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Application of Shear Wave Velocity for Characterizing Clays from Coastal Regions

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Abstract

Demand for infrastructure development in coastal regions where the subsurface is often composed of soft clays, results from the desire for rapid industrialization. The frequently encountered coastal soil conditions often call for suitable ground improvement and modification techniques to prepare soft deposits for foundation construction. For appropriate site improvement, several engineering soil properties are desired which require expensive and time consuming field and laboratory testing. Under such circumstances, empirical correlations based on routinely determined soil properties would be very useful and economical in planning of the project. With this as a goal, undisturbed samples from three on-shore and off-shore sites of the coastal regions in India, where major infrastructure projects are being executed, were collected. These samples were tested for physical, chemical, mineralogical and geotechnical characteristics. In addition, shear wave velocity of specimens of these soils was determined by using bender elements. Based on this data, empirical correlations between void ratio, compression index and undrained shear strength with shear wave velocity of clays were developed. Because shear wave velocity can be easily determined in the laboratory, these correlations are found to be valuable for preliminary planning of the project. The utility of these correlations for preliminary characterization of the soft soil has also been demonstrated.

Keywords: *soft clays, shear wave velocity, void ratio, compression index, undrained shear strength*

1. Introduction

Demand for infrastructure development in coastal regions, where primarily clays are found in the subsurface, is continuously increasing to cater requirements of rapid industrialization. This leads to development of infrastructure (such as ports, harbor, roads and many offshore structures) on soft clays, which are found in abundance in these regions. Therefore, it is often essential to improve the existing soil so that moderate to heavy loading can be sustained by them. This often calls for adoption of suitable ground improvement techniques, which require engineering characteristics of these clays as a major input.

To achieve this, either in-situ testing or laboratory tests on undisturbed or remolded samples of clays are conducted. Results obtained from the Dynamic Cone Penetration Test, DCPT, and Static Cone Penetration test, SCPT, are employed to characterize soft clay strata (Jarvis and Knight, 2000; Anagnostopoulos *et al*., 2003). In addition, dilatometer tests (Marchetti, 1980) and field vane shear tests (Bjerrum, 1993) have been found to be quite useful for this purpose. Though, in-situ tests (viz., cone penetration test, standard penetration test and dilatometer test) are found to be quite useful and yielding excellent results, they are extremely time-consuming and cost-intensive.

This necessitates development of alternate methodologies that are easy to adopt, less time-consuming and cost-intensive for determining characteristics of the coastal clays. Hence, development of empirical correlations, based on the overall properties of clays, seems to be quite prudent for planning and execution of the project (Dewoolkar and Huzjak, 2005). In this direction, efforts have been made to develop correlations between the moisture content, specific surface area and proportions of clay minerals by earlier researchers (Bojana, 2006). Empirical correlations between hygroscopic moisture content of the soil with its surface area, cation exchange capacity, liquid limit, swelling potential and electrical properties (conductivity and dielectric constant) have also been proposed by the earlier researchers who have studied the influence of depositional and post-depositional geochemistry and its correlation with the geotechnical properties of the marine clays (Ohtsubo *et al*., 1995; Shah and Singh, 2006).

Researchers have developed correlations to estimate hydraulic conductivity of soft clays (Bryant *et al*., 1981; Tavenas *et al*., 1983). Based on the results obtained for clays from different

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locations, relationships have also been developed between the void ratio and permeability, which can also be employed for evaluating in-situ permeability (Bryant *et al*., 1981). Moreover, attempts have been made in the past to profile the shear strength, water content and bulk density of the sub soils occurring in the coastal regions (Bryant *et al*., 1981; Watabe *et al*., 2002). Researchers have also proposed several relationships between the undrained shear strength, void ratio and liquidity index of the fine-grained alluvial soils (Yilmaz, 2000) and it has been found that the normalized shear strength increases with increase in plasticity index (Skempton and Bjerrum, 1957). Similarly, compression index of soil can also be correlated to other physical and geotechnical parameters. Some of the researchers have found that compression index predicted by a simple multiple regression analysis involving the natural water content, the natural void ratio and liquid limit can reasonably evaluate the real soil compression index (Azzous *et al*., 1976; Koppula, 1981; Yoon *et al*., 2006). It has been demonstrated that the compression index can also be estimated from the plasticity index and the specific gravity of the soil (Rendon-Herrero, 1983). In addition, correlations between shear wave velocity and other geotechnical properties of the soils have been proposed by earlier researchers (Ohta and Goto, 1978; Blake, 1996). Though, these relationships are found to be of immense help to practicing engineers and research fraternity, they are mostly site-specific and limited to correlations between certain parameters.

With this in view, clay samples from distinctly located areas in the coastal region of India, where several infrastructure projects are being executed, were collected. It is worth mentioning here that these samples are mostly normally consolidated in nature. These samples were characterized for physical, chemical, mineralogical and geotechnical properties (viz., bulk density, dry density, void ratio, compression index, undrained shear strength and shear wave velocity). Based on these results, empirical correlations, which can be employed for estimating void ratio, compression index and undrained shear strength of clays, by employing shear wave velocity, have been developed. These correlations have been validated by using the data available in the literature. An attempt has been made to highlight the utility of shear wave velocity for estimating the parameters that are useful for preliminary planning of infrastructure projects, and evolving suitable ground modification scheme(s), on the substrata containing soft clays.

2. Experimental Investigations

Undisturbed samples of soils were procured from Western, Eastern and Southern coastal regions of India. These samples (designated as Sample A, Sample B and Sample C, respectively, as depicted in Table 1) were characterized for their physical and geotechnical characteristics. Moreover, since the sampling has been done from three different coastal regions, mineralogical and chemical characterization was done as described in the following sections, in order to know the composition of the clay samples.

Table 1. Details of the Clay Samples Used in the Present Study

Coastal region	Sample designation	Depth (m)	Location	Number of samples
Western	A	0-9	Offshore and Onshore	55
Eastern	B	0-51	Offshore and Onshore	61
Southern	C	0-44	Onshore	14

2.1 Mineralogical Characterization

The mineralogical composition of the sample was determined by conducting X-Ray Diffraction (XRD) Spectrometer (Phillips 2404, Holland) studies, by using a graphite monochromator and Cu-Kα radiation. The sample was scanned for 2θ ranging from 5^o to 80^o and the presence of minerals in it was confirmed with the help of the data files presented by the Joint Committee on Powder Diffraction Standards (JCPDS, 1994). XRD analysis of Samples A and C indicates presence of Anorthite, Quartz and Montmorillonite However, Samples B is found to contain mainly Quartz, Albite and Halloysite.

2.2 Chemical Characterization

Chemical composition of the sample, in the form of major oxides, was determined using an X-Ray Fluorescence setup (Phillips 1410, Holland) following the methodology proposed by Kolay and Singh (2001). The range of chemical composition (by % weight) of different samples is presented in Table 2. These samples contain silica (45 to 66%), alumina (11 to 19%) and ferric oxide (2 to 26%). It is quite interesting to note that these samples, though being located thousands of miles away from each other, exhibit identical chemical compositions.

The chloride and sulphite contents of the soil were obtained on an extract of 2:1 water to soil ratio (by weight), with the help of an Indion Easy test kit (Ion Exchange, India Ltd.). A water

Table 2. Chemical Composition (by % weight) of Different Samples

Oxide	Sample		
	A	B	C
SiO_2	45-53	58-66	50-60
Al_2O_3	11-15	14-17	17-19
Fe_2O_3	9-13	2-13	12-26
TiO_2	1-2	2-3	1-2
CaO	8-24	4-6	0.3-2
K_2O	1-2	1-2	1-2
MgO	2-3	2-3	1-2
P_2O_5	0.1-0.2	0-0.1	0.2-0.5
SrO	0.1-0.2	0-0.01	0.02-0.04
Na_2O	4-6	3-4	0.4-3
MnO	0-0.1	0-0.1	0-0.1
LOI	10-18	4-12	11-15

