Geotechnical History of the Development of the Suvarnabhumi International Airport

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1. Introduction

The construction of Suvarnabhumi International Airport (or Second Bangkok International Airport) has been planned since 1960 to accommodate the rapid growth of air traffic in this region. The Suvarnabhumi International Airport (SIA) will not simply provide additional airport capacity to supplement the existing Bangkok International Airport at Don Muang, but will also develop the Bangkok into an international aviation hub in Southeast Asia. The new airport project since the initial planning in 1961 has passed 16 Prime Ministers and 30 Cabinets and was finally approved for construction in May 1991. The first phase of Suvarnabhumi International Airport is scheduled to open in September 2005 with capacity to deal with 40 million passengers and 1.46 million tons of cargo per year. In the future, the new airport will be able to serve 100 million passengers and 6.40 million tons of cargo annually. The New Bangkok International Airport Company Limited (NBIA), a state-enterprise under the Ministry of Transportation and Communications, was formed in February 1996 to implement the SIA construction. Total construction cost is estimated to be more than 120 billion Thai Baht and in which, over 60% will be used for engineering cost. In December 2001, the construction of passenger terminal building, a major milestone, has been finally launched. Figure 1 shows the construction schedule of major SIA facilities.

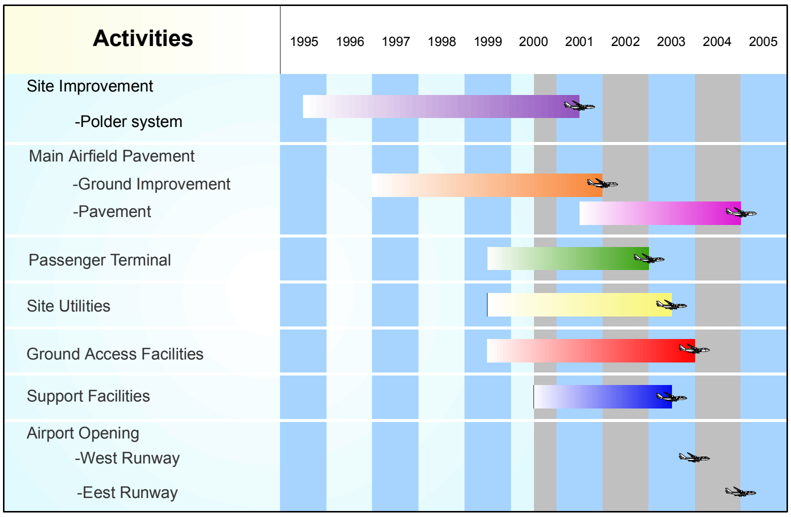


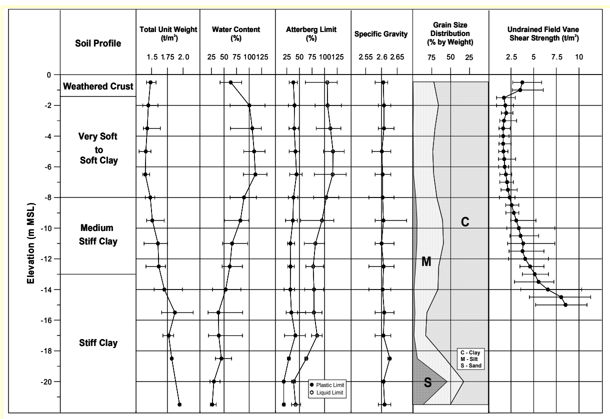
Figure 1 Construction Schedule of Major SIA Activities

The SIA is located at Nong Ngu Hao (means "Cobra Swamp" in Thai), about 30 km to the east of Bangkok Metropolis as shown in Figure 2. The SIA site is 8 km long

and 4 km wide with a total area of 32,000,000 sq. m approximately. The new airport site is situated on the swampy land in flat marine deltaic deposit and most of areas were covered by ponds of shrimp farms or agricultural usages with several crossing canals. Due to the underlying high compressibility and low strength soft marine clay, ground improvement becomes necessary prior to the construction of permanent airport facilities to reduce the maintenance cost. Geotechnical study with field-testing program on evaluation of ground improvement techniques at Nong Ngu Hao was first conducted by Northrop and Asian Institute of Technology (AIT) in 1972. Varied engineering studies were then continued at site until 1997, the implementation of first large-scale ground improvement project – "Ground Improvement for Airside Pavements", which was then completed successfully in June 2002.

2. Subsoil Condition

The subsoil condition at the SIA site is relatively uniform consisting of weathered crust, very soft to soft clay (the famous "Bangkok Clay"), medium stiff clay and stiff clay within the depth of 20m. Underlying the stiff clay, the first dense sand layer is expected below 25m depth. Changes of physical properties with depth are associated with increasing silt or fine sand content and decreasing clay fractions. The major concern for the airport construction is the 8 to 10 m thick layer of very soft Bangkok Clay, which usually has over 100% natural water content with very low bearing strength. The general soil properties including total unit weight, natural water content, Atterberg limits, specific gravity, grain size distribution, undrained field vane shear strength and consolidation parameters are summarized in Figure 3.



