ESTIMATING UNDRAINED SHEAR STRENGTH OF SOFT CLAY GROUND IMPROVED BY PRE-LOADING WITH PVD —CASE HISTORY IN BANGKOK—

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ABSTRACT

The installment of pre-fabricated vertical drain (PVD) combined with pre-loading is an efficient way to gain the prescribed strength and compressibility of very soft clay over a short period of time. This note describes case histories of such improvement performed in Bangkok. The change in undrained shear strength, s_u , before and after the ground improvement was investigated at two sites in Bangkok area by means of field vane test (FVT) and flat dilatometer test (DMT). The results of FVT showed that the s_u value of soft Bangkok clay increased over nearly the full depth down to the tip of the PVDs. It is also demonstrated that DMT may be a useful tool for promptly assessing the change of s_u in soft clay due to pre-loading. Applicability of the SHANSEP method in estimating the profile of s_u with depth in recently overconsolidated clay ground after pre-loading is also discussed.

Key words: case history, clay, dilatometer test, field vane test, ground improvement, shear strength (IGC: D5/D6/D7/C2/C6/B1)

INTRODUCTION

The vertical drain technique is often employed in order to accelerate the settlement of soft soils (e.g., Holtz, 1991). The vacuum pre-loading method (Kjellman, 1952) has also been widely used in Asian region for land reclamation and soil improvement work over the last two decades or so (Chen and Bao, 1983; Bergado et al., 1998; Shang et al., 1998; Chu and Yang, 2000 among others). Recently, pre-fabricated vertical drains (PVDs), instead of sand drains are frequently used together with (vacuum) pre-loading (or surcharge). In the Bangkok area, where the sandy soils are scarce, the vacuum pre-loading method with PVDs is particularly cost-effective in improving the engineering properties of soft Bangkok clay (Balasubramaniam et al., 1996; Bergado et al., 1996a).

In the ground improvement schemes for soft clay ground, the variation of undrained shear strength, s_u , with time is a primary concern for the short-term stability of the embankment. Quite a few case records have been reported in the literature, indicating that the s_u value of soft soils increases over almost the full depth down to the tip of PVDs (Hansbo et al., 1981; Bergado et al., 1996a; Tang and Shang, 2000; Chu and Yang, 2000 among others). The field vane test (FVT) is most often employed for investigating the change of s_u in soft clay after the treatment. The cone penetration test is occasionally em-

ployed, however the cone tip-resistance is not as reactive as the direct s_u measurement. On the other hand, the strength tests in the laboratory are not well suited for achieving this specific purpose, since it is time-consuming, cost-ineffective and discontinuous in data as compared to in-situ tests. More fundamentally, in common practice seen in most Asian countries, the s_u value from laboratory tests comprises the effects of ungovernable sample disturbance, which in turn mask any detectable change in s_u before and after the ground improvement (Tanaka et al., 1996; Shibuya et al., 2000a).

Accordingly, it is urgently necessary to establish an alternative procedure for promptly, reliably and continuously evaluating the profile of s_u with depth by means of in-situ testing other than FVT. It is internationally accepted that dilatometer test (DMT) can provide nearly continuous data in a prompt manner with remarkable repeatability (Jamiołkowski et al., 1985). In this paper, the feasibility of DMT in use for evaluating the change of s_u after the pre-loading was examined in case study performed in soft Bangkok clay. In addition, predicting s_u by SHANSEP method (Ladd and Foott, 1974) was also attempted by having the data on the effective stress history of soft Bangkok clay during the vacuum pre-loading.

SITES AND TESTS PERFORMED

As seen in Fig. 1, the case study was carried out at two

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sites, i.e., Nong Ngu Hao (NNH) and a site along Outer Bangkok Ring Road (OBRR), where the PVD ground improvement technique was applied in soft Bangkok clay.

The soil profiles at NNH and OBRR sites are very similar to each other (Shibuya et al., 2000a and b). As seen in Fig. 2, the subsoil at the NNH site, for example, is characterized by sub-layers consisting of upper 2 m-thick layer of weathered clay overlying so-called soft Bangkok clay

Fig. 1. Location of Nong Ngu Hao (NNH) and Outer Bangkok Ring Road (OBRR) sites in central Bangkok area

with a gray colour extending down to the depth of 15.5 m. The stiff clay lies below this, with a light-brown colour extending to about 22 m depth. A dense sand layer is situated below the stiff clay. As described in detail by Balasubramaniam et al. (1996) and Bergado et al. (1996a), the piezometric pressure distribution of the original subsoil was lower than the hydrostatic pressure below 6 m due to the excessive pumping in the sand layer underneath the stiff clay.

Figure 3 shows the profile of the embankment, together with the pattern of PVD installment at each site. In common with both the sites, a series of PVD was installed down to each designed depth; i.e., 12 m and 9 m down from the original ground surface at the NNH and OBRR sites, respectively. The horizontal drainage using a geotextile combined with a perforated pipe was placed on top of the PVDs. The embankment was afterwards constructed in stages in order to provide the surcharge. The full-scale test embankment at the NNH site had dimensions of 40×40 m at the base with a height of 2.5 m, including a sand mat 0.8 m thick at the base (see Fig. 3a). At the OBRR site, the PVDs were installed using a spacing of 1.2 m in a triangular pattern, and the embankment was 44 m wide and 3 m in height at the end of construction (see Fig. 3b).

