Smart Structures and Systems, Vol. 7, No. 4 (2011) 289-302
DOI: http://dx.doi.org/10.12989/sss.2011.7.4.289

An experimental procedure for evaluating the consolidation state of marine clay deposits using shear wave velocity

Ilhan Chang[1], Tae-Hyuk Kwon[2] and Gye-Chun Cho*[3]

[1]*Department of Civil and Environmental Engineering, KAIST, 291 Daehak-no, Yuseong-gu, Daejeon 305-701, Korea*
[2]*Earth Sciences Division, Lawrence Berkeley National Laboratory, Berkeley CA 94720, USA*
[3]*Department of Civil and Environmental Engineering, KAIST, 291 Daehak-no, Yuseong-gu, Daejeon 305-701, Korea*

(Received July 1, 2010, Accepted September 1, 2010)

Abstract. In marine clay deposits, naturally formed or artificially reclaimed, the evaluation and monitoring of the consolidation process has been a critical issue in civil engineering practices due to the time frame required for completing the consolidation process, which range from several days to several years. While complementing the conventional iconographic method suggested by Casagrande and recently developed in-situ techniques that measure the shear wave, this study suggests an alternative experimental procedure that can be used to evaluate the consolidation state of marine clay deposits using the shear wave velocity. A laboratory consolidation testing apparatus was implemented with bimorph-type piezoelectric bender elements to determine the effective stress-shear wave velocity (σ'-V_s) relationship with the marine clays of interest. The in-situ consolidation state was then evaluated by comparing the in-situ shear wave velocity data with the effective stress-shear wave velocity relationships obtained from laboratory experiments. The suggested methodology was applied and verified at three different sites in South Korea, i.e., a foreshore site in Incheon, a submarine deposit in Busan, and an estuary delta deposit in Busan. It is found that the shear wave-based experimental procedure presented in this paper can be effectively and reliably used to evaluate the consolidation state of marine clay deposits.

Keywords: bender element; consolidation state; effective stress; marine clay; shear wave velocity; under-consolidation.

1. Introduction

As the need for infra-structures expands to offshore areas, marine soft clay deposits–either naturally occurring sediments or artificially reclaimed sediments-are in need to be characterized from an engineering perspectives. Moreover, when an additional load, such as a heavy offshore structure is applied, the time-dependent volumetric compression process occurs in a clay deposit. This is known as *consolidation*. The consolidation process may take days-to-years depending on the deposit depth and boundary drainage condition. Therefore, evaluating and

*Corresponding Author, Associate Professor, E-mail: gyechun@kaist.edu
[1]Research Assistant Professor
[2]Postdoctoral Research Fellow

monitoring of the consolidation process has been an issue in many civil engineering practices (Shang *et al.* 2009, Kahn *et al.* 2010).

A clay deposit memorizes its stress history-the pre-consolidation effective stress (σ'_p) that is defined as the maximum effective stress the clay has previously experienced. In fact, the stress history and current stress state determine geotechnical engineering properties and behavioral characteristics of clays. According to the relativity among the current effective stress in field (σ'_o), the previous maximum effective stress (pre-consolidation effective stress, σ'_p), and the expected overburden effective stress (i.e., simply determined by the gravitational weight of the overburden deposits for the normally-consolidated state, σ'_{nc}-as described further in following sections); the in-situ consolidation state of a clay deposit is categorized into three statuses: over-consolidated (OC, when $\sigma'_{nc} < \sigma'_o < \sigma'_p$), normally-consolidated (NC, when $\sigma'_o \approx \sigma'_{nc}$ with no observable σ'_p), and under-consolidated (UC, when $\sigma'_o < \sigma'_{nc}$). The stress history of a clay deposit has been determined by the iconographic method suggested by Casagrande (1936) where the inflection point of the void ratio-effective stress (e-log σ') curve determined by a consolidation test is defined as the pre-consolidation effective stress (σ'_p) during incremental loading and unloading processes. The commonly used iconographic method leads to an inevitable error because the inflection point of the curve depends considerably on visual observation and personal judgment. However, the conventional consolidation test and its interpretation only allow estimation of the temporal scale for completion of primary consolidation: they do not provide the real-time monitoring consolidation state, particularly in an under-consolidation state due to the lack of real-time information in situ.

Despite the rapid recently advances in field testing techniques, such as uCPT (Elsworth *et al.* 2006, Lee *et al.* 2009), assessing the in-situ effective stress state remains immature due to the low signal-to-noise characteristics of pore water pressure profiles (Elsworth and Lee 2007), soil disturbance induced by invasive field testing probes (Richards 1988, Landon *et al.* 2007), and the spatial limitation of probes that represent only the local area (Lunne *et al.* 1997). In contrast, assessing the in-situ stress state using the shear wave velocity (V_s) of clays has advantages and a great potential, because the shear wave propagates only through soil skeletons, so that the shear wave velocity of a soil medium can reflect the soil composition and the effective stress state (Hardin and Richart 1963, Jovicic *et al.* 1996, Pennington *et al.* 2001, Santamarina *et al.* 2001, Klein and Santamarina 2003, Leong et al. 2005, Landon *et al.* 2007). In addition, the shear wave velocity measured by field testing techniques, such as cross-hole logging, downhole logging and SP suspension (SPS) logging methods, represents meter-scale measuring, thus minimizing measurement error such as that from soil disturbance. A few studies (e.g., Kwon and Cho 2005, Chang and Cho 2010) presented methods which incorporate the laboratory-determined shear wave velocity data as a reference to interpret shear wave velocity profiles measured in situ. Particularly, Chang and Cho (2010) presented the predictability and applicability of the laboratory-measured shear wave velocity to the sedimentation and consolidation behavior of reclaimed clay.

