PRECONSOLIDATION OF SOFT BANGKOK CLAY BY NON-DISPLACEMENT SAND DRAINS AND SURCHARGE

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SUMMARY The major geotechnical problem for the proposed development of an international airport for Bangkok at Nong Ngu Hao, a possible site selected by the Department of Aviation of the Government of Thailand, is the low strength and high compressibility of the subsoils underlying the site. Study of the feasibility of soil improvement scheme was one of the major item of work for the Master Plan Study, Design and Construction Phasing of the proposed airport.

In addition to theoretical study and analyses, three test sections were designed, constructed and monitored to evaluate the effectiveness of soil improvement. Non-displacement sand drains of 26 cm diameter at different spacings were installed under the test sections to approximately 14.5m in depth, i.e. to the underlying first stiff clay stratum. Surcharge loading by fill was used at one of the test sections. Vacuum preloading and groundwater lowering by pumping were used at the other two sections. The test sections were well instrumented to monitor the vertical settlements, lateral movements, changes in porewater pressures in the subsoils under the test area. This paper reports the results obtained by using surcharge fill load with sand drains for soil improvement. Comparisons of the actual soil behavior are made with those from theoretical predictions as well as those obtained from test sections studies without sand drains. It was found that the rate of settlement of the Bangkok clay at Nong Ngu Hao can be effectively accelerated by the use of non-displacement type sand drains.

INTRODUCTION

In 1983, a field test program was carried out at the proposed Nong Hgu Hao site for the Second Bangkok International Airport (SBIA) in Thailand. The purpose of the trial tests was to determine the effectiveness of using non-displacement type sand drains for accelerating consolidation settlement of the soft clay underlying the site. The test program includes three test areas, one with embankment surcharge fill, one using vacuum loading and the third one using groundwater lowering technique by pumping. The three test areas were well instrumented to monitor the vertical settlements, lateral movements and pore pressure changes.

This paper describes the construction of the test embankment and presents the major monitoring results. The field performance of the trial embankment is appraised. In addition, the results are compared with the performance of an embankment without sand drains. The use of non-displacement type sand drains with surcharge fill is proved to be an effective way to accelerate the settlement rate in the Bangkok Clay.

In the past 15 years, several types of sand drain have been tried in Bangkok. For example, displacement-type sand drain at the Tha Chang Bridge site (CIRIDON, 1973), small diameter sand-wick drain at the Naval Dockyard site (AIT, 1977), and sand drains at the Asian Institute of Technology campus (RAHMAN, 1980). However, the non-displacement sand drain used in the present study is the first trial ever carried out in Thailand.

Geotechnical Conditions of Site

The test site is located at the Nong Ngu Hao area which is 25km to the east of the Bangkok Metropolis. It is a piece of flat, marine deltaic deposited plain having an average ground elevation of about 1.4m above the MSL. The typical soil profile and soil properties obtained from the site are shown in Fig. 1. Five different soil strata can be identified within the top 35m zone. They are:

(a) Weathered Clay (0-1.5m)

(b) Very Soft to Soft Clay (1.5-11m)

(c) Soft to Medium Clay (11-15m)

(d) Stiff Clay (15-25m)

(e) Dense Sand (below 25m)

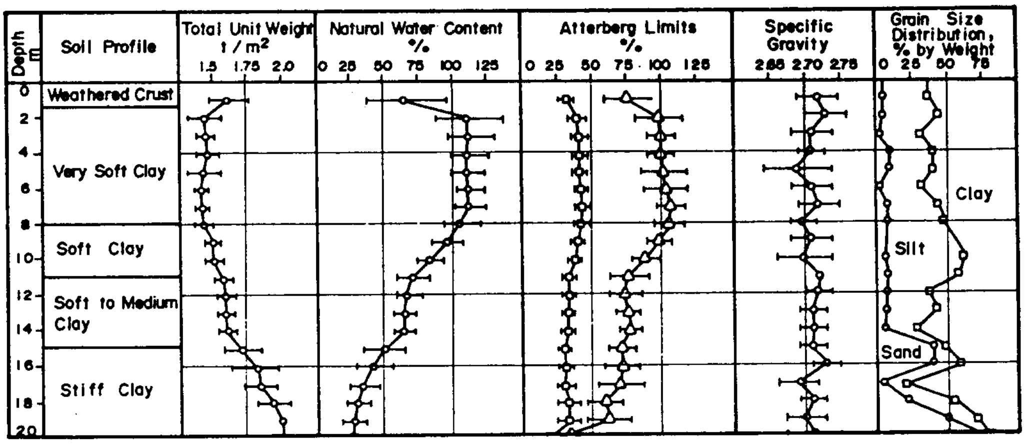

Fig. 1 Typical Soil Profile and Soil Properties

It is worthy to note that the very soft clay existing at depth of 1.5m to 11m below the ground surface has very high natural water content of about 112% and high plasticity. The changes of physical properties with depth are associated with the increasing silt and decreasing clay fractions. The undrained shear strength, coefficient of permeability and consolidation parameters for the subsoil layers are shown in Fig. 2. These data were used to calculate the settlement rate and slope stability of the trial embankment.

