Numerical modeling for highway embankment on soft clay with PVD improvement in Red River Delta

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ABSTRACT

The Red River Delta is one of the major deltas in Vietnam. Sedimentation by Red River at the Pleistocene to Holocene epoch created unique soft clay deposits. Soil formation in the area is critical in geotechnical engineering practice with respect to settlement and land subsidence issues caused by consolidation of the subsoils. Research on the consolidation settlement issue within the delta is limited, which creates a drawback for engineers to design and construct roadways or buildings in the area. This paper presents the results of consolidation settlement analyses for one of the most important constructions, Hanoi – Hai Phong Expressway, within the Red River Delta. Prefabricated vertical drains (PVDs) were used to accelerate the dissipation of the excess pore water pressure within the subsoil due to the placement of the highway embankment. The PVDs were installed with a spacing of 1.4 – 1.8 m and a length of 10 – 16 m depending on the thickness of the soft clay. Monitoring instruments, including settlement plates, piezometers, and inclinometers, were installed and monitored from the beginning of construction to the time when approximately 90% of the degree of consolidation of the soft clay has reached. Numerical modeling was conducted using the Plaxis 2D software to evaluate the consolidation settlement and the dissipation of the pore water pressure of the soft clay with the installation of the PVDs. An evaluation of lateral displacement due to the placement of the highway embankment was also conducted in this study. The predicted data obtained from the models were compared to the field measured values, and differences in the comparison were discussed in the paper. It is shown that the predicted values match reasonably well with the monitoring data. Therefore, the numerical modeling approach used in this paper can be utilized for future projects within the area.

Keywords: Numerical Analyses, Ground Improvement, Prefabricated Vertical Drain, Red River Delta.

1 INTRODUCTION

The Red River Delta (RRD) is the largest delta in the north of Vietnam with an area of 150,000 km^2 and is considered as one of the most dynamic economic centers in Southeast Asia. Most of the surface area of the delta is covered by sediment formation. The youngest formation with around 3,000 years of age is mainly original from lacustrine and shallow-sea sediment (Kirov and Truc, 2011). According to Giao and Hien (2007), most of the area is young soft clay deposits (Pleistocene to Holocene Epoch). The subsoils of the area mainly consist of soft to medium clay, underlain by sand layer, medium to stiff clay to a depth of 20 m (Giao and Hien, 2007).

One of the most important constructions in this area is Ha Noi – Hai Phong Expressway which connects Ha Noi capital and Hai Phong city. Hai Phong is the biggest seaport in Northern Vietnam. It plays a significant role in importing and exporting products for all of Northern Vietnam. Hanoi is the capital and is also the economic, cultural, and political center of Vietnam. With the construction of the expressway, time for traveling from Ha Noi to Hai Phong is reduced from 2.5 to 1.5 hours. The freeway is the first-Class A-Expressway in Vietnam, designed to meet the International Expressway Standard. The freeway has a total length of 105.5 km and a design speed of 120 km/h. The project starts from Ring Road No. III in Hanoi city (Km 0+00) and ends at the Dinh Vu dam in Hai Phong city (Km 105+500). The whole project was divided into several packages. Package EX-3 (Km 19+000 – Km 33+000) located in Hung Yen province was chosen for our analyses. The study area is shown in Figure 1.

Considering various ground improvement methods related to construction time, estimated costs, technique effectiveness, and limitation in construction, prefabricated vertical drain (PVD) is widely adopted as one of the most cost-effective methods for soft clay (Bergado et al., 1996a, b; Chai et al., 2001; Shen et al., 2005; Chu and Yan, 2005; Rowe and Taechakumthorn, 2008, Abuel-Naga et al., 2012; Ong et al., 2012; Deng et al., 2013; Cascone and Biondi, 2013; and Bari and Shahin, 2014). In the past, the design of the vertical drains was commonly conducted by using the Terzaghi 1-D consolidation theory (Barron, 1948; Hansbo, 1981). Numerical modeling has been utilized to provide a more rigorous design of the vertical drains (e.g., Chai et al., 2001; Borges, 2004; Rujikiatkamjorn and Indraratna, 2006; Rujikiatkamjorn et al., 2008; Abuel-Naga et al., 2012; Bari and Shahin, 2014; Lam et al., 2015). In this study, a 2-D plane strain finite element modeling using the Plaxis software was conducted to evaluate the effect of the PVD on the consolidation of the soft clay at the Package EX-3 site. Comparison of the modeling results to the monitoring data was made, and the results of the comparison are included in the paper.