The present study suggests an alternative means of consolidation state evaluation that analyzes the laboratory V_s data plus in-situ V_s profiles. First, a laboratory testing method was developed to evaluate the consolidation state of in-situ soft clay deposits. An oedometric device instrumented with piezoelectric bender element sensors was used to evaluate the effective stress-shear wave velocity (σ'-V_s) relationship of an undisturbed clay specimen sampled from the deposit of interest. In-situ shear wave velocity data collected from field seismic explorations were compared with the σ'-V_s relationship to determine the in-situ consolidation state.

2. Evaluation of consolidation state using shear wave velocity

In this section, the relationship between the effective stress and shear wave velocity (σ'-V_s relationship) and the concept of the consolidation state are addressed and then the concept of evaluating the consolidation state by the shear wave velocity is presented.

2.1 Effective stress and shear wave velocity

The shear wave velocity of particulate materials under zero lateral strain loading (K_0 condition, one-dimensional consolidation) can be expressed in terms of the vertical effective stress as follows (Hardin and Richart 1963, Santamarina *et al.* 2001)

$$V_s = \alpha_1\left(\frac{\sigma'_m}{1\ \text{kPa}}\right)^{\beta} = \alpha_1\left(\frac{(1+K_o)\sigma'_v}{2\ \text{kPa}}\right)^{\beta} = \alpha\left(\frac{\sigma'_v}{1\ \text{kPa}}\right)^{\beta} \tag{1}$$

where σ'_m is the mean effective stress that is defined as the average of the horizontal and vertical stresses, σ'_v is the vertical effective stress, K_0 is the coefficient of earth pressure at rest, and α and β are experimentally defined parameters. Generally, a higher plasticity index of soils renders higher sensitivity to stress, such that it is expressed by the higher β exponent and lower a factor in Eq. (1). For a given single soil specimen, α and β remains constant during a virgin compressional loading process; thus, one set of the parameters α, β and the equation can be derived for a normally consolidated condition (Santamarina *et al.* 2001, Chang and Cho 2010).

2.2 Expected in-situ effective stress σ'_{nc} and shear wave velocity $V_{s\text{-}nc}$ for NC condition

An expected in-situ effective stress state at a certain depth z for the normally-consolidated state (NC state) is evaluated by integrating the in-situ density profile of overburden soils. Depending on the location of the ground water table (h_w), the density of the soils above the water table uses the total density (p_t), while the submerged density ($p_{sub} = p_{sat} - p_w$) is used for soils below the water table. The expected effective stress (σ'_{nc}) at the depth of interest in the NC state is calculated as

$$\sigma'_{nc} = \int_0^H \gamma'\, dz = \int_0^{h_w} \rho_t \cdot g\, dz + \int_{h_w}^H (\rho_{sat} - \rho_w) g\, dz \tag{2}$$

where γ' is the effective unit weight, H is the overburden depth, h_w is the ground water table depth, ρ_w is the water density (1 g/cm^3), and g is the acceleration of gravity. The calculated stress is generally expected to be the final vertical effective stress at the depth of interest when the primary consolidation process ends. Thus, the expected in-situ shear wave velocity under the NC condition ($V_{s\text{-}nc}$) can be estimated from Eqs. (1) and (2)

$$V_{s-nc} = \alpha \frac{\sigma'_{nc}}{1\ \text{kPa}} \tag{3}$$

In such a case, the experimentally determined parameters α and β are valid for the clay specimen tested, which represents a clay deposit at the retrieved depth. The laboratory testing method used to determine parameters α and β is explained in Section 3.

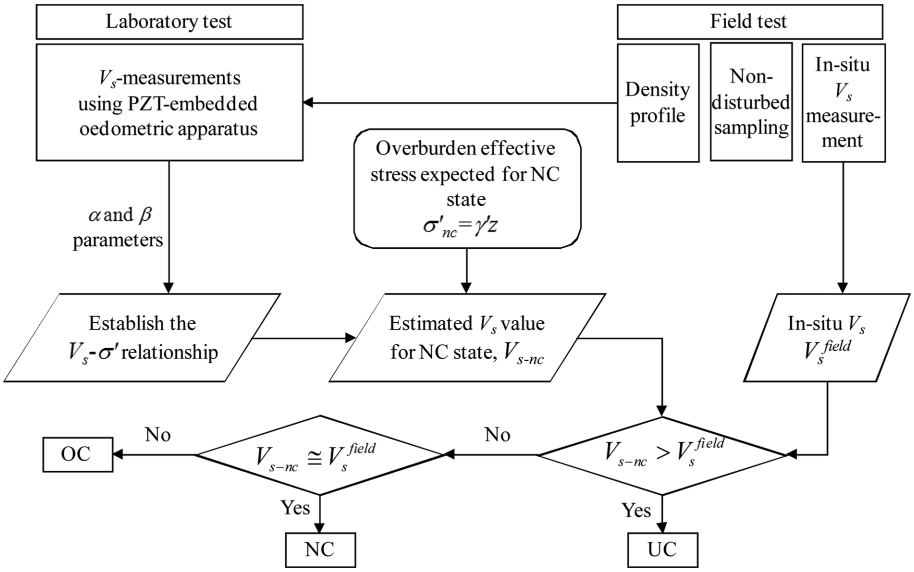


Fig. 1 Conceptual flow chart for evaluation of the in-situ consolidation state

2.3 Consolidation state evaluation

The main idea for the in-situ consolidation state evaluation using the shear wave velocity is to compare the expected in-situ shear wave velocity under the NC condition ($V_{s\text{-}nc}$ in Eq. 3) as estimated from laboratory experiments to the real shear wave velocity measured in-situ (V_s^{field}), which represents the in-situ effective stress state at the time when the in-situ measurement is performed. Therefore, the in-situ shear wave velocity profile is required for this presented concept. The in-situ consolidation state can then be evaluated by comparing the values of V_s^{field} and $V_{s\text{-}nc}$. This is summarized in the flow chart shown in Fig. 1.

