THE SOUTHEAST ASIAN GEOTECHNICAL SOCIETY (SEAGS)

The Southeast Asian Geotechnical Society was founded in 1967 by Dr. Za-Chieh Moh as a Regional Society encompassing countries or territories in Southeast Asia, not fully fledged in the National Society of the then International Society for Soil Mechanics and Foundation Engineering (ISSMFE). At that time, the Society was called the Southeast Asian Society of Soil Engineering (SEASSE). The countries which originally composed this Regional Society were Thailand, Malaysia, Singapore, Philippines, Indonesia, Hong Kong and Taiwan with members from Korea, Vietnam, Nepal, Bangladesh, Burma and Pakistan.

As each country began to develop, they formed their own National Societies. Thus, we now have National Societies in Indonesia, Korea, Vietnam, Pakistan, Bangladesh and Nepal. However, there are still many members from these countries who retain their membership in SEAGS. Additionally, the Southeast Asia Region is very dynamic in its development and as such, many Geotechnical Engineers and Companies have interest in the region and many of them worked in Southeast Asia. Thus, nearly a third of SEAGS members coming from Japan, Australia, New Zealand, Germany, France, UK, Norway, Sweden, Switzerland, Canada, USA and the former countries in which there were no National Societies in Asia. Over the years, Thailand, Malaysia, Singapore, and Taiwan continue to remain as full partners with SEAGS.

SEAGS arranges regular Southeast Asian Conferences once in two to three years, publishes a Journal and prepares Newsletters as well as liaise with ISSMFE. Dr. Edward W. Brand was the Journal Editor who conceived the Journal in 1970 and continued to be the Editor until 1978. The current editor of the Journal is Dr. Noppadol Phienwej. The previous editors were Dr. V.K. Campbell, Dr. J.S. Younger, Dr. D.R. Greenway, Mr. P.G.D. Whiteside, Dr. C.A.M. Franks and Prof. D.T. Bergado.

The President of the Society is rotated among the member countries: Dr. Za-Chieh Moh (1967-1970 and 1970-1973); the late Prof. Chin Fung Kee (1973-1975); the late Prof. Peter Lumb (1975-1977); the late Dr. Tan Swan Beng (1977-1980); Dr. E.W. Brand (1980-1982); Dr. Ting Wen Hui (1982-1985); Dr. A.S. Balasubramaniam (1985-1987); Prof. Seng-Lip Lee (1987-1990); Dr. Chin Der Ou (1990-1992); Dr. Ooi Teik Aun (1992-1995); Dr. Surachat Sambhandaraksa (1995-1998); Dr. John C. Li (1998-2001). The current President of SEAGS is Prof. K.Y. Yong and the current Secretary-General is Prof. D.T. Bergado.

ENGINEERING INSTITUTE OF THAILAND (E.I.T.)

The Engineering Institute of Thailand celebrated her 60th Anniversary in 2003. It is a world-class professional institution devoted to the task of formulating and promoting the engineering profession. Numerous engineering standards and professional practices, including textbooks, magazines, and journals have been published by the Engineering Institute of Thailand.
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PREFACE

This year, the Second Bangkok International (Suvarnabhumi) Airport will be opened for normal operations. Envisioned to stand as one of Thailand’s landmarks, the airport covers an area of 3,220 hectares. This modern airport can accommodate 45 million passengers and 3 million tons of cargo annually. Originally conceived in 1965, the project site has been subjected to over 40 years of numerous geotechnical investigations and evaluations. It has been well-known that the airport site is situated on a lowland marshy area underlain by the world-famous soft Bangkok clay. The low strength and high compressibility of the soft foundation make this megastructure vulnerable to vertical settlements and lateral deformations. Moreover, the site is also subjected to ground subsidence caused by the excessive withdrawal of the groundwater from underlying aquifer.

Significant works include construction of flood protection dikes, ground improvement using PVD in areas of runways and taxiways; cement column improvement of soft clay for foundation of roadways, transition areas, fuel tanks and parking lots; tunnel construction for trains and utilities; pile foundation construction of terminal building, traffic control tower and many other structures.

The successful construction of the Suvarnabhumi Airport is a triumph of new technologies and creative engineering. To celebrate this success, the Southeast Asian Geotechnical Society (SEAGS) in cooperation with the Engineering Institute of Thailand (EIT) is organizing this International (Suvarnabhumi) Airport. This Symposium will serve as a forum for engineers, consultants, contractors, suppliers and academics as well as government officials to discuss the new techniques and methodologies, lessons learned, systematic solutions to numerous problems, the geotechnical properties of the underlying soft ground, etc. This gathering also marks the milestone of achieving the difficult task of undertaking the creation of the largest International Airport in Southeast Asia.

On behalf of SEAGS and EIT, we would like to welcome all Distinguished Guests, Sponsors, Exhibitors, Speakers, and Participants to this Symposium on Geotechnical Aspects of Second Bangkok International Airport.

Dr. Za-Chieh Moh
Prof. Dennes T. Bergado
Dr. Noppadol Phienwej
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Z.C. Moh and P.C. Lin
MAA Group Consulting Engineers, Taipei, Taiwan

ABSTRACT

Planning for the construction of a second international airport in Greater Bangkok area started as early as 1960. Final government approval for the construction was made in 1991, and the ground improvement was started in 1997. The airport, named the Suvarnabhumi International Airport, is expected to start operation in late 2006.