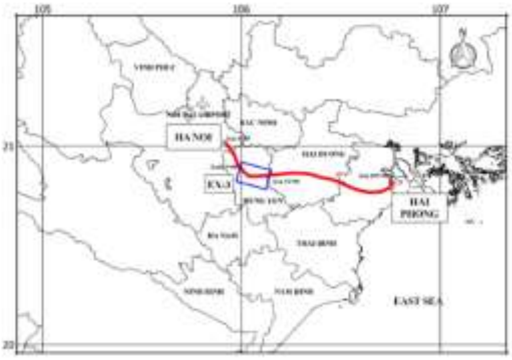

Fig. 1 Location of study area

2 NUMERICAL ANALYSES OF PREFABRICATED VERTICAL DRAIN

The finite-element method (FEM) is commonly used to simulate the effectiveness of vertical drains installed in soft clays. As the effect of each drain is in a three-dimensional (3D) manner, it is ideal that the vertical drain is simulated with a 3D model (Borges, 2004; Rujikiatkamjorn and Indraratna, 2006; Rujikiatkamjorn et al., 2008; Wang and Chen, 2004). On the other hand, there is a number of civil engineering structures, such as road and railway embankments, that can be analyzed by a 2D plane strain model. Researches indicate that plane strain in numerical analysis can be readily adapted for most real conditions (Hird et al., 1995, Indraratna and Redana, 2000). The equivalent plane strain for a vertical drain system was illustrated by Indraratna and Redana (1997), as plotted in Figure 2.

According to Hansbo (1981), the average degree of consolidation, U_h, for axisymmetric flow on a horizontal plane at a depth z and at time t can be predicted from:

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

Where T_h = time factor for horizontal drain; F = factor of the PVD geometry.

Indraratna and Redana (1997) showed that the degree of consolidation, U_{hp}, at a depth *z* in a plane strain condition could be represented by:

$$U_{hp} = 1 - \exp\left(\frac{-8T_{hp}}{F_p}\right) \tag{2}$$

Where T_{hp} is a time factor in a plane strain condition

At a given stress level, to maintain the same degree of consolidation at each time step, the average degree of consolidation, U_h, for the axisymmetric condition and the average degree of consolidation, U_{hp}, for the equivalent plane strain condition are made to be equal:

$$\frac{T_h}{F} = \frac{T_{hp}}{F_p} \tag{3}$$

Where

$$T_h = \frac{c_h t}{4r_e^2}\ ;\ T_{hp} = \frac{c_{hp} t}{4B^2}$$
$$F = \ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k_s}\right)\ln s - \frac{3}{4}$$
$$F_p = \frac{2}{3} + \frac{2k_h}{Bq_{wp}}(2lz - z^2)$$

k_h = horizontal permeability in axisymmetric flow; n = D_e/d_w (D_e is the diameter of the unit cell, d_w is the diameter of a drain); s = d_s/d_w (d_s is the diameter of the smear zone); k_s = horizontal permeability of smear zone; B = drain spacing in-plane strain condition; r_e = drain spacing in axisymmetric flow; c_h = coefficient of consolidation in axisymmetric flow; and c_{hp} = coefficient of consolidation in-plane strain condition.

The horizontal permeability, k_{hp}, for the in-plane strain condition can be derived by assuming that the drain spacings B and r_e for the plane strain condition and the axisymmetric condition, respectively, are the same. The equation for k_{hp} is shown as follows.

$$k_{hp} = \frac{2k_h}{3\left[\ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k_s}\right)\ln(s) - \frac{3}{4}\right]} \tag{4}$$

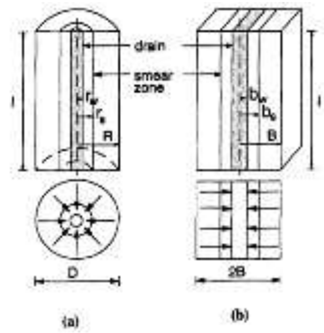

Fig. 2 Equivalent plane strain from axisymmetric unit cell (Indraratna and Redana, 1997)

3 GEOTECHNICAL CHARACTERIZATION OF THE STUDY SITE

3.1 Cross sections used in the analyses

There are 68 boreholes conducted for the soil investigation in the package. The soil profile along Package EX3 from Km 19+000 to 33+000 was created based on the boring logs and is shown in Figure 3. Figure 3 indicates that the soils at the site consist of a layer of soft to firm clay, underlain by interbedded clayey sand and soft clay, and loose to medium dense clayey sand.

