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Ilhan Chang[1] *and Gye-Chun Cho*[2]

A New Alternative for Estimation of Geotechnical Engineering Parameters in Reclaimed Clays by Using Shear Wave Velocity

ABSTRACT: The consolidation behavior of reclaimed clay can be categorized as large strain deformation. Findings from previous studies indicate that the effective stress and the void ratio are important geotechnical engineering parameters for the characterization of large strain consolidation behavior. However, existing in situ consolidation characterization methods of reclaimed clay cannot adequately estimate changes of the effective stress and void ratio during a consolidation process. This paper suggests an alternative method for estimating the geotechnical engineering parameters of reclaimed clays using a shear wave. An in situ self-weight consolidation process of reclaimed clay is simulated in laboratory while shear wave velocity is continuously measured. Experimental results show that there are single trends in relationships among the shear wave velocity, effective stress, void ratio, and geotechnical engineering parameters for a normally consolidated clay (e.g., reclaimed clay). As a practical application, the in situ parameters and the expected settlement are predicted by incorporating the obtained relationships with the in situ shear wave velocity. The predicted values are in good accordance with the values measured in field. Therefore, the proposed method can be used effectively for geotechnical engineering parameter estimations of reclaimed clay during/after self-weight consolidation with the aid of in situ seismic exploration techniques.

KEYWORDS: consolidation, geotechnical engineering parameter, reclaimed clay, shear wave, effective stress, void ratio

Introduction

Since Terzaghi (1923) introduced the one-dimensional consolidation theory, notable improvements have been made to overcome its assumptions and limitations (e.g., Mikasa 1963, Gibson et al. 1967, Gibson et al. 1981, Pradhan et al. 1995, Pane and Schiffman 1997, among others). In particular, the large strain consolidation model is suitable for describing soft soil behaviors in the field, such as the self-weight consolidation process of dredged and reclaimed clays. The effective stress and void ratio are governing geotechnical engineering parameters for large strain consolidation behavior characterizations.

The consolidation characterization of dredged and reclaimed clay deposits is important for the prediction of their long-term settlement and strength when an additional load (e.g., offshore structure construction) is applied. However, conventional in situ consolidation characterization methods have difficulty in monitoring the change of effective stress and consolidation state during consolidation process. Indeed, it is more difficult to simulate the in situ self-weight consolidation process of dredged and reclaimed clay in laboratory.

In field tests, settlement monitoring, pore pressure measurement, and cone penetration tests with pore pressure measurements (CPTu) are commonly used for in situ consolidation state estimations. However, the settlement tendency is discordant with the degree of pore pressure dissipation (Schiffman et al. 1984) and settlement monitoring cannot capture the local variation of the consolidation process. In addition, pore pressure measurement is associated with adverse factors stemming from the installation and control of the pore pressure gages, leading to unsatisfactory results (Tanaka and Sakagami 1989). Although the CPTu is suitable for exploring the behavior of soft ground, especially reclaimed clay (Lunne et al. 1997), the pore pressure dissipation testing in CPTu is time-consuming and thus cannot easily be applied to wide reclaimed sites. Accordingly, an alternative method of estimating geotechnical engineering parameters in reclaimed clays is needed.

In this study, the in situ self-weight consolidation process of reclaimed clay is simulated in laboratory while shear wave velocity is continuously measured. Given the difficulty of obtaining undisturbed soil samples from reclaimed sites, a laboratory sedimentation test is performed to duplicate an in situ soil composition. The in situ self-weight consolidation process is reproduced through laboratory consolidation testing. High depth conditions are readily replicated at this small specimen scale. The shear wave velocity is continuously measured inside the soil specimen so as to estimate the change of the effective stress and the void ratio during self-weight consolidation. For verification of the presented method, predicted geotechnical engineering parameters are compared with the parameters measured from in situ testing.

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[1]Ph.D. Student, Dept. of Civil and Environmental Engineering, Korea Advanced Institute of Science and Technology (KAIST), Daejeon 305-701, Korea, e-mail: ilhanchang@kaist.ac.kr

[2]Associate Professor, Dept. of Civil and Environmental Engineering, Korea Advanced Institute of Science and Technology (KAIST), Daejeon 305-701, Korea (Corresponding author), e-mail: gyechun@kaist.edu

Characteristics of Clay Reclamation

In Situ Reclamation Behavior

Dredged and reclaimed clays, in general, follow sedimentation and self-weight consolidation processes. The sedimentation process refers to the motion of particles in suspensions according to gravitational and electrical forces (Kynch 1952). Dispersed particles floc-

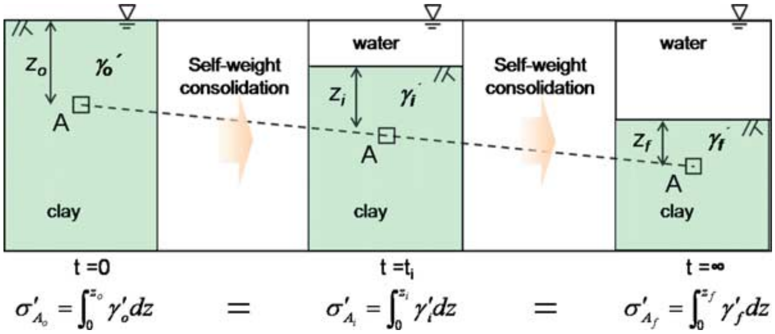

FIG. 1—*Concept of the destined effective stress condition during self-weight consolidation process.*

culate when the Coulombic force brings together oppositely charged surfaces and edges on particles thereby forming larger aggregates. The electrostatic attraction and repulsion between particles stem from excess charges on the surfaces of the clay particles and form the surrounding fluid chemistry (Rand et al. 1980). After accumulation and aggregation, particles undergo a gravity and buoyancy-induced free-falling process, and finally form the sediment. The electric accumulation and free-fall settling govern the initial conditions of sediments such as the fabric, particle packing, and density of the soil (Van Olphen 1977; Santamarina et al. 2001). The settling stage depends on the material type and initial circumstances, such as the water content, ionic concentration, and pH, at the beginning of sedimentation. Finer particles, with larger specific surfaces, have a longer settling stage.

The consolidation process is an applied stress-induced gradual squeezing out of water, which causes the soil particles to become packed together more tightly (Terzaghi 1923; Lambe and Whitman 1969). After sedimentation, the volume of the sediment decreases while the density increases subsequently. Specifically, the weight of the overburden soil behaves as an applied load on the underlying soil. This process is defined as self-weight consolidation.

Generally, once clay is dredged and reclaimed, sedimentation occurs first. The self-weight consolidation subsequently occurs below the transitional boundary between the free-fall settling and consolidation (Imai 1980; Been and Sills 1981). Thus, although particles over the transitional boundary flocculates and settles, the settled layer under the boundary undergoes consolidation. In the case of kaolinite-type clay, which is the most common type of clay in Korea, the free-fall settling process is relatively short; therefore, the self-weight consolidation process is particularly significant for the characterization of reclaimed deposits. As the sedimentation process transits into the self-weight consolidation process, particles are stabilized because they become irreversible. Although volumetric contraction occurs during consolidation, the number of particles above a clay element remains constant. Thus, the final effective stress (σ'_f) received by an in situ clay element after self-weight consolidation is determined by its initial location at the beginning of the formation of the soil structure, rendering a constant ("destined") effective stress condition (Fig. 1). That is, during the consolidation process, the current effective stress changes while the σ'_f remains constant. Moreover, the final effective stress can be estimated from the in situ density profile at any state of the in situ consolidation process.

