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Compressibility and Permeability Characteristics of Selected Coastal Soils of Bangladesh

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Introduction

A geotechnical engineer is concerned with the compressibility characteristics of soil because he has to know by how much a soil will be compressed under external load and at what rate the compression will take place. On the other hand, permeability governs such important engineering problems as the ground water regime in layered deposits, or near natural and excavated slopes, the consolidation of clay foundations under applied loads and the flow of water through or around engineering structures.

In recent times several coastal regions of Bangladesh have been affected by severe cyclone 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 needed to be built in large quantities in these coastal regions. It is essential to understand the mechanical properties and behaviour of these soils which are in general silty clays of low to high plasticity.

This paper presents the compressibility and permeability characteristics of four coastal soils. Attempt has also been made to evaluate the essential intrinsic properties of these soils.

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Compressibility and Permeability Characteristics of Clays

Expression for rate of consolidation has been first developed by Terzaghi (1943). Based oil Terzaghi's theory an ASTM standard is formulated (1979). Recently, two other one-dimensional consolidation test procedures have been developed which are much faster and provide reasonably good results (Lowe, 1969; Smith and Wahls, 1969). For a given stress increment, the coefficient of consolidation c_v can be determined from laboratory observations of time versus dial reading. Two graphical procedures are commonly used for this are the logarithmic of time method proposed by Casagrande and Fadum (1940) and the square root of time method proposed by Taylor (1942).

$$c_v = \frac{0.197\, H^2}{t_{50}} \quad \text{and} \quad c_v = \frac{0.848\, H^2}{t_{90}} \tag{1}$$

where, H is the average length of drainage path, i.e., half the height of sample at the end of 100% consolidation for a given stress increment. There are two other useful methods, which are proposed by Su (1958) and Sivaram and Swamee (1977).

Several researchers have suggested functions for the relationship between compression index and void ratio for the undisturbed samples of coastal soils of Bangladesh (Serajuddin and Ahmed, 1967; Amin et al., 1987; Kabir et al., 1992). The empirical correlations between C_c and. e_0 for undisturbed samples, as proposed by Serajuddin (1969) and Amin et al. (1987) are given by the following Eqns. (2a) and (2b) respectively:

$$C_c = 0.50(e_0 - 0.50) \tag{2a}$$

$$C_c = 0.33(e_0 - 0.35) \tag{2b}$$

Samarasinghe et al. (1982) defined the void ratio e_1 as the value of void ratio when the effective stress is 1 tsf (95.7 kPa). Samarasinghe et al. (1982) termed C_c and e_1 as compressibility parameters. The values of e_1, C_c, C_s and c_v obtained from reconstituted soil samples studied by several researchers are summarized in Table 1. Using C_c and e_1 from Raymond (1966), Samarasinghe et al. (1982) and a number of regional soils of Bangladesh, Siddique (1986) proposed the following relation between C_c and e_1 for reconstituted samples

$$C_c = 0.39(e_1 - 0.14) \tag{3}$$

Table 1
Values if LL, PI, e_1, Compression Index, Swelling Index and Coefficient of Consolidation of some Reconstituted Samples

Location	LL	PI	e_1	C_c	C_s	c_v (10^{-4} cm²/cm)	
						log t fitting	$\sqrt{t}$ fitting
Don Valley Clay[1]	.41	22	0.78	0.26	–	–	–
New Liskeard[1]	69	40	1.14	0.41	–	–	–
Greyish Sandy Clay[2]	27	14	0.57	0.16	–	3 to 7	4 to 9
Gazipur Clay[3]	33	6	0.93	0.29	–	70 to 180	100 to 250
London Clay[4]	69	45	1.16	0.55	0.15	0.8 to 4	1 to 6
Dhaka Clay[5]	40	20	0.84	0.28	–	5 to 10	7 to 14

1: Raymond (1966); 2: Samarasinghe et al. (1982); 3: Siddique (1986);
4: Siddique and Clayton (1994); 5: Siddique and Safiullah (1995)

Burland (1990) introduced a new normalizing parameter termed as the void index to aid in correlating the compression characteristics of various clays. Later Kamaluddin (1990) established the same relationship for Dhaka clay. Burland (1990), termed the properties of reconstituted clays as 'intrinsic' properties since they are inherent to the soil and independent of natural state. The term 'intrinsic' has been used to describe the properties of clays which have been reconstituted at a water content of between w_L and $1.5w_L$ and then consolidated under one-dimensional condition.

The compressibility and strength characteristics of reconstituted clays were used as a basic frame of reference for interpreting the corresponding characteristics of natural sedimentary clays. The intrinsic properties provide a frame of reference for assessing the in-situ state of a natural clay and the influence of structure on its in-situ properties. One dimensional compression curve for reconstituted natural clay is normalized by assigning fixed values to e^*_{100} and e^*_{1000}. The parameters e^*_{100} and e^*_{1000} are the intrinsic void ratios corresponding to consolidation pressures, $\sigma'_v = 100$ kPa and $\sigma'_v = 1000$ kPa, respectively. The normalized parameter chosen has been defined as the void index, I_v such that,

$$I_v = \frac{e - e^*_{100}}{e^*_{100} - e^*_{1000}} = \frac{e - e^*_{100}}{C^*_c} \tag{4}$$

The intrinsic compression index, C^*_c is defined as the difference in void ratio between e^*_{100} and e^*_{1000}.

Following Terzaghi (1925), the parameters e^*_{100} and C^*_c are called as the constant of intrinsic compressibility. When $e = e^*_{100}$, then $I_v = 0$ and when $e = e^*_{1000}$, then $I_v = -1$. The void index may be thought of as a measure of the intrinsic compactness of a sediment. When I_v is less than zero, the sediment is compact and when I_v is greater than zero, the sediment is loose. It has been seen that a reasonably unique line is achieved when void index, I_v versus $\log v$ is plotted for clays covering a wide range of liquid limits and of pressures. This line has been termed as intrinsic compression line (ICL). Burland has given a cubic equation as follows which may represent the coordinates of the ICL with sufficient accuracy

$$I_v = 2.45 - 1.285x + 0.015x^3 \tag{5}$$

where $x = \log p'_o$ in kPa

The intrinsic compression line may either be measured directly for a clay or if the values of e^*_{1000} and C^*_c are known, for the clay, the ICL may be constructed using the above Eqn. (5). If it is required to plot the ICL in terms of e versus $\log p'_o$, then the values of e corresponding to various values of $\log p'_o$ may be obtained from the following expression:

$$e = I_v C^*_c + e^*_{100} \tag{6}$$

In the above Eqn. (6), the values of I_v may be obtained from Eqn. (5).

