From Cobra Swamp to International Airport
"The Ground Improvement at Suvarnabhumi International Airport"

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Abstract

The construction of Suvarnabhumi International Airport has been planned since 1960 to accommodate the rapid growth of air traffic in this region. The 41 years project was finally approved for the construction by the Thai Government in 1991 and is scheduled to open in December 2004 with capacity to deal with 30 million passengers and 1.46 million tons of cargo per year. Due to the underlying high compressibility and low strength soft marine clay, ground improvement is necessary to reduce the post-construction settlement prior to the permanent airport facilities construction. Two ground improvement projects as "Airside Pavements" and "Landside Road System" are currently being implemented at the new airport site. This paper discusses the design concept, construction method, and up-to-date ground improvement performance of both projects.

Keywords: Airport, Soft Clay, Ground Improvement, PVD, Preloading, Monitoring

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 the construction by the Thai Government in May 1991. The first phase of Suvarnabhumi International Airport is scheduled to open in December 2004 with capacity to deal with 30 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.

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 Fig. 1. 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.

Figure 1 Location Map of SIA

2. Previous Engineering Study at Airport Site

The past major engineering studies with soil investigation at SIA site can be divided into five phases:

Phase I – Performance study of test sections by Northrop/AIT (1972~1974)

A total of 28 soil borings and 64 vane shear borings were taken at site during this study. The test embankments included two test fills with one embankment constructed to failure (up to 3.4m or 61 kPa). No ground treatment was applied under the embankment.

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

A total of 11 boreholes, 40 electric cone penetration tests and 40 pore pressure probe tests were carried out during phase II study. Field testing program also included three testing embankments, one with embankment surcharge fill, one using vacuum loading and the third one using groundwater lowering technique by pumping. Non-displacement sand drains (0.26m diameter) were installed to 14.5m deep at 2m spacing in triangular pattern. Hydraulic connection induced by sand drains to the underneath sand layer was first observed during this study.

Phase III – Independent soil engineering study by STS/NGI (1992)

A total of 51 boreholes, 100 vane shear borings and over 80 open tube (stand pipe) piezometers were carried out in Phase III. 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. Preloading with vertical drains was finally recommended based on the comparison of cost, schedule and technical limitations.

Phase IV – Full scale PVD test embankments by AIT (1993~1995)

A total of 3 boreholes and 6 vane shear borings were carried out at site before test embankment construction. Three 4.2m high (75 kPa) test embankments with PVD spacing of 1.0m, 1.2m and 1.5m in square pattern to 12m deep were constructed to evaluate the ground improvement technique by using PVD. It has been concluded that PVD is a suitable technique for accelerating consolidation of the SIA soft clay under a careful design.

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 (Ground Improvement) by ADG (1995~1996)

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 ground improvement design of Airside Pavement.

Landside Road System Design (Ground Improvement) by MAA (1996)

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 the 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 Fig. 2.