Laboratory Simulation

The in situ conditions of reclaimed clay, including the dredged soil type, the initial water content, the local concentration of particles, and the ionic concentration, affect its sedimentation process, rendering different fabrics and structures in the clay (Imai 1980; Toorman 1996; Toorman 1999; Lee 2007). Undisturbed sampling of reclaimed clay is very difficult due to its fragility during sampling. Thus, large-scale laboratory sedimentation tests are often performed to simulate in situ reclamation processes. However, this approach is inefficient and cumbersome. Furthermore, it is restricted to low-stress conditions (shallow depth simulation). As a new and alternative method, in situ reclamation/self-weight consolidation can be reproduced in a laboratory by performing small-scale sedimentation tests for reconstructing representative (elementary) specimens, which can be followed by conventional consolidation tests to simulate self-weight consolidation separately.

Representative specimens (identical to in situ soils) can be reconstructed in a laboratory by performing conventional small-scale sedimentation tests on an in situ soil sample while preserving its in situ conditions. Consolidation tests can then be performed on the reconstructed specimens by applying the final (destined) effective stresses of interest at once, simulating the in situ self-weight consolidation process.

Characteristics of Shear Wave Velocity in Soft Clay

In a continuous medium, the state of confining stress has a minimal effect on the stiffness of the material. On the other hand, the stiffness of a structure consisting of particulate materials is governed mainly by the state of the effective stress, as the shear wave propagates only through the soil skeleton (Santamarina et al. 2001). The vertical shear wave velocity of particulate materials under zero lateral strain loading (i.e., K_o loading, one-dimensional consolidation) can be expressed in terms of the vertical effective stress as follows (Hardin and Richart 1963; Santamarina et al. 2001):

$$V_{s-v} = \alpha_1(\sigma'_m)^\beta = \alpha_1\left(\frac{(1+K_o)\sigma'_v}{2}\right)^\beta = \alpha_1\left(\frac{1+K_o}{2}\right)^\beta(\sigma'_v)^\beta = \alpha(\sigma'_v)^\beta \tag{1}$$

where

V_{s-v} = the vertical shear wave velocity,
σ'_m = the mean effective stress at the polarized plane of the shear wave propagation,
σ'_v = the vertical effective stress,
K_o = the coefficient of earth pressure at rest, and
α_1, α factors (i.e., the shear wave velocity at 1kPa) and the β

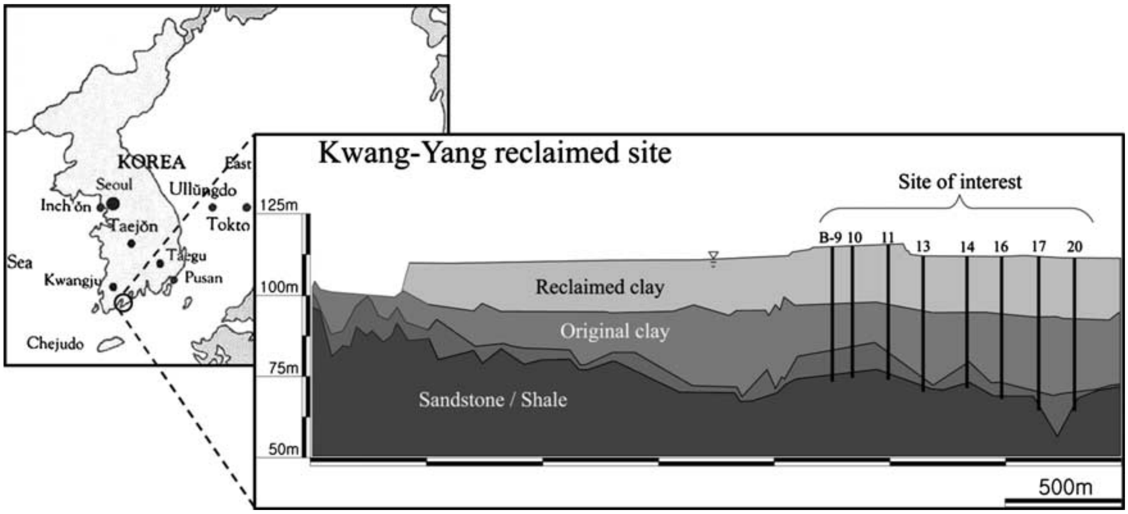

FIG. 2—*Schematic drawing of the Kwang-Yang reclaimed site.*

exponent (i.e., the sensitivity of the shear wave velocity with respect to the applied loading) are experimentally determined. Generally, in clays, a higher plasticity index is associated with a higher β exponent and a lower α factor. The α factor in Eq 1 involves two different properties: Those of the grains and those of the packing effect. The packing effect is related to the void ratio and the coordination number (i.e., the number of particle contacts). Therefore, the shear wave velocity of clay is correlated with both the effective stress and the void ratio (Hardin and Richart 1963; Hardin and Drnevich 1972).

Meanwhile, the effective stress-void ratio relationship of normally consolidated clay is linear on the semilogarithmic scale. Thus, a single equation can be derived for the shear wave velocity/void ratio relationship of normally consolidated reclaimed deposits using Eq 1. The void ratio-shear wave velocity relationship of kaolinite has a linear trend and is independent of the ionic concentration of the initial mixed slurry (Santamarina et al. 2001). This finding is in agreement with Imai's sedimentation and self-weight consolidation theory. Therefore, reclaimed clay is expected to show a unique relationship between the void ratio and the shear wave velocity.

Consolidation evolves at medium to large strains whereas shear wave propagation is a small strain phenomenon. A direct correlation between the shear wave velocity (or shear modulus) and the consolidation-related parameters may appear to lack physical justification. However, the state of the effective stress and void ratio greatly impacts (or is directly related to) both the shear wave velocity and the state of consolidation. Thus, this paper formulates indirect relationships between the shear wave velocity and the geotechnical engineering parameters via an effective stress and void ratio.

Experimental Program

Site of Interest and Soil Tested

The site of interest in this study is close to Kwang-Yang in South Korea. It is intended for a railroad construction project that will connect an established train system to Kwang-Yang harbor for freight transportation (Fig. 2). The site was reclaimed by dumping dredged kaolinite slurry with an initial water content of 300 % on the original clay two years prior to this study. The thickness of the in situ reclaimed deposit of interest is approximately 13 m. As the reclaimed clay was suspected to be still under consolidation, its current geotechnical engineering parameters and consolidation state were considered to be very important in the design of the current construction project.

Several attempts were made to take undisturbed soil samples at different depths/locations in the reclaimed zone. However, the retrieved samples were partly or completely disturbed owing to their weakness and high water contents. As the overall clay types and conditions were very identical, average values were used to estimate the geotechnical engineering parameters of the in situ soil sample, as documented in Table 1.

Sedimentation Test

The sampled clay was washed and dried at 105°C. The dried clay (300 g) was mixed with distilled water (900 g), making a clay slurry with a water content of 300 %, which is identical to the initial in situ water content when the dredged clay was reclaimed in the field. Salts were added to the slurry to equate the in situ ionic concentration (c=3.5 %). The prepared suspension was poured into four

TABLE 1—*Average properties of the in situ soil sample. (Notation: G_s=specific gravity, w=average water content (ASTM D2216-05 (2007)), e=void ratio, F_{200} =percent finer than #200 sieve, c=ionic concentration, ρ=density, LL=liquid limit (ASTM D4318-05 (2007)), PL=plastic limit (ASTM D4318-05 (2007)), PI =plasticity index, USCS=soil type.)*

Average Property	G_s	w (%)	e	F_{200} (%)	D_{50} (mm)	c (%)	ρ (g/cm^3)	LL (%)	PL (%)	PI (%)	USCS
Measured data	2.68	82	2.2	92	0.015	3.5	1.52	60	29	31	CH

