The Influence of Piezometric Draw Down on Settlements in Test Embankments with Vertical Drains and Surcharge in the Bangkok Sub-Soils

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Summary: The Bangkok sub-soils which is a part of the larger Chao Praya Plain consists of a broad basin filled with sedimentary soil deposits which form alternate layers of sand, gravel and clay. The profile of the surface of the bedrock is still undetermined, but its level in the Bangkok area is known to be between 550 m to 2000m. The aquifer system beneath the city area is undoubtedly very complex and the deep well pumping from the aquifer over the last fifty years or so have caused substantial piezometric draw down in the upper clay layer, which is soft and highly compressible. In this paper the influence of the piezometric drawdown on the performance of vertical drains with surcharge in ground improvement works is examined in detail on two sites for construction of a Naval Dockyard and a new international airport. Also , the earlier work of Eide (1977)on a test section at a third site in the Bangkok-Siracha Highway is briefly included to complete the full presentation of the performance of test embankments with vertical drains in the Bangkok sub-soils. Test embankments with sophisticated instrumentation were built and the performance of sandwicks, large diameter sand drains and prefabricated vertical drains (PVD) were examined. A novel interpretation technique was adopted in evaluating the settlements from pore pressure dissipation and to compare with those measured directly. Also, the long term settlements and the rate of decay in settlements and lateral movements were also determined to illustrate the progress of primary consolidation and the onset of secondary consolidation. This is to illustrate that the primary cause for the settlement on a long term basis is due to secondary consolidation, rather than due to hydraulic connections or undrained creep.

INTRODUCTION

The city of Bangkok is situated about 40 km from the sea in the Chao Phraya Plain. The plain consists of a deep basin filled with sedimentary soil deposits which form alternate layers of clay, sand, gravel and clay. The profile of the bedrock subcrop is still undetermined, but its depth in the Bangkok area is known to be between 550 to 2000 m. The detailed study carried out at the Asian Institute of Technology by Prof. Prinya Nutalya and his team (AIT, 1981) indicate that the aquifer system beneath the city is very complex (see Figure 1) and the available data suggest the existence of eight aquifers separated by layers of stiff and hard clays. This study also revealed that there are over 1500 wells tapping water from the aquifers. At the New International Airport site, continuous monitoring of the piezometric draw down was carried out for a long time and these data are compiled by the AIT team of researchers during and prior to the 1995 study (see Figure 2a). Consequently, the layers of soils below 6m or so are experiencing increases in effective stress at times at the order of 300 kN/m^2, which in turn cause the subsoil layers to consolidate resulting in subsidence. Figure 2b shows the compressibility characteristics of the clay layers below the Plain at the AIT campus (Jiann, 1977). The upper 10 to 20 m of the sub-soil is highly compressible clay with low shear strength. Since the land is low-lying, most of the areas are prone to heavy flooding during the rainy season and most development projects are on filled ground to above the flood level. The surface subsidence and the high consolidation and yielding settlement of the soft clays means most roads and expressways are constructed on embankments. In the past the road embankments were constructed on untreated ground and as such needed re-filling and reshaping at intervals of ten to 15 years. Also, the numerous culverts and small bridges were supported on piles. Special embankment piles needed to be designed in order to provide a smooth transition from the piled areas to the non-piled areas. In the last ten years or so, ground improvement techniques and in particular surcharge fills with vertical drains were tried to increase the shear strength of the soft clays and to reduce long term settlement. Initially, large diameter sand drains were tried and then small diameter sandwicks. Finally prefabricated vertical drains (PVD) were found to be suitable and used in large scale projects, eg airport runways and highway constructions. Also, sand is difficult to obtain in the Bangkok Plain and the use of sandfill was found to be expensive as surcharge material. Therefore attempts were made to use a vacuum type of drainage system (Moh and Woo 1987) to minimize the volume of sand used as surcharge material. This paper illustrates some of the geotechnical experience related to the use of sand drains, sandwicks, PVD and also vacuum drainage. The effect of piezometric draw down and the subsidence in the interpretation of the measured field data for settlement analysis is highlighted.