During the past 45 years, many studies including site characterization, evaluation of soil properties, feasibility of various ground improvement techniques, full scale field tests, and sophisticated research work were carried out by a number of consulting companies and academic institutions. This paper summaries some of the major work done.

INTRODUCTION

Construction of the 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 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 the Suvarnabhumi International Airport is scheduled to open in late 2006 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% are being 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 the major SIA facilities.
<table>
<thead>
<tr>
<th>Activities</th>
<th>Year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Improvement</td>
<td></td>
</tr>
<tr>
<td>- Polder System</td>
<td></td>
</tr>
<tr>
<td>Main Airfield Pavement</td>
<td></td>
</tr>
<tr>
<td>- Ground Improvement</td>
<td></td>
</tr>
<tr>
<td>- Pavement</td>
<td></td>
</tr>
<tr>
<td>Passenger Terminal</td>
<td></td>
</tr>
<tr>
<td>Site Utilities</td>
<td></td>
</tr>
<tr>
<td>Ground Access Facilities</td>
<td></td>
</tr>
<tr>
<td>Airport Opening</td>
<td></td>
</tr>
</tbody>
</table>

Source: PMC

Fig. 1 Construction Schedule of Major SIA Activities (based on opening on 29 Sept. 2005)

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 swampy lands in flat marine deltaic deposit and most of the 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 site ground characterization at Nong Ngu Hao was first conducted by Northrop and Asian Institute of Technology (AIT) in 1972. Various engineering studies including evaluation of ground improvement techniques were continued at the site until 1997. Implementation of the first large-scale ground improvement project – “Ground Improvement for Airside Pavements”, was completed successfully in June 2002. Ground improvement work for the Landside Road System was completed in 2005. Additional ground improvement for the proposed 3rd Runway, the First Midfield Satellite Aprons and their connections to facilities already completed or under construction was started in December 2004 and is continuing at the time of this Conference (May 2006). In addition to works commissioned by the airport authorities, more than 45 master’s and doctoral thesis research were carried out by students at the Asian Institute of Technology. This paper presents a summary history of major geotechnical studies carried out by various agencies at the site in the past 30 years. Details of the studies have been published elsewhere as well as presented at this Conference.
SUBSOIL CONDITION

The subsoil condition at the SIA site is relatively uniform consisting of weathered crust, very soft to soft clay (the so-called "Bangkok Clay"), medium stiff clay and stiff clay within the depth of 20m. Underlying the stiff clay, the first dense sand layer is found 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.

Fig. 2 Location Map of SIA

Fig. 3 General Soil Profile at Nong Ngu Hao
SUBSIDENCE AND GROUNDWATER AT SIA SITE

Deep well pumping has been a common practice for the shrimp farms and agriculture lands in the 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 the rainy season annually. Most of the subsidence was expected to take place in layers deeper than 30m. Earlier studies indicated that the subsidence rate at the Nong Ngai 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 4, a total of 600mm subsidence has occurred at Station 29 during the past 20 years. The average ground elevation at the SIA site 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 somewhat contrast results at the two stations. Further study to establish reliable data in subsidence surrounding the SIA site becomes essential.

![Figure 4: Ground Subsidence at SIA Site (Source: Royal Thai Survey Department)](image)

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 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 recent 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 Figure 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 piezometer installation, the water pressure varied lineally with depth.

Fig. 5 Variation of Dummy Pore Water Pressure with Depth

HISTORY OF GEOTECHNICAL STUDIES AT THE SIA SITE

History of major geotechnical studies with comprehensive field-testing program and subsoil investigation at the SIA site are summarized in the following sections.

Performance Study of Test Sections by Northrop/Asian Institute of Technology, 1972–1974 (AIT, 1974)

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. A total of 28 soil borings and 64 vane shear borings were made at the site during this study. Four test sections without ground treatment underneath had been carried out by AIT in 1973 (AIT, 1974) which included:

Test Section A - A 200 m long embankment with varying height of 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/m² 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. Figure 6 shows comparison of predicted surface settlement with measured settlement.

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.

![Fig. 6 Comparison of Predicted Surface Settlement with Measured Settlement – Northrop/ATI Test Section B](image)


During this preliminary master plan study, field testing program including three full scale test sections to evaluate the effectiveness of non-displacement sand drains as soil
improvement technique (Engineers for Second Bangkok International Airport, 1984; Moh et al, 1987; Woo et al, 1989). As shown in Figure 7, two of the test sections, i.e. TSI and TSIi each 40m by 40m in plan, had sand drains installed in triangular pattern at 2.0m spacing (or one drain per every 3.50 sq.m of ground surface area). TSIii had one sand drain per every 5.0 sq.m. Each sand drain which was installed by jet-bailer method had a minimum nominal diameter of 26 cm and 14.5 m in length. The three test sections were loaded by two different methods, they are TSI and TSIii using vacuum pumping, and TSIi with embankment fill surcharge. The test sections were well instrumented and regularly monitored. The instruments installed within the test sections included piezometers, settlement plates, inclinometers, sondex settlement gauges and hydrostatic profile gauges. Some instruments were also placed in a dummy area to monitor the natural variation of the subsurface conditions with time during the study period. Figure 8 shows a typical instrumentation plan.