At the NNH site, the pre-loading was provided not only by the embankment but also by applying a partial vacuum pressure (Bergado et al., 1998). Figure 4 shows the time-history of the pre-loading, together with the settlement-time curves at different depths in the subsoil right below the center of the test fill. It should be mentioned that a partial vacuum of not less than 70 kPa at the source was applied over the sand mat. Note that the settlements showed a common tendency to stabilize with time after about five months.

The distribution of pore pressures with depth at differ-

Fig. 2. Soil properties before and after improvement at NNH site

Fig. 3. Embankment, together with PVD installment: a) NNH site and b) OBRR site

Fig. 4. Settlement of subsoil due to pre-loading at NNH site

ent stages is shown in Fig. 5. The measurements were made based on the readings of five piezometers installed vertically in-between the PVDs (Bergado, 1998). As stated earlier, the piezometric pressure was lower than the hydrostatic pressure below 6 m depth due to the excessive pumping in the lower sand layer. When the PVDs were installed, the piezometric pore pressure shifted close to the hydrostatic line as a result of water recharge along the drains. On the application of a partial vacuum, the pore pressure decreased dramatically throughout the entire depths examined (refer also to Hanh and Shibuya, 1999). The upper bound of σ'_v shown in Fig. 2 refers to the maximum effective overburden pressure experienced during the vacuum pre-loading. As can be seen in Fig. 5, the maximum value of σ'_v was reached at around 45 days after the vacuum application, and the pore pressures increased gradually back to the atmospheric pressure over the entire PVD-treated zone. Note also that after the improvement, the total unit weight increased, involving the decrease of natural water content over the full depth (see Fig. 2).

At each site, two kinds of in-situ tests; i.e., field vane test (FVT) and flat dilatometer test (DMT), were performed before and after the ground treatment. The vane had the dimensions of 50 mm in diameter and 100 mm in height. It was rotated at an approximate rate of 0.1 degrees per second. In this note, the s_u value from FVT refers to the corrected value by applying correction factors proposed by Bjerrum (1973). The ratio of correct-

Fig. 5. Pore pressure change with time during vacuum pre-loading at NNH site

ed strength to the measured strength ranged between 0.67 and 0.8, depending on plasticity index (see Fig. 2).

The DMT was carried out by using the conventional

Marchetti's dilatometer with a blade 94 mm wide, 15 mm thick and 235 mm long. Interpretation of DMT is made using pressure readings at two instants; i.e., the contact pressure, P_0 and the pressure at 1 mm expansion, P_1 (Marchetti, 1980; Marchetti and Crapps, 1981). The material index, I_D , horizontal stress index, K_D , and dilatometer modulus, E_D , are given in the following expressions, respectively;

$$I_D = (P_1 - P_0) / (P_1 - u_0) \quad (1)$$

$$K_D = (P_0 - u_0) / \sigma'_v \quad (2)$$

$$E_D = 34.7(P_1 - P_0) \quad (3)$$

where the symbols of u_0 and σ'_v represent the current in-situ pore pressure and vertical effective overburden pressure, respectively.

When considering the dilatometer loading in elastic half space, Young's modulus, E' , of the assumed elastic soil mass may be given by

$$E' = E_D(1 - \nu'^2) \quad (4)$$

where ν' denotes Poisson's ratio. Accordingly, the undrained Young's modulus, E_u , may be estimated as $0.75 E_D$ with $\nu_u = 0.5$ in Eq. (4) (Marchetti and Crapps, 1981).

In the case study at NNH, a series of unconfined compression (UC) test and unconsolidated-undrained (UU) test was performed using a rate of axial straining of 1 percent per minute on the clay samples retrieved by the Japanese fixed-piston thin-wall sampler.

RESULTS AND DISCUSSION

Figures 6 and 7 show the s_u profile with depth before and after the ground improvement, noting that the comparison at the NNH site includes the results from laboratory shear tests. The results of the direct s_u measurements show considerable increases throughout the depths examined. It should be mentioned that the FVT after the improvement was carried out by completely removing the surcharge at both NNH and OBRR sites. Accordingly, the s_u values after the ground improvement can be regarded to mean the strengths of very recently overconsolidated clay ground.

The DMT results before and after the PVD improvement are compared in Figs. 8 and 9 for the NNH and OBRR sites, respectively. It should be reminded that the results after the improvement were obtained when the surcharge was completely removed. Accordingly, it is important to note that the properties change in each site after and before the improvement work can be practically attributed to the overconsolidation history during the improvement work.