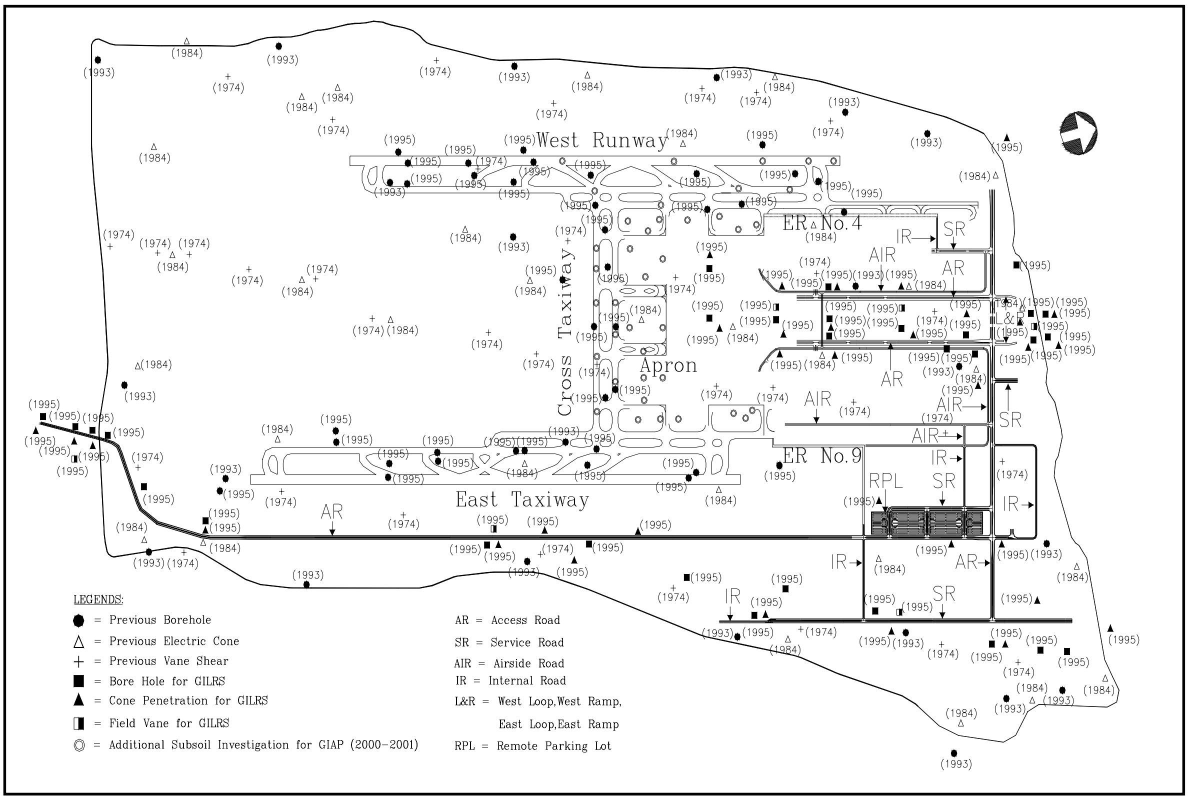


Figure 2 Locations of Soil Investigation at SIA Site

3. Project Outline

Two ground improvement projects, “Ground Improvement for Airside Pavements (GIAP)” and “Ground Improvement for Landside Road System (GILRS)”, are currently being implemented at the SIA site. Preloading with installation of Prefabricated Vertical Drain (PVD) are adopted by the NBIA for both projects to accelerate the consolidation process and reduce post-construction settlement. The design, construction and financial source of both projects are different from each other. MAA is responsible for the design of GILRS and construction supervision of both projects. Major project data of GIAP and GILRS are summarized in Table 1.

The GIAP includes West and East (partial) Runways, Apron, Taxiways and two Emergency Access Roads with a total area of 3,080,000 sq. m. The construction of “Reference Section” (a trial section located at West Taxiway) was required in the initial stage in order to confirm the design assumption and contractor’s working method. Total construction progress in financial term for GIAP is 98.69% at end of 2001 and the remaining areas including south end of west Runway, east Taxiway and Emergency Roads are all under final waiting period.

Table 1 Project Data

	GIAP	GILRS
Total Construction Cost (Thai Baht)	8,419,205,000	1,767,488,000
Financial Source	Government Budget	OECF Loan (Japan)
Construction Period	01/11/97~30/04/02 (54 mos.)	01/12/00~19/04/03 (29 mos.)
Design	Airside Design Group	Moh and Associates
Construction Supervision	TEC/MAA/SIGEC/UIC/MTL	TEC/MAA/NK
G.I. Area (sq. m)	3,080,000	1,320,000
PVD (m)	33,580,000	10,889,600
Sand Blanket (cu. m)	4,550,000	899,600
Preloading Material (cu. m)	2,890,000 (Crushed Rock)	1,722,800 (Sand)

The GILRS covers twenty-three internal access roads and the remote parking lot with a total improved area of 1,320,000 sq. m. There are three types of preloading embankments with height of 2.2m, 3.5m and 4.5m in the GILRS. Total length of Type I, II & III embankment is 13,203.28m (44.1%), 15,708.4m (52.4%) and 1,040m (3.5%), respectively. Except for the remote parking area with Type II embankment only, Type I and II preloading embankment appeared in every 200m alternatively according to the sub-grade profile in GILRS. The varied road profile along the Landside Road System is mainly to obtain horizontal drainage function according to the design. Although the construction progress (in financial term) has been over 90% in GILRS currently, only less than 10% of preloading embankments has satisfied the surcharge removing criteria. The remaining areas are either under waiting period or waiting for surcharge fill removed from the completed section due to the cycle loading plan.

4. Subsoil Conditions

The subsoil condition at the SIA site is relatively uniform consisting of weathered crust, very soft to soft clay (Bangkok Clay), medium stiff clay and stiff clay within the depth of 20m. Underlying the stiff clay, the first dense sand layer is expected after 25m depth. The changes of physical properties with depth are associated with the increasing silt or fine sand content and decreasing clay fractions. The major concern for the airport construction is the 8 to 10 m thick 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 Fig. 3.

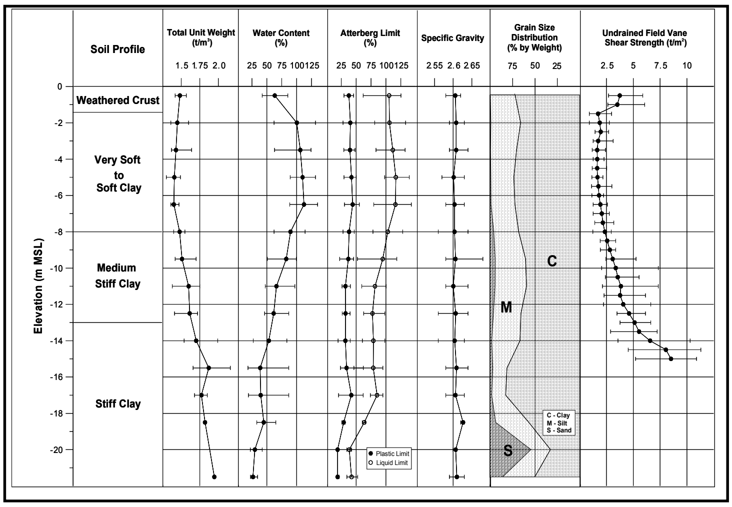


Figure 3 General Soil Properties at SIA Site

5. Subsidence and Ground Water Conditions

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 ground water pumping by the government in 1983. Based on the monitoring stations around the SIA site, as shown in Fig. 4, 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 in GIAP and GILRS. Unfortunately, there is no data from 1996 to 2000 at Station 20 and the survey data in 2001 showed contrast results at two stations. Further study to establish reliable data in subsidence surrounding the SIA site becomes essential.