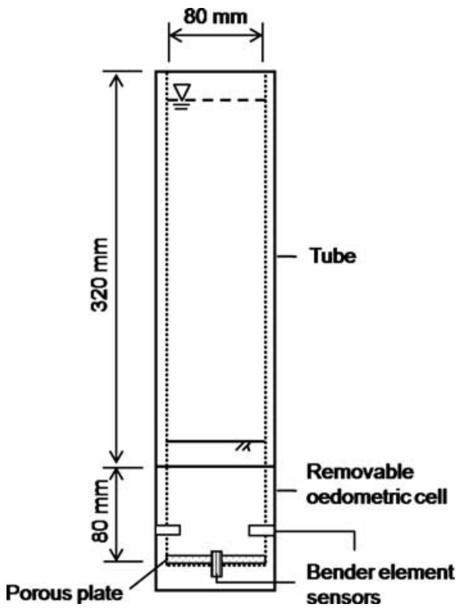

FIG. 3—*Sedimentation test device: Sedimentation tube with a removable oedometric cell.*

sedimentation tubes with a removable oedometric cell at its base (Fig. 3). In an analogy to conventional sedimentation tests, each sedimentation tube was vigorously shaken by applying 60 end-over-end cycles during a 2 min period. It was then immediately replaced in the upright position. After the settlement converged, the oedometric cell was then carefully dismantled from the sedimentation tube. The specimen reconstituted in the oedometric cell reflects the initial state of the in situ self-weight consolidation process.

Consolidation Test

Bimorph type piezoelectric bender elements 12 mm in length, 8 mm in width, and 0.6 mm in thickness were used to generate and receive shear waves. The anode and cathode wires of a coaxial cable were soldered to each side of the bender element. This assembly constituted series-type bender elements. Polyurethane was coated around the surface of the bender element for waterproofing. Conductive paste was then layered on the surface to shield against the effect of the coupling and cross-talking induced by unwanted electromagnetism between the source and receiver bender elements. Finally, the bender elements were mounted in the oedometric cell and on the load cap and were fixed with epoxy (Fig. 4).

The separated oedometric cell specimen was placed on an oedometric testing device and porous stones at the top and bottom of the specimen allowed the pore water to drain in two directions during the loading process. The load cap with a bender element was placed on the top of the specimen prior to loading.

The in situ self-weight consolidation process was reproduced by applying the σ_f' to the oedometric cell specimen, which was calculated with multiplying the simulated depth with an average unit weight of clay. Four specimens were prepared to represent different depths: 3.0 m ($\sigma_f'=15.4$ kPa), 6.0 m ($\sigma_f'=30.8$ kPa), 9.0 m ($\sigma_f'=46.2$ kPa), and 12.0 m ($\sigma_f'=61.7$ kPa).

The shear wave velocity was measured in the vertical direction (the direction of loading) for all specimens and in the horizontal direction for the 9.0 m specimen during the consolidation tests. A single step input signal was generated with an amplitude of 5 V at a frequency of 5 kHz. The band pass (100 Hz high pass and 50 kHz low pass) filter was applied to remove unwanted noises from received signals. Figure 5 shows the typical results of received signals at different depths. For each case, the first arrival of a shear wave is measured at the rising point (zero amplitude) of a maximum amplitude curve in the received signal, as marked in Fig. 5. Details of signal interpretation can be found in Lee and Santamarina (2005).

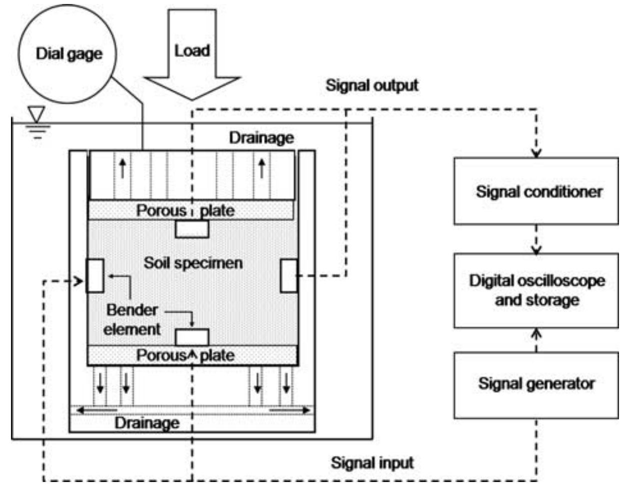

FIG. 4—*Consolidation test setup.*

Unloading and Vane Shear Test

After consolidation was completed, four specimens were unloaded to the same stress level (i.e., 1 kPa) to make different void ratios. Upon convergence of the shear wave velocity and volumetric expansion, the load cap was removed from the oedometric cell and a laboratory vane shear test (the vane blade size is 12.7 mm long and 12.7 mm wide, ASTM D4648-05 (2007)) was immediately performed to estimate the undrained shear strength of each specimen.

In Situ Test

For the in situ geotechnical engineering parameter evaluations, CPTu (ASTM D5778-95 (2007)) was performed at several field sites (i.e., B-9, 10, 11, 13, 14, 16, and 20 in Fig. 2). The degree of consolidation (U_z) and coefficient of consolidation (C_v) were

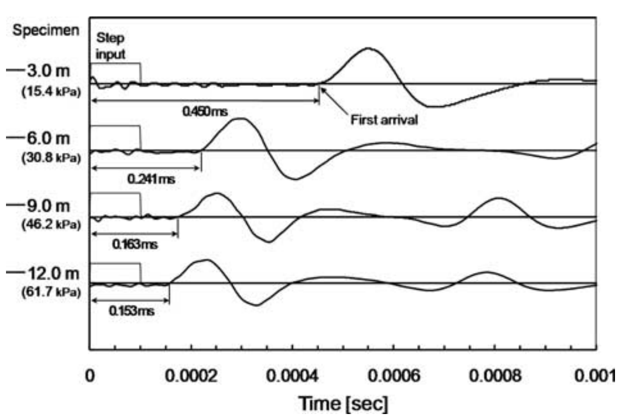

FIG. 5—*Example of shear wave travel time interpretation.*

TABLE 2—*Experimental conditions and results.*

Specimen			Tube A	Tube B	Tube C	Tube D
Simulating depth			3.0 m	6.0 m	9.0 m	12.0 m
Sedimentation test	Dry soil (g)		300			
	Void ratio	Initial	7.80			
		Final[a]	3.17	3.18	3.12	3.16
Consolidation test	Applied vertical stress (kPa)		15.4	30.8	46.2	61.7
	Void ratio	Before loading[b]	2.47	2.56	2.54	2.61
		After loading	1.68	1.55	1.48	1.40
	Vertical shear wave velocity (m/s)	Before loading	22.6	22.0	19.5	20.1
		After loading	68.0	106.3	124.0	153.0
Unloading/laboratory vane shear test	Void ratio		1.71	1.58	1.55	1.45
	Vertical shear wave velocity (m/s)		50.0	85.0	90.0	120.0
	Undrained shear strength (kPa)		2.5	6.5	8.0	11.0
In situ shear wave velocity (m/s)			73	71	120	140

[a]The discordance between the final void ratio after sedimentation is due to the initial confinement applied by the cap load.

[b]The initial void ratio before loading is due to the initial confinement applied by the cap load.

evaluated by pore pressure dissipation tests, and the undrained shear strength (S_u) was estimated using the total cone resistance relationship (Lunne et al. 1997). The hydraulic conductivity (k) was measured by the slug test method (ASTM D5912-96 (2007)) inside boreholes B-9, 13, and 20 at the site. The in situ vertical shear wave velocity was measured at borehole B-17 by performing a suspension P-S (SPS) logging test, which provides vertical shear wave propagation identical to that of laboratory testing.

Laboratory Experimental Test Results

Sedimentation and Consolidation Test

The conditions and results of the laboratory experimental tests are summarized in Table 2. The sedimentation test results show that the sediment height decreases with time and approaches a constant value (Fig. 6). Four specimens show a similar trend of sedimentation, suggesting that they can be considered as identical.