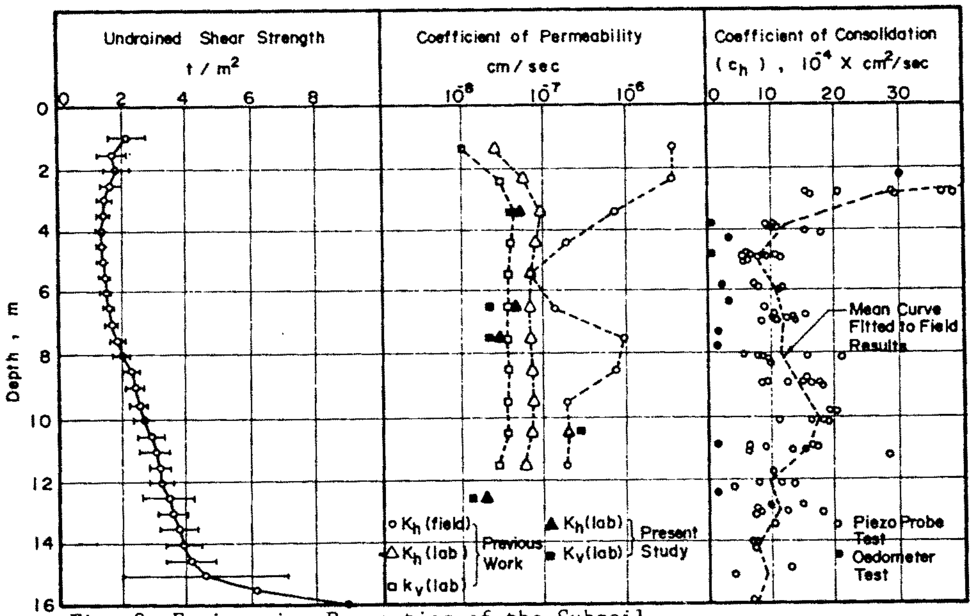

Fig. 2 Engineering Properties of the Subsoil

Construction of Test Area

The plan area of the Test Section was 40m by 42.6m. Sand drains of 26cm in diameter were installed to a depth of about 14.5m in triangular pattern at 2.0m spacing. A sand blanket, 0.8m thick and made up of well-graded sand was placed on the ground surface to serve as a working platform and for lateral drainage.

Water jetting method was used to install the sand drains. The installation machine consists of a crawler crane, a high water pressure pump and a 9m long steel bailer with a 26cm diameter cutting shoe attached to the bottom of the bailer. During the formation of a hole, a steel cylinder guide was placed at the prescribed sand drain location which was set out in advance. The bailer was vertically hung over the guide by the crawler crane. When the bailer just touched the ground, high pressure water (up to 100 ton/m^2) was discharged through its nozzle. The bailer was then allowed to sink and form a cylindrical hole in the clay by its own weight. By raising and lowering the bailer gently, the clay cuttings were brought out from the hole by the discharge water until the hole had reached its depth. Water was allowed to flush out from the bottom of the hole until the discharge water became only slightly muddy with a specific gravity of not greater than 1.032. This specific gravity of the discharged muddy water was adopted because the clay content in the sand of the drain would then not be greater than 5% even when the sand is mixed with the muddy water. After the hole was formed, the bailer was then slowly pulled out from the hole and a steel wire ball of 26cm diameter was lowered into the hole to check the exact bottom elevation and to ensure that the hole has not collapsed after the bailer was withdrawn. A trial test on the stability of a sand drain hole proved that the hole could stand open for eight hours without collapsing. The hole was then filled up with sand to the top of the sand blanket. The quantity of sand poured into each sand drain hole was measured by the number of shovels.

The clayey fill embankment was built in two stages. The first stage was up to a height of 2.85m with material having unit weight of 1.48 ton/m^3. The final embankment height was 4.2m corresponding to a vertical pressure of 6.3 ton/m^2. All the soil was placed in layers, each about 40cm thick. Three sides of the embankment had a slope gradient of 1 (V) to 1.5 (H) and the fourth side 1 to 3. This was chosen for evaluating the effect of slope gradient on stability and lateral movement of the subsoil.

Instrumentation

The test section was well instrumented and regularly monitored. The instruments installed within the test section included piezometers, settlement plates, inclinometers, sondex settlement gauges and hydrostatic profile gauges. Some instruments were also placed in the dummy area to monitor the natural variation of the subsurface conditons with time during the study period.

Figure 3 shows the instrumentation plan for the test section.