LOI: Loss on ignition

Table 3(a). Physical Properties of Samples A

Sample Designation	z (m)	G_s		% Fraction			LL (%)	PL (%)	PI (%)	γ (kN/m^3)	γ_d (kN/m^3)
		$G_{s(UP)}$	$G_{s(DB)}$	clay	silt	sand					
A1	2.25	2.77	2.85	44	49	7	50	33	17	14.9	7.9
A2	4.25	2.65	2.76	22	67	11	47	29	18	15.5	7.9
A3	2.50	2.81	2.86	35	65	-	45	31	14	14.9	7.9
A4	5.25	2.71	2.83	33	63	4	58	33	25	13.2	7.4
A5	0.75	2.82	2.86	56	44	-	66	34	32	14.9	7.9
A6	6.25	2.71	2.83	48	50	2	69	36	33	14.3	8.1
A7	2.75	2.76	2.82	31	69	-	63	34	29	14.5	7.7
A8	6.75	2.72	2.80	14	81	5	59	33	26	14.9	8.9
A9	1.75	2.76	2.81	13	87	-	62	34	28	13.4	7.2
A10	3.50	2.69	2.79	20	68	12	62	36	26	14.1	7.9
A11	2.25	2.69	2.85	34	66	-	69	36	33	14.5	7.6
A12	7.25	2.70	2.79	20	52	28	54	32	22	14.5	7.4
A13	1.25	2.70	2.75	20	76	4	58	35	23	14.2	7.6
A14	8.25	2.69	2.79	52	44	4	67	34	33	14.4	7.8
A15	3.35	2.69	2.76	24	72	4	67	34	33	14.4	8.4
A16	5.55	2.68	2.78	30	66	4	62	33	29	16.1	10.3
A17	4.35	2.64	2.77	28	67	5	67	32	35	16.1	10.1
A18	5.85	2.67	2.77	20	76	4	66	35	31	16.2	8.6
A19	1.80	2.69	2.79	33	67	-	56	36	20	12.4	6.6
A20	5.35	2.63	2.75	12	84	4	57	33	24	14.2	7.9
A21	3.25	2.74	2.78	35	60	5	67	37	30	14.9	9.1
A22	5.25	2.71	2.80	33	64	3	66	38	28	15.4	9.9
A23	2.75	2.60	2.75	30	62	8	55	34	21	10.4	5.4
A24	7.25	2.72	2.80	30	64	6	41	32	9	10.9	5.6
A25	3.25	2.69	2.79	33	61	6	56	35	21	12.2	6.6
A26	7.25	2.71	2.80	31	60	9	41	32	9	11.8	6.6
A27	4.25	2.61	2.75	27	70	3	63	36	27	10.9	5.6
A28	6.25	2.85	2.88	32	66	2	66	35	31	12.2	6.4
A29	7.25	2.79	2.84	62	36	2	67	35	32	14.0	8.4
A30	3.25	2.87	2.89	35	61	4	59	32	27	15.9	10.4
A31	7.25	2.86	2.89	41	56	3	48	34	14	13.1	7.6
A32	4.25	2.81	2.86	35	60	5	66	40	26	11.8	6.0
A33	7.25	2.80	2.85	43	53	4	62	36	26	13.6	8.5
A34	3.25	2.76	2.83	38	59	3	62	36	26	13.1	7.6
A35	5.25	2.82	2.86	62	34	4	56	30	26	13.6	8.1
A36	4.25	2.73	2.81	45	48	7	56	31	25	12.7	6.9
A37	7.25	2.85	2.88	44	51	5	58	33	25	13.1	7.9
A38	2.25	2.82	2.86	51	46	3	68	32	36	16.9	11.4
A39	5.35	2.82	2.86	41	53	6	74	34	40	16.7	9.8
A40	5.25	2.84	2.89	46	49	5	65	34	31	13.4	7.3
A41	6.85	2.81	2.86	55	42	3	77	38	39	13.9	7.8
A42	4.25	2.81	2.85	45	52	3	64	34	30	13.9	7.8
A43	4.75	2.83	2.83	53	44	3	63	34	29	14.4	8.3
A44	7.85	2.77	2.83	32	63	5	67	34	33	15.5	8.5
A45	3.25	2.78	2.85	55	44	1	66	34	32	16.1	9.1
A46	8.75	2.74	2.82	44	53	3	60	36	24	16.2	9.2
A47	4.25	2.74	2.88	51	48	1	60	33	27	16.0	9.1
A48	6.75	2.78	2.84	36	61	3	59	32	27	16.1	8.9
A49	4.25	2.75	2.87	57	42	1	59	35	24	16.1	9.2
A50	8.43	2.78	2.84	41	56	3	62	33	29	16.0	8.4
A51	6.20	2.59	2.89	36	56	8	61	33	28	15.2	8.3
A52	12.20	2.56	2.84	39	53	8	71	34	37	18.3	9.9
A53	3.20	2.62	2.76	37	50	13	77	32	45	14.9	10.0
A54	6.20	2.54	2.77	53	41	6	74	32	42	14.9	8.6
A55	9.15	2.53	2.78	58	36	6	75	28	47	16.0	9.7

Table 3 (b). Physical Properties of Samples B

Sample Designation	z (m)	G_s		% Fraction			LL (%)	PL (%)	PI (%)	γ (kN/m^3)	γ_d (kN/m^3)
		$G_{s(UP)}$	$G_{s(DB)}$	clay	silt	sand					
B1	3.50	2.63	2.83	31	58	11	36	18	18	15.5	9.9
B2	4.50	2.49	2.62	71	28	1	76	30	46	15.5	9.6
B3	5.50	2.56	2.68	54	44	1	65	27	38	16.6	10.1
B4	8.50	2.52	2.69	53	45	2	71	29	42	16.0	9.6
B5	11.50	2.53	2.86	57	42	1	71	38	33	15.5	9.7
B6	12.50	2.51	2.77	60	39	1	73	32	41	15.5	9.4
B7	13.50	2.49	2.68	70	29	1	83	38	45	15.3	8.9
B8	14.50	2.61	2.54	52	47	1	73	35	38	14.3	8.8
B9	17.00	2.51	2.62	52	36	12	74	31	43	16.0	10.3
B10	18.50	2.48	2.68	67	32	1	81	34	47	15.5	9.4
B11	20.00	2.48	2.64	66	33	1	87	35	52	15.5	9.3
B12	21.50	2.48	2.66	64	35	1	78	31	47	16.6	10.1
B13	23.00	2.51	2.67	67	32	1	74	30	44	16.0	10.1
B14	24.50	2.45	2.61	67	28	5	93	32	61	15.5	8.9
B15	27.50	2.50	2.68	65	34	1	75	26	49	16.6	10.3
B16	30.50	2.55	2.65	58	40	2	82	33	49	16.0	9.1
B17	32.00	2.49	2.62	72	27	1	88	36	52	16.0	10.1
B18	33.50	2.53	2.59	65	34	1	78	33	45	16.6	10.6
B19	35.00	2.41	2.74	78	17	5	88	34	54	15.5	9.5
B20	36.50	2.51	2.62	69	30	1	85	36	49	15.5	9.9
B21	38.00	2.42	2.60	78	17	5	89	39	50	16.0	10.1
B22	39.50	2.52	2.67	56	41	3	82	35	47	15.5	9.8
B23	41.00	2.48	2.62	69	30	1	87	36	51	16.6	10.3
B24	42.50	2.48	2.57	70	30	0	86	35	51	14.9	8.5
B25	47.50	2.49	2.66	78	17	5	82	34	48	16.0	10.7
B26	48.50	2.41	2.62	73	27	0	91	36	55	16.0	10.2
B27	50.50	2.46	2.65	73	27	0	97	37	60	15.5	10.0
B28	18.90	2.50	2.68	74	15	11	94	44	50	15.0	8.6
B29	23.80	2.51	2.68	65	26	9	80	38	42	16.0	9.1
B30	24.80	2.54	2.70	57	26	17	77	38	39	16.0	9.3
B31	28.80	2.50	2.68	70	12	18	81	38	43	16.0	9.3
B32	29.30	2.51	2.68	74	17	9	78	41	37	16.0	9.6
B33	17.60	2.60	2.73	61	23	16	67	34	33	14.9	8.9
B34	26.80	2.54	2.70	63	25	12	81	39	42	16.0	9.6
B35	27.80	2.52	2.69	64	24	12	81	40	41	16.0	9.5
B36	28.80	2.53	2.69	56	8	36	72	25	47	16.6	9.9
B37	33.80	2.55	2.70	70	22	8	78	38	40	16.6	10.5
B38	45.80	2.57	2.72	59	25	16	62	31	31	18.3	12.9
B39	47.80	2.63	2.75	50	25	25	48	25	23	18.9	14.0
B40	50.60	2.57	2.72	64	21	15	69	32	37	18.9	13.0
B41	9.50	2.71	2.80	44	46	10	64	25	39	17.2	11.7
B42	11.20	2.74	2.82	30	63	7	51	24	27	18.3	13.4
B43	12.40	2.69	2.79	35	48	17	58	27	31	17.7	12.3
B44	13.40	2.88	2.90	41	44	15	58	27	31	17.7	12.2
B45	17.45	2.74	2.82	36	54	10	55	24	31	17.7	12.6
B46	18.30	2.68	2.78	47	51	2	72	29	43	18.3	13.0
B47	19.60	2.67	2.77	45	53	2	61	24	37	17.7	13.1
B48	33.30	2.64	2.76	40	57	3	57	24	33	18.3	13.5
B49	39.60	2.68	2.78	46	48	6	61	29	32	18.3	12.2
B50	57.35	2.62	2.75	43	42	15	65	25	40	18.3	13.6
B51	61.15	2.85	2.88	44	34	22	61	30	31	19.5	15.4
B52	62.43	2.84	2.87	64	34	2	78	36	42	17.7	12.6
B53	63.30	2.78	2.84	57	41	2	72	33	39	17.7	12.8
B54	64.20	2.71	2.80	62	34	4	78	31	47	17.7	12.8
B55	64.40	2.68	2.78	45	37	18	64	28	36	18.3	13.9
B56	37.25	2.74	2.82	29	65	6	48	22	26	17.7	13.4
B57	62.40	2.69	2.79	55	41	4	63	32	31	17.2	12.2
B58	37.25	2.86	2.89	27	66	7	46	21	25	18.9	14.7
B59	42.50	2.91	2.91	47	48	5	65	29	36	17.7	12.9
B60	62.40	2.72	2.80	60	36	4	77	33	44	17.7	12.8
B61	66.30	2.73	2.81	55	40	5	63	27	36	17.7	12.6