For coarse grained soils, permeability has been related to grain size distribution. Similar relationship for fine-grained soils is less successful to predict permeability accurately for engineering purposes. In the past, numerous researchers have suggested functions for the relationship between permeability and void ratio (Taylor, 1948; Raymond, 1966; Lambe and Whitman, 1969). Recently, Samarasinghe et al. (1982) suggested a model to predict permeability of normally consolidated reconstituted clays. The relationship is as follows:

$$k = C\left(\frac{e^n}{1+e}\right) \tag{7}$$

Samarasinghe et al. (1982) termed n and C as permeability parameters. A plot of $\log\left[k(1+e)\right]$ versus $\log e$ results in a straight line of which n is the slope and log C is the intercept. Samarasinghe et al. (1982) carried out both direct permeability tests and incremental loading consolidation tests on an artificially sedimented normally consolidated sandy clay to verify the model represented by Eqn. (7). For any method, n is about the same for each soil.

Table 2
Values of Permeability Parameters of Different Soils Derived from Consolidation Test

Soil	Direct Permeability Method		$\log t$ Fitting Method		$\sqrt{t}$ Fitting Method	
	n	C (10^{-10}m/sec)	n	C (10^{-10}m/sec)	n	C (10^{-10}m/sec)
Don Valley Clay[1]	4.8	29	4.8	15	–	–
New Liskeard Clay[1]	4.9	4	4.7	3.6	–	–
Greyish Sandy Clay[2]	5.2	77	–	–	–	–
Gazipur Clay[3]	5.1	355-	4.8	186	5	282
Dhaka Clay[4]	4.7	12	4.5	8.5	4.8	10

1: Raymond (1966); 2: Samarasinghe et al. (1982); 3: Siddique (1986);
4: Siddique and Safiullah (1995)

However, C varies with the method of tests. Table 2 shows the values of n and C obtained for a number of clays.

Raymond (1966) also conducted hydrostatic consolidation tests and direct permeability tests on the same specimen for three different reconstituted normally consolidated clays of medium to high plasticity. It has been observed that the reconstituted clays investigated by Raymond (1966) behave exactly in conformity with Eqn. (7) as proposed by Samarasinghe et al. (1982).

For both reconstituted and natural clays linear plots of void ratio versus logarithm of permeability have also been suggested by other researchers (Tavenas et al., 1983; Tababa and Wood, 1987; Leroueil et al., 1992; Nagaraj et al., 1993). Tavenas et al. (1983) assessed the applicability of Eqn. (7) for a number of undisturbed natural soft clays. The experimental data obtained in the investigation suggest that the relation, $\log e - \log[k(1+e)]$ is suitable to represent permeability - void ratio relation in natural clays.

Soils Used

Disturbed samples were collected from four selected locations of coastal region from shallow depths. Undisturbed block samples were also collected from these locations. Figure 1 shows the location of the studied sites. From borehole records and from inspection of open cuts nearby, it was established that the sub-soil at this locations are more or less uniform in the upper layer. Table 3 shows the index properties and classification of coastal soils used.

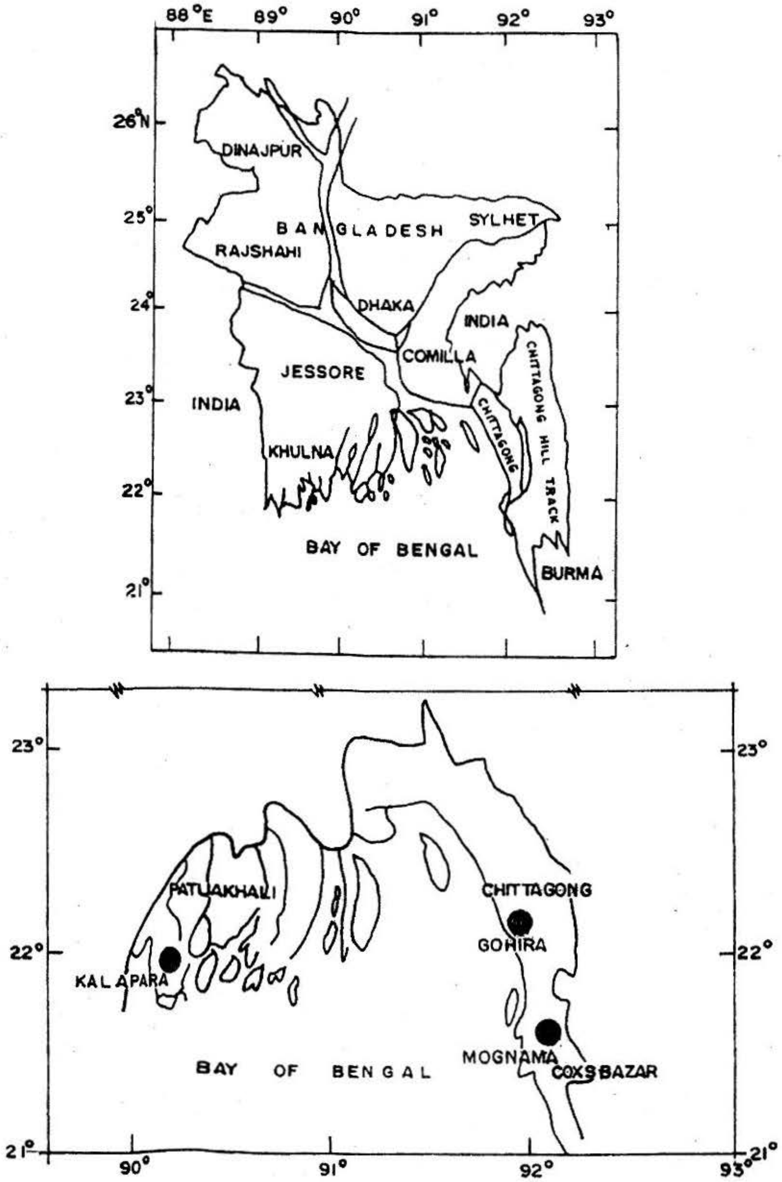

FIGURE 1 : Map of Bangladesh showing the Location of Coastal Sites

Table 3
Index Properties and Classification of the Coastal Soils Used

Location	Sample Desig.	G_s	LL %	PI %	Clay %	Silt %	Sand %	USCS symbol
Gohira (1.5m) Chittagong	G1	2.79	48	24	41	58	1	CL
Gohira (3.5m) Chittagong	G3	2.71	36	16	38	61	1	CL
Kalapara (GL) Patuakhali	KA	2.65	45	18	30	70	0	ML
Mognama (1m) Cox's Bazar	M	2.73	49	26	39	56	5	CL