In both the sites, the contact pressure P_0 increased noticeably, whereas P_1 did not. As a result of it, the K_D value (see Eq. (2)) increased after the ground improvement over the whole range down to the tip of the PVDs. It is a well established fact that the K_0 value of clay increases as the overconsolidation ratio, OCR, increases in

Fig. 6. Undrained shear strength profile at NNH site

Fig. 7. Undrained shear strength profile at OBRR site

value, and this has been confirmed in the behaviour of soft Bangkok clay (Shibuya and Tamrakar, 1999). Considering the OCR effect on the K_0 value, the horizontal effective stress after the improvement work must have been larger than before, which in turn was responsible for some increase of the contact pressure, P_0 .

The in-situ OCR and the coefficient of earth pressure at

Fig. 8. DMT test results before and after pre-loading at NNH site

Fig. 9. DMT test results before and after pre-loading at OBRR site

rest, K_0 , may be estimated by using the following empirical expressions (Lacasse and Lunne, 1988);

$$\text{OCR} = 0.34(K_D)^{1.56} \quad (5)$$

$$K_0 = (K_D / 1.5)^{0.47} - 0.6 \quad (6)$$

According to Marchetti (1980), the undrained strength of cohesive soils could be closely related to K_D . On the other hand, the effect of mechanical overconsolidation on the undrained strength ratio, s_u / σ'_{vc} , of overconsolidated (OC) clay may be expressed in the following form (Ladd et al., 1977);

$$(s_u / \sigma'_{vc})_{OC} = (s_u / \sigma'_{vc})_{NC} (\text{OCR})^m \quad (7)$$

where m is a power approximately equal to 0.8 (refer also to Mitachi and Kitago, 1976).

Mesri (1975) suggested that the average strength at failure underneath an embankment is equal to 0.22 times the pre-consolidation pressure. Combining Eqs. (5) and (7) with $m=0.8$ and $(s_u / \sigma'_{vc})_{NC}=0.22$, we obtain;

$$(s_u / \sigma'_{vc})_{OC} = 0.09(K_D)^{1.25} \quad (8)$$

The behaviour of OCR and K_0 both estimated by using Eqs. (5) and (6), respectively exhibited some increases down to the tip of PVDs. Conversely, the estimated E_u value showed considerable decrease after the ground improvement. Similarly, the s_u value estimated using Eq. (8)

Fig. 10. Ratios of s_u and K_D before and after pre-loading at NNH site

Fig. 11. Ratios of s_u and K_D before and after pre-loading at OBRR site

also showed some decrease below 7–8 m depth. These results suggest that the estimates of E_u and s_u from the DMT were not successful by considering the fact that the s_u value from the FVT, for example, exhibited steady increase after the treatment to the tip of the PVDs (see Figs. 6 and 7).

The ratio of s_u measured by FVT before and after the pre-loading is plotted against the original depth in Figs. 10 and 11, in which the ratio of K_D before and after the pre-loading is also shown for comparison. It should be pointed out that i) the s_u value from FVT exhibited some increase over nearly the full depth down to the tip of PVDs, ii) the gain in strength was noticeably larger in the near surface area, and iii) the pattern of change in the ratio of s_u from FVT was similar to that of K_D from DMT. Despite that the case is very limited, the results strongly

Fig. 12. Comparison between estimated and measured values of s_u

encourage the use of DMT as a tool for promptly investigating the undrained strength change before, during and after the pre-loading on soft clays.

In estimating the s_u value of the original ground before and after the surcharge, the SHANSEP method (Ladd and Foot, 1974) was employed. The calculation was made by using Eq. (7) with the parameters of $m=0.8$ and $(s_u/\sigma'_{vc})_{NC}=0.22$, the applicability of these parameters to Bangkok clay has been validated by Bergado et al. (1996b). As can be seen in Fig. 12, the estimated s_u value was in good agreement with the measured data except for the near surface region (see Figs. 6 and 7). The results from the case study strongly suggest that the SHANSEP method may be useful when estimating the profile of s_u with depth of soft clay ground subjected to recent over-consolidation stress history due to pre-loading.

CONCLUSIONS

Case histories in Bangkok described in this note have manifested some engineering aspects to be further investigated in conjunction with soft clay ground improvement by pre-loading with PVD.

- 1) The performance of PVD installment with surcharge seems satisfactory in that the consolidation time decreased substantially, involving steady increase of s_u over almost the full depth down to the tip of the PVDs.
- 2) The vacuum pre-loading with PVD in soft clay ground was also effective in that the pore water pressures in soft clay decreased immediately after the application of a partial vacuum over the full depth down to the tip of the PVDs. By applying this technique, the construction time of an embankment may be reduced considerably by approximately half.
- 3) The use of the flat dilatometer test (DMT) as well as the field vane test (FVT) was both successful for promptly estimating the change in s_u of soft Bangkok clay. The ratio of $(s_u \text{ after}/s_u \text{ before})$ from FVT was approximately

equal to (K_D after/ K_D before) from DMT.

4) The SHANSEP method may be useful in estimating the profile of s_u with depth in recently overconsolidated soft clay ground subjected to pre-loading. The application of SHANSEP method using $m=0.8$ and $(s_u/\sigma'_{vc})_{NC}=0.22$ was successful when estimating the corrected vane strength of Bangkok clay, bearing in mind that the change of pore pressures with time ought to be measured to define the in-situ OCR value in the case of improvement work with the vacuum pre-loading.

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