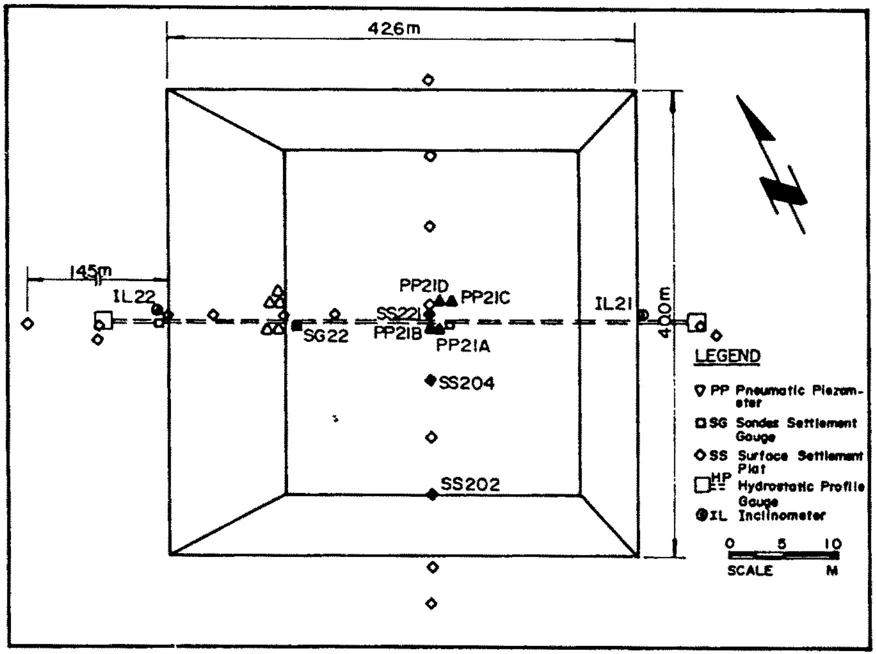

Fig. 3 Instrumentation Plan

RESULTS OF STUDY

After completion of installation of the sand drains and the field instruments in October 1983, there was a heavy flood in the entire Bangkok area. Because of the access road being flooded, the preloading operation with surcharge fill was delayed till early February 1984. However, the test area was well protected from flood water by a bund and the monitoring of instruments was continued without interruption. Most of the instruments were monitored for half a year after full loading, and some instruments were monitored longer.

The instrumentation results of the test section can be summarized mainly into three categories, i.e. the vertical settlements, the lateral movements and the pore water pressure changes in the soil. Figure 4 shows the settlement at ground surface versus time at three measurement points. It was found that settlement occurred rapidly under the sand blanket and surcharge fill. After the final stage of loading, the ground continued to settle for 60-80 cm in six months' time. The total settlement under the test section was not uniform. This is shown in the cross-sectional settlement profile, as measured by a hydrostatic casing, in Fig. 5. It is seen that maximum settlement did not occur at the center of the test section. This is due to the influence of groundwater underpressure in the deeper layer as will be discussed later in this paper.

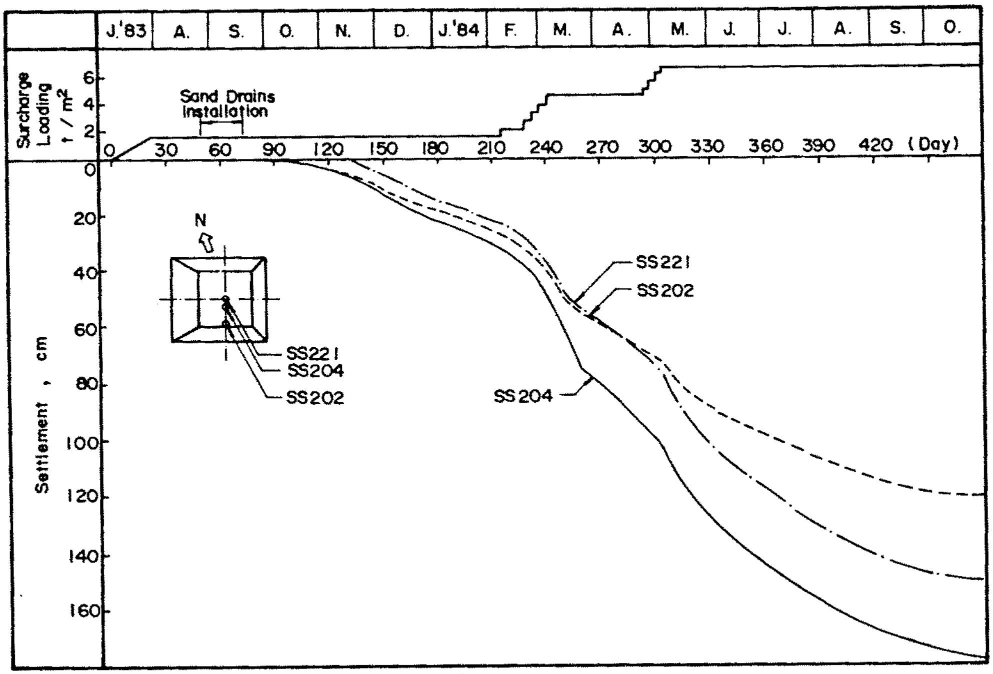

Fig. 4 Observed Settlement Results

At a few locations, vertical settlements of the soil layer at various depths were monitored by Sondex tubes. The data as shown in Fig. 6 was measured at point SG22 in Fig. 3. Settlement for an individual subsoil layer is readily available by subtracting the settlement at a point below the said layer from the settlement measured at a point above it.