USCS: Unified Soil Classification System

Regional Geology

The coastal region comprising Barisal, Chittagong, Cox's Bazar, Khulna, Noakhali and Patuakhali are underlain by floodplain and meandering deposits laid down by the rivers and their tributaries. The river gradient is flat, and is one of the causes for the reduction of the velocity and deposition of fine eroded particles like silt and clay. The formation is subjected to flooding to a varying depth. The soil profile is generally stratified but their continuity is limited to short distances. Coastal soils can not be classed either as recent or terrace deposits. These are usually estuarine and tidal deposits.

Ground water in the coastal area is strongly influenced by saline water. Tube wells of more than 200 m deep are dug in and around the Chittagong hills and hills near Moheskhali Island to avoid saline water intrusion. Some flowing artesian wells are also observed in these areas. The well depth in other parts of the coastal area of around 300 m is generally much deeper. Some wells near Noakhali are more than 400 m in depth (MCSP, 1992). Most soil studies in the coastal area in the past have dealt with the soil types upto approximately 20 m below the ground surface. The surface layer mainly consists of silt and clay and has a thickness of some 50 m, except at the mouth of the Meghna river where the thickness is reduced to some 10 m. A more detailed examination reveals that the soil texture of the surface layer differs from one area to another in both the horizontal and vertical directions. The grain size, density and consistency also largely differ from one area to another. These differences reflect the sedimentation environment and are caused by frequent changes of the well-developed river and water channel courses. In general, the deposits of the major rivers are coarser than those of the sea currents. A generalized stratigraphic succession of rocks in the area is presented in Table 4.

Table 4
General Stratigraphic Sequence of Rocks In Coastal Region (Khan, 1991)

Approximate age	Group	Formation	Lithology
Recent		Beach sand Coastal deposit Alluvium	Sand containing heavy mineral Sand, silt and clay Silt, clay and fine sand
	....	Unconformity	....
Mio-Pliocene	Dupi Tila		Coarse ferruginous soft sandstone; clay (sandy and mottled) at intervals; pebbly sandstone and conglomerate beds
	....	Unconformity	....
		Girujan Clay	Dark grey and bluish grey clay with mottlings, lower part consists of ferruginous sandstone
Upper Miocene	Tipam	Tipam Sandstone	Coarse and gritty ferruginous sandstone with interlayer of clay; sandstones are massive, cross bedded and yellowish brown in colour; conglomerate in pocket, fossil wood and lignite are present
		Upper Unit	Bluish grey shales with conchoidal fractures; lenses of hard calcareous sandstone and shale
Middle Miocene	Boka Bil	Middle Unit	Alternations of massive and bedded sandstone with thin layers of shales and siltstones; Lenticular and spherical concretions; Conglomerates are occasionally found
	Surma	Lower Unit	Thin bedded, bluish grey shales with intercalations of shaly siltstone
	....	Unconformity	....
		Upper Unit	Shales predominate in the lower part with progressively more sand intercalation towards the top: Fine sandstone in the uppermost part
Lower Miocene	Bhuban	Middle Unit	Standstone with subordinate shale Mainly an alternation of shales, sandy shales, and fine grained sandstone
		Lower Unit	Mainly sandy shale, pebbly sandstone and silty shale
	....	Base not known	....

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 (Jardine, 1985). Reconstituted samples of the four coastal soils were prepared in the laboratory by K_0-consolidation of a uniform slurry of the clay in a cylindrical consolidation cell of 153 mm diameter and 178 mm height using a consolidation pressure of 100 kPa to get a clay of soft consistency. The slurry had a water content of approximately 1.5 to 2 times the liquid limit of the soil.

Test Results And Discussion

Consolidation tests were carried out using a stress increment ratio of 1, (i.e., a load ratio of 2) was used. The vertical stresses applied during consolidation were 26.8 kPa, 53.6 kPa, 107.2 kPa, 214.5 kPa, 429 kPa, 858 kPa and 1287 kPa. The samples were also allowed to swell under stresses of 322 kPa and 10.7 kPa. Duration of each loading step was twenty four hours. During all these tests drainage was permitted from top and bottom of the sample. These tests were carried out in accordance with the procedure specified in ASTM (1979) Standards. For each loading step deformation was recorded by a dial gauge at specified intervals of time.

Time – deformation curves are plotted for each pressure increment and from these plots times corresponding to 50% consolidation i.e., t_{50} and 90% consolidation, i.e., t_{90} were determined using Casagrande's Curve Fitting Method (Casagrande and Fadum, 1940) and Taylor's Curve Fitting Method (Taylor, 1942) respectively. Coefficients of consolidation, c_v, were calculated for each stress increment using the expression shown in Eqn. (1).

Coefficient of permeability of the samples were determined indirectly from one-dimensional consolidation tests. The coefficient of permeability, k was computed using the following expression (Taylor, 1948):

$$k = c_v \, m_v \, \gamma_w \tag{8}$$

where m_v = coefficient of volume compressibility, and
γ_w = unit weight of water.