3. History of Geotechnical Study at SIA Site

History of major geotechnical studies with comprehensive field-testing program and subsoil investigation at the SIA site can be summarized in the following:

<u>Performance study of test sections by Northrop/AIT (1972~1974)</u>

Access to the SIA site in the early year was extremely difficult since most of the area were swamps and canals. The engineering team sometimes had to utilize bamboos to build temporary bridges for crossing the canals and hut to work and live in (Figure 4). A total of 28 soil borings and 64 vane shear borings were taken at site during this study. Four test sections without ground treatment underneath had been carried out by AIT in 1973 which are:

Figure 4 Nong Ngu Hao in 1973

Test Section A - A 200 m long embankment with varying height at 50 cm, 120 cm, 150 cm and 290 cm was built for the observation of long term settlements.

Test Section B - A 100 m long embankment with maximum vertical stress of 5.05 t/m^2 was built for the observation of creep and long-term settlements. A maximum settlement of 50 cm was recorded 130 days after reaching the final height.

Test Section C - Embankment with 1(v) to 2.5(h) side slope built rapidly to failure (up to 3.4 m or 61 kPa) for stability analysis. A reduction factor of 0.7 was found suitable to be applied to the field vane strength and SHANSEP strength profile to obtain a calculated factor of safety of 1.0 against slope failure.

Test Section D - Excavation pit to 4m deep with 1(v) to 2.5(h) side slope below the ground for observation of slope stability. Since there was no failure during the excavation, average effective strength parameters were used in the stability analysis to obtain a minimum factor of safety of 1.1 against slope failure.

Master plan study, design and construction phasing by NACO/MAA (1983~1984)

During this preliminary master plan study, field testing program including three testing embankments: TSI using groundwater lowering technique by pumping with installed instruments, TSII with embankment surcharge fill and TSIII using vacuum loading, were conducted at the SIA site. A total of 11 boreholes, 40 electric cone penetration tests and 40 pore pressure probe tests were carried out. Non-displacement sand drains (0.26m diameter) were installed to 14.5m depth at 2m spacing in triangular pattern (Figure 5). Maximum settlements of 86cm, 126cm and 66cm were recorded for embankment TSI, TSII and TSIII with loading ranged from 6 t/m^2 to $8t/m^2$, respectively. This study concluded that the suitable sand drain length and spacing at Nong Ngu Hao site should be at 15 m and 2 m, respectively, with maximum preloading period of 6 month. Agreement between the predicted and actual settlement (Figure 6) was exceptionally bad, which may be due to the hydraulic connection from sand drains to the underlying sand layer. After the ground improvement, 30 to 40 percent of water content decreasing in the Bangkok Clay was observed.

Figure 5 Sand Drains Installation (1983~1984)

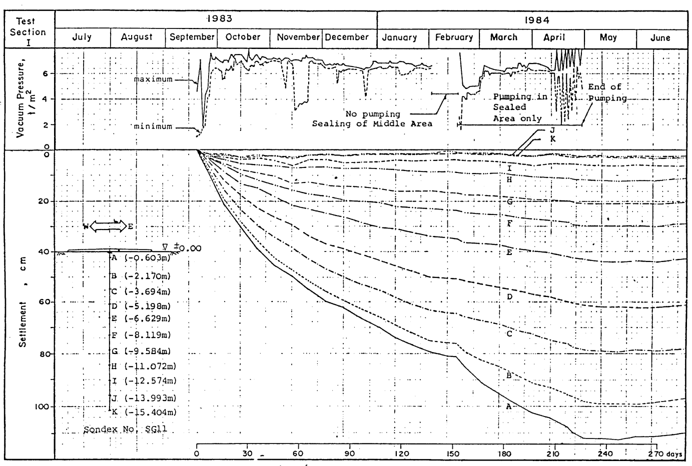


Figure 6 Field Settlement-Time Curve in TSI

Independent soil engineering study by STS/NGI (1992)

Major purpose of this study was to evaluate the past field-testing results in order to select a most suitable ground improvement method to be adopted for SIA construction.

A total of 51 boreholes, 100 vane shear borings and over 80 open tube (stand pipe) piezometers were carried out in this study. Several ground improvement alternatives including preloading with vertical drains, deep soil improvement, piling support with a free spanning plate, relief piles with caps and soil reinforcement and light weight fills were studied as summarized in Table 1. Preloading with vertical drains was finally recommended based on comparison of cost, schedule and technical limitations. The possibility of local hydraulic connection between the vertical drains and the deeper sand layers resulted in excess settlement was also noted after study of pore pressure data obtained from previous studies.

Table 1 Comparison of Design Alternative – Cost and Schedule

Cost & Schedule / Construction Method	Cost for soil improvement Baht/m^2	Total cost including pavement Baht/m^2	Construction Schedule
PVD and soil fill	1,800	2,980[4]	4 years
Deep soil improvement	3,700	5,150[4]	3 years
Piles supporting a free spanning plate	1,000[1]	4,500	2 years
Relief piles with caps and soil reinforcement	2,945[2]	4,410[4]	2 years
Light weight fill, LECA	2,223[3]	4,300	Little extra time

Notes:
1. Cost for installed piles
2. Cost for installed piles, caps and reinforcement
3. Cost for LECA
4. Cost for flexible 0.83m thick pavement = 1,076 Baht/m^2

Full-scale PVD test embankments by AIT (1993~1995)