The consolidation test results are shown in Fig. 7. For all specimens during consolidation, the shear wave velocity increase while the void ratio decreases with time. The initial V_{s-v} of the soft clay sediment is low because its particle packing is loose and its current σ'_v is very low. When a vertical load is applied to a specimen, the total stress is initially resisted by the excess pore water pressure. In addition, the hydraulic pressure head difference causes the pore fluid to flow upward and downward through the drainage path; thus, the effective stress increases with a decrease in the pore water pressure. The particle packing thereupon becomes denser as the pore water pressure decrement transfers to an increase in the σ'_v. The rate of pore pressure dissipation is initially high and continuously decreases according to the reduction of the k as the soil becomes denser. In other words, the σ'_v increases on a logarithmic time scale, which agrees with the V_{s-v} variation. The convergence of the V_{s-v} (Fig. 5) implies that the excess pore water pressure has completely dissipated; thus, the amount of σ'_v inside the specimen becomes equal to the applied load. As the applied vertical stress increases, a steep descent of the e and a steep ascent of the shear wave velocity appear at the earlier stage of time; additionally, the final void ratio decreases, and the final shear wave velocity increases.

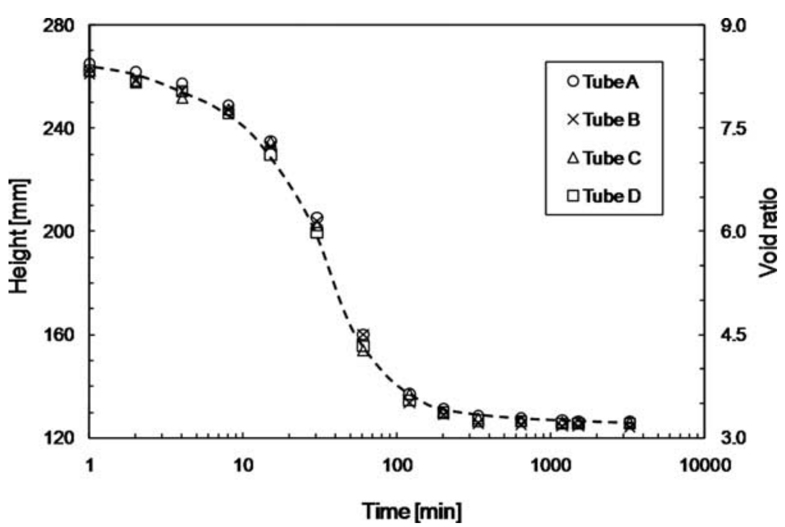

FIG. 6—*Sedimentation test results: Sediment height (or void ratio) versus time.*

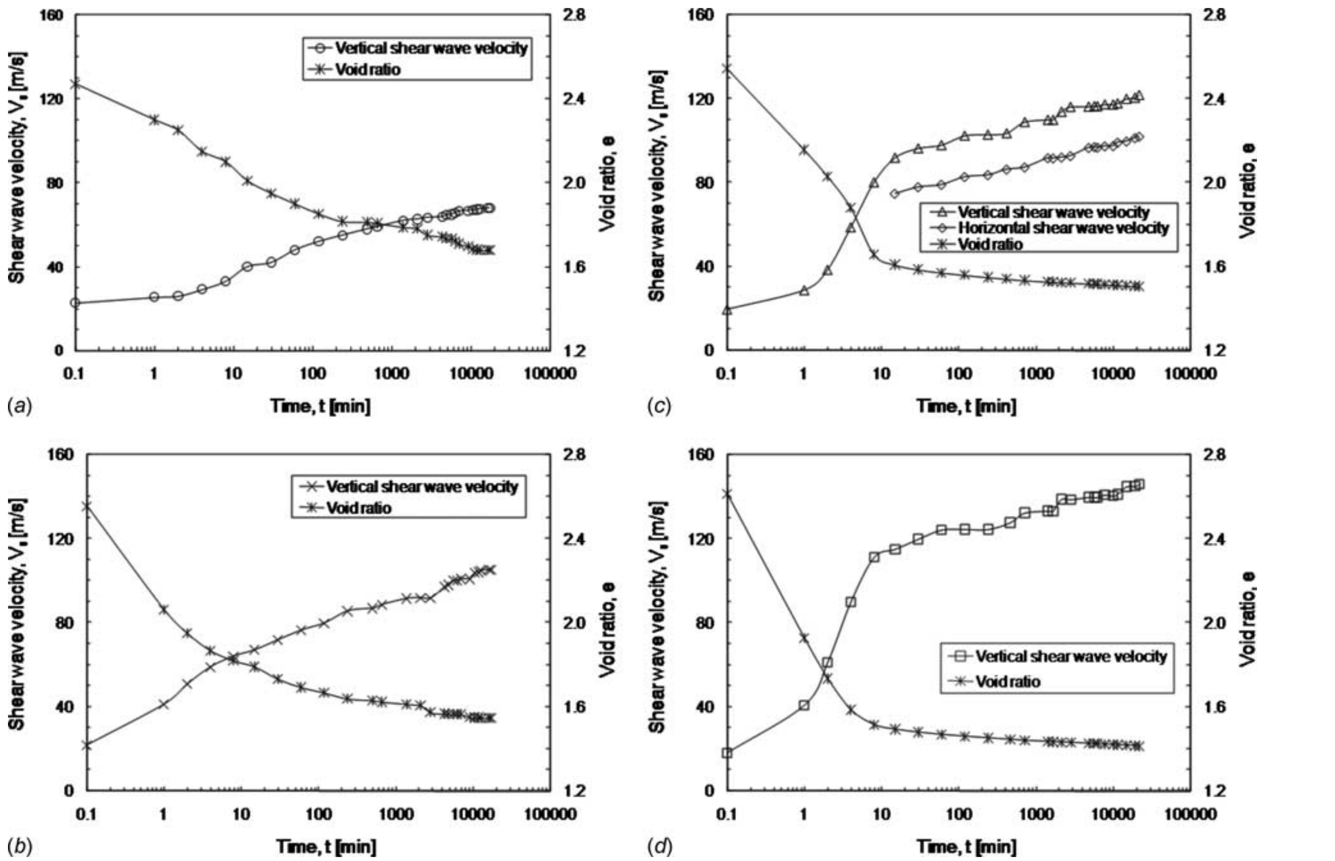

FIG. 7—*Shear wave velocity and void ratio variation during the laboratory consolidation test. Specimens simulate depths of (a) 3.0 m (15.4 kPa), (b) 6.0 m (30.8 kPa), (c) 9.0 m (46.2 kPa), and (d) 12.0 m (61.7 kPa).*

Unloading and Vane Shear Test

After unloading, the converged final void ratio, the V_{s-v} and the S_u were measured for each specimen. These are also summarized in Table 2. Unloading causes the volumetric expansion (i.e., e increase) to be relatively small while decreasing the V_{s-v} significantly, which highlights the effect of the confining pressure on the V_{s-v}. Different shear wave velocities at a vertical stress of 1 kPa (i.e., an unloaded stage) also reflect the effect of the density (i.e., the e) on the V_{s-v}. The V_{s-v} increases as the e decreases. In particular, the undrained shear strengths are plotted versus the corresponding void ratios in Fig. 8. The S_u is linearly related to the e.

Analyses—Estimation of Geotechnical Engineering Parameters

Based on the experimental test results of reclaimed clay (i.e., normally consolidated clay), the shear wave velocity can be correlated with the geotechnical engineering parameters of the vertical effective stress, void ratio, degree of consolidation, coefficient of consolidation, coefficient of earth pressure at rest, hydraulic conductivity, and undrained shear strength.