![Fig. 7 Plan of NACO/MAA Test Sections](image)

**Fig. 7 Plan of NACO/MAA Test Sections**

![Fig. 8 Instrumentation Plan of NACO/MAA Test Section TS II](image)

**Fig. 8 Instrumentation Plan of NACO/MAA Test Section TS II**
Some of the major findings of the tests are: (1) The use of non-displacement sand drains is an effective way to accelerate the settlement rate of the Nong Ngu Hao soils. Figure 9 shows a typical settlement curve with time under surcharge fill load, and Figure 10 shows comparison of center surface settlement at the three test sections. (2) Majority of the consolidation settlement occurred in the very soft to soft clay layers up to a depth of about 11m below the ground surface. (3) The settlement behavior appeared to be independent of the type of loading, i.e. surcharge fill or vacuum, as illustrated in Figure 11. (4) Due to the existence of under pressure of groundwater in the stiff clay layer and the underlying sand layer, sand drains penetrating into these layers may provide paths of water flow and cause additional consolidation of the soft clay. To eliminate this discharge effect, sand drains and also other type of drains at the SIA site should not be more than 11m long.

Fig. 9 Typical Settlement vs Time Curve of NACO/MAA Test Section TS II

![Typical Settlement vs Time Curve of NACO/MAA Test Section TS II](image)

Fig. 10 Comparison of Center Surface Settlement at NACO/MAA Test Sections

![Comparison of Center Surface Settlement at NACO/MAA Test Sections](image)
Fig. 11 Percentage of Settlement Versus Depth – NACO/MAA Test Sections

**Independent Soil Engineering Study by STS Engineering Consultants Co./Norwegian Geotechnical Institute, 1992 (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 the SIA construction (STS/NGI, 1992). 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 schemes, including preloading with prefabricated vertical drains (PVD), deep soil improvement, piling support with free spanning plates, relief piles with caps and soil reinforcement, and light weight fills were studied, as summarized in Table 1. Preloading with prefabricated 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 Alternatives – Cost and Schedule (from STS/NGI, 1992)

<table>
<thead>
<tr>
<th>Cost &amp; Schedule Construction Method</th>
<th>Cost for soil improvement Baht/m²</th>
<th>Total cost including pavement Baht/m²</th>
<th>Construction Schedule</th>
</tr>
</thead>
<tbody>
<tr>
<td>PVD and soil fill</td>
<td>1,800</td>
<td>2,980&lt;sup&gt;4&lt;/sup&gt;</td>
<td>4 years</td>
</tr>
<tr>
<td>Deep soil improvement</td>
<td>3,700</td>
<td>5,150&lt;sup&gt;4&lt;/sup&gt;</td>
<td>3 years</td>
</tr>
<tr>
<td>Piles supporting a free spanning plate</td>
<td>1,000&lt;sup&gt;1&lt;/sup&gt;</td>
<td>4,500</td>
<td>2 years</td>
</tr>
<tr>
<td>Relief piles with caps and soil reinforcement</td>
<td>2,945&lt;sup&gt;2&lt;/sup&gt;</td>
<td>4,410&lt;sup&gt;4&lt;/sup&gt;</td>
<td>2 years</td>
</tr>
<tr>
<td>Light weight fill, LECA</td>
<td>2,223&lt;sup&gt;3&lt;/sup&gt;</td>
<td>4,300</td>
<td>Little extra time</td>
</tr>
</tbody>
</table>

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/m²

Full-Scale PVD Test Embankments by Asian Institute of Technology, 1993~1995 (AIT, 1995)

Full-scale PVD test embankments (Figure 13) were conducted at the SIA site after the conclusions made by STS/NGI during the independent soil engineering study (AIT, 1995). 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 the site before the test embankment construction. In conclusion, the study recommended that PVD is a suitable technique for accelerating consolidation of the Nong Ngu Hao 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 free discharge of excess pore water was also mentioned.

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

Ground improvement by using preloading and PVDs was adopted as part of the Airside Pavement Design contract by the ADG (Consortium of DMJM International, Scott-Wilson-Kirkpatrick, Nonconsult International, SPAN and SEATEC). 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 previous soil data, it was confirmed that the soil conditions across Airside areas are homogenous in both the thickness of the compressible layers (within 10m depth below original ground) and the minimum water contents which showed very little variation. Therefore, the same preloading design was adopted for all airside areas. In the design, PVDs, 10m in length at spacing of 1m in square pattern, with minimum surcharge load of 75kPa and waiting period of 6 months were designed for 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 used in the AIT field tests 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.


In connection with the design of the landside road network system (total length 27.8km) inside the SIA, thirty one boreholes, 6 vane shear tests and 37 cone penetration tests were performed at the site. Some deep boreholes up to 27m were made mainly for pile design purpose. The overall ground improvement scheme is similar to that for the airside, i.e. using PVD with fill preloading. The major difference is the preloading design in view of the characteristics of the roadway requirements. Details of the design philosophy and methodology are presented in Moh and Associates (1996) and Moh & Lin (2003).