Figure 7 AIT Test Embankments at Nong Ngu Hao

Full-scale PVD test embankments (Figure 7) were conducted at the SIA site after the conclusion made by STS/NGI during the independent soil engineering study. Three 4.2m high (75 kPa) test embankments (40m x 40m) with PVD spacing of 1.0m, 1.2m and 1.5m in square pattern to 12m deep were constructed. Instruments including surface and deep settlement gauges, pneumatic/standpipe/hydraulic piezometers and inclinometers were installed to evaluate the ground improvement technique by using PVD. The final measured settlement at 6 months after full fill height was about 152cm, 138cm and 128cm, for PVD spacing of 1.0m, 1.2m and 1.5 m, respectively. A total of 3 boreholes and 6 vane shear borings were carried out at site before test embankment construction. In conclusion, the study recommended that PVD is a suitable technique for the accelerated consolidation of Bangkok Clay at the SIA site provided a very careful design of the PVD system to minimize any possible adverse effect due to hydraulic connection with the first sand layer located at 22m depth. Proper drainage system to allow for the free discharge of excess pore water was also remarked.

The above engineering studies were made mainly on the feasibility study purpose. Accompanying with the ground improvement design contracts with the NBIA, two independent soil investigations were also carried out to confirm the soil data obtained from previous studies:

Airside Pavement Design by Airport Design Group (ADG) (1996)

Ground improvement by using preloading and PVDs was adopted as part of Airside Pavement Design contract by the ADG. Due to the hostility from local villagers and flooding at site, only 50% of the planned soil investigation program including 10 shallow boreholes (20m), 5 deep boreholes (40m), 11 piezocone tests and 11 vane shear tests were carried out during the design.

According to the soil investigation results and the previous soil data, it has been confirmed that the soil condition across Airside areas are homogenous in both the thickness of the compressible layers (within 10m depth below original ground) and the minimum water contents showed very little variation. Therefore, same preloading characteristics were applied for all airside areas. In the design, PVDs, 10m in length at spacing of 1 m in square pattern, with minimum surcharge load of 75kPa and waiting period of 6 months were implemented in all areas except at Aprons near the Terminal Building and Concourses. In those areas, surcharge load of 85 kPa and an 11 months waiting period were required. PVD length was reduced from 12m in AIT field test to 10m mainly to avoid the risk of hydraulic connection to sand layer with low piezometric pressure. Based on estimated settlement values, about 90% of the surcharge material can be removed for re-cycling use after 10 months of preloading. When the required preloading period is 15 months, about 80% of the surcharge fill can be removed due to larger amount of settlement.

Landside Road System Design by Moh and Associates (MAA) (1997)

A total of 21 road network system (27.8km) inside the SIA was designed employing with ground improvement of preloading with PVDs by the MAA. The overall ground improvement scheme is similar to the ADG design. The major difference is the preloading scheme due to project characters and requirement (road embankment). A total of 31 boreholes, 6 vane shear tests and 37 cone penetration tests were performed at site by the designer. Some deep boreholes up to 27m were made mainly for pile design purpose.

As summarized above, a total of 144 boreholes, 236 vane shear tests and 97 cone penetration tests had been carried out at the SIA site since 1970s, with locations shown in Figure 8.

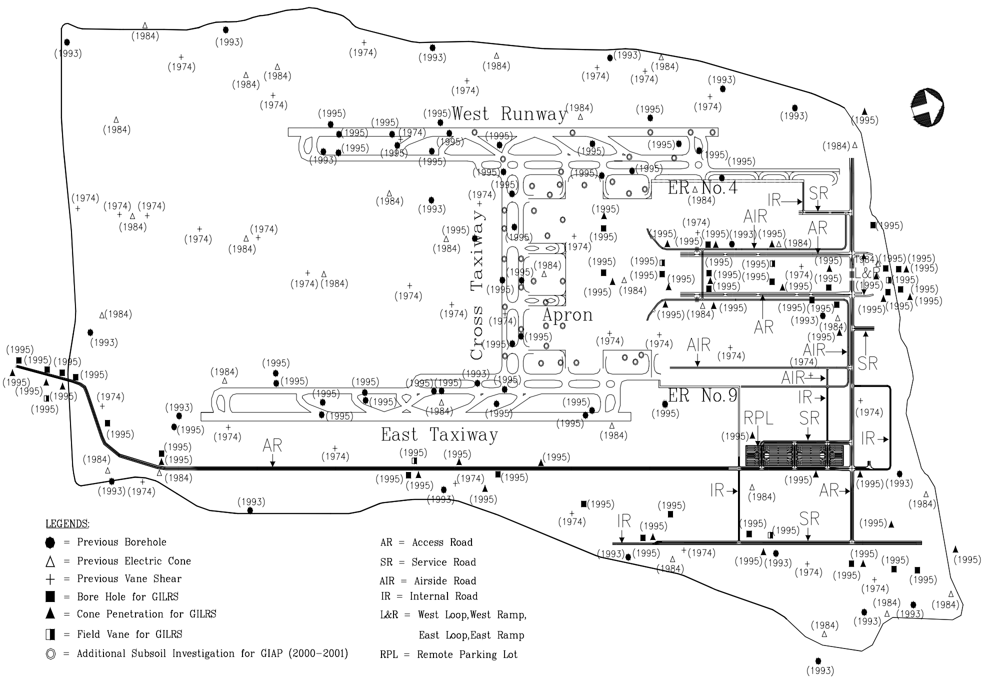
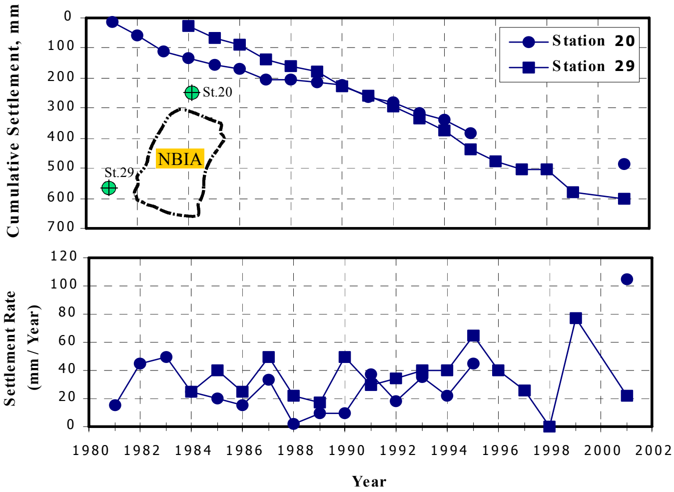