Table 3(c). Physical Properties of Samples C

Sample Designation	z (m)	G_s		% Fraction			LL (%)	PL (%)	PI (%)	γ (kN/m^3)	γ_d (kN/m^3)
		$G_{s(UP)}$	$G_{s(DB)}$	clay	silt	sand					
C1	4.62	2.63	2.71	16	58	36	65	40	25	13.7	6.0
C2	13.11	2.65	2.79	19	68	13	61	38	23	14.9	7.2
C3	16.12	2.53	2.61	16	75	9	65	34	31	14.3	6.7
C4	27.70	2.52	2.66	41	56	3	68	35	33	14.9	8.1
C5	16.80	2.47	2.56	7	84	9	57	36	21	14.3	6.5
C6	19.50	2.53	2.71	13	86	1	66	33	33	14.3	6.4
C7	22.80	2.67	2.71	32	67	1	63	35	28	14.9	7.6
C8	25.80	2.66	2.78	30	69	1	67	33	34	15.5	8.2
C9	28.90	2.60	2.77	35	64	1	61	28	33	16.6	9.6
C10	40.50	2.56	2.73	14	35	51	48	32	16	16.0	9.3
C11	31.75	2.51	2.79	56	38	6	93	34	59	20.0	14.7
C12	34.75	2.52	2.71	62	33	5	75	32	43	16.6	9.8
C13	37.80	2.47	2.72	57	42	1	61	34	27	16.6	10.1
C14	43.80	2.88	2.90	23	28	49	53	29	24	14.9	9.1

quality analyzer (Model PE 136, Elico Ltd., India), with glass electrode, was employed for measuring pH and electrical conductivity (EC) of the solutions of the soil with liquid to solid ratio (L/S=20). The soil solution was stirred continuously, by a magnetic stirrer, before measuring the pH. It was found that these samples contain less sulphite (5 to 30 ppm), however, the chloride content in these soils is quite high (200 to 1000 ppm). The pH of these samples is found to vary from 7 to 9 and their EC values range from 1 to 4 mS/cm.

2.3 Physical Characterization

Specific gravity, G_s, of the sample was determined with the help of an Ultra Pycnometer, (Quantachrome, USA), which utilizes Helium gas as the displacing fluid (ASTM D 5550), and conventional density bottle (ASTM D 554) and the results ($G_{s(UP)}$ and $G_{s(DB)}$, respectively) are presented in Table 3. Pycnometer is specifically useful for determining G_s of soils which either i) contain clay minerals, and hence exhibit swelling due to their interaction with water or ii) contain high organic content or iii) contain high percentages of CaO. A comparison of $G_{s(DB)}$ and $G_{s(UP)}$ for different samples is depicted in Fig. 1, which indicates that the specific gravity obtained from density bottle method is slightly higher than that obtained by employing the pycnometer.

The liquid limit, LL, and plastic limit, PL, of the sample were determined as per ASTM D 4318-93 and the results are listed in Table 3. As depicted in Fig. 2, most of the specimens from samples A, B and C belong to MH or CH group of soil, as per ASTM D 2487-93.

The particle size distribution characteristics of the sample were determined by conducting sieve and hydrometer analysis, as per ASTM D 422-63 and the results are listed in Table 3. The grain size distribution curves for three representative samples from

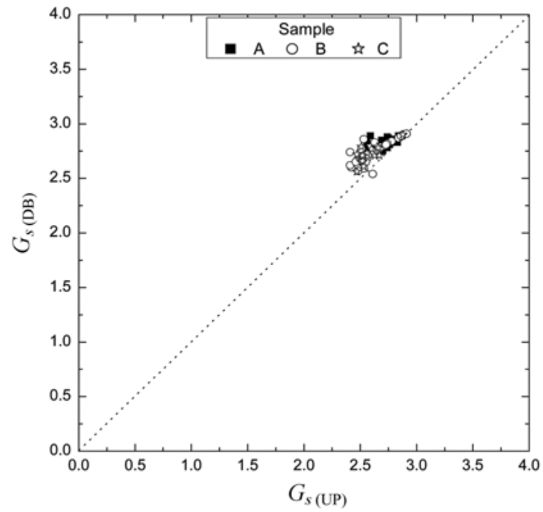

Fig. 1. Comparison of Specific Gravity Values obtained from Helium Gas Pycnometer and Density Bottle Methods

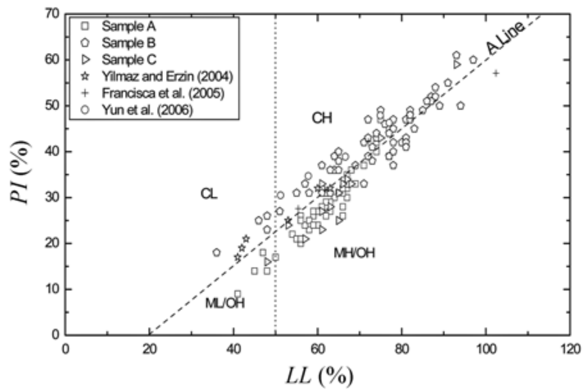

Fig. 2. The Classification of the Soil Samples based on the USCS

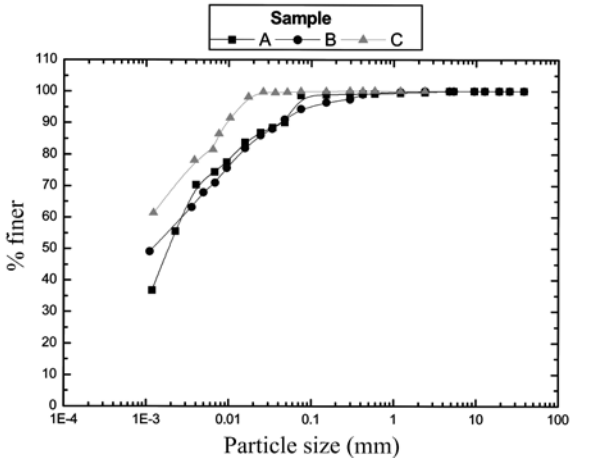

Fig. 3. Grain Size Distribution Curves for Representative Samples

group A, B and C (ref. Table 1) are presented in Fig. 3.

2.4 Geotechnical Characterization

The bulk unit weight, γ, and dry unit weight, γ_d, for these samples are presented in Table 3. Using this data, variations of γ, γ_d, shear wave velocity, V_s, and in-situ void ratio, e_o, along with depth, z, were developed, as depicted in Fig. 4. Except for e_o, which decreases with an increase in z, all other parameters increase as z increases.

Consolidation characteristics of the sample were determined as per ASTM D 2435-03. The consolidation characteristic for a representative sample has been presented in Fig. 5. The compression index, c_c, was computed and is listed in Table 4.

