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Back-analysis of geotechnical parameters on PVD-improved ground in the Mekong Delta

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ABSTRACT

This study presents a back-analysis of geotechnical parameters on prefabricated vertical drain improved ground at a site in the Mekong Delta. Various time – settlement behaviors that reflected different clay thicknesses and loading patterns were observed. The total surface settlement behavior at several monitoring locations was simulated using an updated exponential method that considered staged construction. The analyzed results were validated by substituting the values into a theoretical solution for radial consolidation. The estimated theoretical behaviors were comparable with the monitored behaviors. The geotechnical parameters were back-analyzed by applying the previously analyzed results to various theoretical and empirical formulas. However, the use of extensometer data that were installed at large intervals produced different values of the geotechnical properties. Furthermore, finite element analysis supported the back-analyzed total settlement behaviors and nearly disregarded the application of the geotechnical properties that were obtained using either surface or subsurface settlement data. However, settlements and excess pore pressures in the sublayers were not successfully predicted even when the geotechnical properties were adjusted. Thus, subsurface instruments that can be installed closely in thick clay deposits are required to reliably reevaluate the variations in geotechnical properties along a certain depth.

1. Introduction

Ground improvement of thick clay deposits has been conducted to develop marginal lands in many parts of the world. Prefabricated vertical drain (PVD) techniques are among the most extensively used ground improvement methods. A key issue in PVD techniques is whether field monitoring (e.g., settlement and excess pore pressure behaviors from the initial settlement to the ultimate settlement) and behavior prediction are reliably conducted. Despite numerous studies conducted over the past two or more decades, the precise prediction of consolidation and delayed (long-term) settlements, along with their rates, remains difficult (Bergado et al., 2002; Chu et al., 2006; Indraratna et al., 2012; Jang and Chung, 2014; Karim et al., 2011; Liu and Chu, 2009; Lo et al., 2008; Olsen, 1998; Rowe and Taechakumthorn, 2008; Rujikiatkamjorn and Indraratna, 2015; Saowapakpiboon et al., 2011; Taechakumthorn and Rowe, 2012; Watabe and Leroueil, 2015). The underestimation of settlement and consolidation time have been reported in several projects, such as the Changi Airport project in Singapore (Bo et al., 2003) and the Noksan reclamation project in Korea (Chung, 1999). An unreliable prediction may be related to various factors, including the limitations of the theoretical solutions, evaluated soil, and PVD-related parameters, the construction procedure, deterioration of the installed PVDs. Thus, feedback is essentially used to improve the prediction of settlement behaviors.

Several observational methods can be used to predict the ultimate settlement and the settlement rate (Asaoka, 1978; Debats et al., 2013; Tan et al., 1991; Chung et al., 2014b) and the results can be applied to reevaluate geotechnical parameters (Bergado et al., 1992; Bartlett and Alcorn, 2004; Cao et al., 2001; Chung et al., 2009, 2014b; Chung and Lee, 2010; Leroueil et al., 1990; Magnan et al., 1983; Voottipruex et al., 2014). Numerical analysis is also adopted for similar purposes (Bergado et al., 1993, 1996; Cao et al., 2001; Chai et al., 2001, 2011; Hawlader et al., 2002). Observational methods can easily evaluate the average geotechnical parameters. However, their variations along certain depths are difficult to reflect, whereas numerical analysis exhibits an opposite trend. Back-analyzed geotechnical properties are generally used to compare laboratory and field soil test results and to predict the settlement behaviors of neighboring sites. However, whether the results obtained from observational and numerical methods are comparable with each other is rarely verified (Lam et al., 2015; Rezania et al., 2017). Thus, the two approaches should be simultaneously applied to

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validate the appropriateness of the back-analyzed geotechnical properties.

This study aims to reliably reevaluate the geotechnical properties and to predict the settlement behaviors of a PVD-improved ground in the Mekong Delta, in which the effect of the poorly adopted sampling techniques are compensated. The effects of different thicknesses and stepped loadings on the monitored settlements and excess pore pressures were investigated. An observational method (i.e., the exponential model) was modified to consider the effect of staged construction on the settlement. Various geotechnical properties were determined based on the back-analyzed results. Furthermore, the estimated properties were compared with the laboratory and field test results. Moreover, finite element (FE) analysis was performed using the estimated geotechnical parameters, and the analysis results were compared with the two previous results. Further considerations to improve the back-analysis process were discussed based on the comparison.

2. Observational methods and determination of geotechnical properties

2.1. Observational methods

2.1.1. End of the primary consolidation settlement

The graphical method of Asaoka (1978) is known to produce the most reliable value of the ultimate (ρ_{ult}) or the end of the primary consolidation settlement (ρ_{100}). The ultimate settlement is graphically determined from a special diagram that is plotted using each settlement read out at time interval (Δt) on the ρ-t curve.

$$\rho_{ult} = \frac{\beta_0}{1 - \beta_1} \tag{1}$$

where β_0 and β_1 are the intercept and slope of a straight line in the special diagram, respectively. However, ρ_{ult} (or ρ_{100}) in this method varies depending on the selected time interval (Arulrajah et al., 2004; Asaoka, 1978; Chung et al., 2014b; Edil et al., 1991). The ultimate settlement is also estimated using the hyperbolic relationship of ρ-t, as follows (Tan et al., 1991):

$$\rho = \frac{t}{\alpha + \beta t} \tag{2a}$$

$$\lim_{t\to\infty}\rho = \lim_{t\to\infty}\frac{1}{\alpha/t + \beta} = \rho_{ult} = \frac{1}{\beta} \tag{2b}$$

The hyperbolic methods overestimate the ultimate value, which depends on the percentage of data (Chung et al., 2014a, b).