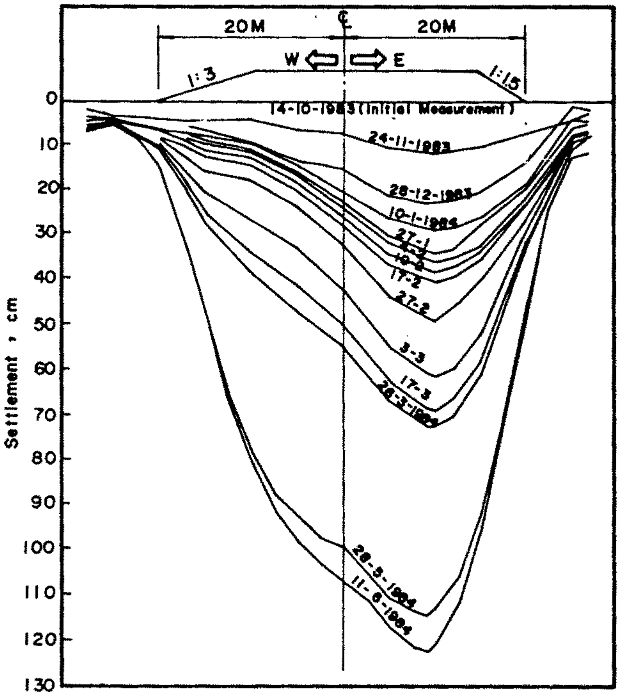

Fig. 5 Settlement Profile

Under the surcharge fill load, lateral displacements of the soil were monitored by inclinometer casings. The maximum lateral displacement observed was 10-12 cm, at the top of the very soft clay layer just below the weathered crust. The lateral displacement profile showing the changes of displacement with time is given in Fig. 7. The lateral displacement of soil is believed to have significantly contributed to the immediate settlement measured at ground surface.

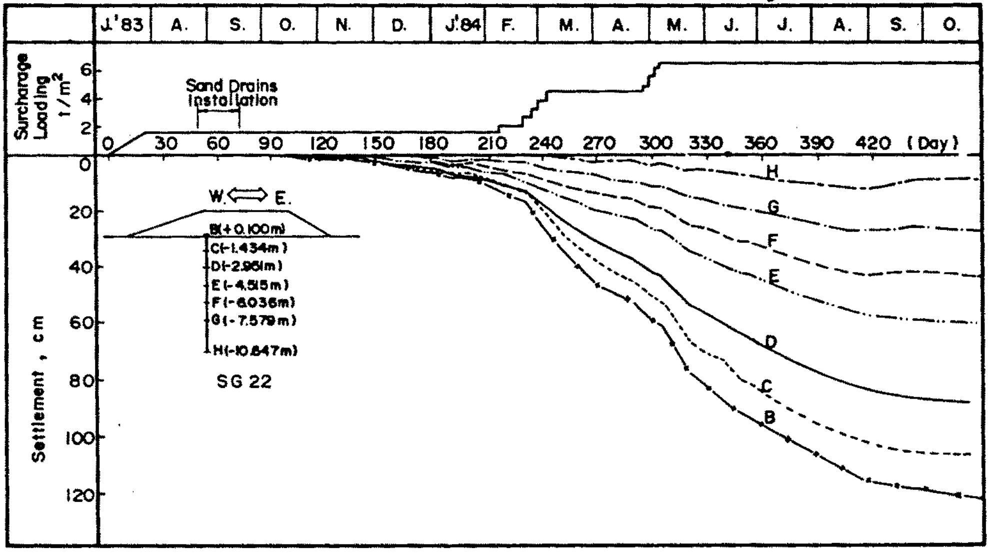

Fig. 6 Vertical Settlement of Soil Layers

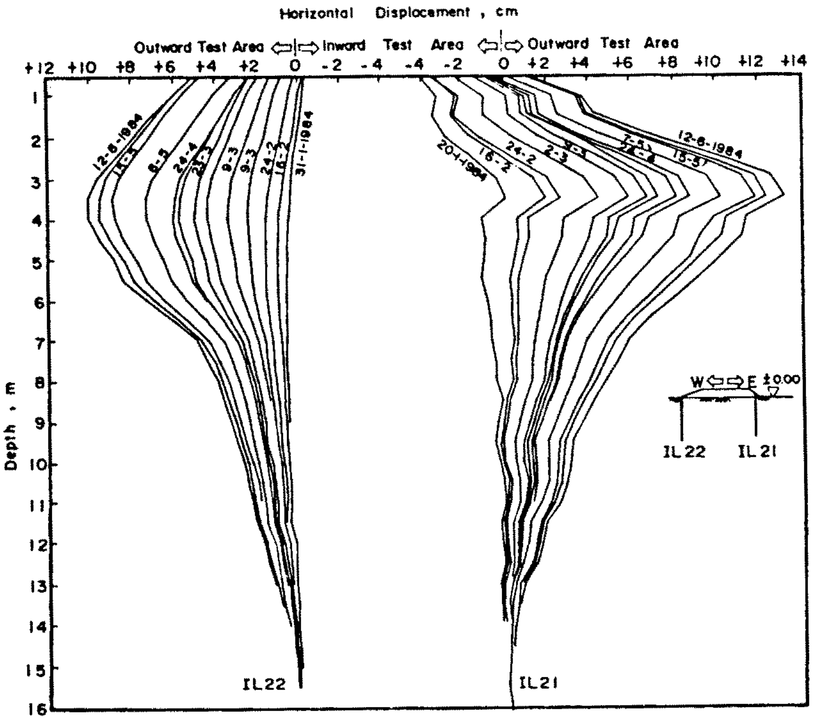

Fig. 7 Results of Lateral Displacement

The pore water pressure of the subsoil under the test section was measured by means of pneumatic piezometers installed at several depths and locations. Figure 8 presents the measured pore water pressures at one location at four depths. At each stage of loading, all the piezometers responded very consistently. The final pore water pressure at PP21D which was 17m below ground surface shows a lower value than piezometer PP21C at 11m below ground surface indicating the non hydrostatic water pressure distribution in this soil layer. Further discussion will be made on this topic in the following sections.