2.3.1 NC state

When $V_s^{field} \approx V_{s\text{-}nc}$, the site condition is determined as the NC state where the current in-situ effective stress at the depth (σ'_o) is expected to be approximately equal to the expected effective stress under the NC condition (σ'_{nc}) as shown in Eqs. 2 and 3. Hereinafter, the *current* state, e.g., the current effective stress state, implies the time when the in-situ measurement is conducted.

2.3.2 UC state

When $V_s^{field} < V_{s\text{-}nc}$, the site can be defined as UC state. A slower in-situ shear wave velocity V_s^{field} than the expected in-situ shear wave velocity under the NC condition $V_{s\text{-}nc}$ indicates that the current in-situ effective stress σ'_o (calculated from V_s^{field} and Eq. 1) is lower than the expected effective stress under the NC condition σ'_{nc} (from Eq. 2), given that the parameters α and β for the σ'-V_s relationship are valid. However, the final destination of the in-situ effective stress, after complete dissipation of the excess pore pressure, will become σ'_{nc}, assuming that there is no significant diagenesis, e.g., mineral precipitation or cementation. Therefore, the degree of consolidation (U_z) can be evaluated by dividing the current effective stress value (σ'_o) by the expected final effective stress value (σ'_{nc})

$$U_z = \frac{\sigma'_o}{\sigma'_{nc}} = \frac{(V_s^{field}/\alpha)^{1/\beta}}{\sigma'_{nc}} \tag{4}$$

2.3.3 OC state

When the wave velocity relationship is $V_s^{field} > V_{s\text{-}nc}$., it is expected that the site has undergone stress unloading (e.g., erosion, uplift, or tidal effect), desiccation (e.g., drying process), or diagenetic cementation (e.g., aging or mineral precipitation). While the diagenesis may require a geologic time scale (e.g., more than 1000 years), the unloading stress history of the deposit of interest is presumed to be the most likely cause of the higher in-situ shear wave velocity V_s^{field} within an engineering time frame (i.e., days-to-years). Thus, when the wave relationship is $V_s^{field} > V_{s\text{-}nc}$, the site is likely to be in the OC state.

3. Laboratory experimental program

3.1 PZT-embedded oedometric apparatus

Bender element-type piezo-electric transducers (PZT) of 12 mm in length, 8 mm in width, and 0.6 mm in thickness were applied to measure the shear wave velocity of laboratory specimens. Each side of a bender element was connected to the anode and cathode wires of a coaxial cable, resulting in a series-type assembly. The surface of the bender element was coated for waterproofing with polyurethane. 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 bottom and top plates of the oedometric cell and were fixed with epoxy.

The configuration of the PZT-embedded oedometric top and bottom plates are shown in Fig. 2. The bottom plate was designed to house a bender element and to fix a Shelby tube. The top plate was designed to apply uniformly distributed stress on the specimen in a Shelby tube (i.e., inner diameter = 74 mm) and to fix a bender element. Both the bottom and top plates were designed to have vertical drainage holes and porous filters to allow water-flowing during a consolidation process.

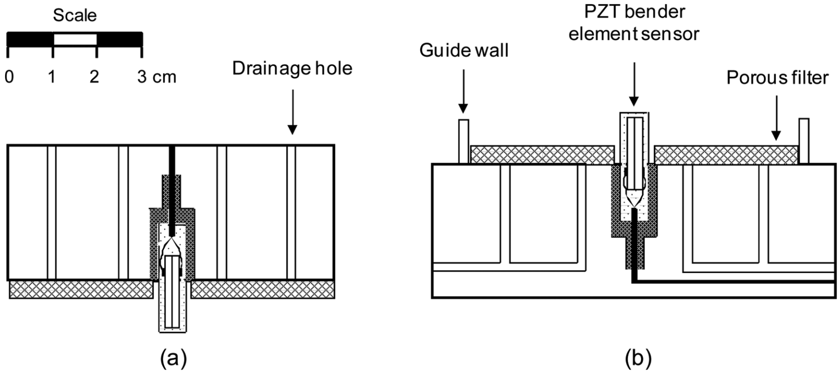


Fig. 2 Configuration of the PZT-embedded oedometric top plate (a) and bottom plate (b)

3.2 Undisturbed specimen preparation

Conventional consolidation testing involves unavoidable disturbance of a specimen by extruding the soil specimen from a Shelby tube and placing the specimen into an oedometric cell. Therefore, minimizing sample disturbance becomes important for accurate in-situ consolidation state evaluation. In this study, the Shelby tube wall itself, which was used for sampling soils from sites of interest, was used as a laboratory oedometric cell wall to minimize sample disturbance. The Shelby tube was cut into a 40 mm long piece without extruding the soil inside. The cutting process was carefully performed using a tube cutter and a steel wire to ensure clean edges, preventing vibration and local stress concentration.

The initial length of the specimens was determined by considering specimen compressibility (ε_v= 0.25-0.30) and the near-field effect of the wave propagation in soil samples (Arroyo *et al.* 2003). To assure sufficient spacing between the tips of the bender elements after loading, the initial specimen height was set to 40 mm. As a result of the greater height of the specimen in this case (i.e., 40 mm) compared to the standard height (i.e., 20 mm) specified for a conventional consolidation test, the required time for excess pore pressure dissipation in this study is expected to be four times longer than that in the conventional testing method.