Vertical Effective Stress

The final (converged) vertical shear wave velocities of four specimens are plotted versus their corresponding applied σ'_v values in Fig. 9. The V_{s-v} increases with the σ'_v. For a given type of reclaimed clay, the σ'_v can be correlated with the V_{s-v} by best curve-fitting the experimental data with Eq 1, as follows:

$$V_{s-v} = 16.5(\sigma'_v)^{0.56} \quad (2)$$

Here, V_{s-v} is in m/s and σ'_v is in kPa. Thus, the in situ current vertical effective stress can be estimated from the in situ vertical shear wave velocity using Eq 2 and vice versa.

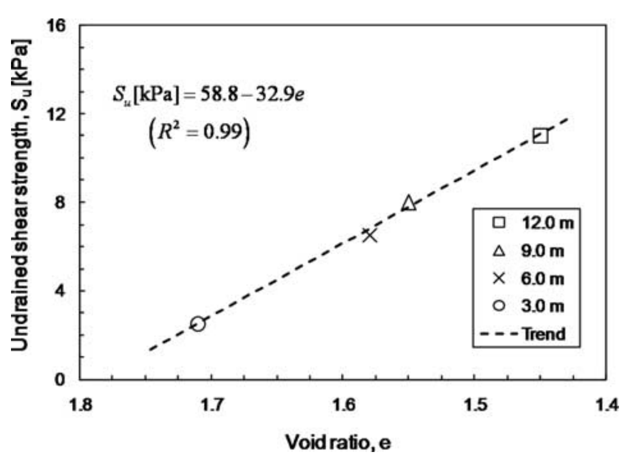

FIG. 8—*Relation of undrained shear strength and void ratio.*

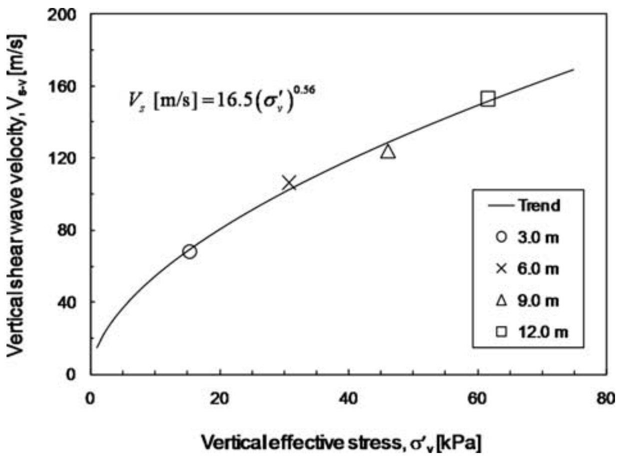

FIG. 9—*Relation between the vertical effective stress and vertical shear wave velocity from consolidation test (after loading).*

Void Ratio

The e and its corresponding V_{s-v} measured during consolidation are plotted together in Fig. 10(a). There is a unique one-to-one relationship between the e and the V_{s-v} of the given normally consolidated reclaimed clay, regardless of the depth of the clay. An approximate equation of the trend in Fig. 10(a) is as follows:

$$e = 3.93 - 1.18 \log V_{s-v} \tag{3}$$

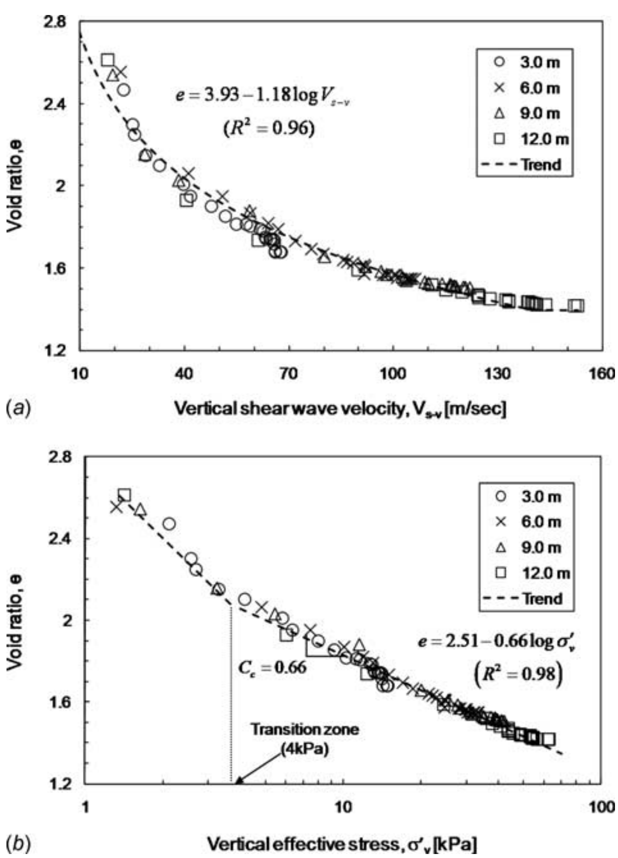

FIG. 10—*Variation of the void ratio with vertical shear wave velocity and vertical effective stress. (a) $e - V_{s-v}$ relation. (b) $e - \sigma_v'$ relation.*

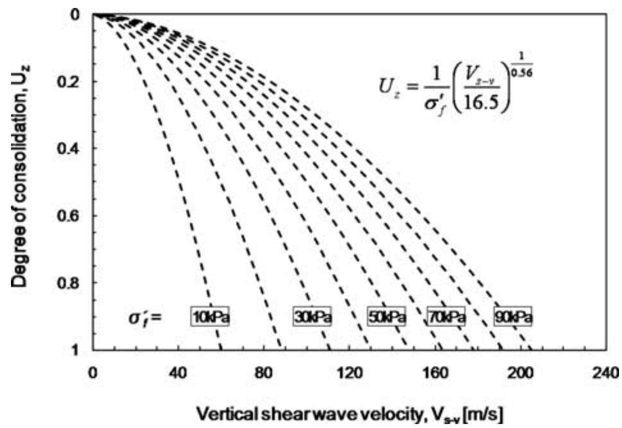

FIG. 11—*Degree of consolidation-vertical shear wave velocity relationship.*

where V_{s-v} is in m/s. This relationship can be used effectively for estimating the in situ void ratios in the reclaimed site from in situ vertical shear wave velocities.

It is known that the e is linearly related to the effective stress at the semilogarithmic scale. In that context, the e can be expressed in terms of the σ_v' by substituting Eq 2 for V_{s-v} in Eq 3 as follows:

$$e = 2.51 - 0.66 \log \sigma_v' \tag{4}$$

The e is then plotted versus its corresponding σ_v', as shown in Fig. 10(b). The e-log(σ_v') trend shows a bilinear form at a breakpoint of 4 kPa.

Degree of Consolidation

The U_z is one of the most important geotechnical engineering parameters of reclaimed deposits because it allows us an estimation of the additional settlement expected in field. The U_z at any depth z is defined as the ratio of the amount of excess dissipated pore water pressure ($u_i - u$) and the initial maximum excess pore water pressure (u_i),

$$U_z = \frac{u_i - u}{u_i} = \frac{\Delta\sigma_v'}{\sigma_f'} \tag{5}$$

where

u = the current excess pore water pressure,

$\Delta\sigma_v'$ = the increased effective stress during the self-weight consolidation ($\Delta\sigma_v' = u_i - u$), and

the initial maximum excess pore water pressure(u_i)=equal to the final effective stress (σ_f') upon completion of the self-weight consolidation.