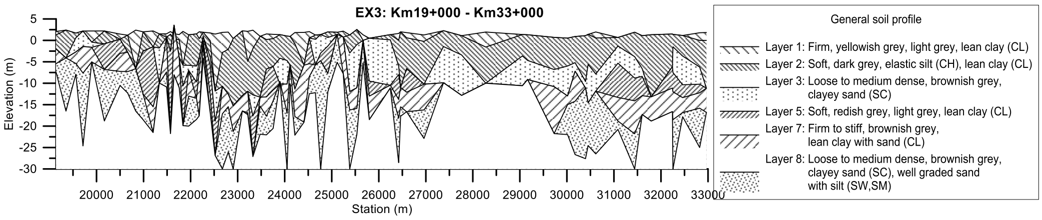

Fig. 3 Soil profile along package EX3

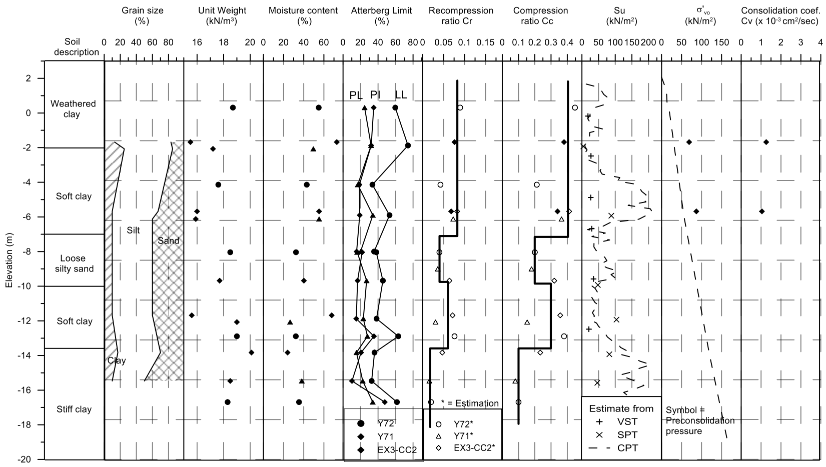

Fig. 5 Soil profile at Km 21+980-22+190

The Atterberg limits of the soft and stiff clays were measured for the samples obtained from the boreholes. The results of the test were plotted in Figure 4. It is concluded that Atterberg limits of the soft and stiff clays are quite similar. As shown in Figure 4, the liquid limit was measured from approximately 30 to 80%, and the plasticity index was measured from approxiamtely 15 to 45%. The majority of the soft and stiff clays were classified as either CH or CL.

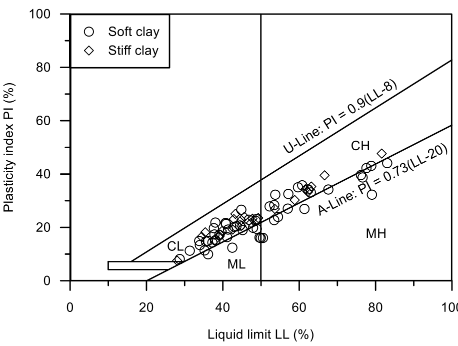

Fig. 4 Plasticity chart of soft and stiff clays along Package EX3

A section from Km 21+980 to 22+190 was chosen for the numerical analyses. Soil properties was determined based on the information obtained from boreholes Y71, Y72, and EX3-CC2, as shown in Figure 5. The soils above a depth of 16 m mainly consist of clay and silt and exhibit fairly uniform soil properties. Below 16 m, the soil consists of sand with increasing strength.