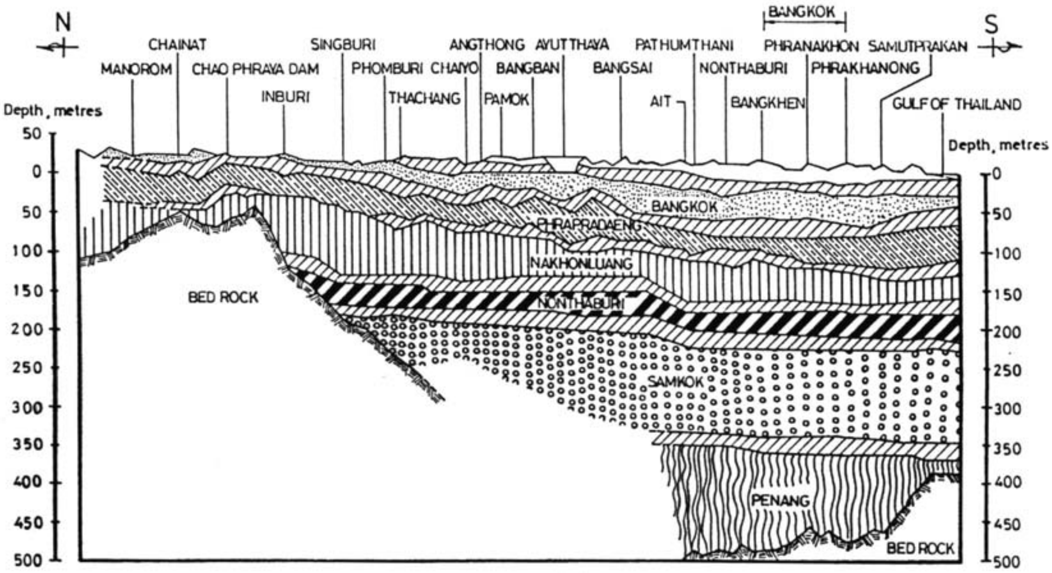

Figure 1 Hydrological profile of Bangkok aquifer system in the north-south direction (after Brand and Arbhabhirama, 1973)

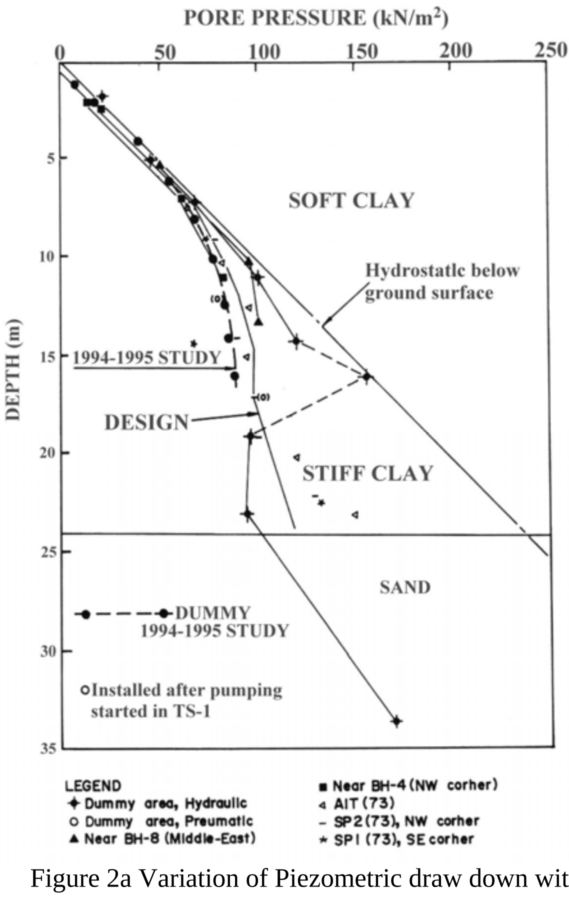

Figure 2a Variation of Piezometric draw down with depth

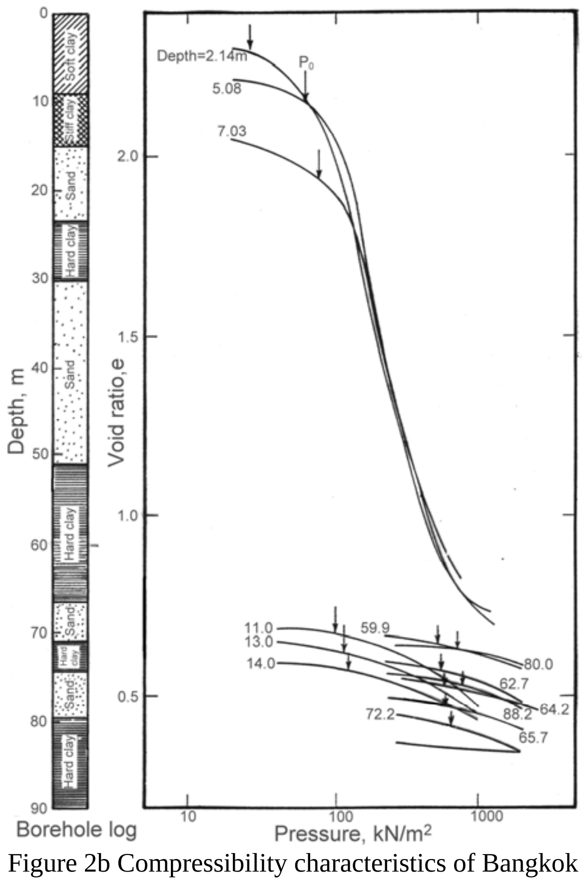

Figure 2b Compressibility characteristics of Bangkok clays (Jiann, 1977)