Thus, the U_z can be rewritten in terms of the V_{s-v} by combining Eqs 2 and 5 as

$$U_z = \frac{1}{\sigma_f'}\left(\frac{V_{s-v}}{16.5}\right)^{1/0.56} \tag{6}$$

where σ_f' is in kPa and V_{s-v} is in m/s. Figure 11 shows the relationship between the U_z and the V_{s-v} for a certain applied effective stress ($\sigma_f' = 10, 20 \ldots 90$ kPa) as obtained from Eq 6. Thus, it is possible to estimate the in situ degree of consolidation from the in

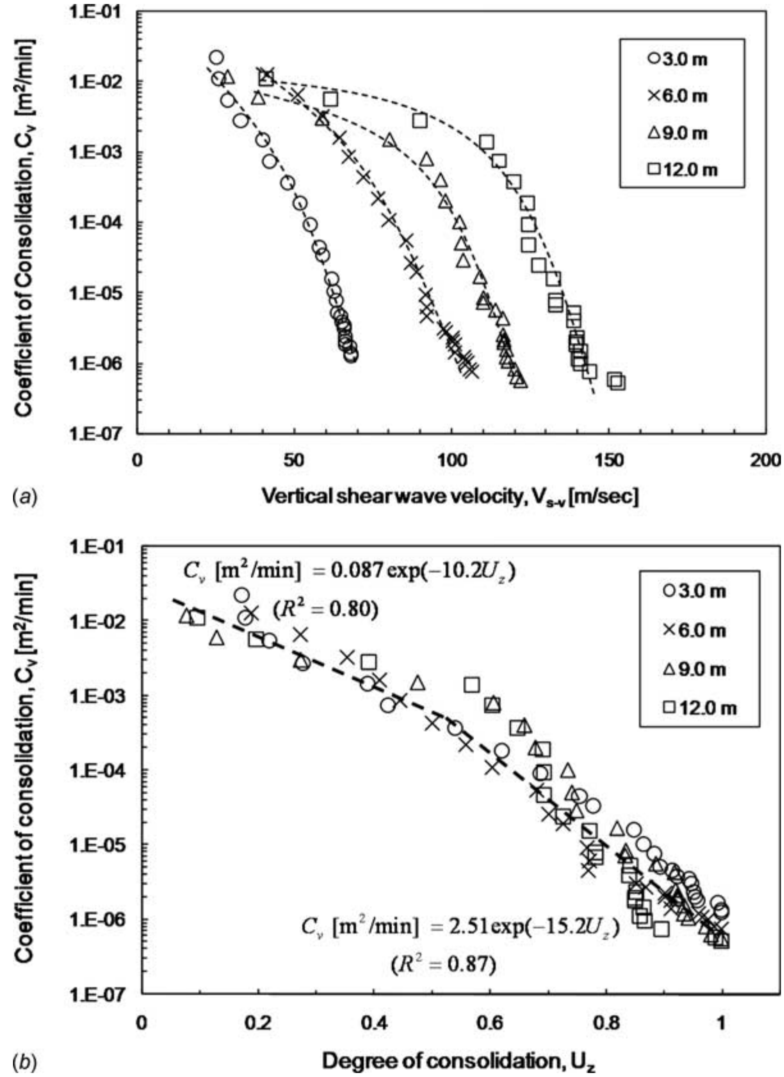

FIG. 12—*Variation of the coefficient of consolidation. (a) $C_v - V_{s-v}$ relationship. (b) $C_v - U_z$ relationship.*

situ vertical shear wave velocity using Eq 6 or Fig. 11, which is one of considerable merits for site characterization in reclaimed areas.

Coefficient of Consolidation

In classical soil mechanics, the C_v for clay deposits is assumed to be constant. However, the k decreases as the density increases during consolidation and the ability of pore water to drain decreases. Therefore, the C_v should decrease as the U_z increases.

The C_v can be evaluated using the variation of the specimen height ($H(t)$) and the evaluated degree of the consolidation-vertical shear wave velocity relationship ($U_z - V_{s-v}$). As both upward and downward drainages are allowed, the length of the maximum drainage path (H_{dr}) is half of the specimen height. As the time factor (T_v) is a function of U_z and U_z is a function of V_{s-v} (Eq 6), the time factor can be expressed in terms of the shear wave velocity:

$$T_v = \frac{\pi}{4}(U_z)^2 = \frac{\pi}{4}\left(\frac{1}{\sigma'_f}\frac{V_{s-v}}{16.5}\right)^{2/0.56} = \frac{\pi}{4}\left(\frac{1}{\sigma'_f}\frac{V_{s-v}}{16.5}\right)^{3.57} \quad \text{for } U_z = 0-0.53 \tag{7a}$$

$$T_v = 1.781 - 0.933[2 + \log(1 - U_z)] = -0.085 - 0.933\log\left[1 - \frac{1}{\sigma'_f}\left(\frac{V_{s-v}}{16.5}\right)^{1/0.56}\right] \quad \text{for } U_z = 0.53-1 \tag{7b}$$

Thus, the C_v can be defined as follows:

$$C_v = \frac{H_{dr}{}^2 T_v}{t} = \frac{\pi}{16}\frac{(H(t))^2}{t}\left(\frac{1}{\sigma'_f}\frac{V_{s-v}}{16.5}\right)^{3.57} \quad \text{for } U_z = 0-0.53 \tag{8a}$$

$$C_v = \frac{H_{dr}{}^2 T_v}{t} = -\frac{(H(t))^2}{4t}\left[0.085 + 0.933\log\left\{1 - \frac{1}{\sigma'_f}\left(\frac{V_{s-v}}{16.5}\right)^{1/0.56}\right\}\right] \quad \text{for } U_z = 0.53-1 \tag{8b}$$

From Eqs 8a and 8b and the measurement data during the laboratory consolidation test, the C_v for each specimen is plotted against the corresponding V_{s-v} in Fig. 12(*a*). Thus, the in situ coefficient of consolidation can be estimated from the in situ vertical shear wave

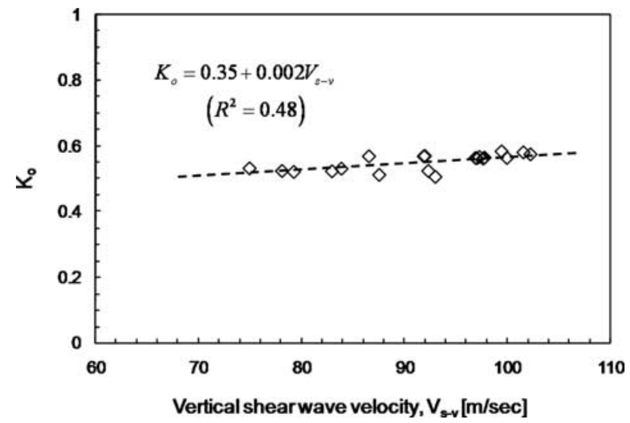

FIG. 13—*Variation of the coefficient of earth pressure at rest with vertical shear wave velocity.*

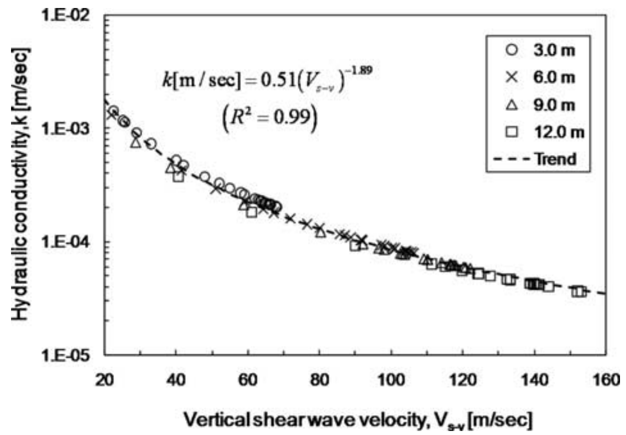

FIG. 14—*Hydraulic conductivity variation with vertical shear wave velocity.*

velocity. Furthermore, to identify the relationship between U_z and C_v, the U_z-V_{s-v} data (Fig. 11) and C_v-V_{s-v} data (Fig. 12(*a*)) are plotted on a C_v-U_z plane by canceling V_{s-v} in Fig. 12(*b*). This result shows that C_v decreases with an increase in U_z and that is a single trend between C_v and U_z regardless of their depths. The approximated equation for the C_v-U_z relationship is given below,

$$C_v[\mathrm{m}^2/\mathrm{min}] = 0.087\ \exp(-10.2U_z) \quad \text{for} \quad U_z = 0 - 0.53 \tag{9a}$$

$$C_v[\mathrm{m}^2/\mathrm{min}] = 2.51\ \exp(-15.2U_z) \quad \text{for} \quad \mathrm{U_z} = 0.53 - 1 \tag{9b}$$