The PVDs were installed to a depth of 9 m with a spacing of 1.8 m and arranged in a rectangular pattern. The height of the embankment at this location is 4.36 m. The embankment was constructed in 6 loading stages. The embankment was also constructed with a 2H:1V side slope. The groundwater table was detected at a depth of about 1.7 m.

3.2 Soil input parameters used in the analyses

Figures 5 summarizes the soil input parameters used in the analyses. The compression index, C_c, was estimated using Equation (5) proposed by Skempton (1944) as follows:

$$C_c = 0.009(LL - 10) \tag{5}$$

The recompression index, C_r, was assumed using the equation shown below (Ladd, 1973):

$$C_r = (0.1 - 0.2)C_c \tag{6}$$

The pre-consolidation pressure or yield point, σ'_p, was estimated using Equations (7) and (8) proposed by Nguyen (2017) for the Red River Delta (RRD) clay.

$$\sigma'_p = \sigma'_{v0} + (20 \div 45)\text{ kPa with } z \le 20\text{m} \tag{7}$$

$$\sigma'_p = \sigma'_{v0} + (60 \div 85)\text{ kPa with } z > 20\text{m} \tag{8}$$

Where σ'_p = pre-consolidation pressure or yield point (kPa); σ'_{v0} = effective stress (kPa); z = depth of sample (m).

Mayne and Mitchell (1988) derived the following empirical relationship to estimate the over-consolidation ratio (OCR) of a natural clay deposit based on the undrained shear strength:

$$\text{OCR} = \beta\frac{S_{u(\text{Field})}}{\sigma'_v} \tag{9}$$

Where $S_{u(Field)}$ is the undrained shear strength measured from vane shear test (VST), σ_v' is the effective overburden pressure, and β is the function of plasticity index as follows:

$$\beta = 22 I_p^{-0.48} \tag{10}$$

The values of the plasticity index, I_p, and effective overburden pressure, σ_v', of the samples taken from a borehole next to the vane shear test were used to determine the OCR using Equation (9). The estimated OCR values are shown in Figure 6.

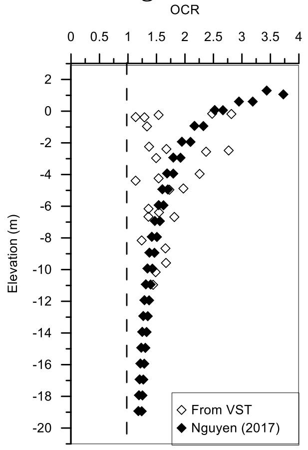

Fig. 6 Estimated OCR values based on VST and Nguyen (2017)

3.3 Monitoring scheme

To monitor the performance of the embankment, the following instruments were installed at the site: settlementplates, piezometers, inclinometers, observation wells, and wooden alignment stakes. The locations of the instruments that were installed at the site are presented in Figure 7. The instruments were monitored from the beginning of construction to the end of the required degree of consolidation.

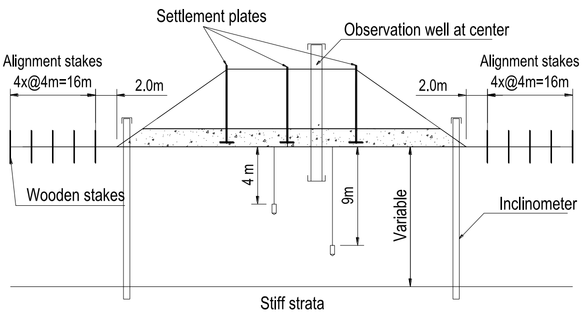

Fig. 7 Monitoring scheme installed at the site

4 NUMERICAL MODELING

Finite element numerical modeling was conducted using the commercial program PLAXIS Version 8 to simulate the effect of the embankment loading on the RRD clay with the installation of the PVDs. The configuration of the finite element meshes is shown in Figure 8. The embankment loading sequence following the measured embankment heights was simulated in the model. The Modified Cam Clay (Soft Soil) model was used in the simulation. The horizontal permeability, k_{hp}, for the plane strain condition calculated using Equation (4) for each soil layer was used initially in the model and then calibrated to match the predicted settlement to the monitoring data.