SAND DRAINS AND SANDWICKS

Eide (1977) reported the results of a test section on the Bangkok-Siracha Highway. The ground condition along the route was considered to be very soft. Sand drains of 0.2m diameter were installed by the displacement method. They were placed in triangular pattern at 2 m spacing. The observed settlement during two-year period after the construction is shown in Figure 3. Sand drains increased the rate of settlement but not to a sufficient degree. The most negative aspect quoted by Eide (1977) was that, even though the sand drain accelerated the consolidation in the first 18 months, yet even at the end of this period the rate of settlement was still as much as 0.03m per month, which was considered to be high. Possibly due to the low factor of safety, a substantial part of the total settlement was probably due continuous undrained creep without volumetric strain, when the stress states approach close to failure.

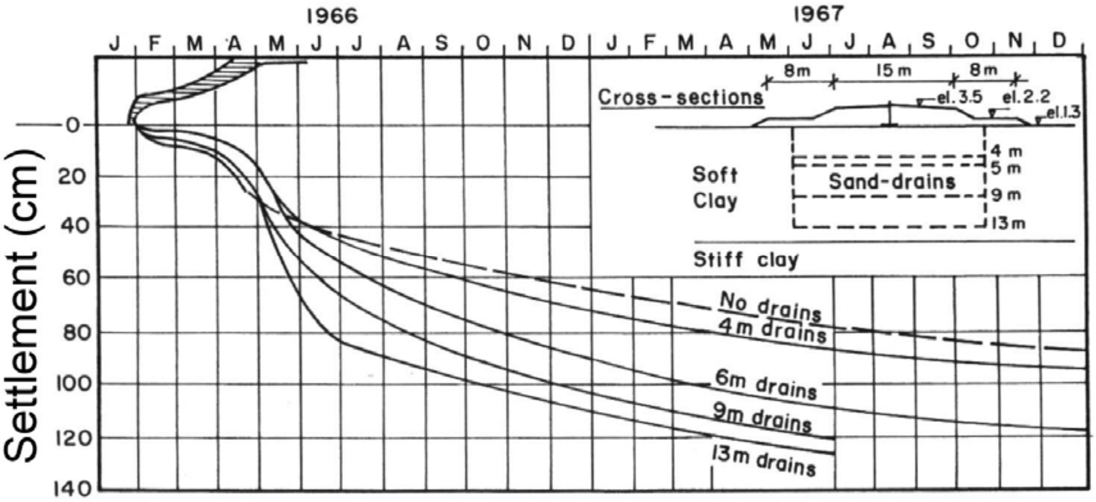

Figure 3 Observed settlements on test section with sand drains (after Eide, 1977)