Figure 8 Summary of Historical Subsoil Investigation at Nong Ngu Hao Sit

4. Subsidence and Groundwater at SIA Site

Deep well pumping has been a common practice for the shrimp farms and agriculture lands in great Bangkok area for many years. Serious ground subsidence due to the exploitation of groundwater has been observed for about 30 years, which had caused flooding in Bangkok city during raining season annually. Most of the subsidence were expected to take place in layers deeper than 30m. Earlier study indicated that the subsidence rate at the Nong Ngu Hao area was estimated to be about 30mm~50mm per year. To reduce the subsidence rate, remedial measures were taken to control groundwater pumping by the government in 1983. Based on the monitoring stations around the SIA site, as shown in Figure 9, a total of 600mm subsidence has occurred at Station 29 during the past 20 years. The average ground elevation at the SIA site was changed from about Elev. +0.5 during the earlier study period to MSL used by ADG and MAA in their design. Unfortunately, there was no data from 1996 to 2000 at Station 20 and the survey data in 2001 showed contrast results at the two stations. Further study to establish reliable data in subsidence surrounding the SIA site becomes essential.



The phenomenon of under-hydrostatic water pressure within the depth of 10m to 20m (soft to stiff clay) was first observed in 1973 and further confirmed during the study in 1984. This was most probably due to decrease of piezometric head in the sand layer caused by deep well pumping. Figure 10 summarizes the recorded dummy readings of water pressure since 1973. The water pressure data below 20m were obtained from open-tube piezometers. Based on recently dummy piezometer data obtained from Landside Road design, underpressure became more significant due to increase of deep well pumping recently by comparing the average pore pressure data from 1973 to 2001. Due to the installed PVD in the dummy area (Landside Road System), the water pressure tends to be close to the hydrostatic up to the depth of PVD installation as observed from Fig. 5. A lower pore pressure below 10m depth within the clay layer was also observed if comparing with the previous data. Zero pore pressure was first observed at 20m depth by ADG in 1995, and it was found to be at about 18m during the airside pavement construction (Ground Improvement). From depth 18m to the maximum 35m depth of open tube

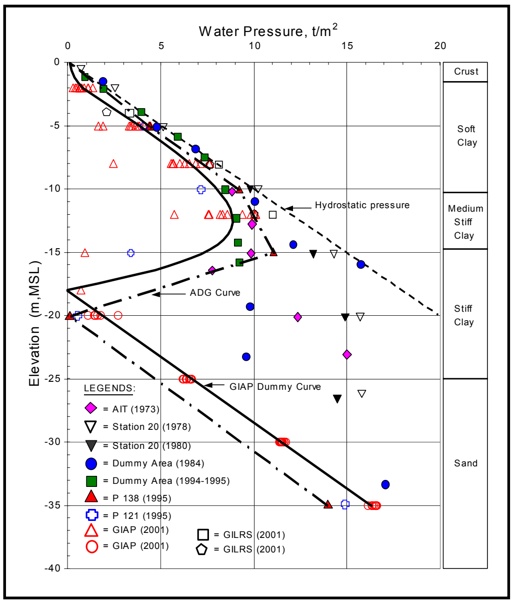


Figure 10 Dummy Pore Water Pressure with Depth

piezometer installation, the water pressure varied lineally with depth.

5. Performance of Ground Improvement Work at Airside Pavements

5.1 Project Data

Ground improvement work at Airside Pavements includes West Runway, Taxiways, Apron and two Emergency Access Roads with total improved area of 3,080,000 sq. m. After reaching the removing criteria, the preloading embankments were then removed to M.S.L. for future pavement construction. Flowchart of ground improvement procedure is illustrated in Figure 11 with project data as summarized below:

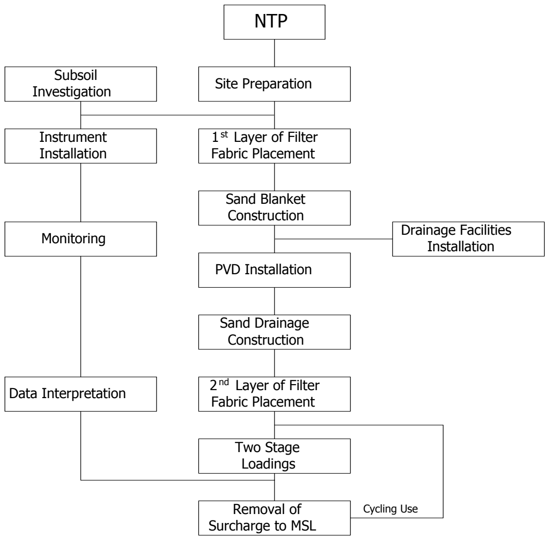


Figure 11 Ground Improvement Sequence

Design Criteria:	A min. 80% of the primary consolidation should be reached.
Sand Blanket:	150cm Thickness
PVD:	10m deep with 1.0m spacing in square pattern
Filter Fabric:	Below and above sand blanket
Preloading Material:	Crushed Rock
Stage Loading:	Two stages with 3 months waiting period in between
Embankment Thickness:	3.8m (75 kPa) & 4.2m (85 kPa)
Counterweight Berm:	15m wide & 1.7m high with 1:4 side slope
Removing Criteria:	1. Min. 6 (75 kPa) or 11 (85 kPa) months waiting period; 2. Min. 80% consolidation 3. Max. 2% (85 kPa) or 4% (75kPa) settlement ratio (monthly settlement to accumulated settlement)

Total quantity of preloading material used during construction included 4,447,453 m^3 of drainage sand, 6,793,294 m^2 of filter fabric, 31,288,708 m of PVD, 2,947,205 m^3 of imported crushed rock, 255,755 m of subdrainage pipe, 12,799 of collector pipe and 146 nos. of manhole. Instrumentation includes surface settlement plates (1,724 nos.), surface settlement monuments (553 nos.), permanent benchmarks (2 nos.), inclinometers (56 nos.), deep settlement gauges (122 nos.), electric piezometers (490 nos.), AIT-type piezometers (40 nos.) and observation wells (1,722 nos.) with typical cross section as shown in Figure 12. Figure 13 shows some filed photos at project site. Ground improvement at East Runway is not included in this contract.