3.3 Shear wave velocity measurements during consolidation test

For the consolidation test, a Shelby tube specimen was placed on the bottom plate, with the top plate above, while the direction of both the bottom and top bender elements were parallel. The bottom bender element sensor was connected to a signal generator (Agilent 33120A) to generate single step signals with a peak-to-peak amplitude of 5 volts at a frequency of 5 kHz. The top bender element sensor was connected to a signal conditioner (Krohn-Hite 3944) for filtering (band pass filtering; 100 Hz - 50 kHz). Both the input and output signals were displayed with a digital oscilloscope (Agilent 54622D). The experimental system is shown in Fig. 3.

Step loading was applied for the laboratory consolidation test. Generally, six to eight loading steps are recommended due to the sensitivity of the shear wave velocity increment under low stress

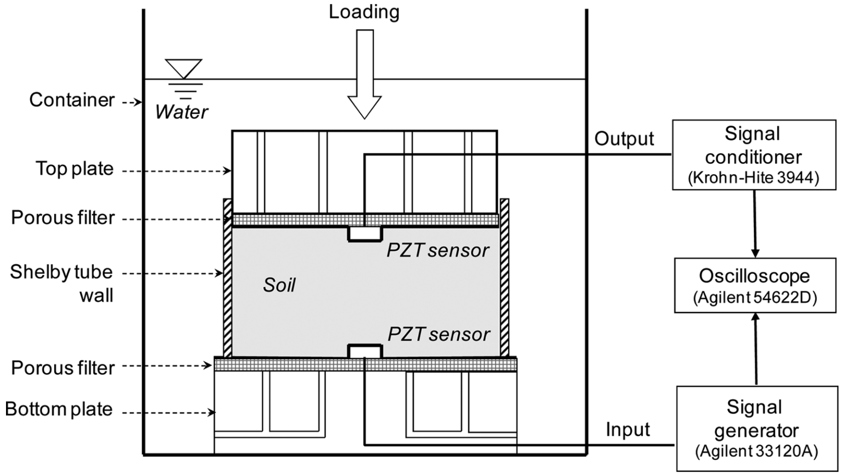


Fig. 3 Laboratory setup for shear wave velocity measurement during consolidation test

conditions. The range of the loading step was determined to cover the expected in-situ effective stress for the NC state (σ'_{nc}) and to consider any additional load, such as a surcharge or the weight of a structure during/after construction at the site.

During the loading process, the travel time of the shear wave between the two bender elements embedded in the top and bottom plates was determined using the first zero crossover arrival time determination method (Kawaguchi *et al.* 2001, Lee and Santamarina 2005, Lu *et al.* 2010). The vertical deformation of the specimen was measured by a dial gauge. Thus, the vertical shear wave velocity of the specimen was measured continuously during the entire consolidation process.

Finally, the σ'-V_s relationship (α and β parameters of Eq. 1) of each specimen was calculated from the final shear wave velocity data for each loading step via least squares best fitting, in which the final shear wave velocity indicates the shear wave velocity converged after the complete dissipation of the excess pore water pressure in a clay specimen. At the same time, the vertical effective stress that the specimen undergoes is equal to the applied load. Therefore, the σ'-V_s relationship was evaluated for each single specimen independently.

4. Practical applications - case study

4.1 Sites of interest

Silty kaolinite clay is the most common type of soft soil in Korea. Three different sites in Korea were studied: a foreshore site (Incheon), a submarine site (Busan), and a site in an estuary delta (Busan). The in-situ density and in-situ shear wave velocity profiles of the study sites are shown in

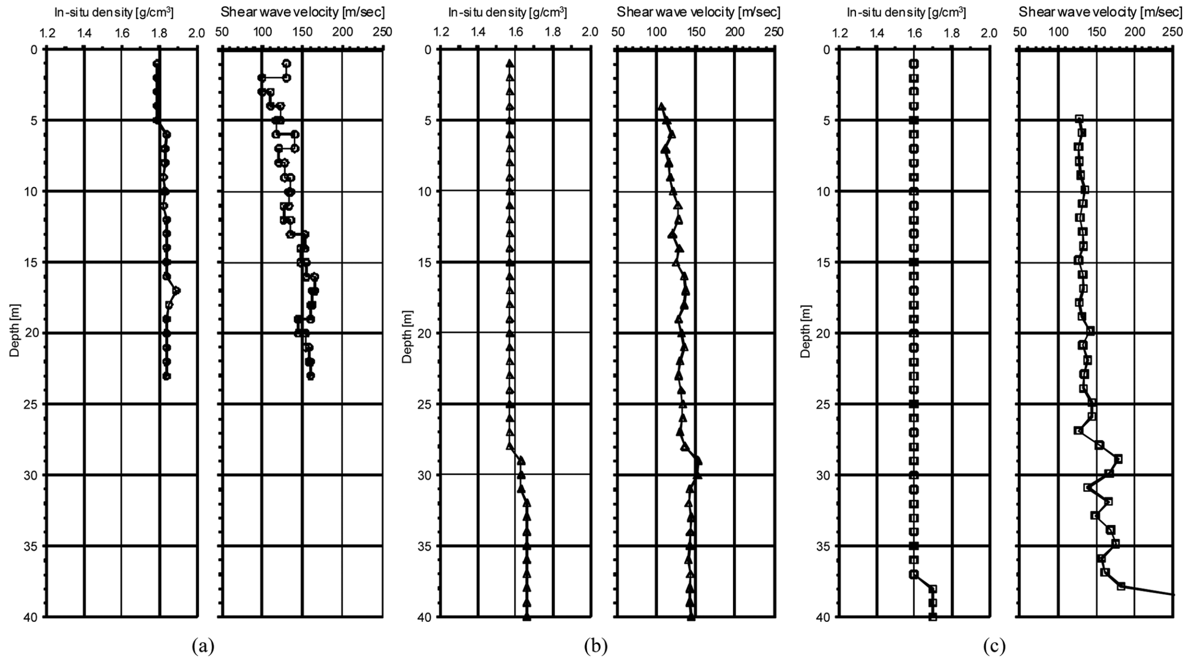


Fig. 4 In-situ density profiles and in-situ shear wave velocity profiles of the study sites. (a) Incheon foreshore, (b) Busan submarine, (c) Busan estuary delta

Fig. 4. Details of each site are as follows.