m_v can be estimated from the following equation:

$$m_v = \frac{C_c}{2.303(1+e)\sigma'} \tag{9}$$

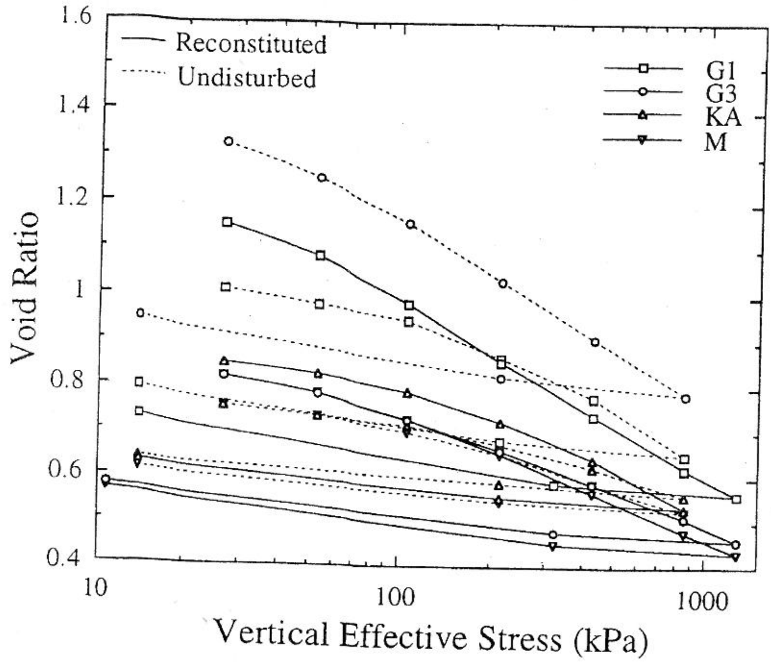

FIGURE 2 : Comparison of Void Ratio Vs. Effective Overburden Pressure Curves between Reconstituted and Undisturbed Samples of Four Coastal Sites

In the following sections compressibility and permeability characteristics of reconstituted and undisturbed samples of four coastal soils obtained from one dimensional consolidation test results are presented and discussed.

Compressibility And Expansibility Characteristics

The compressibility characteristics of reconstituted and undisturbed samples of the four coastal soils undergoing incremental loading in an oedometer are presented in Fig. 2 in which void ratios (at the end of each loading and unloading stages) versus $\log \sigma_{vo}'$, (logarithmic of vertical effective consolidation stress) curves are shown. The values of compression index (C_c) and swelling index (C_s) have been determined from the loading and unloading portion respectively of the $e - \log \sigma_{vo}'$ curves. The values of C_c and C_s for the four coastal soils are listed in Table 5. It can be seen from Table 5 that the values of C_c and C_s for reconstituted samples varied from 0.29 to 0.40 and 0.028 to 0.046, and for undisturbed samples varied from 0.22 to 0.42 and 0.037 to 0.062 respectively. It can also be seen from Table 5 that for samples of Gohira which has high natural water content the C_c values of the undisturbed samples are higher than those of reconstituted samples. For the samples from Kalapara and Mognama which have comparatively low natural

Table 5
Values of w_n, w_l, e_l, Compression Index, Swelling Index of Four Coastal Soils

Site	Undisturbed Sample				Reconstituted Sample			
	w_n (%)	e_l	C_c	C_S	w_l (%)	e_l	Cc	Cs
G 1	38	0.95	0.42	0.054	37	1.02	0.40	0.046
G3	52	1.17	0.42	0.062	27	0.73	0.29	0.028
KA	28	0.72	0.22	0.043	30	0.80	0.37	0.041
M	26	0.71	0.22	0.037	27	0.73	0.29	0.031

water content, however, the C_c values of the undisturbed samples are lower than those of reconstituted samples. It is also evident from Table 5 that C_s of undisturbed samples of Gohira are considerably higher than those of reconstituted samples while the C_s values of undisturbed samples of Kalapara and Mognama are slightly higher than that of reconstituted samples. Farooq (1995) investigated the compressibility properties of three reconstituted soils from Chittagong belt. The values of C_c and C_s reported by Farooq (1995) varied from 0.26 to 0.30 and 0.02 to 0.03 respectively.

Figure 3 shows the compressibility and expansibility characteristics of reconstituted and undisturbed samples of a typical coastal soil (G1: Gohira 1.5m) undergoing incremental loading in an oedometer. In this figure, coefficient of volume compressibility, m_v and coefficient of volume increase, m_s have been plotted against $\log\sigma_{vo}'$. It can be seen froin Fig. 3 that in case of reconstituted sample, for loading up to consolidation stress of 300 kPa, m_v changes significantly. However, above 300 kPa, m_v decreases slightly with increasing consolidation stress. During unloading from 1287 kPa to 13.5 kPa, the values of m_s increased. For reconstituted London Clay (LL = 69, PI = 45), Siddique and Clayton (1994) found that m_v– values reduced sharply beyond preconsolidation pressure and m_s– values increased during unloading. Farooq (1995) found m_v values between 1.13×10^{-4} to 6.1×10^{-4} cm^2/gm for three reconstituted samples of coastal soils from Chittagong. Figure 3 also shows the relation between coefficient of volume compressibility and volume expansibility with effective vertical stress for undisturbed sample. It is evident from Fig. 3 that the values of m_v and m_s of reconstituted sample are comparatively higher than those for the undisturbed sample.

Figure 4 shows typical plottings of coefficient of consolidation, c_v versus vertical effective consolidation stress for reconstituted and undisturbed samples of Gohira. It is also evident from Fig. 4 that for the reconstituted samples c_v value generally increased with increasing levels of consolidation stress. Increase in c_v with increasing levels of vertical effective consolidation stress has been reported by Samarasinghe et al. (1982) and Siddique (1986) for

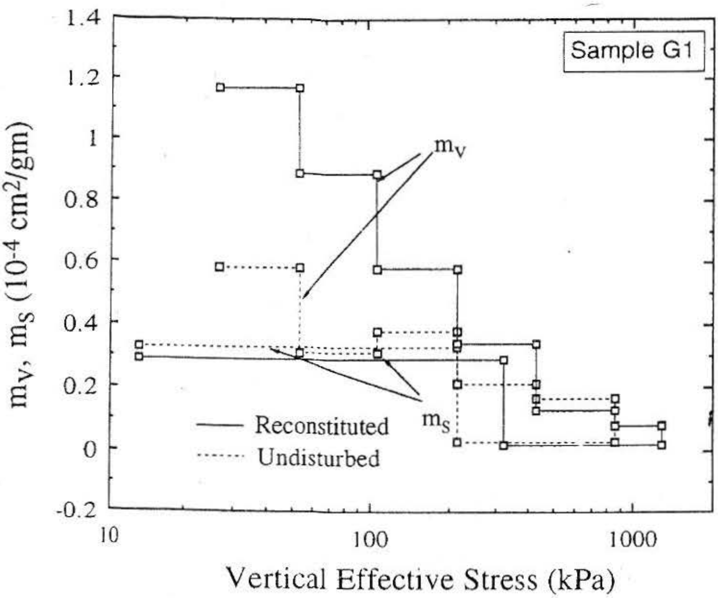

FIGURE 3 : Coefficient of Volume Compressibility and Volume Expansibility Vs. log of Vertical Effective Stress Plots for a Typical Coastal Soil