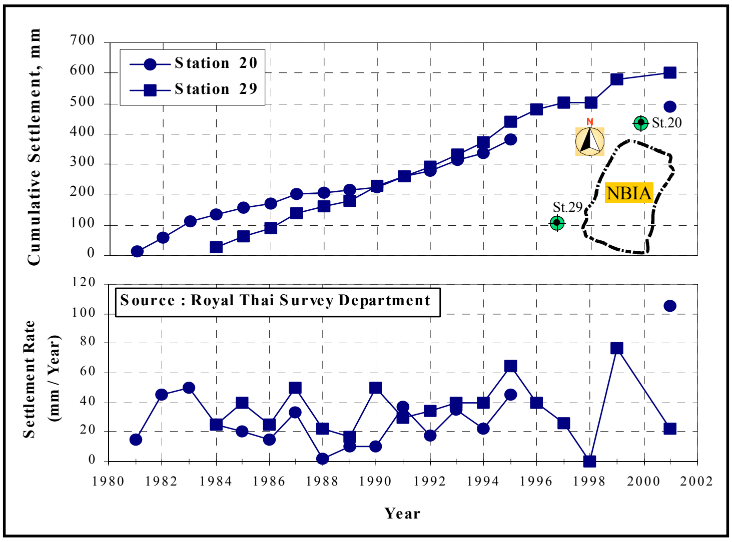


Figure 4 Ground Subsidence at SIA Site

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 Phase II study in 1984. This was most probably due to decrease of piezometric head in the sand layer caused by deep well pumping. Fig. 5 summarizes the recorded dummy readings of water pressure since 1973. The water pressure data below 20m were obtained from open-tube piezometers. Based on the dummy piezometer data in GIAP, underpressure became more significant due to the 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 in GILRS, 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 during GIAP design in 1995, and it was found to be at about 18m during the GIAP construction. From depth 18m to the maximum 35m deep of open tube piezometer installation, the water pressure varied linearly with depth.

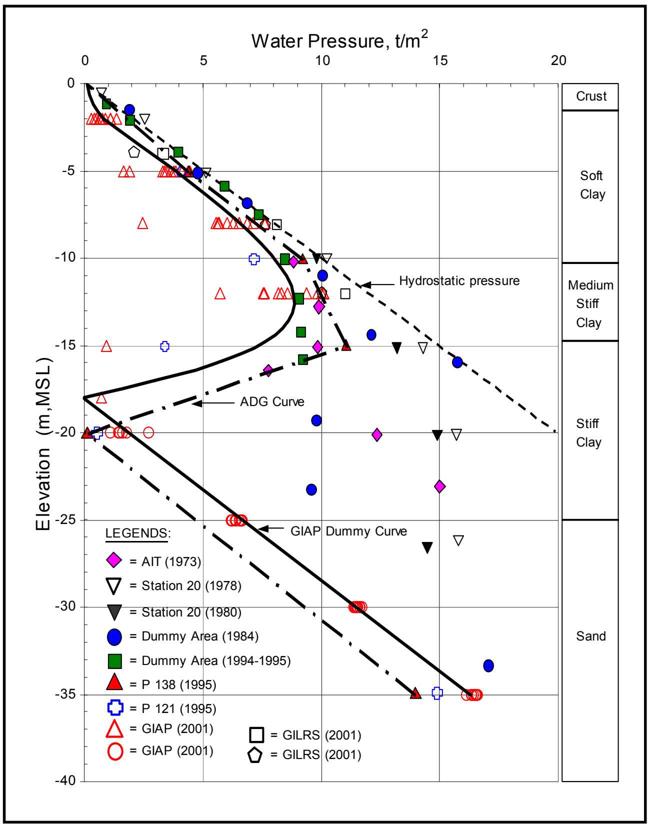


Figure 5 Dummy Pore Water Pressures

6. Ground Improvement Scheme

Ground improvement with preloading is to pre-consolidate the underlying soft soil in the subject area by applying temporary surcharge load, which should be greater than the permanent load in order to reduce the post-construction settlement. Ground improvement sequence in GIAP and GILRS, as shown in Fig. 6, includes construction of sand blanket, drainage facilities, instrumentation, PVD and preloading. Concept of cycle loadings by using removed surcharge material was planned in both projects to reduce the construction cost. The GIAP mainly deals with large areas such as apron and runways. All surcharge fills have to be removed to MSL after ground improvement. The GILRS is for the road embankments only (except for the remote parking area). Surcharge fill will be part of the road structure (removed to sub-grade level). The major design consideration for the GILRS is that the maximum monthly settlement of the road embankment prior to pavement construction should not be more than 0.5cm and whilst for the GIAP, the objective of ground improvement was to achieve minimum 80% primary consolidation of the underlying soft soil. Comparison of ground improvement design in the two projects is summarized in Table 2.

In order to estimate the degree of primary consolidation, a graphical method developed by Asaoka in