In 2004, the New Bangkok International Airport Co. Ltd. decided to accelerate the development of the SIA and commissioned the Third Runway Design Group (Consortium of Scott-Wilson-Kirkpatrick Thailand, Span, MAA Consultants, Southeast Asia Technology and
Norconsult Civil Engineering Co.) to carry out design of the 3rd runway, which is located immediately west of the West Runway, and the 1st Midfield Satellite Aprons. There are many interface areas between these new works and the ongoing airfield pavements construction (West & East runways, aprons, and associated taxiways). As the new airport was originally scheduled for opening at the end of third quarter of 2005, any new construction within the operational safety zone must be completed before the airport starting operation. Because of the many design and construction constraints, particularly time limitation and effect on adjacent construction, different ground improvement schemes were evaluated. Vacuum consolidation with PVD was adopted for those works in the current safety zone. For the works in areas without special constraints, PVD with surcharge fill preloading will be used (TRDG, 2004).

For this project, 15 borings to the depth of 25m below ground surface and 15 in situ vane tests were carried out. Special tests including constant rate of strain consolidation tests with radial drainage were performed to evaluate the horizontal coefficient of consolidation of the soft clay.

As summarized above, a total of 159 boreholes, 251 vane shear tests and 97 cone penetration tests had been carried out at the SIA site since 1970s.

**PERFORMANCE OF GROUND IMPROVEMENT WORK AT AIRSIDE PAVEMENTS**

Ground improvement works for the Airside Pavements and Landside Road System areas were completed in 2005. The following sections present a summary of the results of ground improvement by PVD and fill preloading at the Airside Pavements area to illustrate the effectiveness of this method of ground improvement at the SIA site. The construction supervision work was undertaken by a consortium consists of Thai Engineering Consultants Co., MAA Consultants Co., Siam General Engineering Consultants Co., Upham International Corp., and Meinhardt (Thailand) Co.

**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 12 with project data summarized below:
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 cu m of drainage sand, 6,793,294 sq m of filter fabric, 31,288,708 m of PVD, 2,947,205 cu m of imported crushed rock, 255,755 m of subdrainage pipe, 12,799 of collector pipe and 146 nos. of manhole. Instrumentation included 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.

Fig. 12 Typical Cross-Section of Instrumentation for Ground Improvement at Airside Pavement Areas
MONITORING RESULTS

Monitoring results of the instrumentation including vertical and lateral movement as well as the pore pressures during ground improvement work are summarized as below:

Surface Settlement

In general, field surface settlement was about 10 to 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 13, is 5 to 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

<table>
<thead>
<tr>
<th>Location</th>
<th>Settlement at end waiting period (mm)</th>
<th>Settlement before surcharge removal (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference Section1</td>
<td>1365</td>
<td>1522</td>
</tr>
<tr>
<td>Apron (75 kPa)</td>
<td>1229</td>
<td>1398</td>
</tr>
<tr>
<td>Apron (85 kPa)2</td>
<td>1539</td>
<td>1841</td>
</tr>
<tr>
<td>Cross Taxiway</td>
<td>1239</td>
<td>1447</td>
</tr>
<tr>
<td>West Runway</td>
<td>1275</td>
<td>1531</td>
</tr>
<tr>
<td>West Runway (85kPa)3</td>
<td>1340</td>
<td>1544</td>
</tr>
<tr>
<td>East Taxiway</td>
<td>1226</td>
<td>1495</td>
</tr>
<tr>
<td>Emergency Rd. 4 &amp; 9</td>
<td>1113</td>
<td>1280</td>
</tr>
<tr>
<td>Average4</td>
<td>1241</td>
<td>1445</td>
</tr>
</tbody>
</table>

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.
Deep Settlement

Settlement at various depths was obtained from deep settlement gauges 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 to 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 varied somewhat at different locations, especially at depth of 8 m.

Lateral Movement

Lateral movement data obtained from inclinometers indicate continuous lateral movement along the depth under embankment. The main objective to obtain the lateral movement data is to ensure a safe embankment construction by providing pre-warning 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 in the soft clay. Narrow road configuration such as taxiways and runways usually encountered larger lateral movement than Apron. Figure 14 shows a typical settlement and lateral movement profile.
Fig. 14 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 15 summarizes the ratios obtained from the various areas. The ratio of 0.25 appeared to be a reasonable criterion to be used according to the performance results.

Fig. 15 Lateral Movements vs Vertical Settlement

**Excess Pore Pressure**

Excess pore water pressures, calculated on the basis of 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 during 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 decreased with increasing effective stress. Fluctuated data were often observed especially in the rainy season, which may be due to local variations in the permeability of the clay and 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 16 shows a typical excess pore pressure dissipation curve with time.

![Graph showing pore pressure dissipation](image)

**Fig. 16 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 rose 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.

**CHANGE OF SOIL PROPERTIES**

Beside the instrumentation monitoring data, changes of soil properties within the soft clay layer of ground improvement area are also important indicators to evaluate the ground improvement performance. Major considerations were given 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 17 shows the comparison of soil properties including water content, total unit weight and undrained field vane shear strength before and after ground improvement areas.