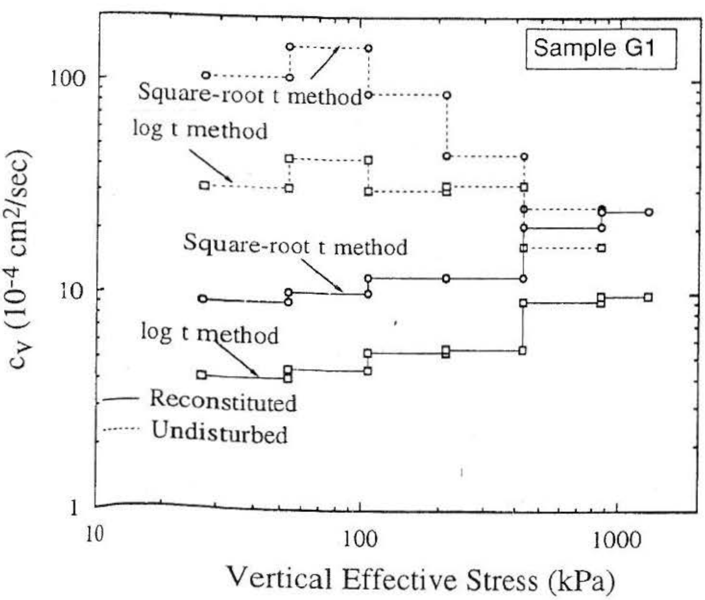

FIGURE 4 : Coefficient of Consolidation Vs. log of Vertical Effective Stress Plots for Reconstituted and Undisturbed Samples of a Typical Coastal Soil

reconstituted normally consolidated clays. Figure 4 shows that for both reconstituted and undisturbed samples, the values of c_v obtained using $\log t$ method are less than those obtained using $\sqrt{t}$ method. Figure 4 also shows that for the undisturbed samples, the values of c_v, in general, reduced with the increase in consolidation stress. The values of c_v for the undisturbed samples have been found to be significantly higher than those for reconstituted samples. The values of c_v for the four reconstituted and undisturbed samples of the coastal soils varied from 3.3×10^{-4} to 30.2×10^{-4} cm^2/sec and 10.1×10^{-4} to 224.4×10^{-4} cm^2/sec respectively. Serajuddin (1969) reported c_v values between 0.2×10^{-4} to 200×10^{-4} cm^2/sec for large number of undisturbed samples of coastal soils while Farooq (1995) reported c_v values between 20×10^{-4} to 57×10^{-4} cm^2/sec for three reconstituted samples of Chittagong coastal region.

Attempts have also been made to relate compressibility parameters C_c and e_1 for the reconstituted. samples. Based on regression analysis of these data the following correlation between C_c and e_1 has been derived:

$$C_c = 0.384(e_1 + 0.06) \tag{10}$$

The above relationship is presented in Fig. 5. In Fig. 5 the relation between C_c and e_1 as proposed by Siddique (1986) and given in Eqn. (3) is

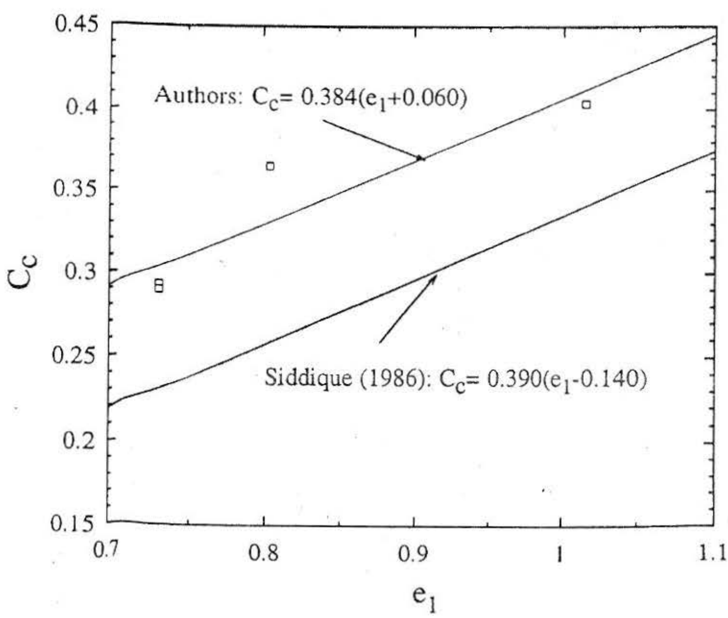

FIGURE 5 : Plot of C_c Vs. e_1 for Reconstituted Samples

also shown for comparison. It can be seen from Fig. 5 that the line representing relationship between C_c and e_1 obtained for the soils studied is parallel to that proposed by Siddique (1986) for a number of reconstituted samples and lies above the line proposed by Siddique (1986).

Void Index and Intrinsic Compression Lines

The compression curves shown in Fig. 2 represent the intrinsic compression curves for the four locations of coastal region, since the samples were reconstituted at water content of 1.5 times the liquid limit. It is useful to normalize these laboratory compression curves with respect to void ratio. Using Burland's concept (1990), the intrinsic compression curves for the coastal regions shown in Fig. 2 have been normalized by plotting void index, I_v versus $\log\sigma_v'$ and which are shown in Fig. 6. The intrinsic void ratio e^*_{100}, intrinsic compressibility, C^*_c, void ratio at the liquid limit, e_L, the equation of void index for the four locations of the coastal region are shown in Table 6. Using the equations given for the void index in Table 6, intrinsic compression lines for the four coastal soils are shown in Fig. 6. It can be seen from Fig. 6 that, the intrinsic compression lines for the samples of the coastal soils investigated compared favorably with that proposed by Burland (1990). Intrinsic compression line for Dhaka clay (Kamaluddin, 1990) also agreed with that proposed by Burland (1990).