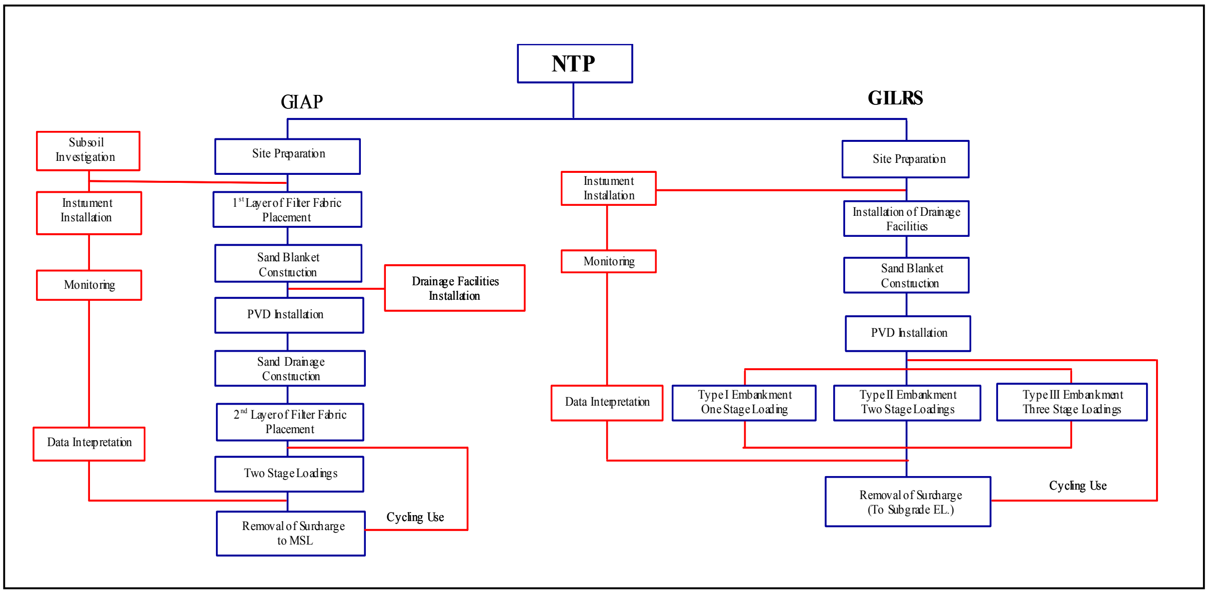
1978 was adopted to predict the ultimate primary consolidation settlement from field data. Length of PVD in both projects was limited to 10m in order to reduce the risk of hydraulic contact to permeable layers with low pore water pressures based on previous study at SIA site. Most of field embankment thickness in GIAP were determined by the surcharge load (75kPa & 85kPa) during construction instead of design thickness to reduce the construction cost. The actual fill thickness was about 10cm less than the design.

Table 2 Comparison of Ground Improvement Design Between GIAP and GILRS

Project Item	GIAP	GILRS		
		Type I	Type II	Type III
Design Criteria	A min. 80% of the primary consolidation should be reached.	Rate of consolidation settlement of the subsoil should be less than 0.5 cm/mo. before pavement construction.		
Sand Blanket	150cm	50cm	80cm	130cm
PVD	10m deep with 1.0m spacing in square pattern	10m deep with1.0m spacing in triangular pattern		
Filter Fabric	Below and above sand blanket	None		
Preloading Material	Crushed Rock	Sand		
Stage Loading	Two**	One	Two**	Three**
Embankment Thickness	3.8m & 4.2m	2.2m	3.5m	4.5m
Berm	15m wide & 1.7m high with 1:4 side slope	No berm with 1:3 side slope		
Design Load	75 kPa & 85 kPa	41.8 kPa*	66.5 kPa*	85.5 kPa*
Removing Criteria	Min. 6 (or 11) months waiting period, min. 80% consolidation & 2%~4% settlement ratio	Min. 6 months waiting period with max. 3 cm monthly settlement rate		

*: Embankment unit weight is assumed to be 19 KN/m^3.

**: Three months waiting period is required before next stage loading construction.



Besides the 6 months final waiting period (or 11 months for Aircraft Stands in the GIAP), other surcharge removing criteria in the two projects were also different. Minimum 80% primary consolidation and maximum 4% (under 75kPa) and 2% (under 85kPa) settlement ratio, which is the ratio of last month settlement to accumulated settlement, are required in the GIAP. The GILRS specifies a maximum settlement rate of 3cm per month prior to surcharge removal.

7. Instrumentation

Instrumentation plays an important role in ground improvement project. Proper and prompt monitoring data can provide effective safety control during embankment construction. Type of instruments installed in both projects are similar, which consists of surface settlement plates, deep settlement gauges, pneumatic (GILRS) or electric (GIAP) piezometers, inclinometers and observation/pumping wells. Surface settlement plates were placed after PVD installation in the GIAP. Surface settlement monuments to monitor fill settlement and two 150m deep permanent benchmarks were also installed in the GIAP. There were some dummy instruments installed outside of the preloading embankment area in both projects. For the GIAP, dummy instruments include deep settlement gauges and electric/AIT-type (open tube) piezometers. Two sets of dummy instruments including surface settlement plates and piezometers were planned together with PVD installation in the GILRS. Typical cross-section of instrumentation and total quantity in the two projects are shown in Fig. 7 and Table 3, respectively.