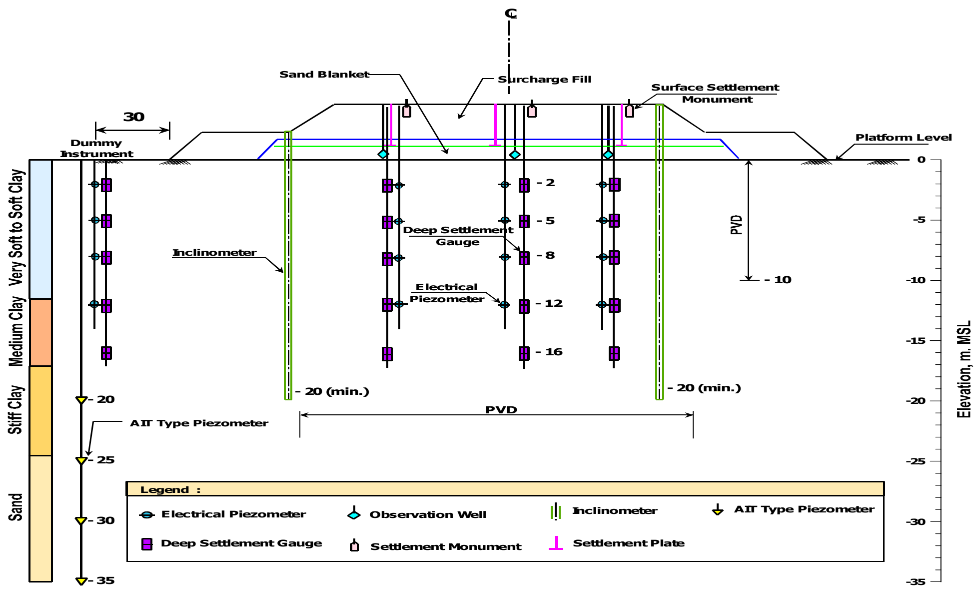


Figure 12 Typical Cross-Section of Instrumentation

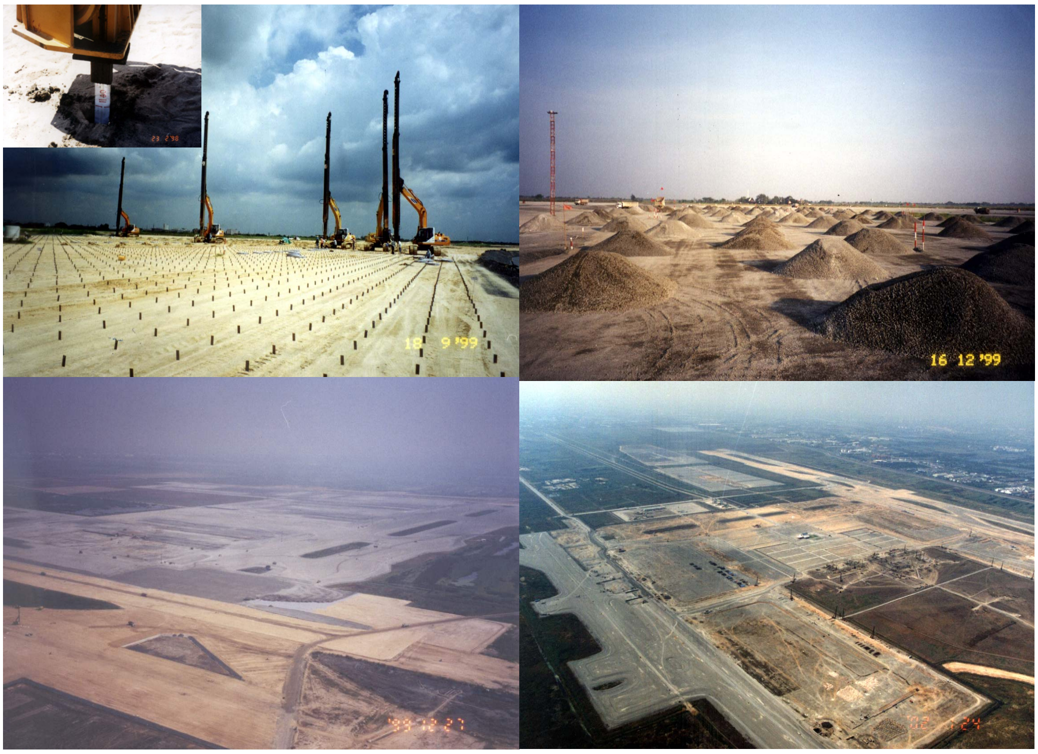


Figure 13 Field Photos of Ground Improvement Work at Airside Pavements

5.2 Monitoring Results

Monitoring data including vertical and lateral movement as well as the pore pressure during ground improvement work are summarized as below:

<u>Surface Settlement</u>

In general, field surface settlement was about 10~20% less than the design estimated settlement at end of waiting period, and the actual time for surcharge removal was about 1 to 2 month(s) longer than the minimum requirement mainly to satisfy the settlement ratio of removing criteria. Therefore, final settlement prior to surcharge removal, as shown in Table 2 and Figure 14, is 5~15% less than the design value. However, uniform settlement over the improved area was observed and the area with higher surcharge load encountered more settlement as expected.

Table 2 Summary of Surface Settlement Data

Location	Settlement at end waiting period (mm)			Settlement before surcharge removal (mm)		
	Avg.	Max.	Min.	Avg.	Max.	Min.
Reference Section[1]	1365	1522	1244	1383	1541	1266
Apron (75 kPa)	1229	1398	1011	1381	1500	1134
Apron (85 kPa)[2]	1539	1841	1095	1594	1854	1203
Cross Taxiway	1239	1447	1016	1291	1484	1102
West Runway	1275	1531	1004	1359	1639	1056
West Runway (85kPa)[3]	1340	1544	1126	1480	1719	1206
East Taxiway	1226	1495	1003	1313	1574	1073
Emergency Rd. 4 & 9	1113	1280	969	1145	1375	980
Average[4]	1241	1445	1041	1302	1519	1102

Notes: 1. under 3.8m fill height or 81 kPa
2. under 11 month waiting period
3. at each end of runway with 6 month waiting period
4. settlement under 85 kPa is not considered.

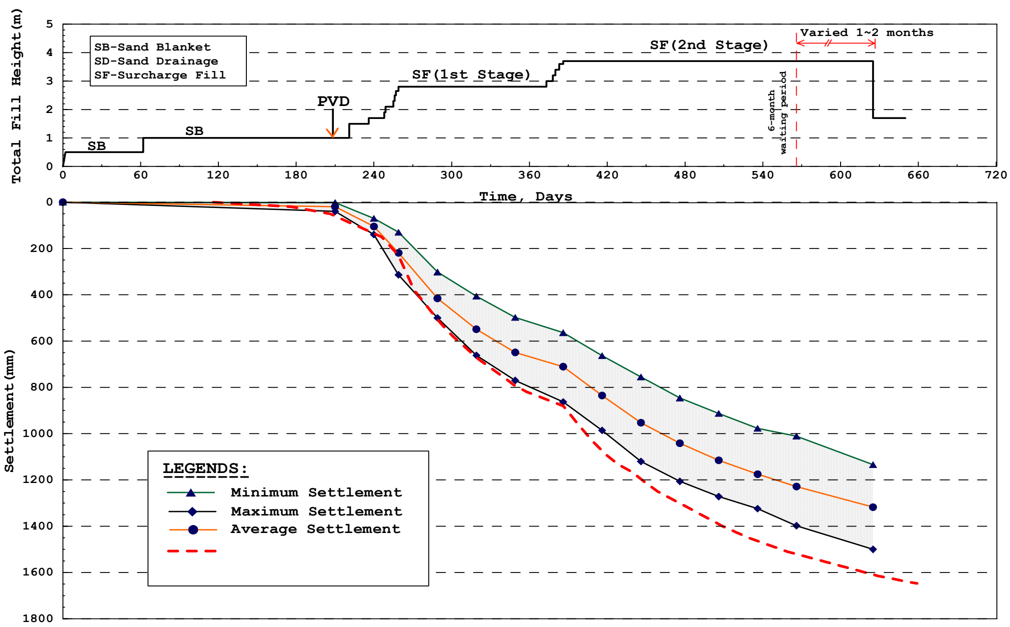


Figure 14 Comparison of Field Settlement and Design Curve at Apron (under 75 kPa)