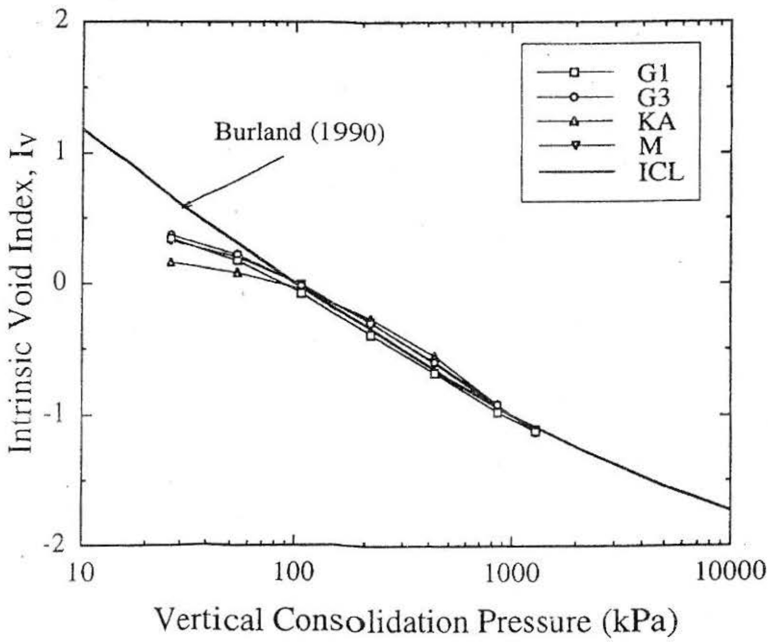

FIGURE 6 : Intrinsic Compression Lines for Samples of Four Coastal Soils

Table 6
Intrinsic Properties of the Soils of Four Locations of Coastal Region

Soil	w_L(%)	e_L	e^*_{100}	C_c^*	Equation for I_v
G1	59	1.65	1.02	0.405	$I_v = (e-1.02)/0.405$
G3	41	1.11	0.73	0.242	$I_v = (e-0.73)/0.242$
KA	45	1.19	0.80	0.295	$I_v = (e-0.80)/0.295$
M	66	1.80	0.73	0.272	$I_v = (e-0.73)/0.272$

Table 7
Coefficient of Permeability for Reconstituted Samples for Four Coastal Soils Determined from One-dimensional Test

Soil Type	Average σ' (kPa)	Average water content (%)	Average void ratio	k (10^{-10}m/sec)	
				log t Fitting Method	Fitting Method
G1	40.2	40	1.120	4.70	10.60
	80.4	37	1.035	3.90	8.90
	160.9	33	0.921	3.10	6.90
	321.8	29	0.800	1.90	4.10
	643.5	24	0.669	1.20	2.70
	1072.5	21	0.591	0.80	2.00
G3	40.2	30	0.804	4.98	7.14
	80.4	28	0.757	4.63	6.33
	160.9	26	0.694	3.48	5.26
	321.8	23	0.624	2.63	3.65
	643.5	20	0.549	1.91	2.74
	1072.5	18	0.485	1.28	1.69
KA	40.2	32	0.840	5.25	8.23
	80.4	31	0.809	4.95	7.70
	160.9	29	0.758	4.54	6.94
	321.8	26	0.684	3.98	5.85
	643.5	22	0.587	3.31	4.62
M	40.2	29	0.804	3.53	5.41
	80.4	28	0.758	2.95	4.68
	160.9	25	0.691	2.25	3.33
	321.8	2	0.610	1.42	2.07
	643.5	19	0.523	0.86	1.18
	1072.5	17	0.455	0.53	0.67

Table 8
Coefficient of Permeability for Undisturbed Samples of Four Coastal Soils determined from One-dimensional Consolidation Test

Soil Type	Average σ' (kPa)	Average water content (%)	Average void ratio	k (10^{-10}m/sec)	
				$\log t$ Fitting Method	Fitting Method
G1	40.2	36	0.997	18.00	60.00
	80.4	35	0.965	13.00	44.12
	160.9	33	0.908	11.52	33.10
	321. 8	30	0.824	6.83	9.50
	643.5	26	0.715	2.78	4.20
G3	40.2	48	1.292	12.00	14.30
	80.4	45	1.207	8.40	19.80
	160.9	40	1.097	7.50	12.40
	321.8	36	0.972	5.30	13.00
	643.5	31	0.848	3.80	4.10
KA	40.2	28	0.747	20.00	54.50
	80.4	27	0.726	18.00	40.00
	160.9	26	0.694	18.30	45.80
	321.8	24	0.648	7.30	24.20
	643.5	22	0.592	5.50	7.70
M	40.2	27	0.747	15.30	99.00
	80.4	26	0.718	13.00	34.90
	160.9	25	0.676	9.60	27.00
	321.8	23	0.620	4.20	13.90
	643.5	20	0.557	3.00	10.80

Permeability Characteristics

Summary of coefficient of permeability derived from consolidation test results for reconstituted samples is presented in Table 7. Permeability determined using $\log t$ method and $\sqrt{t}$ method varies from 0.53×10^{-10} m/s to 5.25×10^{-10} m/s and 0.67×10^{-10} m/s to 10.60×10^{-10} m/s respectively. The values of coefficient of permeability determined using $\sqrt{t}$ fitting method are higher than those determined using $\log t$ fitting method for all the stress ranges. Similar results have also been found by Siddique and Safiullah (1995) for reconstituted Dhaka clay (LL = 40, PI = 20). Table 8 presents summary of coefficient of permeability derived from consolidation test results for undisturbed samples. It can he seen from Table 8 that similar to the reconstituted samples, the values of coefficient of permeability determined using $\sqrt{t}$ fitting method are higher than those determined using $\log t$ fitting method for all the stress ranges.

Permeability determined using $\log t$ method and $\sqrt{t}$ method varied from 3×10^{-10} m/s to 20×10^{-10} m/s and 4×10^{-10} m/s to 99×10^{-10} m/s respectively. Comparing the values of permeability of reconstituted and undisturbed samples, it can be seen from Tables 7 and 8 that, the values of permeability for the undisturbed samples of all the coastal soils are significantly higher than those for the reconstituted samples.

Figures 7 and 8 show the plottings of void ratio against logarithm of permeability determined from consolidation tests for reconstituted and undisturbed samples respectively. It can be seen from Figs. 7 and 8 that permeability decreased with decreasing void ratio and that $e - \log k$ relationships are non-linear. Non-linear $e - \log k$ relations have been reported by Samarasinghe et al. (1982) and Raymond (1966) for reconstituted normally consolidated clays. Non-linear $e - \log k$ relations have also been reported by Ahmed (1977), Tavenas et al. (1983) and Siddique and Safiullah (1995) for undisturbed natural soft Ranjit clay of Bangkok, Matagami clay of Canada and reconstituted Dhaka clay respectively.