4.1.1 Foreshore in Incheon

This site was planned for coastal road construction. Consistent with the tidal effect, the site is saturated during floodtide, while it is partially unsaturated during ebb tide. A 13 m thick clay layer exists between a thin reclaimed surface of 3 m thick and underlying granular layer (below 16 m depth). A seismic cone penetration test (sCPT) was performed to obtain the in-situ shear wave velocity profile.

4.1.2 Submarine in Busan

This site is a natural sedimentary deposit under the seawater level in which reclamation is planned. The clay deposit has a thickness in 30 m, and the bedrock is located directly below the clay deposit. The in-situ shear wave velocity profile was obtained in a suspension (SPS) logging test.

4.1.3 Estuary delta in Busan

The Kim-Hae delta is a thick clay deposit located at the estuary of Nak-Dong River, near Busan. The soft clay layer is 30 m thick with sand, weathered rock, and bedrock layers below in order (Ninjgarav *et al*. 2007). The SPS logging test was performed in the field for in-situ shear wave velocity measurements.

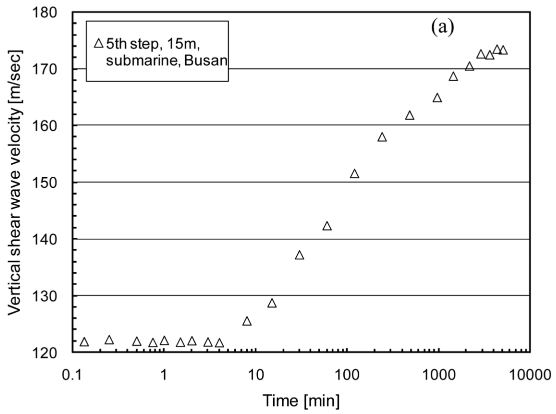
4.2 Results of laboratory tests

The in-situ properties and loading steps of the undisturbed soils sampled for the laboratory testing are summarized in Table 1. According to the sampling depth and in-situ density profile, the expected effective stress value for the NC state (σ'_{nc}) at each point was calculated using Eq. 2.

Table 1 In-situ properties and applied loading steps of tested soil samples

Site	Sample depth [m]	σ'_{nc} [kPa]	V_s^{field} [m/sec]	Loading step [kPa]							
				1st	2nd	3rd	4th	5th	6th	7th	8th
Foreshore (Incheon)	3.0	23.2	110	6.3	12.5	25.2	50.0	100.4	201.1	402.2	-
	4.5	34.8	123	9.6	19.1	38.3	76.6	153.1	306.1	612.3	-
	5.5	42.8	118	9.6	19.1	38.3	76.6	153.1	306.1	612.3	-
	7.5	59.1	121	9.6	19.1	38.3	76.6	153.1	306.1	612.3	-
Submarine (Busan)	5.0	28.0	113	5.7	11.4	22.8	45.6	91.2	182.5	-	-
	15.0	83.9	120	11.4	22.8	45.6	91.2	182.5	365.0	-	-
	28.0	156.6	126	22.8	45.6	91.2	182.5	365.0	730.0	-	-
Estuary delta (Busan)	5.0	29.4	128	6.4	12.9	25.8	51.6	103.2	206.4	412.7	825.4
	10.0	58.9	136	6.4	12.9	25.8	51.6	103.2	206.4	412.7	825.4
	15.0	88.3	127	9.8	19.6	39.2	78.4	156.7	313.4	626.8	1253.5
	20.0	117.7	142	19.6	39.2	78.4	156.7	313.4	626.8	1253.5	-
	25.0	147.2	139	19.6	39.2	78.4	156.7	313.4	626.8	1253.5	-
	30.0	176.6	167	19.6	39.2	78.4	156.7	313.4	626.8	1253.5	-

Note that σ'_{nc} indicates the expected effective stress at the sampled depth under the normally consolidated state, and V_s^{field} means the measured in-situ shear wave velocity at the depth of the sample.



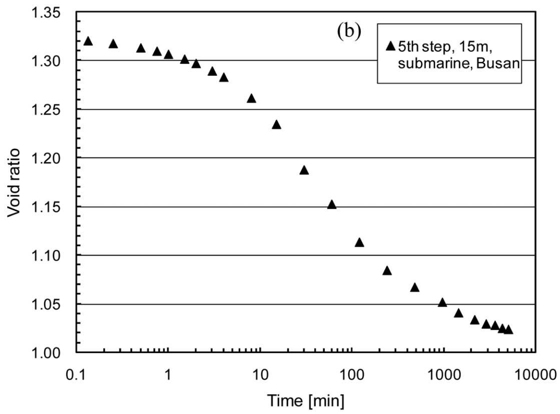


Fig. 5 Changes of vertical shear wave velocity (a) and void ratio (b) in time by a single loading increment. The presented data is extracted from the results of the specimen sampled at the depth of 15 m in submarine deposit, Busan during 5th loading increment step

The changes in the shear wave velocity and void ratio during a single load step are shown in Fig. 5, as an example. The shear wave velocity starts to increase approximately 5 min after the loading. A significant increase in the shear wave velocity occurs between 10 min and 1000 min. Then, the shear wave velocity converges to a certain value after 5000 min, which indicates the dissipation of excess pore water pressure (i.e., this represents a case in which the effective vertical stress is equal to the applied load). Meanwhile, the void ratio decreases continuously with time and converges after 5000 min, which is consistent with the conventional consolidation behavior of clays.