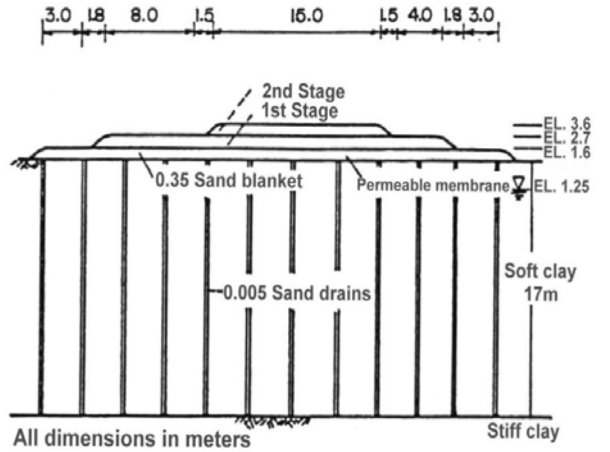

Figure 4 Test embankment with sandwicks

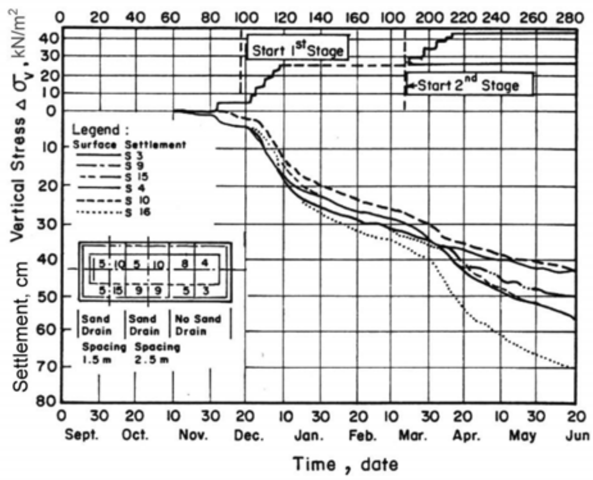

Figure 5 Settlement plots for test embankment with sandwicks

The performance of sandwicks in accelerating the consolidation of soft Bangkok clay was studied in a full-scale test embankment at Pom Prachul Dockyard and was reported by Balasubramaniam (1975) and Balasubramaniam *et al* (1980). The embankment is shown in Figure 4 and was built in three sections, namely: a section without drain, a section with drains of 2.5 m spacing, and a section with 1.5 m spacing. The sand drains consisted of small diameter (0.05m) sandwicks and were installed by the displacement method. The finished sandwicks extended to 17 m depth and the embankment was built in two stages, first to a height of 1.45 m and then raised to 2.35 m. The settlement records are shown in Figure 5 and it appears all three sections virtually showed the same trend and magnitude of settlement. This was the first time, and it was noticed that the sandwicks recharged the water table below the test embankment and to a distance of at least 20m so that below the test embankment the piezometers showed virtually no piezometric draw down. First the piezometers were suspected of faulty readings and more piezometers (nearly 200 of them) were installed. All of them showed that the sandwicks totally erased the piezometric draw down and that the water pressure below the embankment was hydro-static, while at a distance far away from the test embankment, a piezometric draw down existed.

At the airport site in Nong Ngu Hao, the most extensive sand drain studies on test embankments were performed in 1983 (Moh and Woo, 1987) as part of the ground improvement scheme for the runway pavement and other sections of the taxiways and landside roads. Sand drains of minimum diameter 0.26m were installed to a depth of 14.5 m by water jetting. The test program included three test areas, one with surcharge fill, the second with vacuum loading, and a third with ground water lowering. Test Section 1 was 40x40m in plan and sand drains were installed at 2m spacing in triangular pattern. The vacuum load was not successful as several leakages developed and finally the section was covered with plastic shield. Test Section 3 was similar to the Test Section 1 except that the spacing of the drain was increased to 2.4 m. Due to similar problem as in Section 1, the loading was not successful. The test Section 2 was slightly larger than test Section 1 and pre-loading of 60 kN/m^2 was applied in three stages. While difficulties were encountered in maintaining the vacuum load as well as the ground water lowering, the embankment surcharge was found to be a reliable technique when compared to vacuum loading in accelerating the consolidation with sand drains. The settlement observations are presented in Figure 6 and Figure 7 (Moh and Woo, 1987). The field trial was not successful in the sense that: (i) there was a settlement of 0.4m under a sand blanket of 0.7 m after a five-month period (see Figure 6 and Figure 7), and (ii) the settlement across the section was remarkably asymmetric. The observations indicate the possibility of hydraulic connections between the sand drains and the first sand layer located at 25m depths with a piezometric drawdown of 120 kN/m^2. It appears sandwicks (as used at the Naval Dockyard site) recharged the piezometric drawdown in the clay layer while the large diameter sand drains as used in the airport site in 1983 tend to form hydraulic connections with the underlying aquifer and caused additional settlements due to the piezometric drawdown.

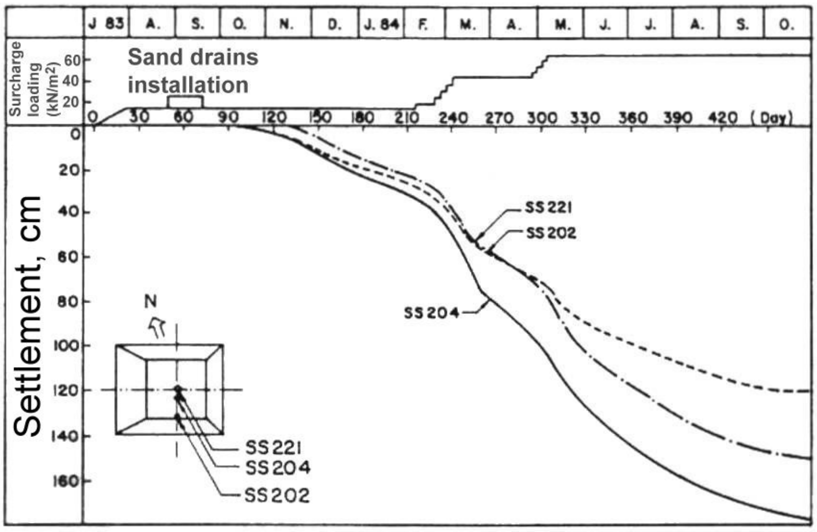

Figure 6 Observed settlements for test embankment with sand drains (Moh and Woo, 1987)