Fig. 17 Comparison of Soil Properties before and after Ground Improvement

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ENGINEERING PROPERTIES OF SOFT BANGKOK CLAY

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ABSTRACT

This paper presented technical informations on engineering properties and behavior of soft Bangkok clay with typical soil profiles and soil conditions. Shear strength and deformation properties were presented. It aims to present practical informations for the design of sand embankment on soft Bangkok clay, and the expected clay’s behaviour, based on recent findings. Soil conditions at Suvanabhumi airport and their behaviour resulting from the nature of soil deposits and problems from locally deep well pumping near the airport were also presented. These were obtained from new findings which were found after the airport construction has been started.

INTRODUCTION

In the sand embankment design, especially the airport runway, is controlled by very limited differential settlement, and total settlement resulting from the low ground elevation. These will certainly require the post construction settlement to be small. It also leads to the necessity for embankment to be design with suitable factor of safety for minimizing time dependent undrained movements which are caused by undrained creep and undrained plastic flow with the subsequently with the occurrence of plastic consolidation. These are considered to occur during consolidation process which is followed by the drained creep (secondary compression), particularly in normally consolidated (NC) range. Part of the findings were publicly presented by Sambhandharaksa et al. (2003) and Sambhandharaksa and Aimdee (2004) for cemented aging soft marine Bangkok clays having plasticity index ($I_p$) about 40-50% and 75% respectively. These clays have the liquidity index ($I_l$) about 1.0, very sensitive and have the water content about 80% for clay with 40% plasticity index and about 100% for clay with 75% plastic index respectively.

The paper concentrated on strength and deformation properties. Emphases were made on engineering effects from aging, viscous, and plastic flow behaviour, including the Fe$_3$O$_3$ cementation effects partly resulting from deep well pumping (Sambhandharaksa and Aimdee, 2004).

GENERAL SOIL PROFILES AND SOIL CONDITIONS IN BANGKOK

Based on the writer experience and recently research results, there are three types of soil profiles and soil conditions in Bangkok area.

Profile A and Type I Clay

Profile A is normally found in the city and the northern part of the city. Type I soft marine clay, having the OCR (overconsolidation) about 1.5 caused by aging, is overlaid by the 2-3 m of the crust layer. The typical total thickness is about 14 m. Type I clay has $I_p$ (plasticity index) about 40%, liquidity index ($I_l$) about 0.8 to 1.0, and the natural water content
about 70 to 80%, and has the older age but lower $I_p$ than the Type II marine clay. The fresh water leaching process, additionally, causes the clay type I to have meta fabric. This process is more than Type II clay, and partly causes the both types of clays become sensitive (Type II clay is described in profile B). Aging process and the chemical cementation of Fe$_2$O$_3$ (Sambhandharaksa and Aimdee, 2004) from deep well pumping are also the mechanisms that cause Type I clay to be very sensitive (sensitivity about 4.0) and liquidity index ($I_L$) around 0.8 to 1.0.

Fig. 1 In Situ Pore Pressure a Suvarnabhumi Airport (Taonthong, 1985)

The deep well pumping usually causes the declination of in situ pore pressure below 8 m. The upper clay’s pore water pressure condition is in hydrostatic condition from the recharge of the surface water from the ground. Shells and organic matter generally found, as the result of the marine deposit and humid climate. The deep well pumping well locates generally below 500 m depth. This causes the ground water to flow downward.
Underlying the soft to medium clay layer is the brownish stiff to very stiff clay layer which is underlain by the normally consolidated (NC) sand layer where the in situ pore pressure is lowest. This is the result of deep well pumping. In locations where water supply can be reached, and no locally deep well pumping, the in situ pore pressure, generally, is constant the first sand layer. Underly the first silty fine sand layer, alternating layers of alluvium clay and coarse sand layer are encountered.

The ground water table is likely to flow downward below the depth of about 10 m, even the high embankment was built and caused the high excess pore pressure. This is due to the significantly low in situ pore pressure in the first sand layer. The distribution in situ pore pressure at Suvarnabhumi airport shown in Figure 1 (Taothong, 1985)

Profile B and Type II clay

Type II clay is the very soft to soft marine clay and causes troublesomes in engineering problems more than Type I clay, resulted from its higher plasticity (IP ~ 75%). It can be found at the outskirt of the city, excepting in the northern part of the city. The high sensitive upper clay has IP about 70 to 80%, Wp about 90 to over 100% and LI is about 1.0 or larger. Type II clay is still has about the same in situ sensitivity with Type I clay, resulted from the lowering of leaching effect, and from higher viscous property from its high IP despite having higher Wp and LI.

Profile B consists of the Type II clay overlying soil layers from Profile A. Profile B’s soft clay’s thickness varies. The total thickness of Type I and Type II clay can be as much as 20 m. This leads to the location of first sand layer to be between 20 to 30 m depths. The variation in clay’s thickness overlying the sand layer, and the differences in pore pressure’s declinations from deep well pumping, can lead to differential settlement resulting from deep well pumping. This is partly from the difference in the rate of consolidation and of the thickness of compressible layers, especial in areas which locate nearby the localizing deep well pumping wells, resulting from the lack of water supply.