In an attempt to examine the applicability of the theoretical relation (Eqn. 7) as proposed by Samarasinghe et al. (1982), $\log[k(1+e)]$ versus $\log e$ have been plotted for the reconstituted samples of four coastal soils. These plottings are shown in Fig. 9. It can be seen from Fig. 9 that approximately linear $\log[k(1+e)] - \log e$ relationships have been obtained for the coastal soils. Linear $\log[k(1+e)] - \log e$ relationships have also been obtained for the undisturbed samples of the four coastal soils. This suggests that the permeability prediction model proposed by Samarasinghe et al. (1982) may be applicable for these coastal soils. The permeability parameters n and C of the reconstituted and undisturbed samples of the four coastal soils studied were determined from the linear plots of $\log[k(1+e)]$ versus $\log e$. The magnitudes of these parameters together with corresponding correlation coefficients are presented in Tables 9 and 10 respectively for reconstituted and undisturbed samples. It can be seen from Table 9 that for reconstituted samples, permeability parameter n varied between 1.7 and 4.1, while permeability parameter C varied from 7.2×10^{-10} m/s to 27.6×10^{-10} m/s. Table 10 shows the values of n and C parameters for the undisturbed samples which varied from 3.1 to 9.1, and from 11.6×10^{-10} m/s to 1264.7×10^{-10} m/s respectively. From the data presented in Tables 9 and 10, it is evident that the values of permeability parameters n and C for the undisturbed samples of the coastal soils are significantly higher than those for the respective reconstituted samples.

Conclusions

Compressibility and permeability characteristics of reconstituted and undisturbed samples of coastal soils of Bangladesh have been studied by

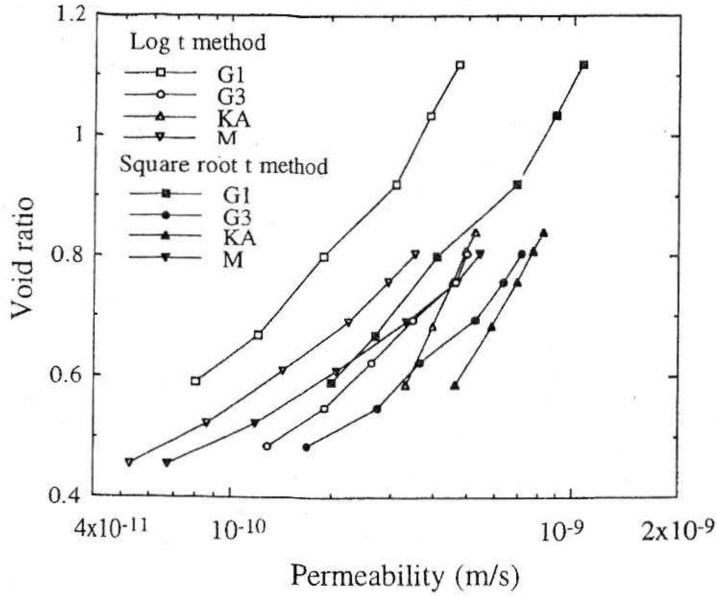

FIGURE 7 : Void Ratio Vs. Log Permeability Plots for Reconstituted Samples of Four Coastal Sites

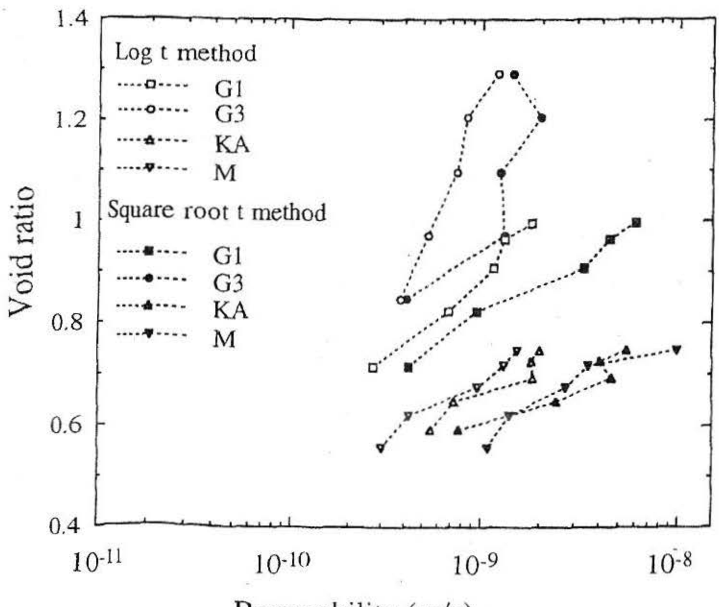

FIGURE 8 : Void Ratio Vs. Log Permeability Plots for Undisturbed Samples of Four Coastal Sites

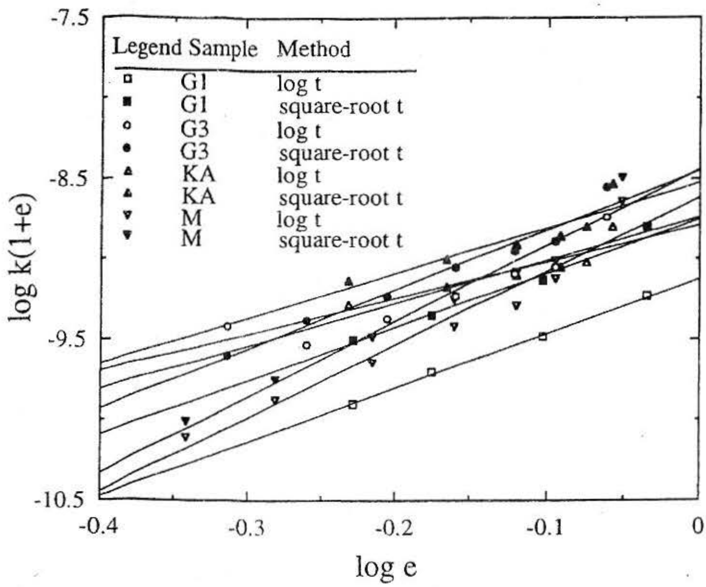

FIGURE 9 : $\log k(1+e)$ Vs. $\log e$ Plots for Reconstituted Samples of Four Coastal Soils

performing incremental loading one-dimensional consolidation tests. The major findings and conclusions are summarized below.