The final shear wave velocity ($V_{s\text{-}nc}$), when the primary consolidation process at each loading step was completed, was measured to determine the σ'-V_s relationship of the NC state. The final shear wave velocity of a tested specimen with respect to the vertical applied stress, which corresponds to the vertical effective stress after the dissipation of the excess pore pressure, is plotted in Figs. 6, 7, and 8 for each site. The σ'-V_s curves were modeled using the experimental parameters α and β with Eq. 1. The in-situ shear wave velocities measured as V_s^{field} at the corresponding depths of the sites and the changes in the void ratio with respect to the applied effective stress are superimposed in Figs. 6, 7, and 8.

4.3 Analysis and discussion

Based on the measured shear wave velocity results shown in Figs. 5, 6 and 7, the σ'_v-V_s relationship parameters α and β (Eq. 1) were determined for each specimen tested and are listed in Table 2. The expected in-situ shear wave velocities under NC conditions ($V_{s\text{-}nc}$, calculated using Eq. 3) and the measured in-situ shear wave velocity (V_s^{field}) are also summarized in Table 2. Accordingly, the consolidation state of each site was evaluated as listed in Table 2.

4.3.1 Foreshore - incheon

The in-situ shear wave velocity of the site V_s^{field} and the expected shear wave velocity for the NC state $V_{s\text{-}nc}$ are plotted in Fig. 9(a). As the V_s^{field} values are smaller than the $V_{s\text{-}nc}$ for all tested depths (i.e., 3 - 7.5 m deep), all layers at this site are considered to be in the UC state, which implies that

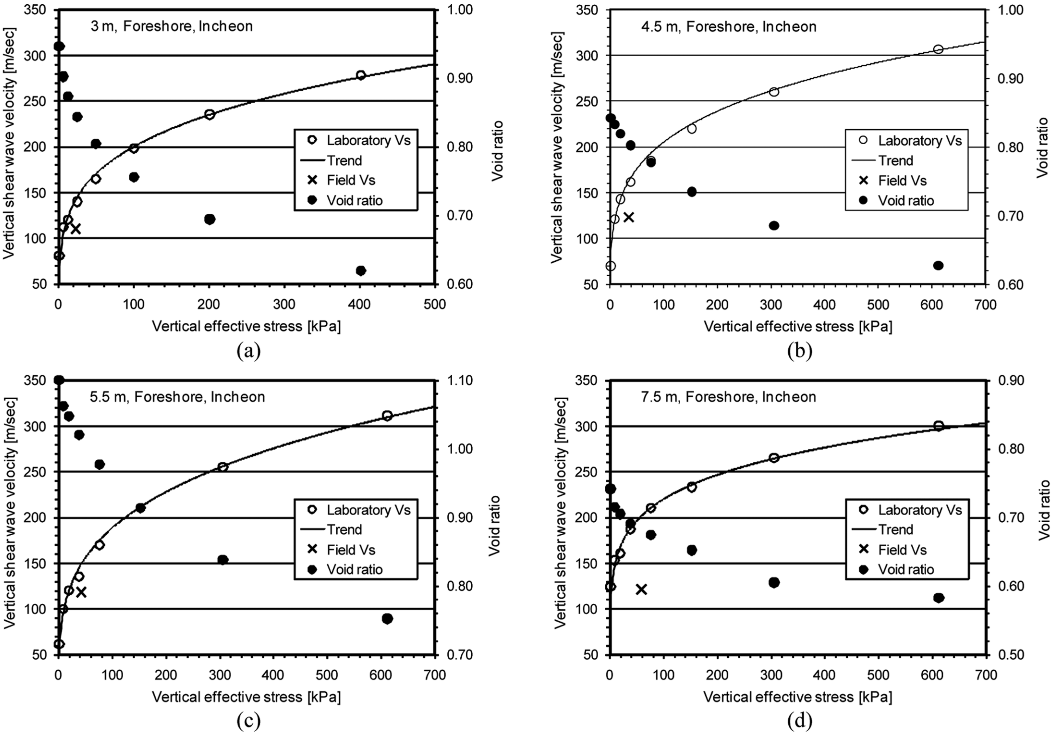


Fig. 6 Laboratory results of shear wave velocity of the clay specimen (open circles) from foreshore site, Incheon from the depth of (a) 3 m, (b) 4.5 m, (c) 5.5 m, and (d) 7.5 m. The σ'-V_s curves (solid lines) are modeled by the experimental parameters α and β, and Eq. 1. The in-situ shear wave velocities measured V_s^{field} at the corresponding depths of the sites (cross points), and the changes in void ratio (solid circles) with respect to applied effective stress are superimposed on the figures

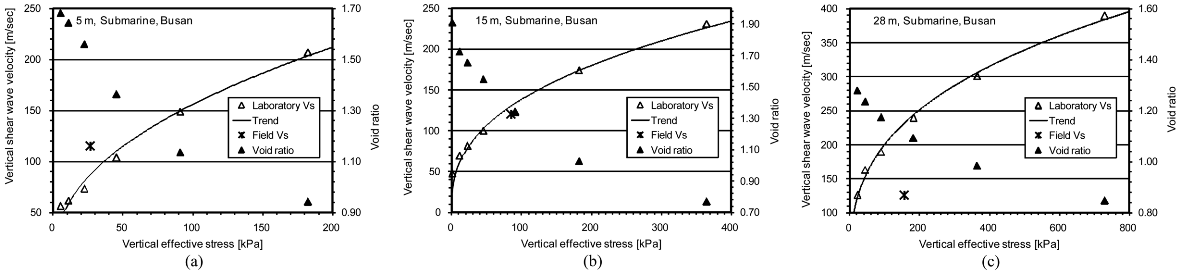


Fig. 7 Laboratory results of shear wave velocity of the clay specimen (open triangles) from submarine site, Busan from the depth of (a) 5 m, (b) 15 m, and (c) 28 m. The σ'-V_s curves (solid lines) are modeled by the experimental parameters α and β, and Eq. 1. The in-situ shear wave velocities measured V_s^{field} at the corresponding depths of the sites (cross points), and the changes in void ratio (solid triangles) with respect to applied effective stress are superimposed on the figures