Recently, an exponential model is used as a rational method to simulate consolidation behavior (Chung et al., 2014b).

$$\rho = \rho_{ult}[1 - \exp(-\eta t)]^{\kappa} \tag{3}$$

where three unknowns (ρ_{ult}, η, and κ) are determined by best fitting the measured data. The data after the end of construction are generally used in observational methods. However, the current study considers stepped loading data as part of the primary consolidation settlement. Thus, the time (t_0) to initiate the primary consolidation in the monitored ρ-t relationships is determined as follows. Eq. (3) is rearranged to consider the initial settlement and time.

$$-\ln\left(1 - \left(\frac{\rho}{\rho_{100}}\right)^{1/\kappa}\right) = \eta\,(t - t_0) \tag{4}$$

where the unknowns (ρ_{100}, η, and κ) are successfully obtained using Excel Solver.

2.1.2. Consolidation coefficient

For thick deposits with small drain spacing, only radial flow may be approximately considered to occur, thereby neglecting the effect of vertical flow (Lee and Chung, 2010). That is,

$$U_h = \left[1 - \exp\left(-\frac{8}{F}T_h\right)\right] \tag{5a}$$

or

$$U_h = \left[1 - \exp\left(-\frac{8}{F_n}T_{h(n)}\right)\right] \tag{5b}$$

where T_h ($=c_h t/d_e^2$) and $T_{h(n)}$ ($= c_{h(n)}t/d_e^2$) are the time factors; d_e is the diameter of the influence zone of each drain; F is a factor that accounts for the combined effects of spacing ($F_n \approx \ln(n) - 0.75$ for $n > 10$), [$F_s = (k_h/k_s - 1)\ln(s)$] is the smear; ($F_r$) is the well resistance; $n = d_e/d_w$, in which d_w is the equivalent diameter of the drain; $s = d_s/d_w$, in which d_s is the equivalent diameter of the smear zone; and k_h and k_s are the coefficients of the horizontal permeability of the undisturbed soil and smear zone, respectively. Thus, a relation is obtained based on Eq. (6) as follows:

$$\frac{c_h}{F} = \frac{c_{h(n)}}{F_n} = \frac{c_{h(n+s)}}{F_{n+s}} \tag{6}$$

where $c_{h(n+s)}$ is the consolidation coefficient that corresponds to F_{n+s}. The radial consolidation coefficient $c_{h(n)}$ for the ideal condition (without the effects of smear and well resistance) can be obtained with a given F_n.

Magnan et al. (1983) proposed a method for estimating the $c_{h(n)}$ of PVD-improved ground based on the analytical solution developed by Asaoka (1978).

$$c_{h(n)} = \frac{d_e^2}{8}\frac{F_n}{\Delta t}\cdot\ln\beta_1 \tag{7}$$

Chung et al. (2009) presented a method for estimating $c_{h(n)}$ based on the hyperbolic method.

$$c_{h(n)} = [0.1333\ln(n) - 0.0906]\cdot\frac{\beta d_e^2}{\alpha} \tag{8}$$

$c_{h(n)}$ that varies with time can be considered with the exponential model (Chung et al., 2014b). That is, the following expression is derived from Eqs. (3) and (5b):

$$c_{\mathrm{h(n)-var}} = \frac{T_{h(n)}}{t}d_e^2 = \frac{\eta F_n d_e^2}{8}\frac{\ln(1 - U_h)}{\ln(1 - U_{\exp}^{1/\kappa})} = \frac{\eta\,d_e^2}{8}\cdot[\ln(n) - 0.75]\cdot\Phi \tag{9}$$

where $\Phi = \ln(1 - U_h)/\ln(1 - U_{\exp}{}^{1/\kappa})$ and $U_{\exp} = \rho/\rho_{100} = U_h$. The Φ value rapidly decreases at the initial part and then gradually decreases with increasing U_h. Thus, the average value of Φ is determined for 30% $\le U_h \le$ 90% as follows:

$$\Phi_{ave} = \frac{\int \Phi dU}{\int dU} = [0.971\kappa^{-0.119} + 0.028\kappa^{-1.978}]^{5.123} \tag{10}$$

where the coefficient of determination $r^2 = 0.9999$. Thus, the average value of $c_{h(n)}$ is obtained by

$$c_{\mathrm{h(n),ave}} = \frac{\eta\,d_e^2}{8}\cdot[\ln(n) - 0.75]\cdot\Phi_{ave} \tag{11}$$

where Φ_{ave} approximately corresponds to Φ at U_h = 60%.

2.2. Determination of geotechnical properties

2.2.1. Compressive parameters

Compressive parameters may be estimated using the applied load, the monitored settlement, and excess pore pressure. Recompression and consolidation settlements for the 1D condition are calculated as follows:

$$\rho_r = \sum_{i=1}^{m}\frac{C_{Si}}{1 + e_{0i}}\Delta h_i\log\left(\frac{\sigma'_{pi}}{\sigma'_{v0i}}\right), \tag{12a}$$