Coefficient of Earth Pressure at Rest

The ratio of horizontal effective (σ_h') stress to σ_v', when horizontal strain is not allowed (i.e., zero-lateral strain condition), is known as the K_o. Thus, the horizontal shear wave velocity (V_{s-h}) represents the K_o condition of the in situ process. The V_{s-h} and σ_v' are related as follows:

$$V_{s-h} = \alpha_1(\sigma_h')^\beta = \alpha_1(K_o\sigma_v')^\beta \tag{10}$$

Here, β corresponds to β=0.56 from Eq 2. From Eqs 1 and 10, the ratio between V_{s-v} and V_{s-h} is expressed as

$$\frac{V_{s-v}}{V_{s-h}} = \frac{\alpha_1\left(\frac{1+K_o}{2}\right)^\beta(\sigma_v')^\beta}{\alpha_1(K_o\sigma_v')^\beta} = \left(\frac{1+K_o}{2K_o}\right)^\beta \tag{11}$$

Finally, from Eqs 2 and 11, K_o can be defined as

$$K_o = \left[2\left(\frac{V_{s-v}}{V_{s-h}}\right)^{1/\beta} - 1\right]^{-1} = \left[2\left(\frac{V_{s-v}}{V_{s-h}}\right)^{1/0.56} - 1\right]^{-1} \tag{12}$$

Figure 13 shows the relationship between K_o and V_{s-v} after substituting the V_{s-v} and V_{s-h} data of Fig. 7(*c*) into Eq 12. The value of K_o increases slightly as V_{s-v} increases. The approximated relationship between K_o and V_{s-v} is as follows:

$$K_o = 0.35 + 0.002(V_{s-v}) \quad \text{for} \quad V_{s-v} > 0 \tag{13}$$

where V_{s-v} is in m/s. Thus, it is possible to estimate the in situ K_o from the in situ vertical shear wave velocity.

Hydraulic conductivity

Been and Sills (1981) suggested a nonlinear $\sigma_v'-k$ equation to derive a constant coefficient of the consolidation equation

$$C_v = -\frac{k}{\rho_f(1+e)}\frac{d\sigma_v'}{de} \tag{14}$$

where

ρ_f = the is the fluid density.

However, the C_v is not constant during the large strain consolidation of reclaimed clay (Fig. 12). The derivation of the effective stress with respect to *e* in Eq 14 can be obtained by differentiating Eq 4 with σ_v'

$$\frac{d\sigma_v'}{de} = -\frac{2.30\sigma_v'}{0.66} = -3.49\sigma_v' \tag{15}$$

Therefore, the *k* can be written by combining Eqs 14 and 15,

$$k = \frac{\rho_f(1+e)C_v}{3.49\sigma_v'} \tag{16}$$

As the *e*, C_v, and σ_v' variations are known, the *k* of each specimen can be estimated. The obtained $V_{s-v}-k$ relationship is shown in Fig. 14. It can be expressed as follows:

$$k[\mathrm{m/s}] = 0.51(V_{s-v})^{-1.89} \tag{17}$$

Thus, the in situ hydraulic conductivity can be estimated from the in situ vertical shear wave velocity using Eq 17.

Undrained Shear Strength

Considering that the S_u of normally consolidated clay is governed by its *e* (Ladd et al. 1977; Larsson 1980; Terzaghi et al. 1996), the in situ undrained shear strengths can be estimated from the in situ current void ratios using the relationship obtained in Fig. 8 as follows:

$$S_u[\mathrm{kPa}] = 58.8 - 32.9e \tag{18}$$

After combining Eqs 3 and 18, the relationship between the S_u and V_{s-v} for reclaimed clay can be expressed as

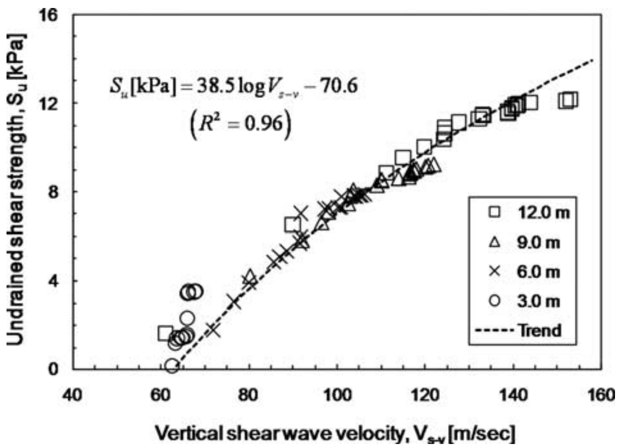

FIG. 15—*Variation of undrained shear strength with vertical shear wave velocity.*

$$S_u[\text{kPa}] = 38.5 \log V_{s-v} - 70.6 \quad (19)$$

The predicted S_u is plotted against the V_{s-v} for each specimen in Fig. 15.

In Situ Parameter Estimation—Verification

The in situ geotechnical engineering parameters of the vertical effective stress state (σ_v'), void ratio (e), degree of consolidation (U_z), coefficient of consolidation (C_v), coefficient of earth pressure (K_o), hydraulic conductivity (k), and undrained shear strength (S_u) were estimated for the four simulating depths by substituting in situ vertical shear wave velocities (Table 2) into the corresponding equations (i.e., Eq 2 for σ_v'; Eq 3 for e; Eq 6 for U_z; Eqs 8a and 8b for C_v; Eq 13 for K_o; Eq 17 for k; and Eq 19 for S_u). The estimated results are summarized in Table 3, showing that the surface layer around a depth of 2 m is overconsolidated. The overconsolidated condition is generally observed near the top surface of reclaimed sites. Desiccation and surface moisture evaporation decrease the water content of the surface soils. The bonding effects and capillary forces between particles increase the interparticle forces without a global load increment (Cargil 1985; Znidarcic and Liu 1989). Therefore, the estimation of the 2 m depth is inapplicable because the simulation in this study applies only to normally consolidated clay deposits due to the linear effective stress-void ratio assumption.

The in situ vertical shear wave velocity profile (Fig. 16(*a*)) is used to predict the in situ profiles of the U_z (Eq 6—Fig. 16(*b*)), C_v (Eqs 8a and 8b—Fig. 16(*c*)), k (Eq 17—Fig. 16(*d*)), and S_u (Eq 19—Fig. 16(*e*)). For verification, the predicted in situ parameters are compared with parameters measured at various locations in situ, as shown in Fig. 16(*b*)–16(*e*). The overall trends of the predicted values are consistent with those of the measured values, suggesting that the method proposed in this study can be used effectively for practical applications.

Most of the site is still under consolidation (Fig. 16(*b*)); thus, additional settlement is expected at the site until the consolidation process is completed. The one-dimensional consolidation settlement S_i of the ith clay layer with a thickness of H_i can be calculated as

$$S_i = \frac{\Delta e}{1+e_o} H_i = \frac{e_o - e_f}{1+e_o} H_i \quad (20)$$

where
e_o = the current void ratio and
e_f = the final void ratio at the end of consolidation.

