $[(\sigma_a' + 2\sigma_r') / 3]$ at the end of K_o – consolidation
s'	=	$(\sigma_a' + \sigma_r') / 2$
t'	=	$(\sigma_a' - \sigma_r') / 2$
ε_p	=	axial strain at peak deviatoric stress
σ_a'	=	axial effective stress
σ_r'	=	radial effective stress
σ_{vc}'	=	vertical effective stress at the end of K_o – consolidation
σ_c'	=	isotropic effective consolidation stress