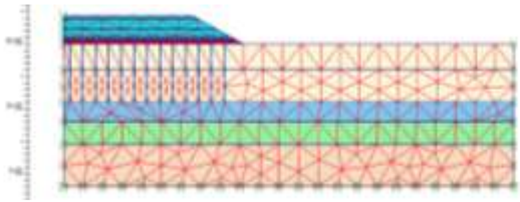

Fig. 8 Finite element mesh (Note to Manh: take out the border of the figure)

5 RESULTS AND DISCUSSIONS

5.1 Settlement

The surface settlement was evaluated by comparing the predicted settlement obtained from the Plaxis 2D models to the observed settlement in the field. The surface settlement values along the centerline predicted by the Plaxis 2D program is presented in Figures 9. It is evident that the predicted settlement values match reasonably well with the monitoring data. The slight difference in the settlement at the beginning of construction could be explained by changes in the consolidation coefficient, C_v, during the loading process. A constant value of C_v was adopted in the analyses. However, at the beginning of loading, the subsoil layers are in the elastic state, which would give a lower value of the consolidation coefficient, C_v, than that for the subsoil layers when the soils are in the plastic state.

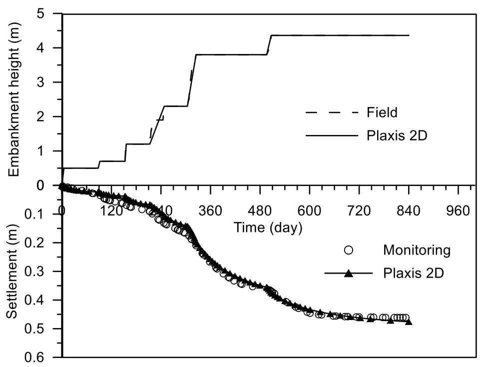

Fig. 9 Settlement at Centerline

5.2 Lateral displacement

In addition to accelerating the vertical consolidation settlement, the placement of the embankment material could cause a lateral displacement of the foundation soil with the installation of the PVDs. The placement of the inclinometers for the monitoring of the lateral movement of the foundation soil is presented in Figure 10.

The lateral displacement profiles with depths predicted by the Plaxis 2D models along with the measured data from the inclinometers are illustrated in

Figure 11. As shown in Figure 11, the predicted lateral displacement values are higher than those measured in the inclinometers in the first year of the monitoring. The predicted lateral displacement values match reasonably well with the field measured data after the first year. The predicted lateral displacement values and the measured data at the end of loading (time = 840 days) is also shown in Figure 11. The poor lateral displacement prediction at the beginning of the loading process could be caused by many factors such as permeability, Poisson's ratio, the anisotropy of the soil, nonlinear stress-strain behavior, and other incorrect assumptions (Poulos, 1972).

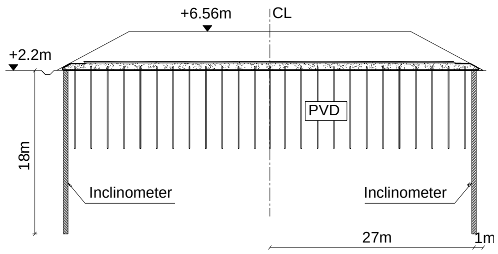

Fig. 10 Installation of the Inclinometers

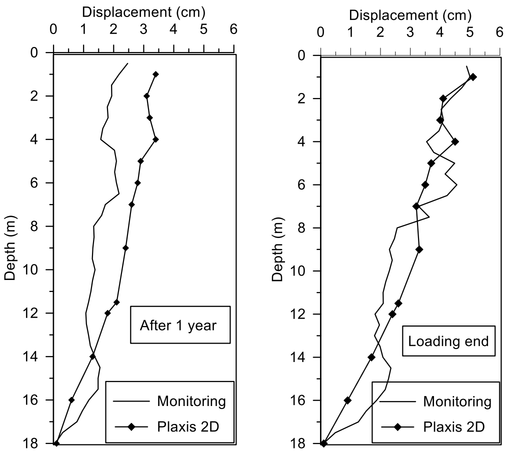

Fig. 11 Lateral displacements at 27m from Centerline

5.3 Pore water pressure

An evaluation of the change in pore water pressure due to the placement of the embankment material was conducted by comparing the predicted pore water pressure predicted in the Plaxis models to the measured data in the piezometers. The comparison was made at a depth of 9 m below the ground surface.