The e_f are summarized in Table 2 and the e_o are shown in Table 3. The top layer is regarded as a desiccated crust, and the deposit is divided into four zones: 2.0–5.0, 5.0–8.0, 8.0–11.0, and 11.0–13.0 m. The estimated e represents the average value of each zone. Hence, the expected total settlement is as follows:

$$S_{\text{total}} = \sum_i S_i = S_1 + S_2 + S_3 + S_4 = 0.08 + 0.23 + 0.03 + 0.02$$
$$= 0.36 \text{ m} \quad (21)$$

The elevation of the surface will decrease by 0.36 m when consolidation is completed (Fig. 17).

Conclusions

This study presented an alternative method for estimating geotechnical engineering parameters of reclaimed clay using shear wave velocity. The laboratory experimental results show that as the shear wave velocity increases, the vertical effective stress and undrained strength increase while the void ratio decreases. These relationships are unique for the given reclaimed clay due to the destined (i.e., constant final effective stress condition) concept of a normally consolidated clay.

Through a series of analyses, the shear wave velocity data was correlated with geotechnical engineering parameters such as effective stress, void ratio, degree of consolidation, coefficient of consolidation, coefficient of earth pressure at rest, hydraulic conductivity, and undrained shear strength. For a given location, as the shear wave velocity increases, the degree of consolidation and coefficient of earth pressure at rest increase while the coefficient of consolidation and hydraulic conductivity decrease. The geotechnical engineering parameters estimated from the shear wave velocity are in good accordance with the values measured from in situ tests, verifying the applicability and reliability of the method suggested in this study. However, all the estimated relationships are site-

TABLE 3—*Estimated in situ geotechnical engineering parameters.*

In situ depth	3.0 m	6.0 m	9.0 m	12.0 m
Effective stress (kPa)	14.6	12.9	24.4	33.8
Void ratio	1.75	1.76	1.50	1.42
Degree of consolidation (%)	93	44	76	75
Coefficient of consolidation (m^2/min)	2.12×10^{-6}	7.89×10^{-4}	1.82×10^{-5}	2.03×10^{-5}
Coefficient of earth pressure	0.50	0.49	0.59	0.63
Hydraulic conductivity (m/s)	1.5×10^{-4}	1.6×10^{-4}	5.7×10^{-5}	4.3×10^{-5}
Undrained shear strength (kPa)	1.14	0.67	9.45	12.03

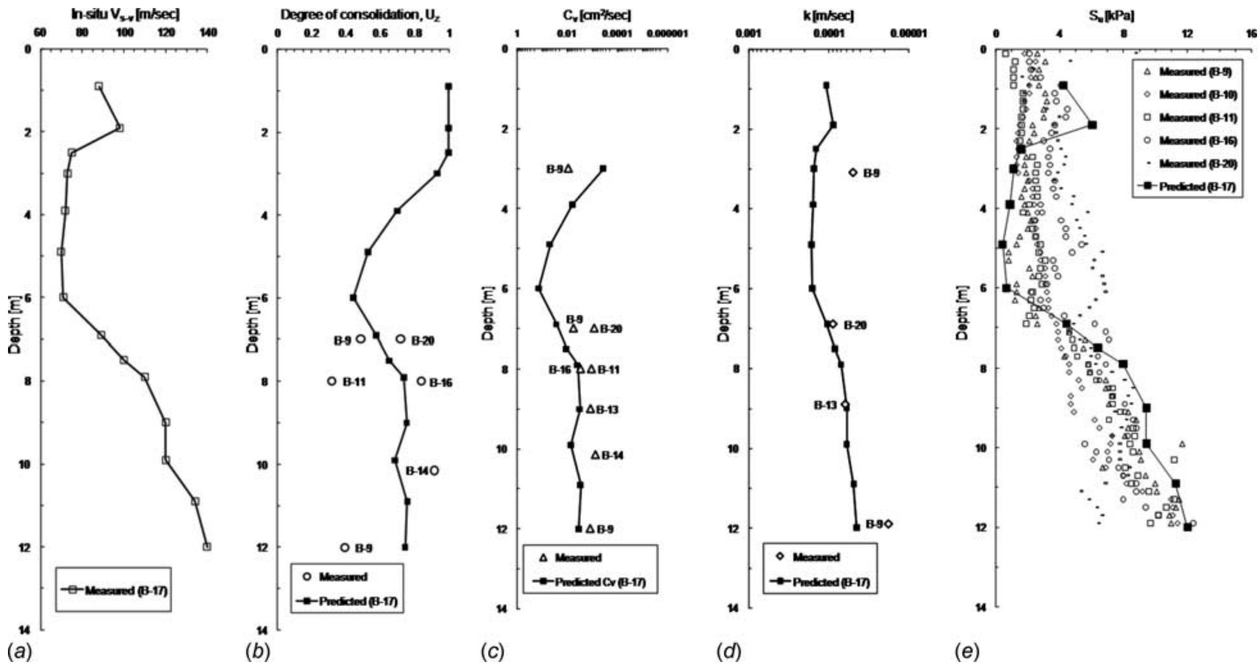

FIG. 16—*Comparison of the predicted parameters (Eqs 6, 8, 17, and 19) with the parameters measured in situ. (a) V_s, (b) U_z, (c) C_v, (d) k, and (e) S_u.*

dependent due to different clay mineralogy and site conditions and should therefore be determined for each site by following the experimental and analytical procedure. Although more field applications can enhance its reliability, this method is only applicable to reclaimed clay, which is normally consolidated.

It appears that the shear wave velocity is a robust and meaningful parameter for estimating the in situ geotechnical engineering parameters of a reclaimed clay. That is, its current state can be effectively estimated by combining laboratory simulation results of a consolidation test with shear wave velocity measurements and in situ shear wave velocity profiles. Moreover, the suggested shear wave velocity-based geotechnical engineering parameter estimation method can be applicable not only for preliminary designs but also for in situ real-time monitoring after reclamation of a soft clay by the aid of various in situ seismic exploration techniques such as SPS logging, seismic cone penetration testing, spectral analysis of surface wave, down-hole and cross-hole tests, and imbedded sensors.

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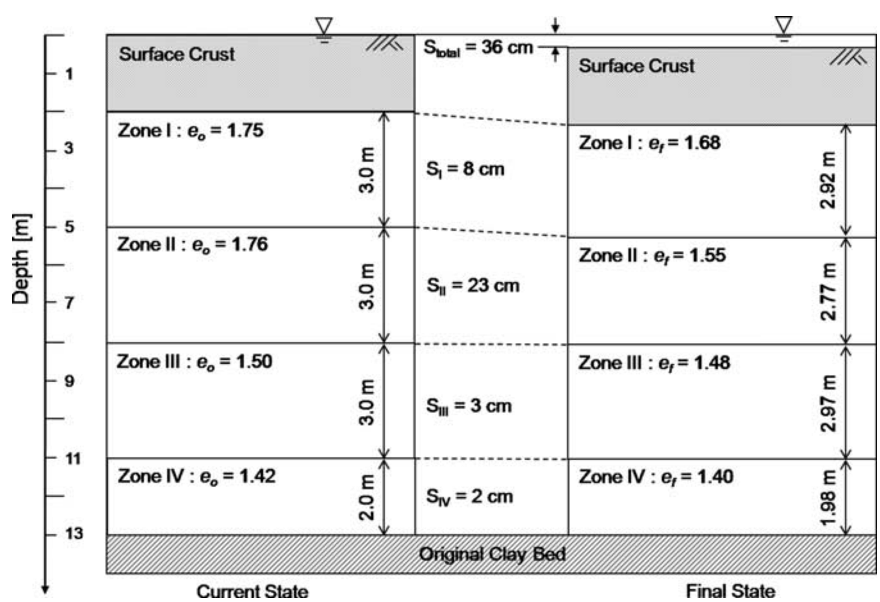

FIG. 17—*Predicted total settlement of the in situ layer.*

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