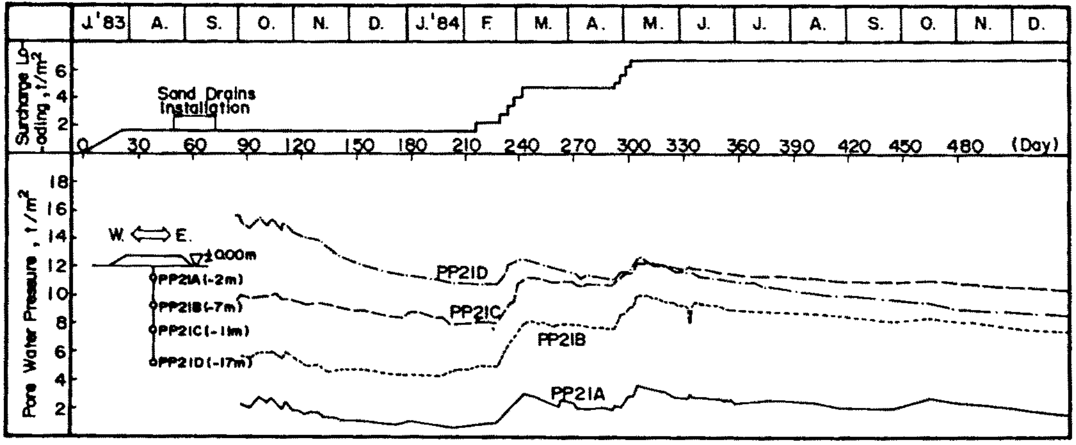

Fig. 8 Measured Pore Water Pressures at Various Depths

Evaluation of Soil Properties After Preloading

To determine the changes in the engineering properties of the soil after preloading, in-situ cone penetration tests and undisturbed sampling were carried out. Figure 9 presents the comparison of water content in the soil before and after preloading. Results of comparison show the soil water content decreased significantly due to improvement. The most obvious decrease of water content is in the very soft layer in the top 6m. The decrease of water content is in the range of 30-40%.

Figure 10 shows the cone resistance increase after soil preloading. The cone resistance of soil after improvement was obtained in field with full surcharge. This surcharge would have caused some additional increase in the cone resistance.

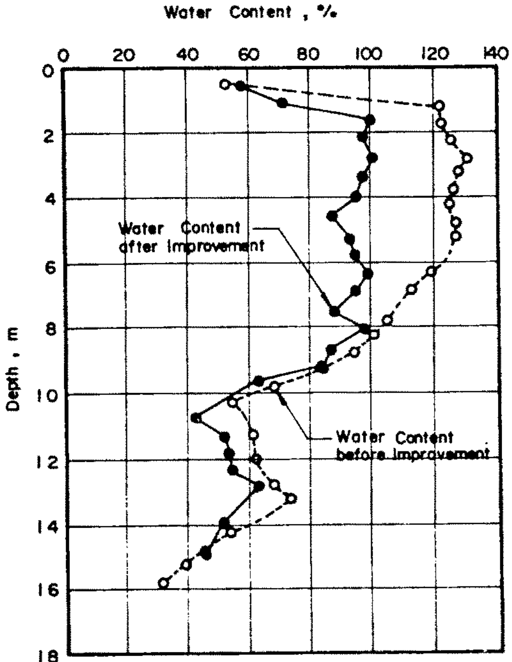

Fig. 9 Water Content of Soil before and after Preloading

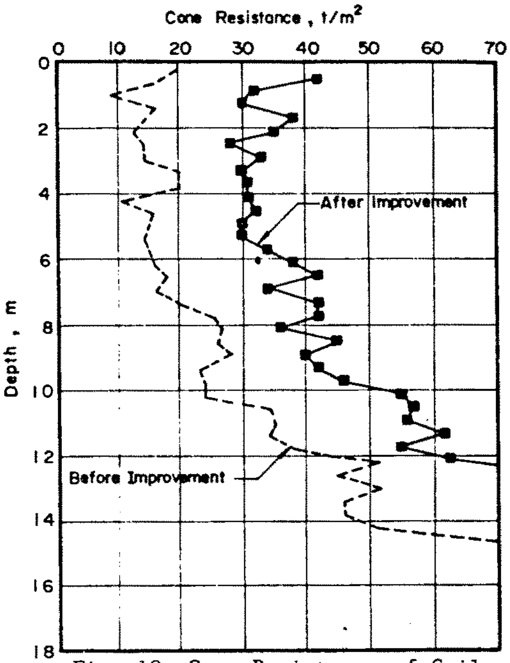

Fig. 10 Cone Resistance of Soil before and after Preloading

DISCUSSION OF RESULTS

Effect of Underpressure

The groundwater pressure in the Bangkok area is known to be non-hydrostatic. This is caused by extraction of groundwater from numerous deep pumping wells. In the trial test area, the field data observed were greatly affected by the existence of the underpressure in the subsoil. In addition to the above, the influence was enhanced by the pumping activities with two adjacent test areas where vacuum pumping and groundwater lowering were applied in the period of September 1983 to April 1984. Though the two test areas were about 100m away from the trial preloading area, the effect of pumping was verified by the water pressure decrease and ground settlement monitored at a 'dummy area' which was at similar distance away from the pumping areas.