Deep Settlement

Settlement at various depths was obtained from deep settlement gauge installed at both dummy and ground improvement areas. The installed deep settlement gauges at dummy area indicated that average settlement at depth of 2 m, 5 m, 8 m, 12 m and 16 m was 47 mm, 30 mm, 16 mm, 12 mm and 8 mm, respectively, a very small amount over the construction period of 40 months. Observed settlement at 2 m depth was close to the surface settlement. The soft clay layer (at depth of 5 m and 8 m) experienced largest portion of settlement as expected. The proportion of settlement at 2 m, 5 m, 8 m, 12 m and 16 m to the surface settlement in average was about 95%, 84%, 43%, 10% and 5%, respectively. However, the amount of settlement still varied greatly at different locations, especially at depth of 8 m.

Lateral Movement

Lateral movement data obtained from inclinometers indicate continuous horizontal movement along the depth under embankment. The main objective to obtain the lateral movement data is to ensure a safe embankment construction by providing the pre-warming notice prior to any shear failure as well as reducing unnecessary plastic flow of soil. Observed maximum lateral movement ranged from 10 to 20 cm and occurred mostly at depths of 3 to 6 m of soft clay. Narrow road configuration such as taxiways and runways usually encountered larger lateral movement than Apron. Figure 15 shows a typical settlement and lateral movement profile.

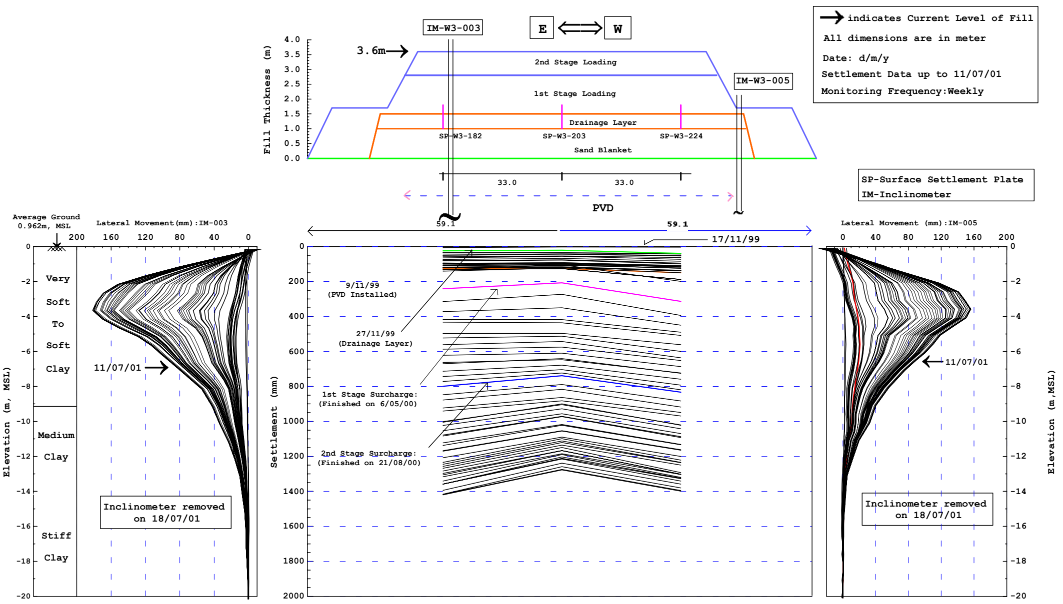
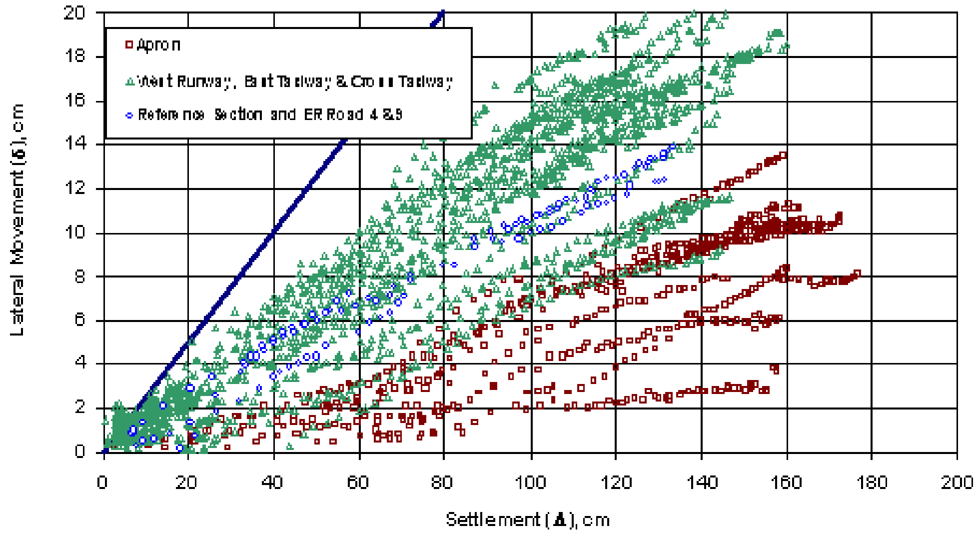


Figure 15 Settlement and Lateral Movement Profile at West Runway

The ratio of maximum lateral movement to maximum surface settlement was used as the criterion for safety control during embankment construction. Special attention was given to the control of rate of construction at the site if the ratio exceeded 0.25, which was obtained from past experience as well as technical publications. Figure 16 summarizes the ratios obtained from various areas. The ratio of 0.25 appeared to be a reasonable criterion to be used according to the performance results.