TEST EMBANKMENT WITH PVD

Following the 1983 study, the Asian Institute of Technology was involved in 1993 in building three test embankments at the same site with three types of PVD (Balasubramaniam and Bergado, 1995). The plan dimensions of the embankments were the same as the earlier study. The locations of the test embankments and the cross-section of embankment TS3 with PVD are shown in Figures 8a and 8b respectively. These embankments were fully instrumented to measure the surface and subsurface settlements and pore pressures, lateral movements and heave. PVD were installed to 12 m depth and the spacing was 1.5, 1.2 and 1.0 m in the three embankments TS1, TS2 and TS3 respectively. All three test embankments performed more or less in the same manner and as such detail discussion will only be based on one (Test embankment TS 3 with PVD spacing at 1m interval). For this embankment the settlement profile with depth and the pore pressure plots at various times are shown in Figure 9 and Figure 10. In Figure 9, the settlement profiles at end of construction (270 days), after 450 days (June 95) and after 660 days (Feb 96) are shown. Settlements were also independently computed from actual pore pressure dissipation. In Figure 10, the dotted curve ABC represents the actual piezometric profile with draw down as observed in September 1994 prior to the construction of the embankment. The full line curve DEF corresponds to the pore pressure profile after the full height of the embankment is reached with a surcharge of 75 kPa and prior to any pore pressure dissipation. The end of construction pore pressure profile is also shown. Similarly the pore pressure profiles in June 95 and in February 1996 are shown. The final pore pressure after the dissipation of the excess pore pressure and the recharged hydrostatic profile is MNPQ (NPQ is the assumed final recharged pore pressure profile, where there are no data points). Settlements were directly computed from these pore pressure dissipation curves.

The degrees of consolidation computed from the pore pressure dissipations are illustrated in Figure 11. Figure 12 compares the degree of consolidation as computed from settlement measurements to those estimated from the pore pressure dissipation. In Figure 12, the ordinate axis U_p refers to the degree of consolidation as computed from the pore pressure dissipation, while the abscissa axis refers to the degree of consolidation U_s as computed from the settlement measurements. With due allowance for a small percentage of secondary settlement and creep, the two degree of consolidations seem to agree well as they are close to the 45 degree line. Due to the limitation in the paper length, the data related to the lateral movements are not presented here. The settlement due to the lateral movements was less than 10% as estimated by the method of Loganathan *et al*. (1993). The immediate settlement computed from the lateral movements as adopted by a method in which the balancing of volume (Loganathan *et al*., 1993) was within ten percent of the measured vertical settlement. The rate of settlement and the rate of lateral movements are plotted in Figures 13a and 13b, and are found to decay with time. Also the settlement log time plots in Figure 14 for the three test embankments were found to be approaching a constant slope. An attempt was made to define the 100% primary consolidation time using

Casagrande type of settlement versus log-time plots. The points P and Q (shown in Figure 14) correspond to the 100% primary consolidation for the Test Section TS3 and TS1 respectively. The data for TS2 is not shown as it will crowd closely with the data from the other two embankments. The final portions of the test data for the two embankments seem to approach the secondary consolidation part as computed from the Casangrande settlement versus log-time plots. These results further confirmed that the PVD did not cause any hydraulic connection with the lower aquifers and the measured final linear settlement is of the same order as the secondary settlement.

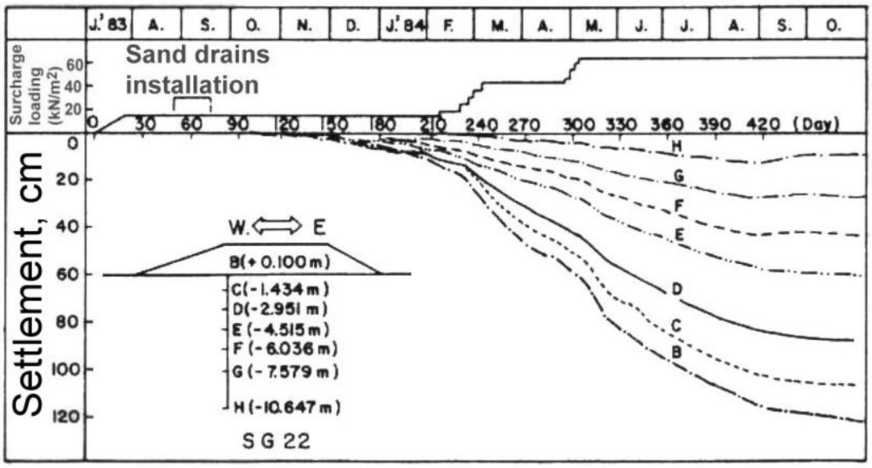

Figure 7 Vertical settlements of soil layers for test embankment with sand drains (Moh and Woo, 1987)