To determine V_s in the sample, the methodology developed by Bartake *et al*., (2008) was employed. From the UDS tubes, a specimen of the soil, which is slightly longer (≈100 mm) than the triaxial specimen was extruded. On both ends of this specimen a pair of bender elements (transmitter and a receiver) was embedded in such a way that they are co-linear. It is worth mentioning here that the bender elements used in this study were procured from the Centre for Offshore Foundation Systems, The University of Western Australia. These elements are constructed by bonding two piezoceramic materials together in such a way that a voltage applied to their faces causes one face to expand

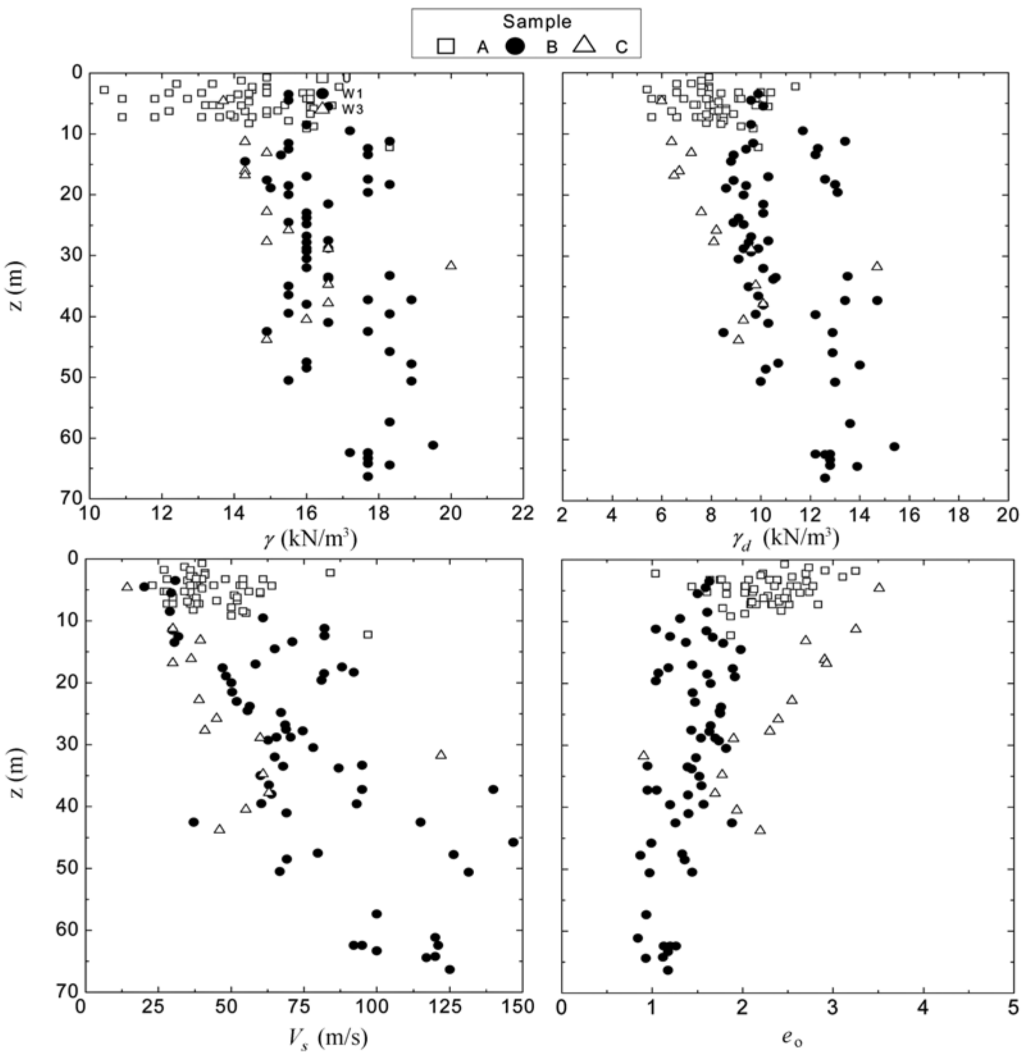

Fig. 4. The Variation of Different Properties of the Sample along with Depth

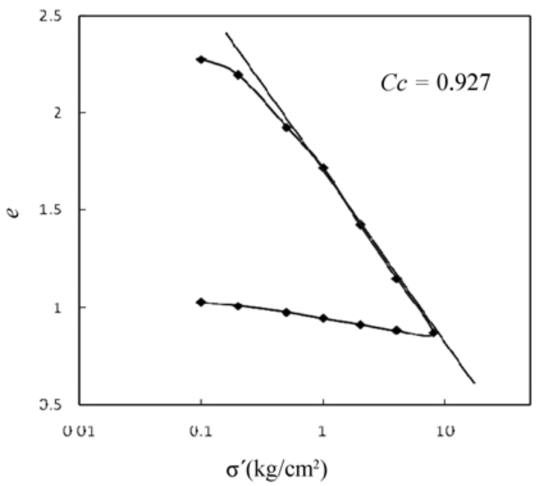

Fig. 5. Consolidation Characteristics of a Representative Sample

while the other face contracts. This causes the entire element to bend and generation of a voltage and vice-versa. These elements are 10 mm long and 5 mm wide. The transmitter is excited with a single Sine-wave of certain amplitude and frequency, *f*, which is generated from a function generator. The receiver is connected to a filter, amplifier circuitry, which is further connected to a digital oscilloscope. The oscilloscope also receives a direct Sine-wave from the function generator. With the help of a digital filter (Matlab 6.5), the Sine-wave recorded by the oscilloscope was processed to determine the travel time, *t*, of the shear wave. In the present study, the first arrival approach of the shear wave in time domain, as suggested by the earlier researchers (Lee and Santamarina, 2005; Bartake *et al*., 2008), has been considered for determining shear wave velocity, V_s. Later, V_s was computed by dividing the tip-to-tip distance of the bender elements with *t*, and results are presented in Table 4. This V_s was multiplied by the overburden stress correction factor, $(P_a/\sigma')^{0.25}$, proposed by Robertson *et al.* (1992) for simulating the in situ conditions, where P_a is the reference stress (=100 kPa) and σ' is the corresponding effective stress (in kPa).

After determining V_s, about 10 mm thick slices were cut from both ends of the soil specimen in order to prepare a standard sized triaxial sample. These slices were used for determining the gravimetric water content, *w*, of the sample as per ASTM D 2216-98. The unconsolidated undrained triaxial tests were conducted on the soil specimen, as per ASTM D 4767-04. These specimens were weighed and their bulk density γ was determined. A confining pressure ranging from 150 to 200 kPa was applied on the specimen for these tests. Later, the specimen was sheared at constant rate of strain of 1.25 mm/minute and its undrained shear strength, S_u, was determined, as listed in Table 4.

3. Interpretation of Test Results and Discussion

Fig. 6 presents the variation of V_s with void ratio, e_o, liquidity index, *LI*, liquid limit, *LL*, water content, *w*, clay content, *CL*, bulk density, γ, undrained shear strength, S_u, compression index,

Table 4(a). Geotechnical Characteristics of Samples A

Designation	z (m)	w (%)	e_o	c_c	S_u (kPa)	V_s (m/s)
A1	2.25	87.1	2.221	0.856	30	41
A2	4.25	95.5	2.531	0.969	17	44
A3	2.50	88.2	2.198	0.559	26	41
A4	5.25	77.8	2.736	0.937	20	27
A5	0.75	88.4	2.464	0.803	19	40
A6	6.25	76.6	2.401	0.686	22	37
A7	2.75	89.1	2.544	0.724	18	37
A8	6.75	66.1	2.093	0.611	29	45
A9	1.75	86.2	2.912	0.803	17	27
A10	3.50	77.1	2.708	0.934	20	35
A11	2.25	89.1	2.699	0.840	17	36
A12	7.25	95.4	2.515	0.871	17	36
A13	1.25	86.0	2.733	0.967	17	34
A14	8.25	83.9	2.425	0.884	20	37
A15	3.35	70.9	2.385	0.600	24	39
A16	5.55	55.7	1.820	0.470	39	60
A17	4.35	59.3	1.436	0.470	46	64
A18	5.85	87.9	2.278	0.927	20	51
A19	1.80	89.1	3.249	0.926	15	36
A20	5.35	79.5	2.635	1.060	40	36
A21	3.25	62.9	1.753	0.770	86	48
A22	5.25	55.2	1.604	0.600	99	56
A23	2.75	93.2	3.105	1.20	21	36
A24	7.25	93.4	2.832	0.950	16	37
A25	3.25	84.6	2.295	0.720	21	40
A26	7.25	79.2	2.322	0.890	22	39
A27	4.25	94.4	2.601	0.770	17	38
A28	6.25	89.8	2.492	0.950	14	38
A29	7.25	66.4	2.090	0.850	22	38
A30	3.25	53.3	1.638	0.540	31	61
A31	7.25	71.2	2.328	1.370	19	28
A32	4.25	95.9	2.776	0.850	14	37
A33	7.25	60.1	2.094	0.770	27	36
A34	3.25	71.2	2.382	1.040	11	28
A35	5.25	68.9	2.111	0.790	27	34
A36	4.25	82.3	2.353	0.980	25	23
A37	7.25	64.2	2.148	0.970	15	30
A38	2.25	48.1	1.035	0.420	30	84
A39	5.35	71.0	2.023	0.680	25	60
A40	5.25	83.0	2.484	0.750	20	28
A41	6.85	79.0	2.102	0.740	24	35
A42	4.25	76.0	1.820	0.570	29	36
A43	4.75	73.0	2.124	0.650	9	40
A44	7.85	83.0	1.778	0.710	11	50
A45	3.25	77.0	2.064	0.960	16	54
A46	8.75	76.0	2.025	0.750	15	55
A47	4.25	77.0	2.024	0.580	19	53
A48	6.75	87.0	2.405	0.680	19	52
A49	4.25	75.0	2.030	0.740	20	54
A50	8.43	90.0	2.030	1.005	19	54
A51	6.20	82.11	2.462	0.930	14	52
A52	12.20	84.75	1.867	0.660	39	97
A53	3.20	49.25	1.765	0.480	15	38
A54	6.20	73.98	2.234	0.590	10	30
A55	9.15	65.12	1.869	0.580	17	50