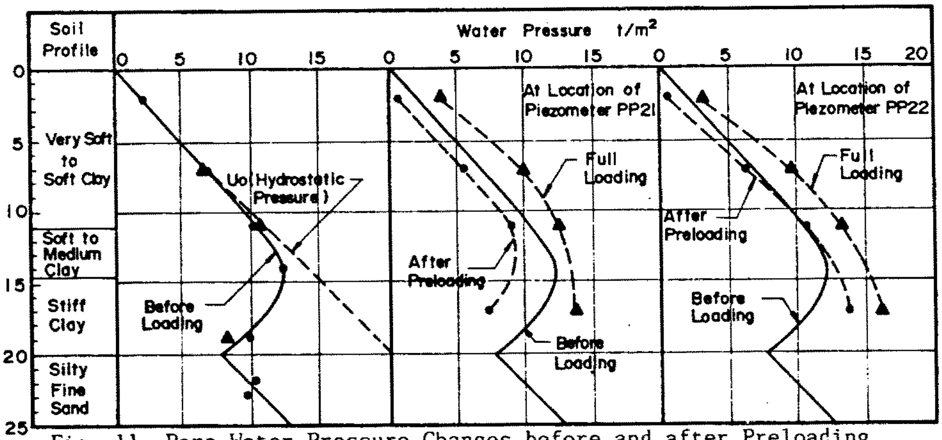

Fig. 11 Pore Water Pressure Changes before and after Preloading

The groundwater pressure distribution at the site prior to the installation of sand drains, as shown in Fig. 11 indicates that the free groundwater table was very close to the existing ground surface, and significant underpressure as much as 12.5m drop below the static head was measured in the stiff clay at depth of 15m and the underlying sand layer.

The length of sand drains in the trial area was designed to be 14.5m which means the tip of drain should be just above the stiff clay layer. However, a few drains near the center portion of the test section were found to be slightly longer, and had actually reached the stiff clay layer. These few drains must have acted as drainage paths linking the stiff clay (or even the sand layer) to the overlying soft clay. The underpressure at the tip of the drains must have drawn water from the drains and some water from the soft clay layer to discharge into the stiff layer. As a result, the soft clay consolidated and surface settlement occurred even there was no surcharge at ground surface.

The above points are reflected by the monitoring results:

(1) Maximum settlement occurred at 4m away from the center (Fig. 5)

(2) The ground surface settled continuously up to 40cm under the 80cm thick sand blanket during the period of September 1983 to January 1984 (Fig. 4)

(3) Comparing the piezometric readings at -17m at two locations, one at the center of the test area and the other near to the side slope, the former always gave lower piezometric pressure. In fact, it even dropped to below its initial

readings which were recorded prior to the sand drain installation. (Fig. 11)

Analysis of Settlement

The observed settlement as shown in Fig. 4 consists of elastic deformation, consolidation settlement (mainly primary consolidation) and settlement induced by underpressure of groundwater at the site. The combination of these factors had made the ground settlement behavior very complicated. To simplify the analysis, measurement data from an observation point (SG22) which was less affected by underpressure were selected for comparison with computed results.

In the analysis, the subsoil layers were further divided into several sublayers to account for the different properties and compressibilities in particular the presence of sand seams. Stress influence factor based on Gray's formula was adopted (GRAY, 1936). To overcome the effect of lateral deformation, a correction factor of 0.75 as proposed by SKEMPTON & BJERRUM (1957) was applied to the ultimate consolidation settlement calculation. At any time after installation of sand drains, a combined radial and vertical flow was assumed and the rate of consolidation settlement was calculated by using the sand drain theory suggested by BARRON (1948). The surcharge loadings of each stage were 1.6 t/m2 (sand blanket), 4.6 t/m2 and 6.3 t/m2 respectively.