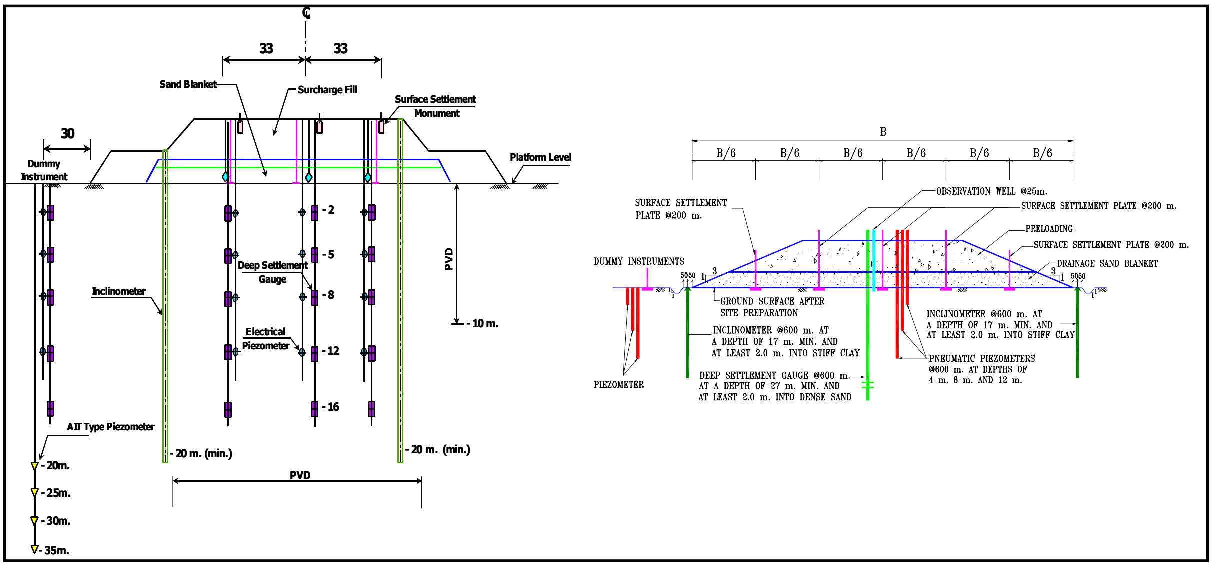
Figure 6 Ground Improvement Sequence

Table 3 Total Quantity of Instrumentation

Item \ Project	GIAP		GILRS	
	Embankment	Dummy	Embankment	Dummy
Surface Settlement Plate	1,724	-	730	4
Surface Settlement Monument	553	-	-	-
Permanent Benchmark	-	2	-	-
Inclinometer	56	-	88	-
Deep Settlement Gauge	111*	11*	53**	-
Pneumatic Piezometer	-	-	159	6
Electric Piezometer	444	46	-	-
AIT-type Piezometer	-	40	-	-
Observation Well	1,722	-	1236***	-

*: One set includes 5 individual deep settlement gauges at 2m, 5m, 8m, 12m & 16m.
**: A deep settlement gauge includes sensor rings at every 2m to min. depth of 27m.
***: Also used as pumping wells

The required monitoring frequency in both projects are a little different. In general, monitoring is required before and after each stage of fill work followed by once a week in GIAP. Three types of monitoring frequency varied from once per week to once per every two weeks were specified in GILRS.



GIAP **GILRS**

Figure 7 Typical Cross Section of Instrumentation

8. Monitoring Data

Monitoring data can be divided into two parts: vertical/lateral movement obtained from surface settlement plates/deep settlement gauges/Inclinometers and pore water pressure obtained from piezometers. Due to the limited pages, the data presented herein mainly focus on the Apron and West Runway of GIAP and South Access Road (SAR), the major and first constructed section in GILRS.

8.1 Vertical/lateral movement

Under similar subsoil condition, variation of vertical and lateral movements were obtained in accordance with the time and height of surcharge fill placement. Marked increase in settlements was observed after placing the 1st or 2nd stage of surcharge fill loading in both projects. Settlement contour in the Apron area of the GIAP is presented in Fig. 8. Uniform settlement has been observed throughout the Apron area with average settlement of 140cm and 170cm under 75kPa and 85kPa loading,

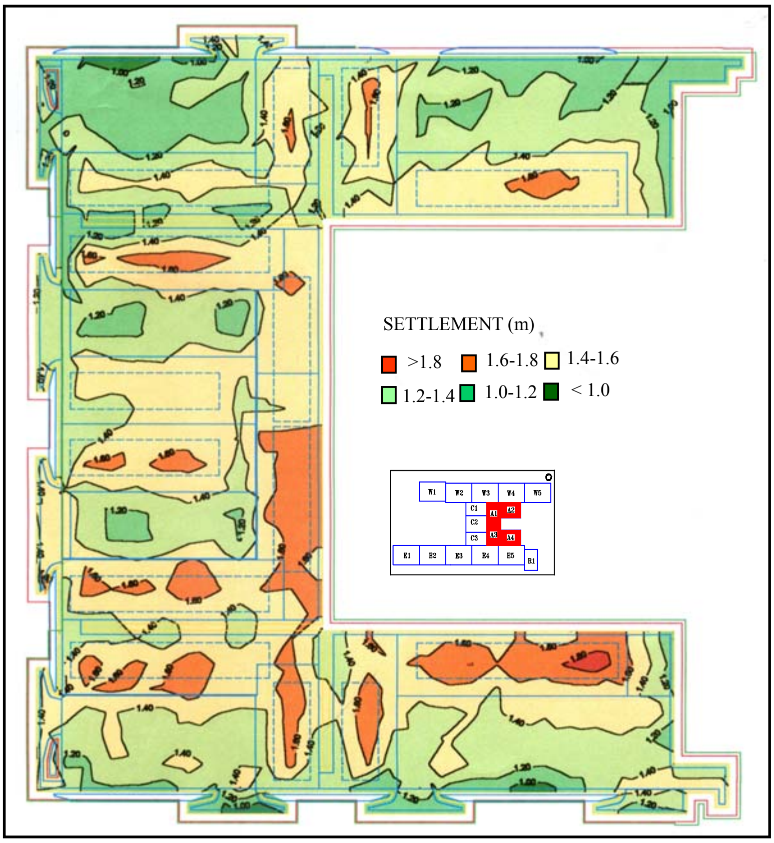


Figure 8 Settlement Contours in the Apron Area

respectively. Settlements at West Runway and Cross Taxiway varied from 110cm to 150cm. A small loop areas in the middle of West Runway had only half of the settlement comparing with other areas under same fill height as shown in Fig. 9. It was later found that PVD was not needed in this area. However, the settlement results have shown the effectiveness of PVD installation. Figure 10 shows typical settlement profiles at various depths as monitored by deep settlement gauges and surface settlement plates. In general, the soft clay layer experienced the largest proportion of settlement as expected. Below the depth of 16m under which stiff clay layer underlies, the amount of settlement was minimal. Similarly, the top hard weathered crust underwent only a small amount of settlement as shown by the nearly equal value of settlement measured by settlement gauges at 2m depth and those at the ground surface. In general, maximum lateral movement at most of inclinometer locations occurred at elevation of 3.0~6.0m below MSL, and less lateral movement was observed at Apron than from other areas. Figure 11 illustrates a typical cross section of settlement and lateral movement profile at West Runway. It is interesting to notice that most of the maximum settlement did not occur at center of the embankment, which may result from contributions of lateral movement and short drainage path to the side settlement plates. Ratio of lateral movement to vertical settlement was used as the criteria for safety control in both projects. Special attention were given to the control of rate of construction at the site if the ratio exceeded 0.25. Figure 12

