Soft Clay Properties and Their Influence in Preloading with PVD and Surcharge

E. Y. N. Oh, A. S. Balasubramaniam, M. Bolton, G. W. K. Chai, M. Braund
School of Engineering, Griffith University, Gold Coast, Queensland, Australia.
y.oh@griffith.edu.au, a.bala@griffith.edu.au, m.bolton@griffith.edu.au, g.chai@griffith.edu.au

V. Wijeyakulasuriya
Queensland Dept. of Main Roads, Herston, Queensland, Australia
vasantha.wijeyakulasuriya@mainroads.qld.gov.au

R. Nithiraj
Zaidun-Leeng Sdn Bhd, Tingkat 5, Bangunan Ming, Jalan Bukit Nenas, Kuala Lumpur 50250, Malaysia.
nithi@zaidun-leeng.com.my

D. T. Bergado
School of Civil Engineering, Asian Institute of Technology, Bangkok, Thailand.
bergado@ait.ac.th

Abstract: Ground improvement techniques are now widely used in soft clays of deltaic, marine and estuarine origin. The water contents of these clays range very widely some time even over 200 percent. Also, some clay has high sensitivity and high organic contents. In addition, geological conditions such as piezometric draw down also have an influence on the method adopted for ground improvement works. In this paper, the role of soil properties, geological conditions and other aspects such as the organic content will be discussed on selected case histories involving ground improvement with surcharge and PVD and other ground improvement schemes. In addition to laboratory methods of determining the soft clay properties, an observational approach in back-calculating soft clay properties from large scaled field tests which form part of the site investigation and construction control program will also be included.

1 INTRODUCTION

In this paper the authors summarize some of their experiences with the use of PVD and surcharge and other ground improvement techniques dependent on the geological conditions and the nature of the soil profile. The experiences are gained from the soft clay deposits in Bangkok, Malaysia at the Muar site and also estuarine clays in Southeast Queensland. In the Bangkok condition, a piezometric drawdown due to ground water pumping exist and as such at times, techniques with surcharge and the use of sand drains and sand wicks seem to generate hydraulic connections with the underlying aquifers which experience continuous piezometric drawdown. In the Muar site in Malaysia no such draw down was experienced yet the predicted settlements indicated the presence of undissipated pore pressures or well effect. An alternate explanation was also made by appealing to smear effects and lower efficiency of the drains. The Bangkok clay is a deltaic deposit while the Muar clay is marine deposit. Experiences with the Southeast Queensland clays indicate that the sensitivity is high and also large secondary settlement effects due to undrained and drained creep. It is thus important that better soil models be developed to account for the sensitivity of the clays as well as time dependent settlements over and above those due to the primary consolidation type. Some of the experiences from these three deposits are presented briefly in this paper. The subsoil profiles at the Muar flat site in Malaysia, at the second international airport site in Bangkok and in the Southern Queensland are presented in Fig. 1. The moisture content in all the three locations are of comparable values, while the estuarine clays in Queensland (Fig. 1(c) and 1(d)) seem to be more sensitive and also possibly exhibit higher secondary consolidation.

2 EMBANKMENTS WITH PRELOADING AND PVD

2.1 Trial Embankments at the Muar Site in Malaysia

Fourteen trial sections were built in one stretch to explore the potential ground improvement schemes which are most useful in road embankments works. All trial sections were fully instrumented to measure vertical settlements. Lateral movements and pore water pressures were measured using settlement markers, settlement gauges, pneumatic piezometers, and inclinometers.

The embankment with vertical drains (Scheme 6/9) was analysed by Ratnayake (1993). This analysis revealed for the single drain, the 90 percent consolidation time as 2500 days without the vertical drains and only 250 days with the drains. For the case with vertical drains, the consolidation settlement resulted from Barron’s theory for the cases of perfect drain, with smear and with smear and well resistance was found to be 1.200m, 1.199m and 1.199m respectively. From the analysis of the whole embankment after 400 days, the immediate and consolidation settlements at the ground surface and along the centre The corresponding maximum lateral deformations were observed as 0.21m and 0.39m respectively were computed as 0.15m and 1.08m while the lateral movement at the embankment toe was 0.08m at the end of the loading period and increased to 0.21 m after 400 days. The observed and predicted behaviour of this embankment for the settlement is presented in Fig. 2(a) and Fig. 2(b). The measured settlements were less than the predicted values when ideal drain conditions were assumed. Possibly the effect of smear and undissipated pore pressures similar to the well effect would have caused the actual settlement to be smaller than the predicted settlement based on ideal conditions.
Fig. 1(a) Geotechnical properties of Muar clay.

Fig. 1(b): Typical soil profile in the Bangkok plain.

Fig. 1(c) Geotechnical properties of clay in Gold Coast Highway (based on Wijeyakulasuriya et al (1999)).

Fig. 1(d) Geotechnical properties of clay in Sunshine motorway (based on Wijeyakulasuriya et al (1999)).
2.2 Trial Embankments in Bangkok

Prefabricated vertical drains were used for the first time on three embankments at the Bangkok second international airport site. The embankments labelled TS1, TS2 and TS3 were 40 m x 40 m in plan dimensions with side slopes 3:1 (as shown in Fig. 2(c)) which shows the elevation of typical section with instrumentation). Initially no berm was used, however, it was installed at a latter date. The PVD were selected from published information and economic considerations and the tests pertaining to the safety of the installation are the puncture resistance and the burst strength. For the post installation performance, discharge capacity tests were carried out not only in the straight conditions but also in twisted and deformed states. Based on these results three types of PVD were selected and installed at spacings of 1.5 m, 1.2 m and 1.0 m respectively in three embankments labelled TS3, TS2 and TS1. The PVD were installed in a square pattern up to a depth of 12 m. A sand blanket of 1.0 m height was laid on the excavated ground (-0.3 m MSL) prior to the installation of PVD. After the PVD installation, the sand blanket was increased to 1.5 m, then clayey sand was used to raise the embankment to 4.2 m in stages (i.e., to 75 kPa of surcharge). During construction, Stage 1 loading was up to 18 kPa, Stage 2 was taken to 45 kPa, followed by Stage 3 to 54 kPa and Stage 4 to 75 kPa (4.2 m fill height). The factor of safety without considering short-term conditions of machinery load was in the range higher than 1.35 and the lowest value was 1.26.