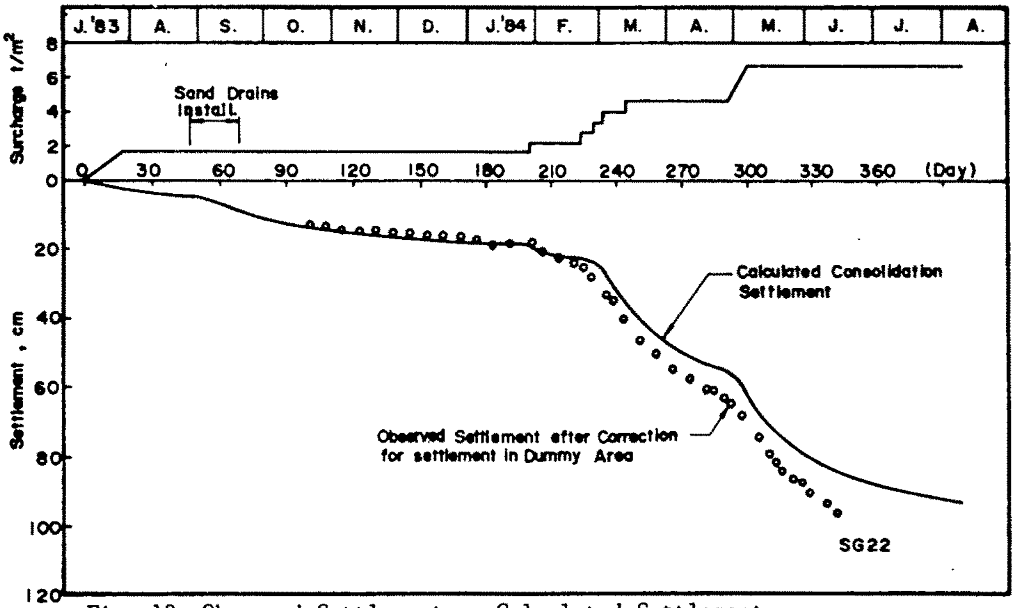

Fig. 12 Observed Settlement vs Calculated Settlement

The calculated consolidation settlement-time curve is presented in Fig. 12. The figure presents the ground surface settlement at location of SG22 before and after sand drain installation and under the various stages of surcharge loading. Actual field settlement monitoring was only started on 5 October 1984, therefore adjustment had to be made for the subsequently observed settlement to match the same computed value on this date. Further correction of observed settlement was made with reference to the observed settlement in the dummy area as induced by pumping tests in the adjacent areas. The corrected observation data plotted with the computed results in Fig. 12 indicate that the results match well at the initial stage, but they deviate more as the preloading increases. The larger observed settlement is believed due to the undrained vertical deformation of the soft clay under the surcharge loading. To confirm this point, the lateral deformations as measured by inclinometers (Fig. 7) were analyzed to find the magnitude of vertical deformation. It is assumed that the total volume of soil displaced laterally would cause a uniform vertical settlement for the whole test area. The undrained vertical deformation so estimated is seen to be identical with the deviation of the two curves in Fig. 12. This indicates that undrained deformation had made partial contribution to the ground settlement at the test area. The computation method adopted can be used to analyze the time-settlement behavior reasonably well.

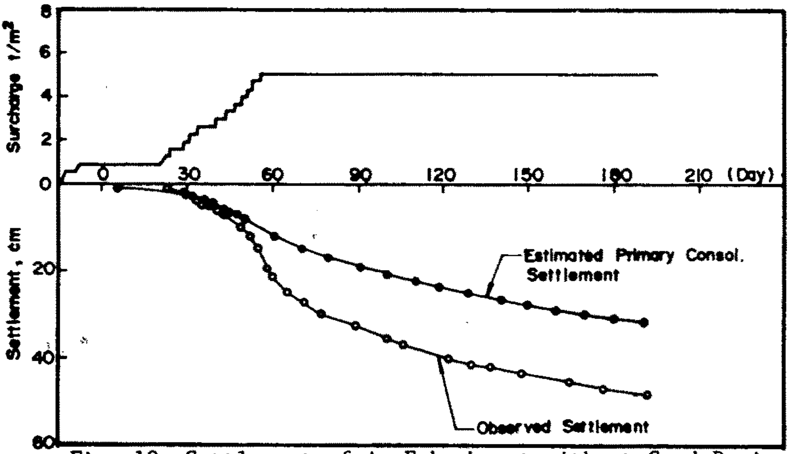

Fig. 13 Settlement of An Embankment without Sand Drains

Comparison With Test Section Without Sand Drains

The settlement behavior of the present test section is compared with another test section carried out at the same site by the Asian Institute of Technology (AIT) in 1973. The AIT test section was constructed on undisturbed ground without sand drains. Consolidation of the clay layers was most likely controlled by the vertical coefficient of consolidation. This differed from the test section with sand drains where the rate of consolidation was controlled by both horizontal and vertical coefficient of consolidation. A comparison made is to show the effectiveness of the sand drains.

Presented in Fig. 13 are the settlement data of the AIT test section together with an estimated primary consolidation settlement curve. In the settlement estimation, the soil profile and the properties were similar to those used for the present test sections with sand drains. As seen in Fig. 13 the observed and recalculated settlement rates agreed very well, indicating that the parameters used are acceptable. Although the rate of settlements observed and calculated are identical, differences are found in the amount of settlement especially during the loading period because of the substantial amount of immediate settlement.

The compression rate of soil layers at various depths are plotted in Fig. 14 . It is seen that the compression of the top 2m weathered clay completed almost immediately after the preloading was placed. This may be attributed to the existence of numerous rootholes and desiccation fissures in this layer. For the very soft clay layer at depth of 2 to 5m, compression was progressive and it contributed to half of the total compression at day 190. Only small amounts of compression were monitored from the clay layers between 5 to 15m. The piezometric pressures measured at 5 and 10m indicate very little dissipation of pore pressure as up to day 190. This clearly show that without sand drains, the clay layer will take very long time to consolidate.