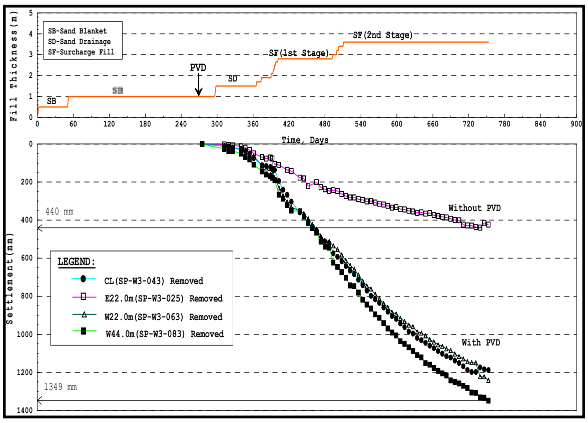


Figure 9 Comparison of Settlement between PVD and Non-PVD Area

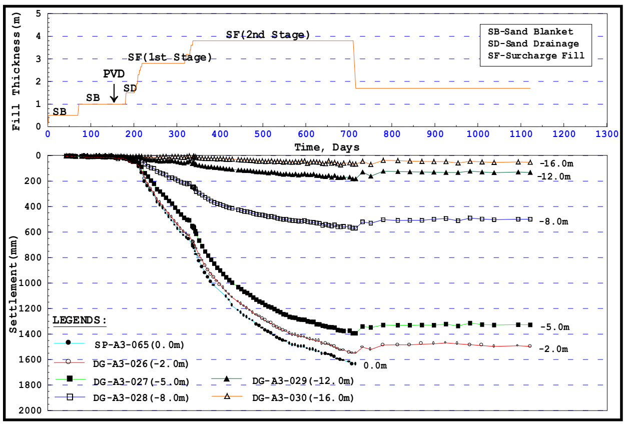


Figure 10 Deep Settlement Profile in the Apron Area

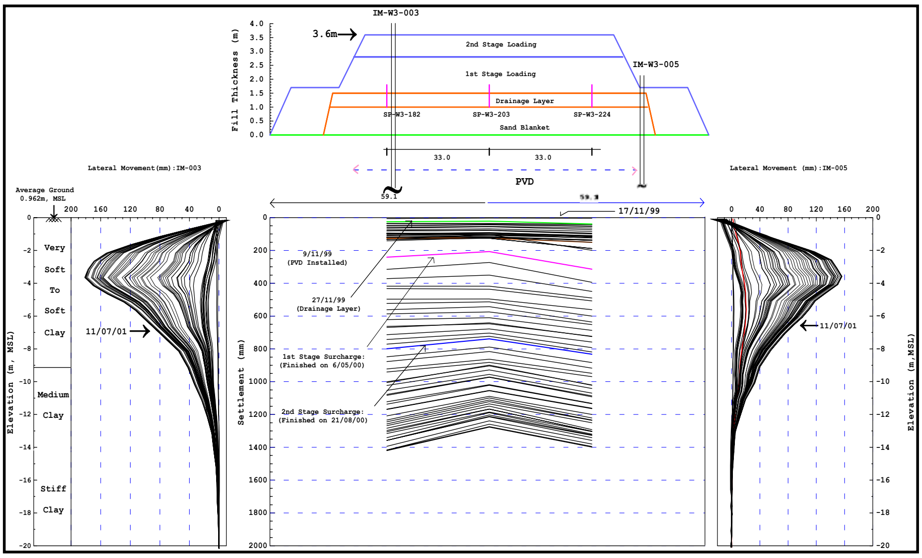


Figure 11 Observed Settlement and Lateral Movement Profile at West Runway

summarized values of the ratio of lateral to vertical movement at Apron and Cross Taxiway during construction. Narrow road configuration generally had higher ratio. The ratio of 0.25 appeared to be a reasonable criterion to be used according to the performance results.

Based on settlement data along the SAR of GILRS, an average of 85cm and 130cm settlement was recorded for type I and II embankment after 6.5 and 3 months waiting period, respectively. Deep settlement data from the sensor-ring type deep settlement gauges, however, showed much smaller settlement within the soft clay layer than the data obtained from the GIAP. It could be contributed to the possible blocking of the sensor rings by the backfill grout. Due to its road embankment configuration, lateral movements at SAR is higher than the GIAP. Ratios of lateral to vertical movement at Types I & II embankments are also plotted in Fig. 12.