The tests pertaining to the safe installation of PVD are (i) grab tensile strength, (ii) trapezoidal shear strength, (iii) puncture resistance, and (iv) burst strength. The basic considerations for optimal performance of the drain is that a PVD must have the ability to permit the pore water from the soil to seep into the drain and should be able to transmit the collected pore water along the length of the drain. These requirements can be classified into two groups: the first pertains to the permeability k of the geotextile, the apparent opening size (AOS) and the criteria for filtration and prevention of clogging. The second group relates to the discharge capacity of the PVD both under straight and deformed conditions with additional factors of reduction due to time as well as filtration and clogging. The predictions of the settlements and pore pressures during loading and the consolidation stages are made by the finite element analysis using the CRISP program. These results are presented in Fig. 2(c) to Fig. 2(f). The strength increase after the primary consolidation is shown in Fig. 2(f) and indicated the positive influence in using the preloading technique with PVD.

3 SOUTHEAST QUEENSLAND EXPERIENCES

Wijeyakulasuriya et al. (1999) presented the behaviour of three trial embankments founded on sensitive soft clays along the eastern coast belt of Queensland. Case study A of the embankment at Mackay was built to 5 m height by stage loading. The construction period was 6 months and the period of preloading was 9 months. Case study B was at the Sunshine Motorway, to assess the feasibility of 2-staged construction within an overall construction period of 300 days. Case study C was at Coombabah Creek in Gold Coast.

Fig. 2(a) Cross section through the centre line of the embankment.

Fig. 2(b) Comparison of consolidation settlement at ground surface along the centre line of the embankment with vertical drains.
The embankment at the Sunshine coast had a top width of 17 m on nominally 1V:2H batters. The berms were 1m high with a width of 5m on one side and 8m on the other side. The trial embankment comprised three 20 m sections, with the end sections installed with wick drains (Section A, 1m spacing and Section C, 2m spacing) and the middle section on undisturbed virgin ground as a control section (Section B). The settlements of the three sections shown in Fig. 3 indicate that the wick drains has not accelerated settlements to any great extent. However the settlements in all sections were in excess of 1.5m. These measurements are now being further analysed.

The trial embankment at Coombabah traversed a swamp with soft clay up to 13 m. Here again the trial embankment consisted 3 sections. Section A had stone columns at 1m spacing and Section C had the stone column spacing as 3m. Section B was without any stone columns. The stone columns extended to 16 m. The height of the trial embankment was 2m with a top width of 12 m on 1V: 2H batters. The stone columns were installed by the jetting process as used in vibro-flotation. The stone columns too were not found to be effective in reducing the settlements (as shown in Fig. 4).

Fig. 2(c) Section view of the test embankment showing the position of instruments at the Second Bangkok International Airport (SBIA).

Fig. 2(d) Measured and computed settlement with different depth at the SBIA site in Bangkok.

Fig. 2(e) Measured and computed pore pressure dissipation of the embankment at the SBIA site in Bangkok.

Fig. 2(f) Improvement in field vane shear strength with consolidation settlement due to surcharge at the SBIA site in Bangkok.

Fig. 3(a) Variation in lateral displacement with settlement in Sunshine motorway trial embankment (based on Wijeyakulasuriya et al (1999))
4 ANALYSES OF SETTLEMENT BEHAVIOURS FOR PEAT

In general, it is very difficult to identify characteristics of soft ground during design stage since embankment is constructed on soft ground which has different strength and behaviour from section to section. As shown in Fig. 5 to Fig. 8, Oedometer tests were carried out on soil samples retrieved. The soil samples consist of peat and can be known as clayey peat. Peat has high moisture content and have high voids ratio from 3 to 11 (shown in Fig. 5). Fig. 6 shows that, peat has high compression index ($C_c$). Due to the high void ratio and compression index, peat has high $m_v$ values in normally consolidation state (as shown in Fig. 7). As shown in Fig. 8, peat has high coefficient of consolidation ($c_v$) of 16 m$^2$/year. These data are now being further analysed.
5 CONCLUSIONS

The authors have extensive experience in the use of PVD and surcharge as ground improvement technique for soft clays. The marine and deltaic deposits of soft clays are found to be highly amenable for improvement with PVD and surcharge. However, recently estuarine clays and clays with high organic content or peat are also being treated with similar technique for improving their properties. Estuarine clays were found to have higher sensitivity and also an increasing component of secondary consolidation. This is an interesting experience by the Queensland Department of Main Roads, where Mr. Wijeyakulasuria is the Principal Geotechnical Engineer. In this deposit, at times the coefficient of consolidation is also very low. Also, in organic clays and peat, the water contents are very high and great reduction in water content takes place under modest loading. The use of ground improvement techniques in such deposits is the current subject of research by the authors and Professor Robert Lo of University of New South Wales in association with Queensland Department of Main Roads.

ACKNOWLEDGEMENTS

The authors, and in particular the second author, are most grateful to Prof. B. Indraratne, and numerous MEng and DEng students who worked with the second author at AIT.

REFERENCES