Table 4(b). Geotechnical Characteristics of Samples B

Designation	z (m)	w (%)	e_o	c_c	S_u (kPa)	V_s (m/s)
B1	3.50	55.1	1.631	0.503	22	31
B2	4.50	60.5	1.589	0.641	27	20
B3	5.50	63.0	1.504	0.605	22	29
B4	8.50	65.5	1.607	0.653	13	29
B5	11.50	59.2	1.599	0.622	15	30
B6	12.50	64.5	1.666	0.665	16	32
B7	13.50	71.2	1.783	0.747	16	31
B8	14.50	63.0	1.977	0.714	24	65
B9	17.00	55.5	1.441	0.585	31	58
B10	18.50	62.8	1.609	0.671	28	82
B11	20.00	64.9	1.643	0.704	33	50
B12	21.50	63.5	1.445	0.635	30	50
B13	23.00	57.6	1.475	0.601	31	52
B14	24.50	73.6	1.746	0.782	33	56
B15	27.50	61.3	1.432	0.614	36	69
B16	30.50	76.7	1.817	0.776	38	78
B17	32.00	59.5	1.483	0.653	45	65
B18	33.50	56.8	1.389	0.594	49	68
B19	35.00	62.3	1.519	0.673	42	60
B20	36.50	56.9	1.544	0.643	46	63
B21	38.00	58.6	1.396	0.637	47	64
B22	39.50	58.1	1.568	0.644	48	60
B23	41.00	60.6	1.401	0.641	60	69
B24	42.50	73.2	1.879	0.783	22	37
B25	47.50	49.3	1.332	0.560	54	80
B26	48.50	56.5	1.359	0.626	55	69
B27	50.50	54.1	1.443	0.648	55	67
B28	18.90	74.9	1.915	0.821	36	48
B29	23.80	75.9	1.759	0.756	35	56
B30	24.80	73.4	1.753	0.734	29	67
B31	28.80	72.4	1.694	0.731	38	70
B32	29.30	74.5	1.737	0.739	29	63
B33	17.60	65.7	1.891	0.693	19	47
B34	26.80	66.6	1.645	0.695	30	68
B35	27.80	67.0	1.630	0.694	37	75
B36	28.80	66.5	1.538	0.648	32	66
B37	33.80	58.5	1.435	0.610	38	87
B38	45.80	41.8	0.991	0.404	98	147
B39	47.80	34.3	0.869	0.305	101	126
B40	50.60	45.0	0.972	0.437	70	132
B41	9.50	46.7	1.309	0.490	72	61
B42	11.20	36.4	1.041	0.355	117	82
B43	12.40	44.4	1.196	0.441	105	82
B44	13.40	46.1	1.374	0.481	101	71
B45	17.45	40.9	1.177	0.412	107	88
B46	18.30	41.4	1.069	0.446	127	92
B47	19.60	35.5	1.041	0.381	99	81
B48	33.30	35.2	0.949	0.351	145	95
B49	39.60	50.1	1.198	0.477	111	93
B50	57.35	34.9	0.934	0.370	138	100
B51	61.15	26.1	0.841	0.301	152	120
B52	62.43	40.9	1.261	0.496	134	121
B53	63.30	38.6	1.175	0.452	100	100
B54	64.20	38.4	1.119	0.459	187	120
B55	64.40	31.7	0.930	0.352	157	117
B56	37.25	32.4	1.050	0.329	179	140
B57	62.40	40.3	1.195	0.437	114	95
B58	37.25	28.9	0.948	0.288	137	95
B59	42.50	37.3	1.257	0.440	147	115
B60	62.40	38.8	1.129	0.460	136	92
B61	66.30	40.9	1.173	0.436	148	125

Table 4(c). Geotechnical Characteristics of Samples C

Designation	z (m)	w (%)	e_o	c_c	S_u (kPa)	V_s (m/s)
C1	4.62	128.0	3.510	1.270	14	14
C2	13.12	108.0	2.699	1.018	25	40
C3	16.12	114.0	2.906	1.096	12	36
C4	27.70	85.0	2.303	0.860	29	41
C5	16.80	119.5	2.930	1.102	24	30
C6	11.25	124.4	3.253	1.210	24	30
C7	22.80	94.9	2.545	0.935	35	39
C8	25.80	89.3	2.395	0.894	28	45
C9	28.90	73.7	1.898	0.714	39	60
C10	40.50	72.0	1.935	0.673	28	55
C11	31.75	36.5	0.904	0.457	112	122
C12	34.75	69.9	1.774	0.715	64	61
C13	37.80	64.5	1.695	0.634	36	63
C14	43.80	64.1	2.194	0.698	77	46

c_c, and effective stress, σ', for Samples A, B and C. The regression coefficient, R^2, for V_s versus LL and CL relationships are found to be extremely poor. However, the V_s versus e_o, LI, w, γ, S_u, c_c and σ' relationships yield R^2 values ranging from 0.6 to 0.74. Though, R^2 for these relationships is also quite less than unity, they can be employed for estimating the required soil parameters with a fair degree of confidence.

However, as e_o, c_c and S_u are the parameters that are most important for determining the response of loading on the soft soils, due to the infrastructure development projects, they were related with V_s as depicted in Fig. 7, Fig. 8 and Fig. 9, respectively. Similar data from literature (Yilmaz and Erzin, 2004; Francisca *et al*., 2005; Yun *et al*., 2006; Landon *et al*., 2007) have also been superimposed in these figures in order to check the validity of these relationships.

As soil samples were retrieved from different depths (up to 70 m), the influence of geological ageing on V_s also becomes important (Hardin and Black, 1968; Humphries and Wahls, 1968; Afifi and Woods, 1971; Anderson *et al*., 1978; Athanasopoulos and Woods, 1985; Schmertmann, 1992). However, as e_o, c_c and S_u, which represent engineering properties of the soil, also equally get affected by the geological aging (Humphries and Wahls, 1968; Schmertmann, 1992); V_s determined from these engineering properties would, implicitly, include the effect of geological ageing. It must be noted that the relationships proposed in the present study are based on the laboratory results obtained from the testing of undisturbed samples, and hence their validity for estimating e_o, c_c or S_u based on in situ measurements of V_s should also be ascertained. However, this is beyond the scope of the present study.

The trend depicted in Fig. 7 can be expressed by Eq. (1), with R^2 and standard error of estimation, SEE, values of 0.65 and