Distribution of excess pore water pressure and pore water pressure at the end of loading (time = 840 days) are illustrated in Figure 12. It shows that the predicted pore water pressure and the predicted excess pore water pressure obtained from the model have similar trends and magnitudes to the monitoring data. It is evident that the pore water pressures increased immediately after the loading of the embankment and decreased gradually after the loading. The discrepancy between the predicted and monitoring data could be explained by the following reasons:

(1) The pore water pressure is very sensitive to the input permeability in the models. The permeability was assumed to be a constant value that fits the overall trend of the measured data. In reality, the permeability varies with time during the consolidation process.

(2) The groundwater table was assumed to be constant in the models. The measurements of the pore water pressure were taken from July 2011 to December 2013. Seasonal fluctuations of the groundwater table during the monitoring period were expected to occur at the site.

(3) At the early stages, the measured pore water pressure and excess pore water pressure are much higher than the predicted values, but later on, they are quite compatible. It could be caused by partial clogging of the drains from the installation of the drains at the beginning of the monitoring. The drains were flushed and functioning more properly later in the monitoring period.

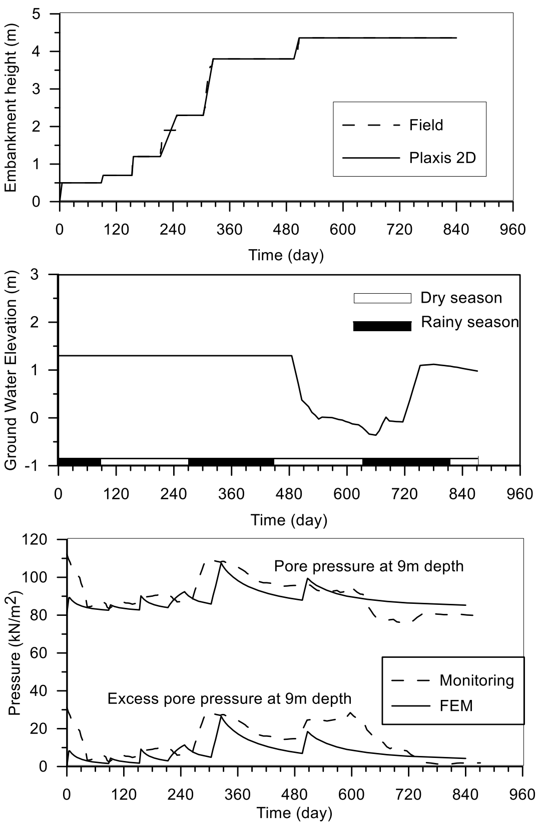

Fig. 12 Variation of Pore pressure and Excess pore pressure with time at 9 m depth

6 CONCLUSIONS

Based on the results of the settlement, horizontal displacement, as well as pore water pressure analyses, the following conclusions and findings are drawn:

a. The performance of highway embankment using the PVDs in the RRD clay could be reasonably modeled using the numerical modeling technique described in the paper. The numerical modeling can be used to assist in the design of the PVDs for the purpose of ground improvement.

b. The calibrated OCR values were found to be 1.8 – 2.2 for the upper soil layers and 1.2 – 1.6 for the lower soil layers with C_h/C_v = 2. The validated values of compressibility and flow parameters for the soil layers were summarized in Table 1. These values of the soil parameters for the RRD clay are useful for use in future projects within the area.

It is difficult to match the predicted pore water pressure to the measured data in the field. Potential factors that could affect the results were listed. It is recommended that these factors be taken into the consideration when doing the numerical modeling for the PVD ground improvement.

Table 1. Consolidation parameters for the RRD clay

Description	C_c	e_0	$C_v \times 10^{-3}$ (cm²/s)
Weathered soil	0.2 ~ 0.4	1.3 ~ 1.6	2
Soft clay	0.3 ~ 0.4	1.2 ~ 1.6	0.5 ~ 3
Medium to Stiff clay	0.1 ~ 0.15	1 ~ 1.2	0.55 ~ 1
Sand	-	0.65 ~ 1	6

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