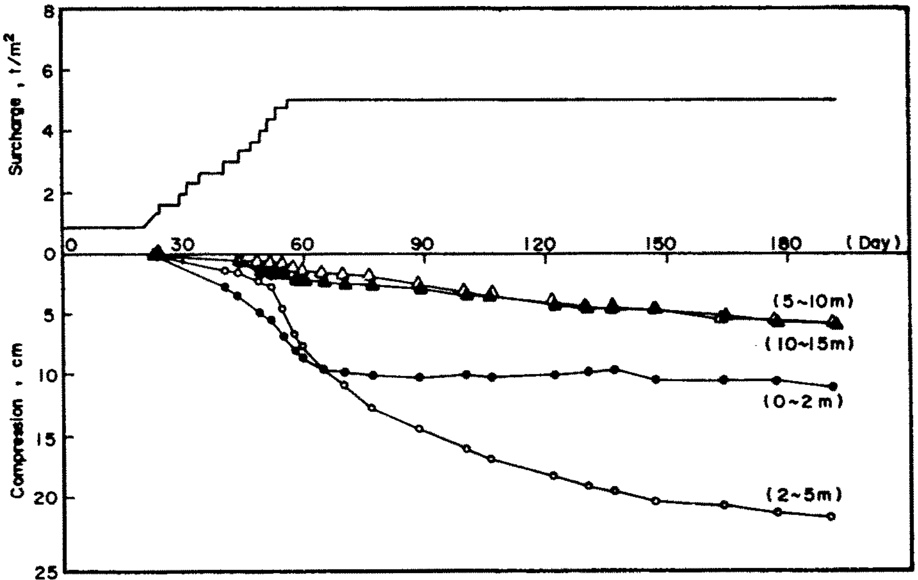

Fig. 14 Compression Rate of Soil Layers without Sand Drains

In the present test section with sand drains, compression of clay at different depths were also monitored. Similar to Fig. 14 , the data from settlement point SG22 are replotted in Fig. 15 . It can be seen that the rate of settlement became much faster by using the sand drains. In fact, consolidation of the entire clay layer (0.1 to 10.65m) completed in less than 8 months after full surcharge. The clays at depths below 10.65m were compressed only slightly because the clay at this depth is less compressible and has many sand seams.

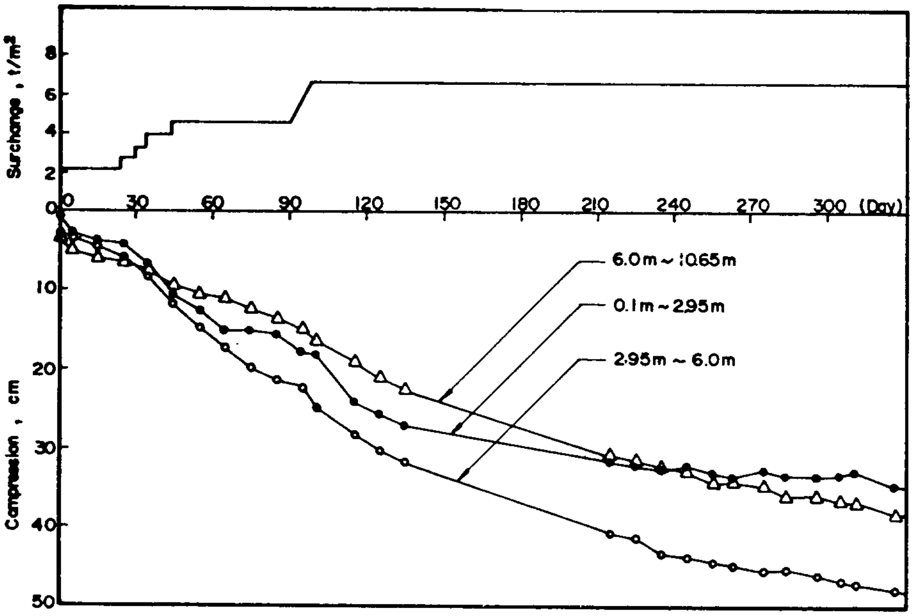

Fig. 15 Compression Rate of Soil Layers with Sand Drains

CONCLUSIONS

Based on the observed performance data of the test section and the analysis of results, the following conclusions are drawn:

1. The use of non-displacement sand drain and surcharge is an effective way to accelerate the settlement rate of the Bangkok clay. The water content of the very soft clay layer can be greatly reduced.

2. Due to the existence of underpressure of groundwater in the stiff clay layer and the underlying sand layer, sand drains penetrating into these layer may provide paths of water flow. That is water may be extracted from the soft clay and discharged to the underpressurized layer. This effect had caused additional settlement to the soft layer. In the Bangkok area, to eliminate this discharge effect, sand drains should not be more than 11m long.

3. The clay layer at depth between 11m to 15m is found to have little contribution to the total settlement. Therefore, sand drains need not be designed below this depth.

4. Settlement behaviors under surcharge loading with or without sand drains were calculated and compared with monitored results. Generally good agreement can be obtained for the rate of settlement. For estimation of immediate settlement, lateral deformation of clay and the rapid compression of the weathered layer should be considered.

ACKNOWLEDGEMENT

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