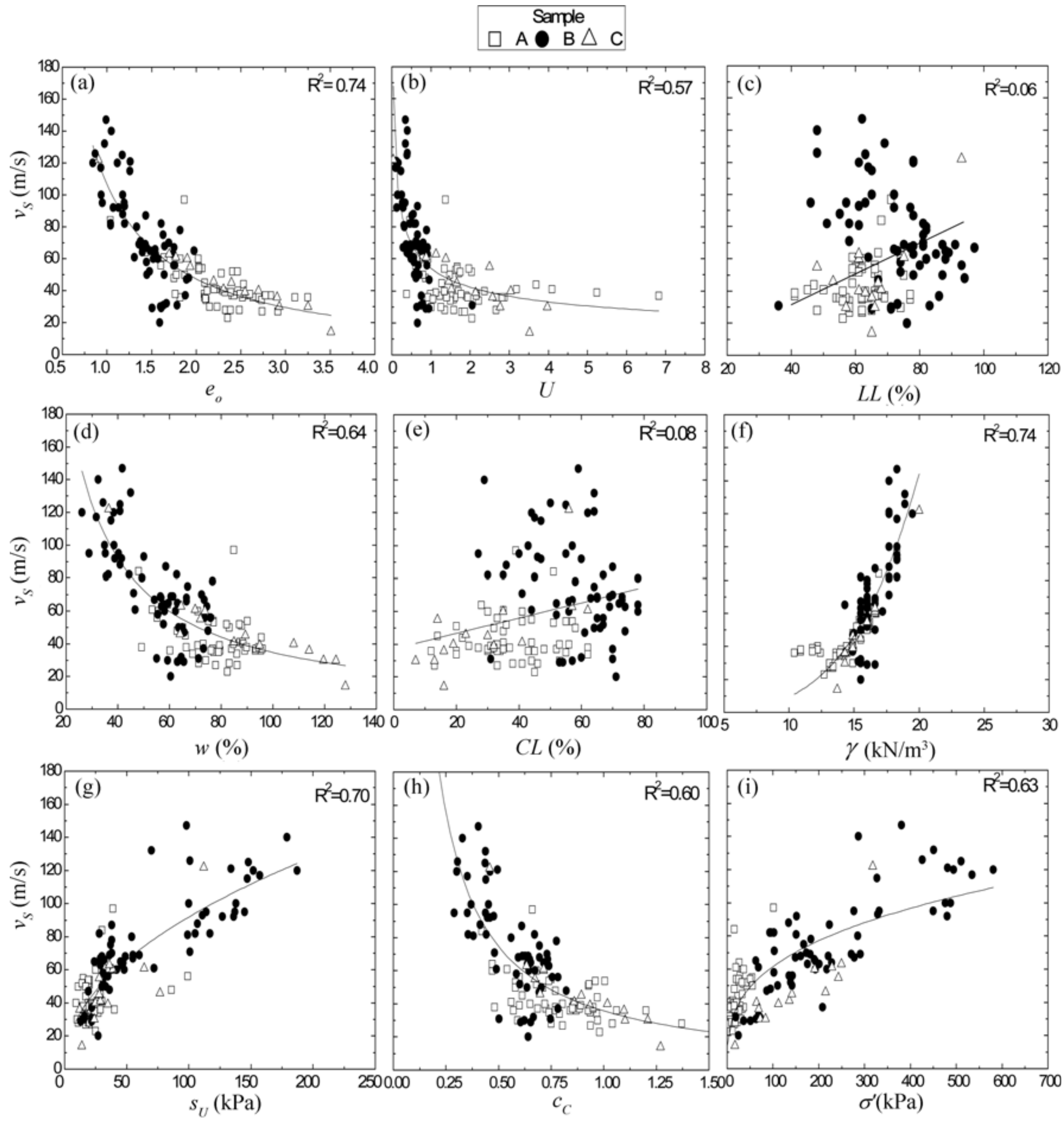

Fig. 6. The Variation of the Shear Wave Velocity with Different Parameters

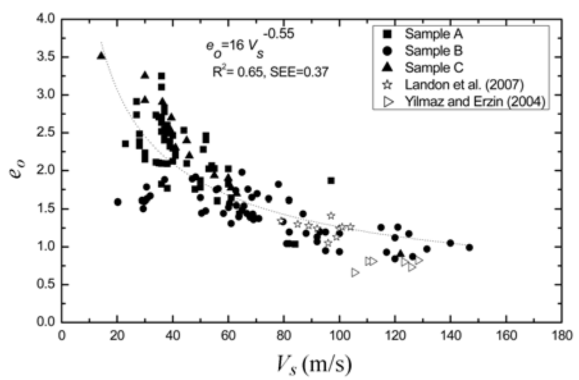

Fig. 7. The Variation of the In situ Void Ratio with the Shear Wave Velocity

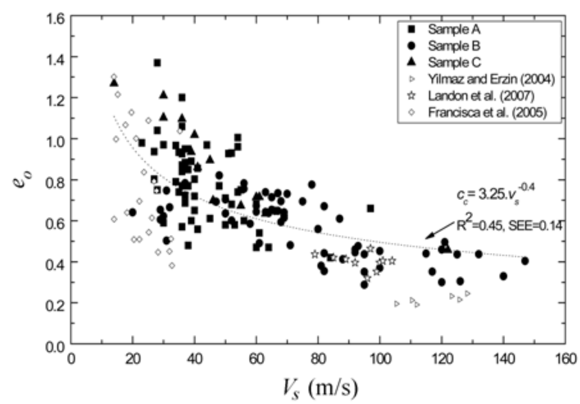

Fig. 8. The Relationship between the Compression Index and the Shear Wave Velocity

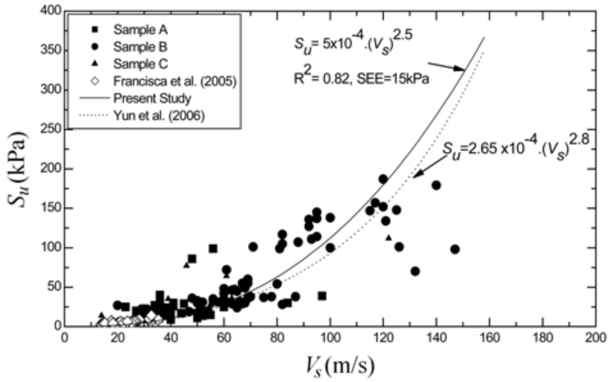

Fig. 9. The Relationship between the Undrained Shear Strength and the Shear Wave Velocity

0.37, respectively.

$$e_o = 16 \cdot V_s^{-0.55} \quad (1)$$

In order to validate Eq. (1), experimentally obtained void ratio, designated as $e_{o(\mathrm{Lit.})}$, for the samples of soft clays (Alba, 2004; Pitilakis *et al*., 2004; Landon *et al*., 2007) were plotted against the computed void ratio, designated as $e_{o(\mathrm{Eq.\ (1)})}$, by using Eq. (1), as depicted in Fig. 10. The resultant coefficient of correlation, CR is found to be 1.1 for the linear fit that passes through the origin. This indicates that under the circumstances when soil samples can not be retrieved from the subsurface, Eq. (1) can be used for determining e_o value quite easily.

A relationship between V_s, e_o and σ', represented by Eq. 2, has been reported in the literature (Hardin and Black, 1968; Hardin and Drnevich, 1972; Hardin, 1978; Wang and Kuwano, 1999; Okur and Ansal, 2007).

$$V_s = [A \cdot f(e) \cdot (\sigma')^n / \rho]^{0.5} \quad (2)$$

where, ρ is the mass density of the sample; A, B and n are empirical parameters and $f(e)$ [$=(B-e)^2/(1+e)$] is a function of the void ratio, which primarily depends upon the type of the soil, and B is generally assumed to be 2.973.

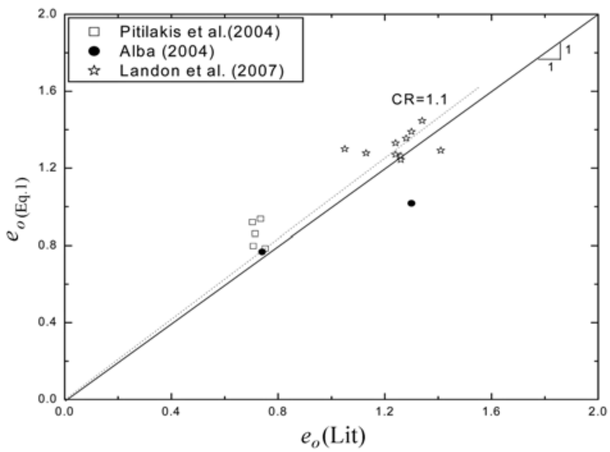

Fig. 10. Validation of Equation 1 using the Data Available in Literature

By fitting laboratory data [represented as $V_{s(\mathrm{Expt})}$] to Eq. (2), with the help of Mathematica4.1, A and n were found to be 600 and 0.5, respectively. It is worth mentioning here that a nonlinear equation can be formed by using the nonlinear fitting option in Mathematica, where the best fit parameters (i.e. coefficients) are chosen by considering the data for regression analysis provided by the mathematical program. In addition, V_s was computed [represented as $V_{s(\mathrm{Estm})}$] by employing Eqs. (1) and (2) and other relationships proposed by the earlier researchers (Hardin and Black, 1968; Hardin and Drnevich, 1972; Hardin, 1978; Wang and Kuwano, 1999; Okur and Ansal, 2007), as depicted in Fig. 11. The resultant coefficient of correlation, CR and regression coefficient, R^2 for these relationships have been obtained from the linear fits passing through zero. The value of the coefficient of correlation, CR, obtained by Eq. (1) and Eq. (2) is close to unity, as the linear fits for these two equations are close to the line which is 45° to the horizontal and the difference in the value of V_s predicted from Eq. (1) and Eq. (2) is limited to 20% only. This shows that both Eq. (1) and Eq. (2) can be used for predicting V_s in clays. The value of CR obtained by employing the relationships proposed by Wang and Kuwano (1999) and Okur and Ansal (2007) is 1.5, whereas it is 2.0 for the relationships proposed by Hardin and Black (1968), Hardin and Drnevich (1972) and Hardin (1978). The value of CR for Eq. (1) and Eq. (2) is found to be 1.15 and 0.85, respectively. Although, the value of CR is different, a reasonably good R^2 value (≈ 0.90) for these relationships has been obtained. As suggested in the literature (Okur and Ansal, 2007), this discrepancy in CR can be attributed to different methodologies (viz., resonant column, cyclic triaxial, bender element test) adopted by different researchers.