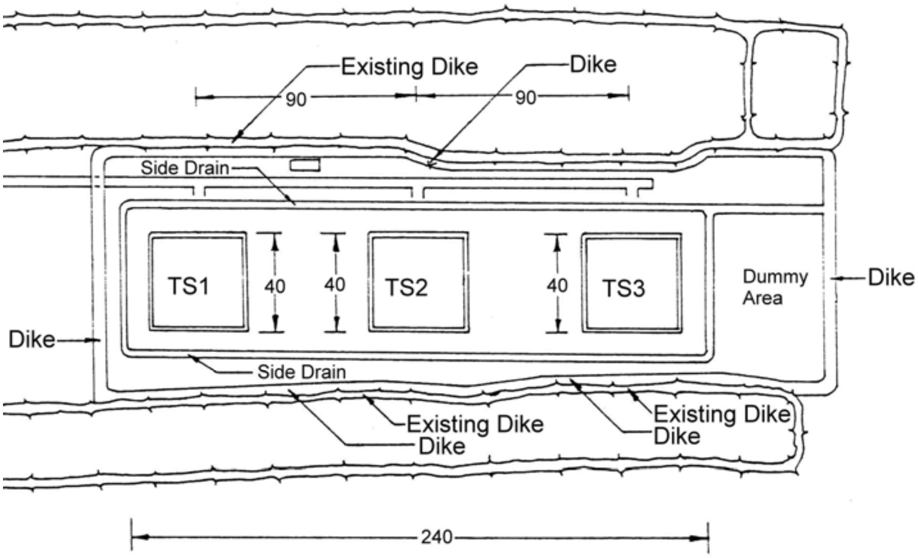

Figure 8a Site plan of test embankments TS1, TS2 and TS3.

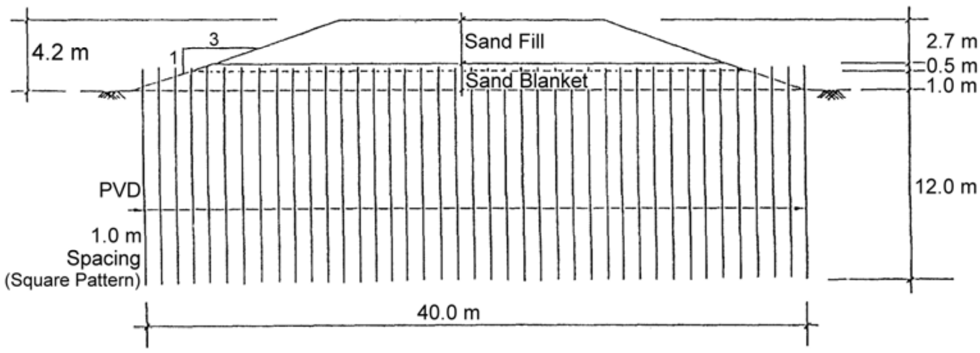

Figure 8b Test section TS3 showing PVD

CONCLUSIONS

The aquifer system below the Bangkok Plain (AIT, 1981) is undoubtedly very complex (eight interconnected and leaky aquifers are identified ranging in depths from 20m to 530 m) and the thickness of sedimentary deposit vary from 550 m to 2000m. The extensive deep well pumping has caused very large piezometric draw down and there is continuous surface subsidence. Even though pre-loading with vertical drains was found to be a suitable ground improvement technique to increase the strength and reduce the long term settlement of the upper soft clay, previous studies by Eide (1977) and Balasubramaniam *et al* (1980) did not give positive results. This paper illustrates the difficulties encountered in the past with hydraulic recharge (Case study at the Dockyard site) and hydraulic connections when large diameter sand drains were tried in the preloading technique (Moh and Woo, 1987). Also, the vacuum drainage was not found to be successful (Moh and Woo, 1987) as it was difficult to maintain the vacuum over long period of time. Finally, suitably spaced pre fabricated drains (PVD) were found to be successful. Settlement computations were made both from direct measurements of settlements as well as from pore pressure dissipation as monitored by a large number of piezometers. These values of settlements were found to agree, confirming that the settlements computed by both methods are the same. The lateral movements (though not discussed here) were found to contribute only less than ten percent of the total vertical settlement. The rate of settlement decay and lateral movement decay were also analysed to confirm that the primary consolidation phase has indeed completed during the preloading period.