Unlike Type I clay, where the effects of chemical bondings from aging and chemical cementation which lead to problems of the breaking down or changes of soil structure, the overlying. Type II clay has, however more problems form viscous effects which lead to more problems from undrained creep, (time dependent undrained deformation), drained creep, thixotropy, and from undrained plastic flow which must be followed by plastic consolidation. Type II clay will, therefore, have more deformation than type I clay during construction. For post construction, Type I and II clay may have significantly long term deformations as the clays may be in normally consolidated state from the influence of the declination of in situ pore pressure from deep well pumping.

The influences of the changes in soil structure, from breaking of bonds from aging (Berjuum, 1973 and Tsuchida, 2001), from chemical cementations of Fe₂O₃ from deep well pumping (Sambhandharaksa et al 2003), and from the leaching of salt concentration with fresh water which lead to meta soil structure, are more in Type I clay. These mechanisms lead to undrained movements and the additional increase in pore pressure during consolidation.

The above effects and undrained creep are considered to cause additional excess pore pressure in Type II clay as well during one dimensional consolidation, in NC range (Sambhandharaksa and Aimdee, 2004), having their maximum effects when σ'w/σ'p is about

25
1.2 to 1.4. This causes the nonlinear stress ($\sigma'_{vc}$) and vertical strain ($\varepsilon$) to be nonlinear, seen in one dimensional consolidation tests of both Type I and Type II clays. With two on three dimensional movement, the influences of time dependent undrained plastic flow is additionally included.

Both Type I and Type II clay, the collapsing of soil structure or sudden changes of soil structure from above mentioned mechanisms, and undrained viscous effects, also lead to the reduction of normalized undrained shear strength ($c'_{v}/\sigma'_{vc}$) in NC state. Such an unexpected reduction leads to unexpectively large time dependent undrained plastic flow, in NC state. Both clay types are considered to be the elasto visco-plastic material. The visco-plastic nature is more in Type II clay, due to it has higher Ip.

In NC state, both Type I and Type II clay will develope nonlinear stress strain characteristics in one dimensional consolidation (Sambhandharaksa et al, 2003, Sambhandharaksa and Aimdee, 2004). This is the result of the additional development of excess pore pressure caused by previously mentioned mechanisms. Following the nonlinear range, the stress ($\sigma'_{vc}$) and strain ($\varepsilon$) become linear. This behaves according to Tsuchida’s (2001) concept.

Upon construction of sand embankment in profile B, the post construction condition usually show the constant pore pressure, hence the constant effective stress, and outward lateral movement. These indicate the existence of time dependent undrained movement behaviour, especially in NC range. The previous mentioned mechanisms explain this behaviour as follows.

The implication of the constant effective stress with time is caused by the reduction in void ratio and pore pressure from consolidation settlement but the occurrences of additional time dependent undrained movements from many previous mentioned mechanisms, cause the increase in pore pressure. The latter causes the reduction in effective stress to compensate the gain in effective stress from consolidation. These are then cause the effective stress to become constant. The increase in shear strength will occur from the reduction of void ratio or water content. But the gain in shear strength will not be as expected resulting from the mechanisms causing time dependent undrained movements. Significant changes in soil structure, breaking of bonds form aging, and the breaking of chemical cementations, are the causes that lead to the unexpected low gain of undrained shear strength from consolidation. This process also induces excess pore pressure and causes the reduction in effective stress gaining from consolidation process as previously explained.

Profile C and Type III soft clay

This profile can be found in areas nearby the Chao Phraya river and places near the natural canals including the swamp areas. It usually has low ground elevation and containing natural canals. The Type III soft clay has about the same index properties with those of Type I soft clay. This Type III clay, however, is underlain by the medium dense silty sand layer or by the sandy clay layer before reaching the marine soils from profile A or B.

At Suvarnabhumi airport, based on 11 bore hole data and soil conditions report form Taothong (1985), the soil conditions contain profile A, B, and also C. Before the construction seem at feasibility stage, the area is covered with 90% of fish ponds. These conditions show very serious difficulties in the design for foundations and airfield as follows.
(1) The differences in soil profiles in the area, lead to non uniform soil conditions; and as the result, it may lead to problems of differential settlements in both pile foundation designs, especially those which are constructed in the condition of using different pile types and construction method in the same constructed facility, upon using the tip depths within 30 m, and air field design with unsuitable ground improvement method. Large settlement from previous mention mechanisms, and the occurrence of differential post construction settlement is important.

(2) Intensive ground water pumping is localized nearby areas near the airport, due to the lack of water supply, can cause not only the total settlement, but also the differential settlement. The differences in the piezometric elevations at the same depth and at different location will lead to different rate of land subsidence from consolidation within the airport. Deep seated settlement from soil layers at great depths also cause these problems in this case.

DEFORMATION CHARACTERISTICS

Consolidation Characteristics

Data from Rowe oedometer of both Type I and Type II soft clay show nonlinear stress strain characteristics (Figures 1 and 2), in NC range between \( \frac{\sigma'_{wc}}{\sigma'_p} \) between 1.2 to about 2 or 3. Addition pore pressure are seen between \( \frac{\sigma'_{wc}}{\sigma'_p} \) between 1.1 to 1.4 from previous mentioned mechanisms. The increase in total horizontal stress to insure the one dimensional movement of saturated clay (Sambhandharaksa and Aimdee, 2004) is required and causes the increase in \( K_0 \) with the increase in \( \frac{\sigma'_{wc}}{\sigma'_p} \).