Fig. 8 indicates that V_s can also be employed for estimating the compression index, c_c, of the soil sample, more easily than performing 1-D consolidation tests. With this in view, an attempt

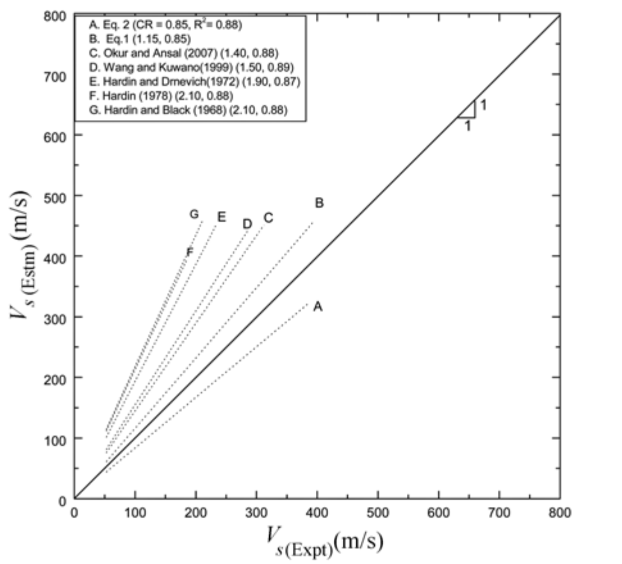

Fig. 11. Comparison of V_s Computed by Employing Eqs. (1) and (2) with the Relationships Suggested by the Earlier Researchers and the Laboratory Results for Different Samples

Table 5. Validation of Different Relationships Used for Computing c_c

Equation	Empirical relationship	Reference	R^2	CR
3	$0.4 \cdot (e_o - 0.25)$	Azzous *et al*. (1976)	0.86	0.93
4	$0.01 \cdot w$	Lav and Ansal (2001)	0.86	0.96
5	$0.141 \cdot G_s^{1.21} \cdot \{(1+e_o)/G_s\}^{2.38}$	Rendon-Herrero (1983)	0.85	0.87
6	$0.37 \cdot (e_o + 0.003 \cdot LL + 0.0004 \cdot w - 0.34)$	Azzous *et al*. (1976)	0.87	0.93
7	$-0.156 + 0.411 \cdot e_o + 0.00058 \cdot LL$	Al-Khafaji and Andersland (1992)	0.86	0.89
8	$0.267 \cdot e^{0.017 \cdot w}$	Yoon *et al*. (2006)	0.63	1.26
9	$0.286 \cdot e^{0.586 \cdot eo}$		0.67	1.29
10	$0.342 \cdot e^{0.014 \cdot LL}$		0.16	1.20
11	$0.2343 \cdot (LL/100) \cdot G_s$	Nagaraj and Murthy (1985)	0.19	0.54
12	$0.009 \cdot w + 0.005 \cdot LL$	Koppula (1981)	0.64	1.29
13	$0.01 \cdot (w - 5)$	Azzous *et al*. (1976)	0.73	0.9
14	$0.01 \cdot (w - 7.549)$	Rendon-Herrero (1983)	0.73	0.86
15	$0.246 + 0.43 \cdot (e_o - 0.25)$	Cozzolino (1961)	0.53	1.31
16	$0.208 \cdot e_o + 0.0083$	Bowles (1989)	0.61	0.56
17	$0.156 \cdot e_o + 0.0107$		0.60	0.43

has been made to estimate c_c from V_s. However, as c_c has not been reported by the researchers (Yilmaz and Erzin, 2004; Landon *et al*., 2007; Francisca *et al*., 2005) who have reported V_s and S_u, Equations 3 to17, presented in Table 5, were employed for estimating c_c. Validation of these relationships was also done by employing experimentally generated data for Samples A, B and C. Table 5 lists R^2 and CR between experimentally obtained c_c and the computed c_c values. It has been observed that only Equations 3 to 7 yield a higher R^2 value and a CR which is quite close to unity. Further, in order to select the most efficient relationship between Eqs. (3) to (7), results reported in literature (Yilmaz and Erzin, 2004; Landon *et al*., 2007) were compared, as depicted in Fig. 12. The resultant coefficient of correlation, CR and regression coefficient, R^2 for these relationships were obtained by the linear fits passing through zero. Eqs. (3) and (4) yield CR values which are very close to unity. As such, these equations were employed for estimating c_c values for the soil samples, which were used in the studies conducted by Landon *et al*. (2007). Later, c_c versus V_s relationship was developed as depicted in Fig. 8. The trend depicted in Fig. 8 can be expressed as:

$$c_c = 3.25 \cdot V_s^{-0.4} \tag{18}$$

Eq. (18) yields R^2 and SEE values of 0.45 and 0.14,

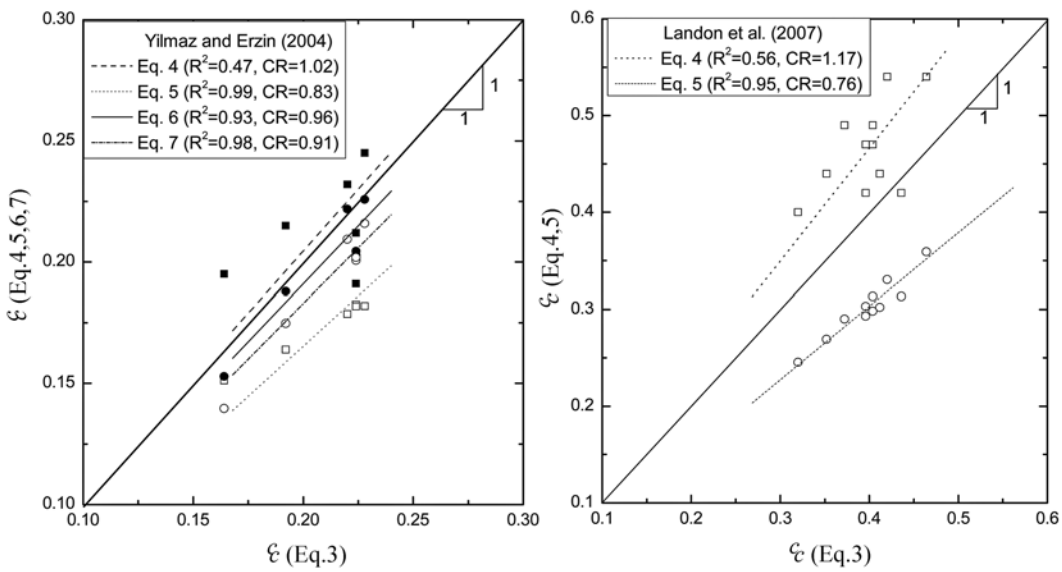

Fig. 12. Comparison of the Compression Indices obtained from Eqs. (3) to (7)

respectively. A very low value of R^2 can be attributed to too much scatter in the value of c_c, particularly for V_s < 30 to 40 m/s. This indicates that under the circumstances when soil samples can not be retrieved from the subsurface, Eq. (18) can be employed for determining c_c value, quite easily.

Earlier researchers (Seed and Idriss, 1970; Anderson *et al*., 1978; Yun *et al*., 2006) have suggested relationships between V_s and S_u. With this in view, an attempt has been made to correlate these two parameters in the present study. As depicted in Fig. 9, V_s can also be employed directly for estimating the undrained shear strength, S_u, of the soil samples. The trend depicted in Fig. 9 can be expressed as:

$$S_u = \alpha \cdot V_s^{\beta} \tag{19}$$

where, S_u is in kPa and V_s is in m/s, $\alpha = 5\times10^{-4}$ and $\beta = 2.5$. Incidentally, these values are found to be matching well with the values reported in the literature (Francisca *et al*., 2005; Yun *et al*., 2006).

Eq. (19) yields R^2 and SEE equal to 0.82 and 15, respectively. This demonstrates that Eq. (19) can be employed for computing S_u of the soft clays if their V_s is known.

Here, it is worth mentioning that the soil samples considered in this study are normally consolidated and hence chances of anisotropic consolidation are less (Blake, 1996). These samples also exhibit linear relationships represented by Eq. (20) and (21), as depicted in Fig. 13 and Fig. 14.

$$\ln(\sigma') = a \cdot \ln(G_{\max}) + b \tag{20}$$

$$S_u = k \cdot \sigma' \tag{21}$$

where $G_{\max}$ is the shear modulus (in kPa) and a, b and k are constant parameters.

Substituting for $G_{\max}$ (=$\rho \cdot V_s^2$, where ρ is the density of the sample), Eq. (20) assumes the following form:

$$S_u = k \cdot \rho^{a} \cdot V_s^{2a} \cdot e^{b} \tag{22}$$

Further, when Eq. (22) is compared with Eq. (23) (Skempton, 1957), R^2 and CR are found to be 0.75 and 0.8 respectively, as depicted in Fig. 15. However, limitations of Eq. (23) are well known as it does not include in it the other soil specific parameters which are responsible for determination of S_u.