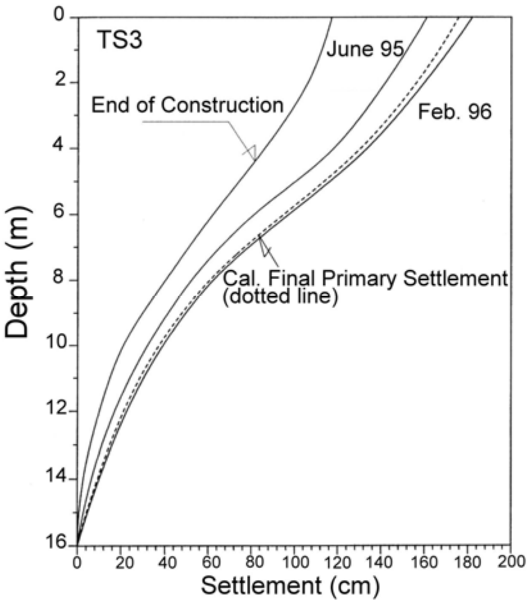

Figure 9 Settlement plot of test embankment with PVD

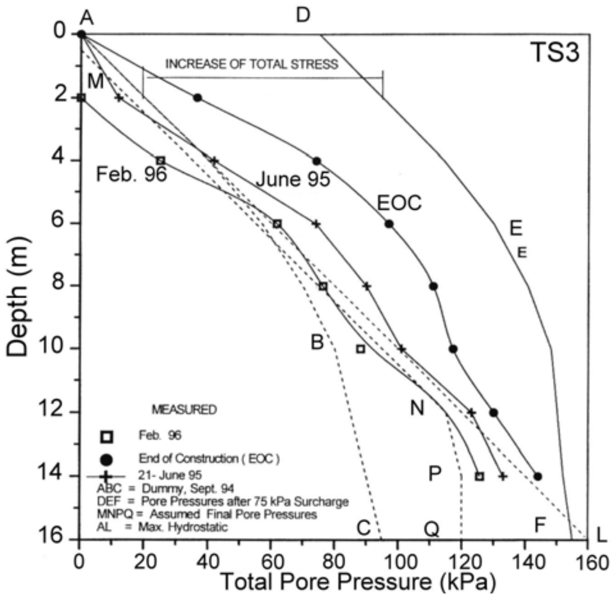

Figure 10 Pore pressure profile of test embankment with PVD

ACKNOWLEDGEMENTS

The work presented in this paper relates to the research work conducted by the first author over a period of 27 years at the Asian Institute of Technology as sponsored research projects and graduate thesis research work. The close collaboration that the senior author enjoyed with having several discussions with the late Ove Eide of the Norwegian Geotechnical Institute, the former colleagues Dr. Za-Chieh Moh, Dr. E. W. Brand, and Prof. Prinya Nutalya are gratefully acknowledged. The Royal Thai Navy and the Airport Authority of Thailand are thanked for engaging the first author and his team in carrying out the elaborate full scaled field tests and the interpretation of the data presented in this paper.

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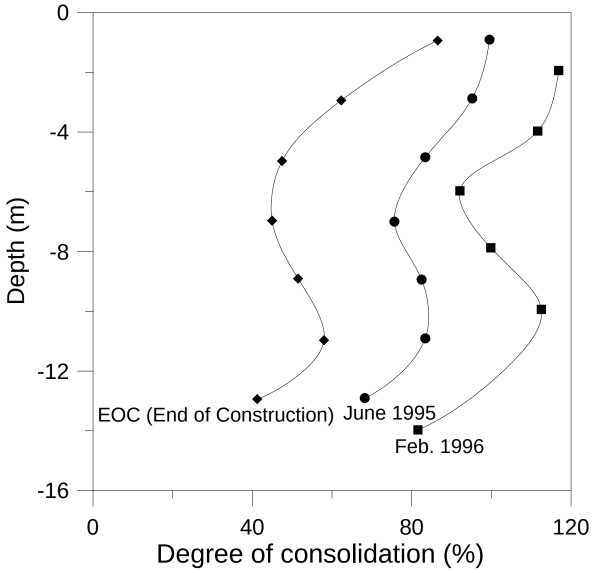

Figure 11 Degree of consolidation from measured pore pressure

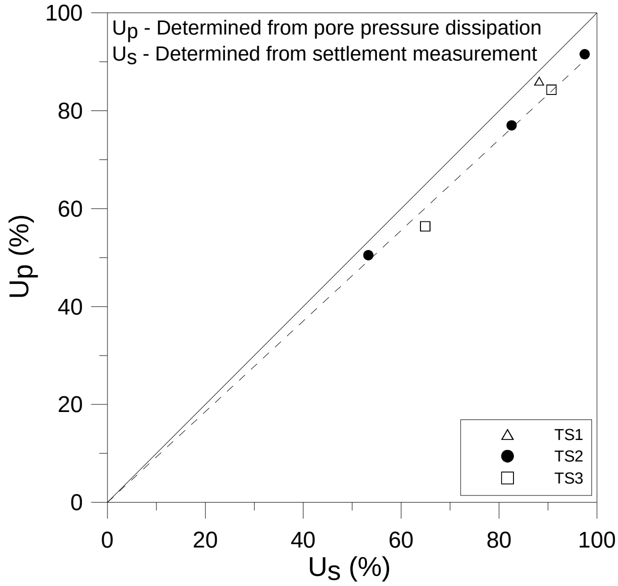

Figure 12 Degree of consolidation computed from pore pressure dissipation and settlement measurements.