the excess pore water pressure induced by the weight of the newly-reclaimed layer remains still. Thus, the degree of consolidation (U_z) of each specimen is estimated using Eq. 4 as follows

$$U_z = \frac{(110/70.4)^{1/0.23}}{23.2} = 0.30 \quad \text{for 3.0 m} \tag{5,1}$$

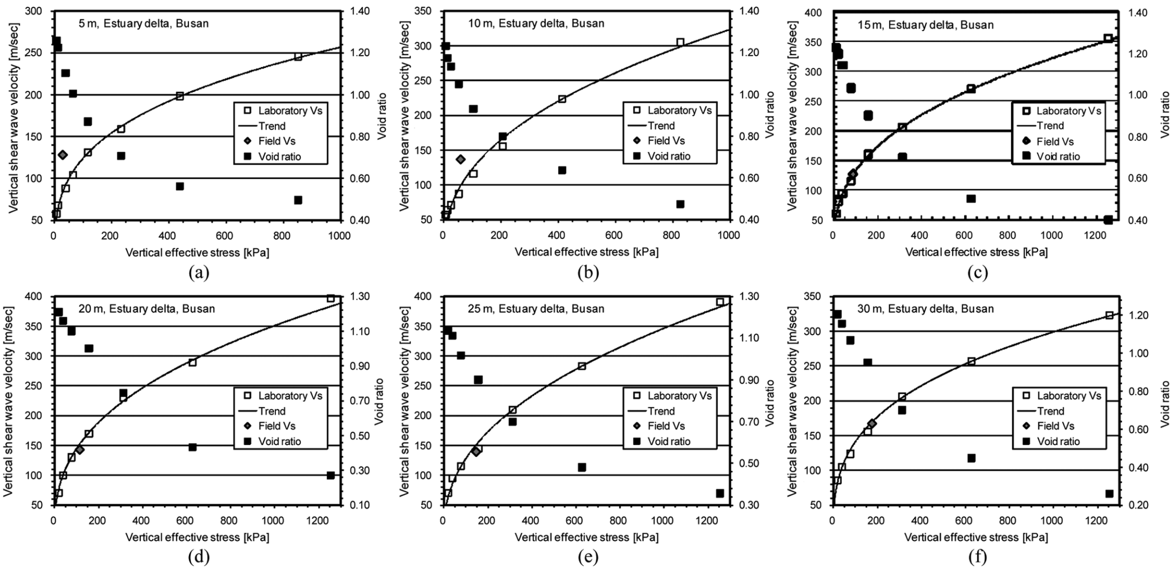


Fig. 8 Laboratory results of shear wave velocity of the clay specimen (open squares) from estuary delta site, Busan from the depth of (a) 5 m, (b) 10 m, (c) 15 m, (d) 20 m, (e) 25 m, and (f) 30 m. The σ'-V_s curves (solid lines) are modeled by the experimental parameters α and β, and Eq. 1. The in-situ shear wave velocities measured V_s^{field} at the corresponding depths of the sites (gray diamonds), and the changes in void ratio (solid squares) with respect to applied effective stress are superimposed on the figures

Table 2 Shear wave velocity parameters and estimated consolidation state

Site	Sample depth [m]	α	β	$V_{s\text{-}nc}$ [m/sec]	V_s^{field} [m/sec]	Consolidation state
Foreshore (Incheon)	3.0	68.72	0.23	145	110	UC
	4.5	74.47	0.22	167	123	UC
	5.5	53.38	0.27	150	118	UC
	7.5	102.94	0.17	206	121	UC
Submarine (Busan)	5.0	21.83	0.44	88	113	OC
	15.0	25.58	0.38	129	120	NC
	28.0	46.84	0.33	230	126	UC
Estuary delta (Busan)	5.0	28.86	0.32	85	128	OC
	10.0	17.87	0.42	98	136	OC
	15.0	22.38	0.39	128	127	NC
	20.0	24.33	0.39	156	142	NC
	25.0	17.32	0.43	148	139	NC
	30.0	31.21	0.33	172	167	NC

Note that α and β are the experimental parameters that determine the σ'-V_s relationship of the clay under the normally consolidated state; $V_{s\text{-}nc}$ is the final shear wave velocity when the primary consolidation at each loading step was completed, correspondently describing the expected in-situ shear wave velocity under the normally consolidated state; V_s^{field} means the measured in-situ shear wave velocity at the depth of the sample; and UC is the under-consolidated clay, NC is the normally consolidated clay, and OC is the over-consolidated clay.

$$U_z = \frac{(123/76.7)^{1/0.22}}{34.8} = 0.25 \quad \text{for 4.5 m} \tag{5,2}$$

$$U_z = \frac{(118/54.5)^{1/0.27}}{42.8} = 0.41 \quad \text{for 5.5 m} \tag{5,3}$$

and

$$U_z = \frac{(121/103.0)^{1/0.17}}{59.1} = 0.04 \quad \text{for 7.5 m} \tag{5,4}$$

The U_z value decreases rapidly below a depth 5 m and approaches zero around 8 m. This indicates that most of the layers require a drainage acceleration treatment from an engineering standpoint.