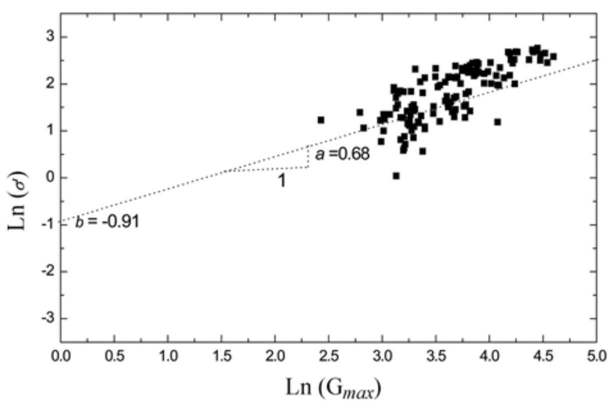

Fig. 13. lnσ' versus ln G_{max} Relationship

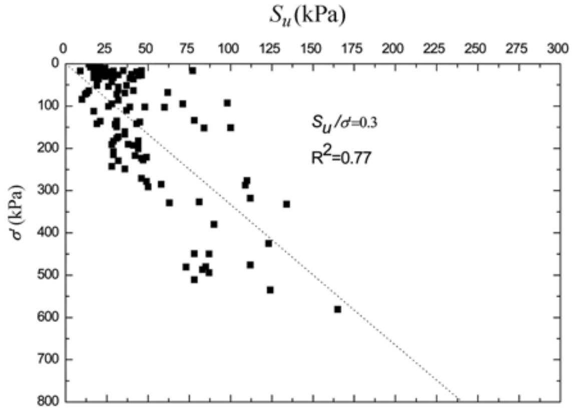

Fig. 14. S_u versus σ' Relationship

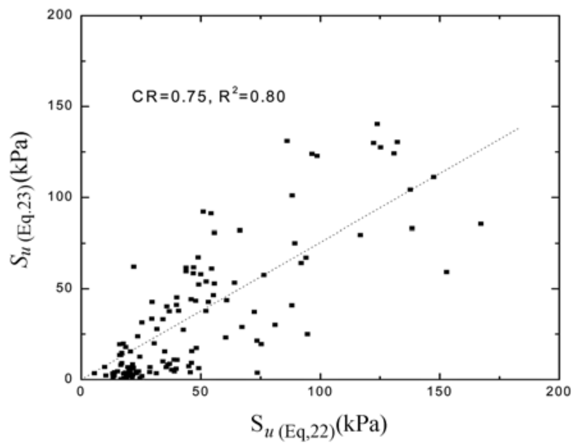

Fig. 15. Validation of Eq. (22)

$$S_u = \sigma'(0.11 + 0.0037PI) \tag{23}$$

Further, using Mathematica 4.1 (2000), a relationship (Eq. 24) between S_u, γ and LL was obtained. The variance for this relationship is found to be 0.06.

$$S_u = 10^{-4} \cdot \gamma^{6} \cdot LL^{-0.86} \tag{24}$$

where S_u, γ and LL are in kPa, kN/m^3 and %, respectively.

Using Eq. (24), the estimated undrained shear strength values, $S_{u(\text{estm.})}$ and $S_{u(\text{expt.})}$, respectively, were obtained and plotted as depicted in Fig. 16. The best fit yields R^2 and SEE of 0.61 and 24 kPa, respectively. However, the entire scatter is reasonably within 95% prediction limits (depicted as XX and X'X').

Mathematica 4.1 has also been used to develop a correlation (Eq. (25)) between the undrained shear strength, shear wave velocity, clay content and the moisture content.

$$S_u = 100 \cdot V_s^{0.9} \cdot w^{-1} \cdot CL^{-0.15} \tag{25}$$

where S_u is in kPa, V_s in m/s, and w and CL are in %.

Fig. 16 shows the best fit line with R^2 and SEE values of 0.83 and 14, respectively. The entire scatter is found to be within 95% prediction limits (depicted as XX and X'X'). It can be noted that

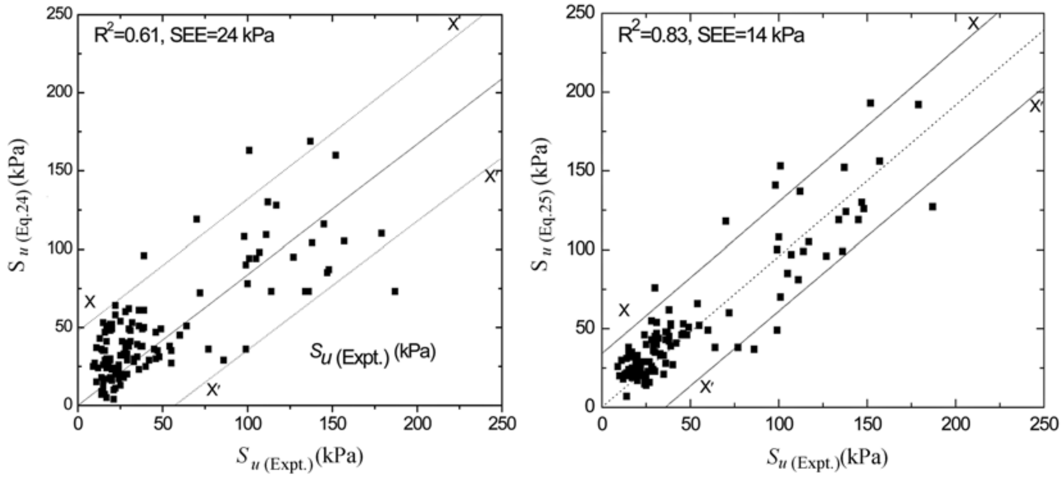

Fig. 16. Validation of Eqs. (24) and (25)

R^2 values for Eqs. (19), (22) and (25) are higher than the R^2 obtained from Eq. (24). This highlights the importance of the V_s for estimating the S_u.

As, Atterberg limits are not only easy to determine, but would facilitate in estimating the undrained shear strength of clays, indirectly (Yilmaz, 2000), the relationship presented by Eq. (26) was developed by plotting S_u for Samples A, B and C with respect to their *LI*, as depicted in Fig. 17.

$$S_u = 140 \cdot e^{(-LI/0.5)} + 16 \tag{26}$$

Further, S_u computed by using Eq. (27) (Yilmaz, 2000) was superimposed on Fig. 17.

$$S_u = e^{(0.026 - 1.21 \cdot LI)} \tag{27}$$

Eq. (27) yields good matching vis-à-vis experimentally obtained results for $0.25 < LI < 2.0$. Hence, Eq. (26) can be considered as a more generalized relationship, as it is based on the experimental results of soils which exhibit a wide range of *LI*, which has a strong bearing on its *LL*, *PI* and the natural moisture content.

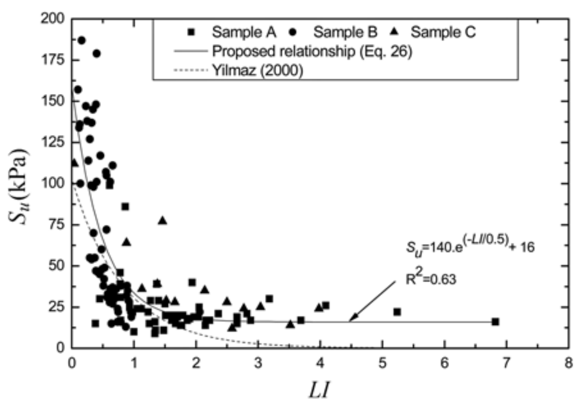

Fig. 17. The Variation of Undrained Shear Strength with the Liquidity Index

4. Conclusions

Based on the investigations on undisturbed samples of clays obtained from different coastal regions of India, and synthesis of the data available in the literature, it has been shown that the shear wave velocity in soils can be employed for estimating in situ void ratio, compression index and undrained shear strength, with a certain degree of confidence. The proposed correlations are found to be quite useful for obtaining the parameters that are essential for preliminary design of various infrastructure projects in the coastal areas. In addition, empirical relationships between the undrained shear strength, liquid limit, clay content, natural moisture content and liquidity index have also been found to be quite useful for soft soil characterization. It must be noted that most of these parameters can easily be determined by conducting simple laboratory tests on undisturbed soil samples.

Notations

A, B, n : Cmpirical parameter
CL : Clay content (%)
c_c : Compression index
e_o : In-situ void ratio
f : Frequency
$f(e)$: Function of the void ratio
G_{max} : Shear modulus
G_s : Specific gravity
LI : Liquidity index
LL : Liquid limit
PI : Plasticity index
PL : Plastic limit
S_u : Undrained shear strength
t : Time lag between receiver and transmitter waves
V_s : Shear wave velocity
w : Natural moisture content
z : Depth

γ : Bulk density
γ_d : Dry density
σ' : Effective stress

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