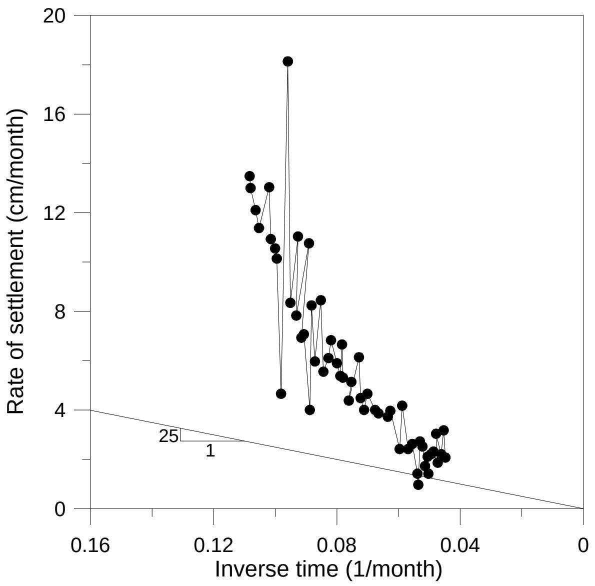

Figure 13a Rate of settlement versus inverse time plot

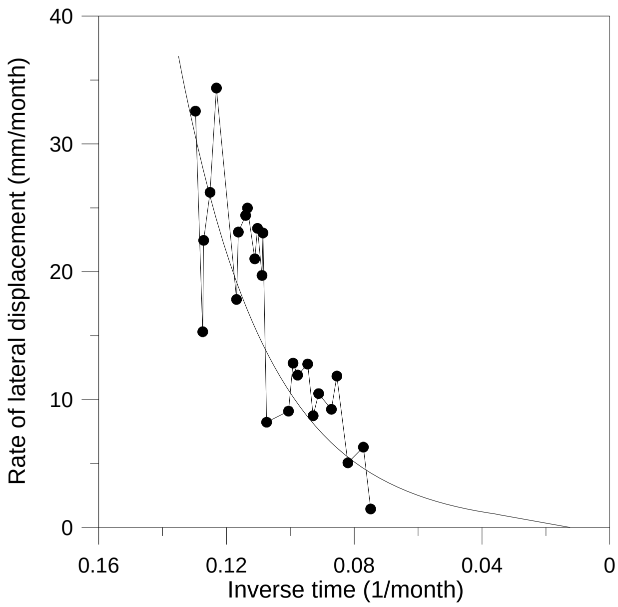

Figure 13b Rate of lateral movement versus inverse time plot

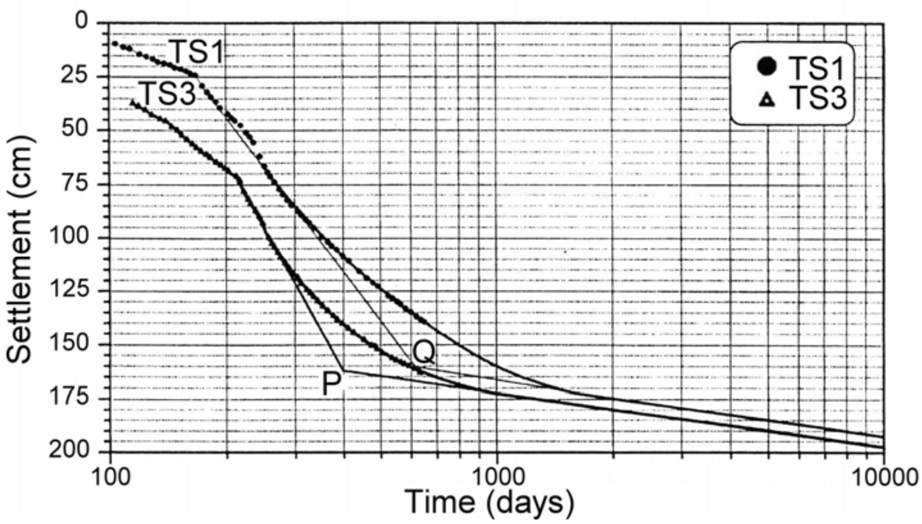

Figure 14 Settlements versus log-time plot for embankments TS1 and TS3.