![Stress-Strain Graph](image)

**Fig. 2 Stress-Strain from Rowe’s Oedometer of Type I Clay, at 7.5 – 8.50 m, (Aimdee, 2002)**

The compression ratio (CR) in nonlinear range is between 0.3 and 0.7. The maximum values are 0.5 for Type I clay and 0.7 for Type II clay. Type II shows more curvature which indicates more effects of undrained movement during consolidation. The ratio of recompression ratio and CR is over 10. In linear range, CR is 0.3.
Fig. 3 Stress-Stain from Rowe’s Oedometer of Type II Clay, at 9.0 – 10.00 m, (Aimdee, 2002)

Fig. 4 $\Delta u/\Delta \sigma_v$ versus $\log t/H^2$, from Rowe Oedometer of Type I Clay, at 6.00-7.00 m depth, $\sigma'_p = 120$ kPa (Aimdee, 2002)

The non-linearity of stress-strain curve leads to the limitations of Asaoka (1978)’s method for finding final consolidation settlement, especially when vertical drains were used. Upon using vertical drain, the hyperbolic method (Tan, 1996) is the better method and yields larger consolidation settlement and longer time for consolidation alone (Houngjing, 1998), based on data from AIT test section. Due to the presence of undrained movement during consolidation, data from lateral movement will effectively indicate the condition of post construction movements.
Fig. 5  $\Delta u/\Delta \sigma_v$ versus Log $t/H^2_{ds0}$ from Rowe Oedometer of Type II Clay, at 9.00-10.00 m depth, $\sigma'_p = 65$ kPa (Aimdee, 2002)

Fig. 6  $c_v$ versus Log $\sigma'_v/\sigma'_p$ from Type I Clay (Aimdee, 2002)

Addition Excess Pore Pressure

Data indicate the existence of additional excess pore pressure is shown in Figures 4 and 5 for Type I and Type II clay, at $\sigma'_v/\sigma'_p$ about 1.2.

Figures 6 and 7 show the values of the coefficient of consolidation ($c_v$) with $\sigma'_v/\sigma'_p$ they show $c_v$ to be constant about 1 m$^2$/year from both clays, in NC range.
**Fig. 7** $c_v$ versus Log $\sigma'_v/\sigma'_p$ from Type II Clay (Aimdee, 2002)

Figures 4 and 5, the term Time/\(H^2d_{so}\) in NC range, indicates the time factor in NC range as $c_v$ is constant. The curves in Figures 4 and 5 show the development of additional excess pore pressure about $\sigma'_v/\sigma'_p$ about 1.1 to 1.5. This indicate the presence of time dependent undrained movements from previous mentioned mechanisms. The increase in total horizontal stress, which leads to the increase in $K_0$ with $\sigma'_v/\sigma'_p$ in NC range (Figure 8), occurs as the results of undrained movement during one dimensional consolidation, and $K_0$ value also becomes constant in $\sigma-\epsilon$ portion which is in linear range. The increase in total horizontal stress is required to keep no lateral movement and the net undrained movement is zero to keep volumetric strain to be equal to axial strain (Sambhandharaks and Aimdee, 2004)

**Fig. 8** $K_0$ versus $\sigma'_v/\sigma'_p$ ratio in normally consolidated range for clay Type I and Type II (Kuroijjanawong, 2002)
Effect of Drainage Length

The existence of undrained creep and the effect drainage path’s length can be shown from comparing the consolidation data which have different drainage length. Figures 9 and 10 show the consolidation test from oedometer tests using conventional oedometer (Figure 9) which has double drainage and Rowe’s oedometer (Figures 10) which has single drainage. Longer drainage length will show more undrained creep effect, shown in Figures 10. This is seen from more nonlinear behavior in Type II clay’s curve. (Figures 10)

Fig. 9 $\sigma$-$e$ from conventional oedometer test of Type II clay at 7.50 - 8.50 m depth (Aimdee, 2002)

Fig. 10 $\sigma$-$e$ from Rowe oedometer test of Type II clay at 7.50-8.50 m depth (Aimdee, 2002)
The longer drainage length sample will require longer time for consolidation, undrained creep will, therefore, occur more and will lead to more curvation (more non linearity) of the stress-strain curve. The $\sigma$-$\epsilon$ curve in Figures 9 and 10, therefore, indicate indirectly the existence of undrained creep during consolidation. The data are compared with the same $\Delta P/P_0$ ratio at EOP duration.

These data, however, show the influence of undrained creep on $\sigma'_p$ value slightly, More undrained creep leads to slightly higher $\sigma'_p$.

Aimdee (2002) presented the data of unload and reload consolidation test using ordinary oedometer test, using EOP loading, but before unloading, 48 hours were allowed for aging, for Type II clay. The unloading process was done in linear range of the consolidation curve. It was found that the unloading curve is practically flat. The reload data show the slightly higher preconsolidation pressure compared to the final vertical stress before loading. However, the slightly non linearity $\sigma$-$\epsilon$ characteristics was still shown in reloading curve.