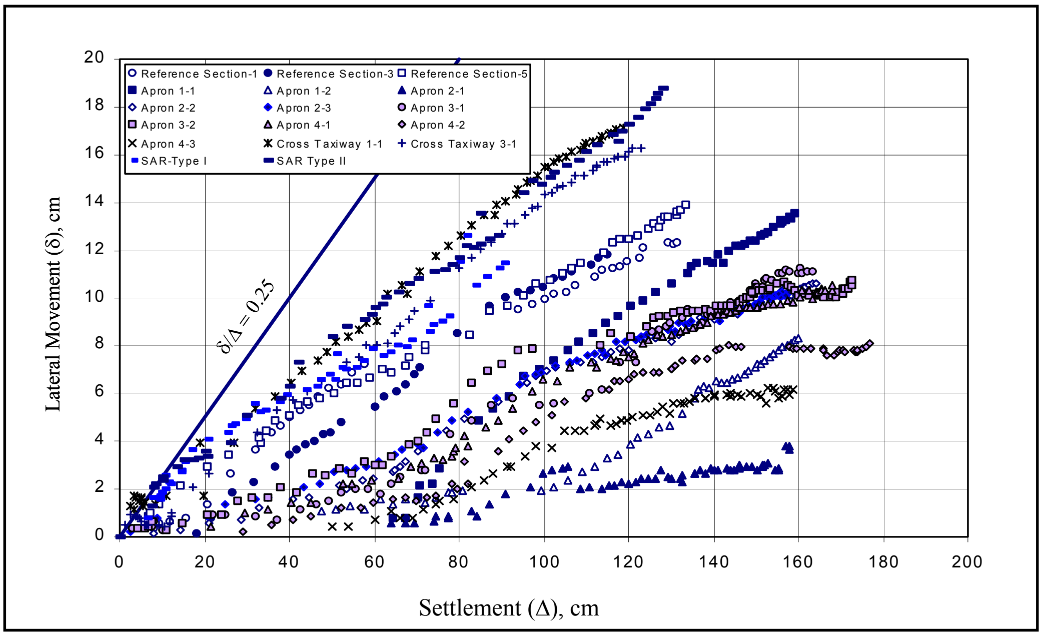


Figure 15 Ratios of Lateral to Vertical Movement at Apron, Cross Taxiway (GIAP) and SAR (GILRS)

8.2 Pore water pressure

Excess pore water pressures were calculated based on the difference between piezometer readings under surcharge loading and dummy readings or the dummy curve as shown in Fig. 5. Elevation of dummy readings was corrected for settlement of the piezometers. In general, excess pore water pressure increased during the fill construction and gradually decreased under the waiting period Fluctuated data were often observed, especially in the raining season. 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. Dissipation of excess pore water pressure at 12.0m depth was slower than others, which may due to the fact that the piezometers were installed below the PVD depth. 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. A typical excess pore pressure dissipation curve with time and the fill status at Apron is shown in Fig. 13.

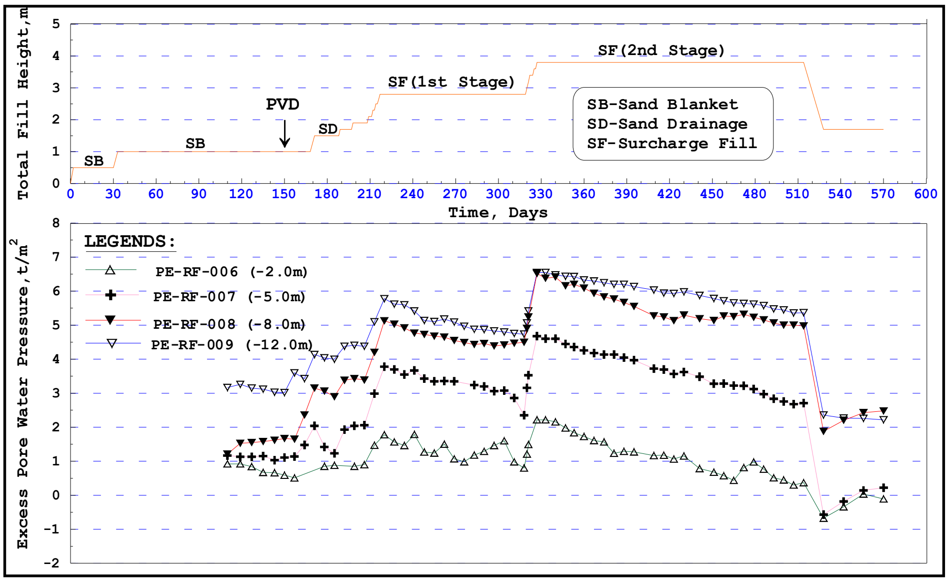


Figure 13 Observed Excess Pore Pressure Distribution with Time and Fill Status at the Reference Section

9. Evaluation of Ground Improvement Performance

Evaluation of the ground improvement performance was made by comparing the settlement data with the design estimate as well as changes in soil properties obtained from additional soil investigation results after ground improvement.

9.1 Based on the design perspective

Before comparing the design settlement to the field data, the following factors had to be taken into consideration:

- Actual waiting period was about 1 to 2 month(s) longer than the minimum requirement of 6 months in both projects mainly due to the settlement rate/ratio criteria;
- The field settlement consists of not only consolidation settlement but also immediate settlement, subsidence and contribution from lateral movement;
- Initial settlement under the first meter of sand blanket in GIAP was ignored since all settlement plates (except the Reference Section) were installed after the PVD installation.

The design settlement at end of final 6 months waiting period was about 150cm, 170cm, 84cm, 140cm and 190cm under 75kPa and 85kPa in GIAP and for Type I, II and III embankment in the GILRS, respectively. The overall field settlement performance (neglecting the initial settlement from the 1m thick sand blanket) in the GIAP was about 10 to 20cm less than the design expectation as shown in Fig. 14. Settlement from the AIT test embankment (PVD in 1m spacing with 12m length) is also included. In the GILRS, settlement of Type I embankment of SAR at end of final waiting period is similar to or a little over the design value. After 3 months waiting period, settlement of Type II embankment was also close to the design estimate under the same period. However, more data are necessary in GILRS for reaching final conclusions.