4.3.2 Submarine - Busan

As shown in Fig. 9(b), the specimens in this site show different results. The upper layer is evaluated as being in the OC state while the degree of consolidation becomes less with a deeper depth. This over-consolidated state of submarine near-seafloor sedimentary deposits is commonly explained by the effects of cementation by mineral precipitation, such as calcite, desiccation, or erosion/slumping of seafloor sediments (Buchan and Smith 1999).

Meanwhile, the specimen at the depth of 28.0 m shows a significant difference between V_s^{field} (i.e., 126 m/s) and $V_{s\text{-}nc}$ (i.e., 230 m/s), which implies that the layer of the depth is in the UC state. Moreover, the U_z value of the 28.0 m specimen is found to be 0.13. This case can arise when rapid sedimentation occurs, such that the excess pore pressure at deeper sediments can be accumulated. However, further studies may be required to identify the cause and effect of the presence of excess pore pressure and the ongoing consolidation process for engineering purposes.

4.3.3 Estuary delta - busan

As shown in Fig. 9(c), the upper layers from 5.0 to 10.0 m are considered to be in the OC condition (i.e., $V_s^{field} > V_{s\text{-}nc}$) while the layers underneath, from 15.0 to 30.0 m, are determined to be

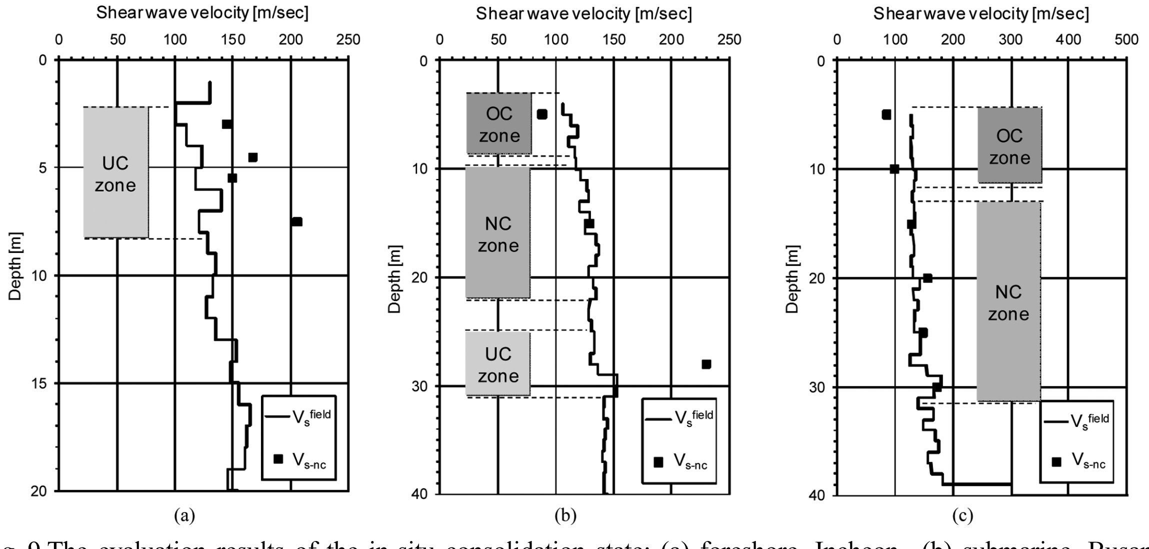


Fig. 9 The evaluation results of the in-situ consolidation state: (a) foreshore, Incheon., (b) submarine, Busan, and (c) estuary delta, Busan

in the NC state. The overconsolidation behavior of Nak-Dong River soil is generally observed (As the upper 10 m thick layer of this estuary delta site is exposed to air, desiccation and surface evaporation, which can harden the soil structure, are presumed to render higher V_s^{field} values and generate an over-consolidation effect (Abuhejleh and Znidarcic 1995).

5. Conclusions

This study presented an alternative method for evaluation of the consolidation state of in-situ clay deposits. The proposed method interprets the laboratory-determined V_s data and in-situ V_s profiles. An oedometric cell instrumented with bender element-type piezoelectric sensors was used for shear wave velocity measurements during a laboratory consolidation process, and to determine the effective stress-shear wave velocity (σ'-V_s) relationship of an undisturbed clay specimen sampled from a deposit of interest. The in-situ consolidation state of the layer in which the specimen was sampled, in turn, was evaluated by comparing the in-situ shear wave velocity data with the estimated effective stress-shear wave velocity trend. A layer at a given depth is classified as over-consolidated (OC) state when the in-situ shear wave velocity at the depth is higher than the laboratory-determined value, i.e., the expected shear wave velocity for the normally consolidated (NC) state under a certain vertical effective stress which corresponds to the same depth. The opposite case is determined as the under-consolidated (UC) state. The site becomes NC when the in-situ shear wave velocity is close to the expected shear wave velocity for the NC state.

For practical applications, the consolidation states of three different sites were evaluated. A foreshore site (Incheon) is evaluated as being in the UC state. A submarine (Busan) and an estuary delta (Busan) deposit show similar distributions: the upper layer shows the OC state, while the deep layers show the NC or UC state. For in-situ consolidation state evaluation, the in-situ density and shear wave velocity profile must be measured first, and undisturbed sampling must be ensured. Thus, the accuracy and reliability of the method proposed in this study depends on the accuracy of the in-situ density profile and shear wave velocity data. The proposed method requires further long-term in-situ monitoring testing that measures the change in the in-situ shear wave velocity during consolidation. This will provide more insightful information regarding a consistent data set of reclaimed or pre-consolidated clay deposits and will enrich the reliability of the presented method.

Acknowledgements

This work was supported by the National Research Foundation of Korea (NRF) grant funded by the Korea government (MEST) (No. R11-2002-101-04005-0).

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