The values of compression Index, C_c and swelling index, C_s for reconstituted samples vary between 0.29 and 0.40 and 0.028 to 0.046, and for undisturbed samples between 0.22 to 0.42 and 0.037 to 0.062 respectively. Samples of Gohira which has high natural water content the C_c

Table 9
Values of Permeability Parameters for Reconstituted Samples of Coastal Soils from One-dimensional Consolidation Test

Soil Type	$\log t$ Fitting Method			$\sqrt{t}$ Fitting Method		
	n	C (10^{-10}m/sec)	Correlation Coefficient r	n	C (10^{-10}m/sec)	Correlation Coefficient r
G1	3.3	7.2	0.999	3.2	16.8	0.992
G3	2.1	13.0	0.909	3.2	27.6	0.997
KA	1.7	12.6	1.0	2.1	21.9	0.997
M	3.9	16.1	0.998	4.1	25.1	1.0

Table 9
Values of Permeability Parameters for Undisturbed Samples of Coastal Soils from One-dimensional Consolidation Test

Soil Type	log t Fitting Method			$\sqrt{t}$ Fitting Method		
	n	C (10^{-10}m/sec)	Correlation Coefficient r	n	C (10^{-10}m/sec)	Correlation Coefficient r
G1	6.1	38.2	0.988	9.1	134.6	0.986
G3	3.1	11.6	0.991	3.5	18.6	0.877
KA	6.5	255.3	0.956	8.5	1264.7	0.958
M	6.4	179.5	0.985	7.2	903.6	0.929

values of the undisturbed samples are higher than those of reconstituted samples. For the samples from Kalapara and Mognama which have comparatively low natural water content, however, the C_c values of the undisturbed samples are lower than those of reconstituted samples. It is also evident that C_s of undisturbed samples of Gohira are considerably higher than those of reconstituted samples while the C_s values of undisturbed samples of Kalapara and Mognama are slightly higher than that of reconstituted samples.

It is evident that the values of m_v and m_s of reconstituted samples are comparatively higher than undisturbed samples. It is also evident that for the reconstituted samples c_v value generally increased with increasing levels of consolidation stress. For both reconstituted and undisturbed samples, the values of c_v obtained using log t method are less than those obtained using $\sqrt{t}$ method. The values of c_v for the four reconstituted and undisturbed samples of the coastal soils varied between 3.3×10^{-4} to 30.2×10^{-4} cm^2/sec and 10.1×10^{-4} to 224.4×10^{-4} cm^2/sec respectively. For the undisturbed samples, the values of c_v, in general, reduced with the increase in consolidation stress. The values of c_v for the undisturbed samples have been found to be significantly higher than those for reconstituted samples.

The range of void ratio at the liquid limit (e_L) for four coastal soils is from 1.10 to 1.80, the range of e^*_{100} is from 0.73 to 1.02 and the range of C^*_c is from 0.24 to 0.40. The intrinsic compression lines (ICL) for samples of the four coastal soils are in good agreement with the ICL proposed by Burland (1990). The compressibility characteristics of reconstituted clays of the soils studied can be used as a basic frame of reference for interpreting the corresponding characteristics of natural coastal soils. Burland (1990) observed that the sedimentation compression curves for most natural clays lie well above the corresponding intrinsic compression curves. The clays which have these characteristics will be more sensitive and brittle than the reconstituted material.

For reconstituted samples, coefficient of permeability determined using $\log t$ method and $\sqrt{t}$ method varied from 0.53×10^{-10} m/s to 5.25×10^{-10} m/s and 0.67×10^{-10} m/s to 10.60×10^{-10} m/s respectively. For undisturbed samples, coefficient of permeability determined using $\log t$ method and $\sqrt{t}$ method varied from 3×10^{-10} m/s to 20×10^{-10} m/s and 4×10^{-10} m/s to 99×10^{-10} m/s respectively. Permeabilities computed from consolidation test using $\sqrt{t}$ fitting method are higher than those determined using $\log t$ fitting method. The values of permeability for the undisturbed samples of all the coastal soils are significantly higher than those for the reconstituted samples.

The $e - \log k$ relationships for samples of the four coastal soils have been found to be non-linear. Non-linear $e - \log k$ relations have been also been reported for intact and reconstituted clays (Tavenas et al., 1983; Samarasinghe et al., 1982). However, approximately linear $\log[k(1+e)] - \log e$ relationships have been obtained for coastal soils. The relation $k = Ce^n/(1+e)$ as proposed by Samarasinghe et al. (1982), therefore, can be used-to predict permeability - void ratio relation for coastal soils. Further research may be carried out using soil samples collected from various other locations of coastal region of Bangladesh in order to confirm the findings of the present work and also to validate the applicability of the above mentioned permeability - void ratio relation in coastal soils. It has been found that compared with the reconstituted samples, the value of permeability parameters n and C of the undisturbed samples are considerably higher. For reconstituted coastal samples investigated, permeability parameter n varied between 1.7 and 4.1, while permeability parameter C varied from 7.2×10^{-10} m/s to 27.6×10^{-10} m/s. For undisturbed coastal samples, permeability parameter n varied between 3.1 and 9.1, while permeability parameter C varied from 11.6×10^{-10} m/s to 1264.7×10^{-10} m/s.

Notations

C = permeability parameter

c_v = coefficient of consolidation

C_c = compression index

C_c^* = intrinsic compression index

C_s = swelling index

e = void ratio

e_L = void ratio at liquid limit

e_0 = in-situ void ratio

e'_1	=	void ratio at consolidation pressure, $\sigma_v' =$ 1 tsf (95.7 kPa)
e^*_{100}	=	void ratio at consolidation pressure, $\sigma_v' =$ 100 kPa
e^*_{1000}	=	void ratio at consolidation pressure, $\sigma_v' =$ 1000 kPa
G_s	=	specific gravity
H	=	average length of drainage path
k	=	coefficient of permeability
LL	=	liquid limit
m_s	=	coefficient of volume expansion
m_v	=	coefficient of volume compressibility
n	=	permeability parameter
PL	=	plastic limit
PI	=	plasticity index
t_{50}	=	time corresponding to 50% consolidation
t_{90}	=	time corresponding to 90% consolidation
I_v	=	void index
σ_v'	=	vertical effective consolidation stress
g_w	=	unit weight of water
w_1	=	water content at consolidation pressure, $\sigma_v' =$ 1tsf (95.7 kPa)
w_L	=	water content at liquid limit
w_n	=	natural water content

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