Excess Pore Pressure

Excess pore water pressures, calculated based on the difference between piezometer readings under surcharge loading and dummy readings, represented the change in pore water pressure under the surcharge load. Elevation of dummy readings was adjusted due to settlement occurred at the piezometer level under surcharge load. In general, excess pore water pressure increased during the fill construction and gradually decreased under the waiting period. The measured dissipation of excess pore pressure during 1st stage loading, which was below the preconsolidation stress, was rapid and the dissipation rate then decreased with increasing effective stress. Fluctuated data were often observed especially in the rainy season, which may due to local variations in the permeability of the clay and the flooding at site. The excess pore water pressure at 5.0m and 8.0m depths, in very soft to soft clay, were higher than that at 2.0m, as expected. Figure 17 shows a typical excess pore pressure dissipation curve with time.

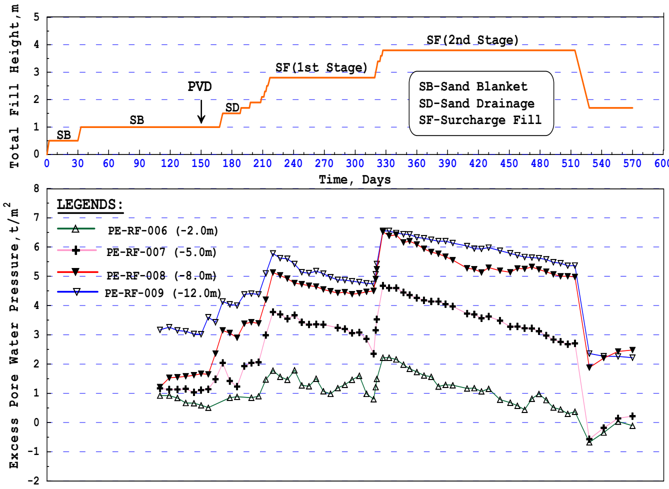


Figure 17 Pore Pressure Dissipation with time at Reference Section

Groundwater level

Groundwater level, obtained from observation wells, indicates the seasonal fluctuation of groundwater condition and the efficiency of drainage facilities. The observation wells were usually installed accompanying with surface settlement plates. In general, the groundwater level rised to its peak during the rainy season and dropped in the dry season. The water level also increased when adding surcharge loads and decreased during the waiting period.

5.3 Change of Soil Properties

Beside the monitoring data, the change of soil properties within the soft clay layer of ground improvement area will also be an important indication to evaluate the ground improvement performance. Major considerations were made to the natural water content, total unit weight, field undrained shear strength and cone resistance of PCPT. In general, reduction of 25 to 35% of natural water content was observed within the soft clay layer (mostly between 2 to 8m depth) after ground improvement. The decrease of water content should also result in increase of total unit weight of the underneath soft clay. In average, a total of 10 to 15% increase in total unit weight after ground improvement was obtained. The undrained field vane shear strength obtained from field vane shear tests increased up to twice of its original value. Uniformed improvement has been observed in all areas. The upper "very soft to soft clay" has been improved to "medium stiff clay" according to the soil properties. Figure 18 shows the comparison of soil properties including water content, total unit weight and undrained field vane shear strength before and after ground improvement areas.

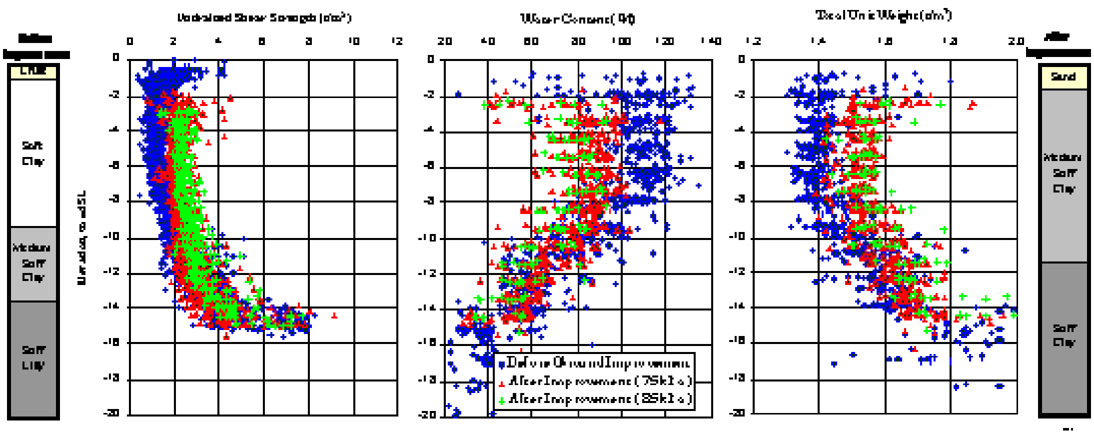


Figure 18 The Comparison of Soil Properties before and after Ground Improvement

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