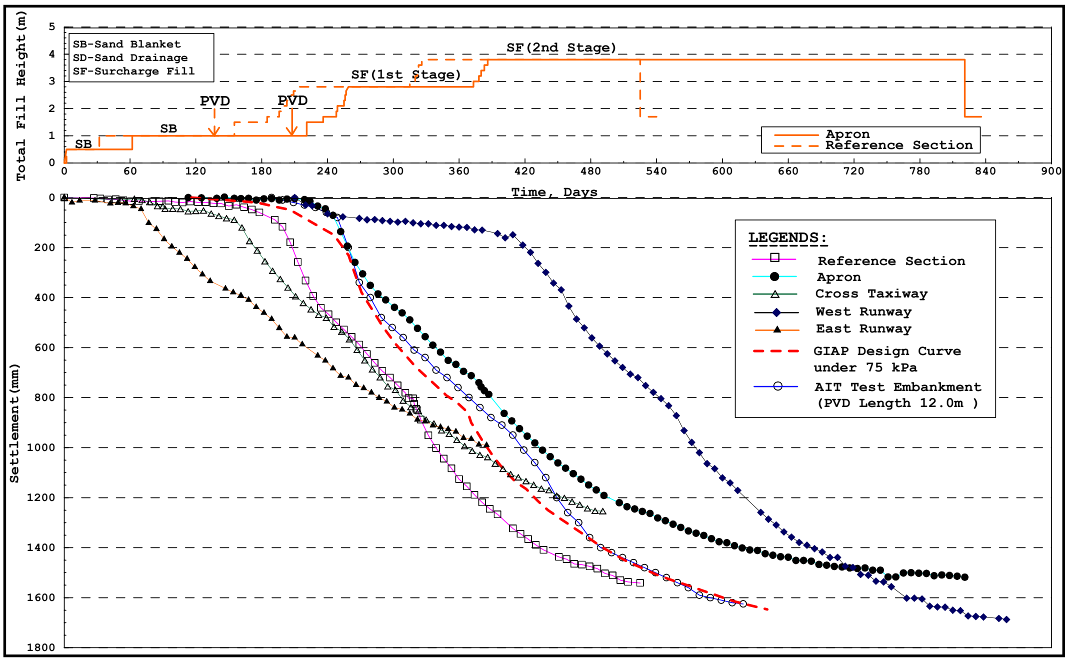


Figure 14 Comparison of Field Settlements with the Design and AIT Test Embankment

9.2 Based on changes in soil properties

Decrease in natural water content and increase in total unit weight and shear strength are expected for the soft clay after ground improvement. Additional subsoil investigation including undisturbed sampling, filed vane shear test and piezocone penetration test has been carried out in the Apron area and also in part of West Runway of GIAP with locations shown in Fig. 2. More soil investigation is expected to be done in both projects in the future. The comparison of soil properties before and after ground improvement is shown in Fig. 15. Water content within the soft clay zone reduced 20~25% whilst the total unit weigh and undrained field vane shear strength increased up to 10% and 100%, respectively. The upper stratum of very soft to soft clay has been improved to medium stiff clay according to the soil properties.

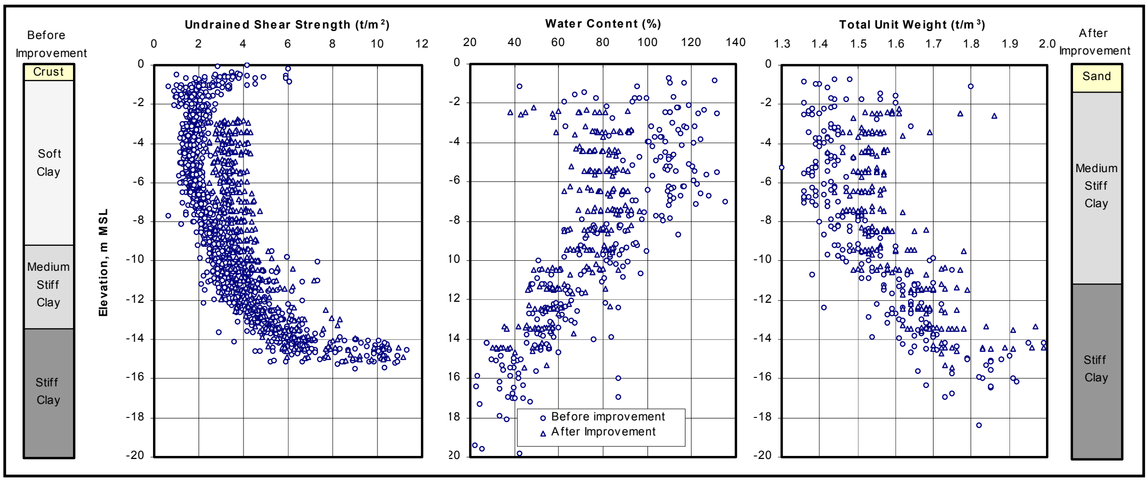


Figure 15 Comparison of Soil Properties Before and After Ground Improvement

10. Conclusions

Some conclusions can be made based on the current performance of GIAP and GILRS as followings:

1. The ground subsidence rate decreased from 1995 to 1998 and increased in 1999 and 2001 based on the survey data near the site. Closely monitored survey and strict control of deep well pumping should be continued.
2. It is further confirmed that the piezometric pressures in the upper 18m of the subsoils at the SIA site exist in an under hydrostatic condition. Further draw down was also observed below the depth of PVD installation.
3. Effective drainage system and proper monitoring are the key for success of ground improvement by preloading with PVD installation.
4. The criterion of 0.25 for the ratio of lateral to vertical movement appears to be reasonable for the embankment safety control.
5. Settlement data are more reliable than the pore pressure values in both projects. In general, field settlements in both projects are close to the design estimates.
6. The overall performance of the ground improvement work in the GIAP is satisfactory based on changes in the soil properties after improvement. More data are necessary in the GILRS for final conclusions.

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