**Creep Characteristics**

Undrained creep in Type I and Type II clay are important. Due to higher plasticity, indicating more viscous behaviour, Type II clay exhibits more creep characteristics than that of Type I clay. Creep's behaviour has two types, consisted of undrained creep and drained creep (secondary compression).

**Undrained Creep Behavior**

Undrained creep test was perform on stress controlled TC at 70% applied shear stress level on both Type I and Type II clays. The findings were as follows.

1. **Type II clay’s sample** failed at 2 days at $\sigma'_{wc}/\sigma'_p$ about 1.2 after creep test. Type I clay also failed at longer time (Figures 11 and 12)

2. The creep (or viscous behaviour) is more in type II clay, especially in NC range (Figure 13). The Shigh and Mitchell (1968)'s model fits very well (Figures 11 and 12). Amazing the minimum "m" value (Figure13) is minimum and also at failure occurred when $\sigma'_{wc}/\sigma'_p$ about 1.2 to 1.4, in normally consolidated range, where the additional excess pore pressure occurs in Rowe's oedometer also occurred.

3. The findings in Figures 11 and 12 indicate the creep strength will be lower than 70% and also lead to the requirement of the design factor of safety to be more than 1.5 for preventing the additional undrained time dependent plastic flow, or the slope failure, in the NC range.

4. In all stress range and stress history, the "m" value is less than 1.0 (Figure 13). This indicates the serious problems of undrained creep and viscous effect in Bangkok clay. Furthermore, undrained creep process can cause failure of slope at post construction stage or the damages of infrastructures.
Drained Creep (Secondary Compression)

Drained creep's behaviour was obtained from Rowe's oedometer and TC after the undrained creep test. Figures 15 and 16 are results from one dimensional movement, and Figure 14 shows data from three dimensional movements. Results can be summarized as follows.

(1) The coefficient of secondary consolidation in term of strain (C_{ac}) is much higher in Type II clay than Type I clay.

(2) Both types of clay show characteristics of high sensitive nature and viscous characteristics, seen form the shape of the curve (Figures 14 to 16).

(3) Three dimensional movement has more drained creep's problems than one dimensional movement about 30%.

(4) $\sigma'_{vc}/\sigma'_p$ ratio of about 1.2 to 1.4 still have the peak value of C_{ac}. These ratios always show the locations of the peak values of CR, RR, and C_{ae} but minimum value of m.

![Graph showing creep strain over time for different OCR values.]

**Fig. 11** Relationship between Rate of Undrained Creep ($\dot{\epsilon}$) and Time of Soft Clay Type I, $\sigma'_p = 88$-94 kPa

The time effect behavior is shown the value of C_{ac}, where the C_{ac} increases time. The data indicate the significantly increase in C_{ac} with time, only in NC range, at $\sigma'_{vc}/\sigma'_p \sim 1.2$, in one dimensional loading.
Fig. 12 Relationship between Rate of Undrained Creep ($\varepsilon_\text{o}$) and Time of Soft Clay Type II, $\sigma'_p = 60-83$ kPa

Fig. 13 Relationship between $m$ and $\sigma'_v/\sigma'_p$ for both Type I and Type II clay
Fig. 14 $C_{ct}$ from TC Test versus log $\sigma' / \sigma'_p$ for Type I and Type II Clay

Fig. 15 Relationship between $C_{ct}$ versus log $\sigma' / \sigma'_p$ of Type I Clay

Fig. 16 Relationship between $C_{ct}$ versus log $\sigma' / \sigma'_p$ of Type II Clay
The equations between \( C_{ox} \) and CR of both types of clays, are shown below in equation 1 and equation 2 for Type I clay and Type II clay respectively.

\[
C_{ox}/CR = 0.037; R^2 = 0.9
\] (1)

\[
C_{ox}/CR = 0.049; R^2 = 0.9
\] (2)

The time effect behavior is shown the value of \( C_{ox} \), where the \( C_{ox} \) increases time. The data indicate the significantly increase in \( C_{ox} \) with time, only in NC range, at \( \sigma'_{wc}/\sigma'_p \sim 1.2 \), in one dimensional loading.

**SHEAR STRENGTH CHARACTERISTICS**

Aging effects, leaching process, and the breaking down of bonds and chemical cementations, which they lead to the changes of soil structure, and viscous effects, including thixotropy, play the important role in the undrained shear strength \( c_u \) behaviour, normalized effective stress envelope (NESE) at \( (\sigma'_i/\sigma'_3)_{max} \) \((q/\sigma'_p \text{ versus } p'/\sigma'_p)\) for both clay types.

**Normalized and Undrained Shear Strength and Its Limitations**

Due to the older age condition, and the more leaching effects, type I clay will show the more influences of soil structure’s changes in the normalized undrained shear strength \( c_u/\sigma'_{vc} \) than that of Type II clay which has more viscous effects and thixotropy.

The above factors lead to the inability for both clays to exhibit normalized shear strength in NC range. However, in the OC range, both clays exhibit normalized behaviour (Sambhandharaksa et al., 1999). The reduction in \( c_u/\sigma'_{vc} \) with the increase in \( \sigma'_{vc}/\sigma'_p \) in NC range, causes the unexpected low gain in shear strength in NC range for stability analysis.

Equation (3) shows the relationship between \( c_u/\sigma'_{vc} \) for stability analysis, obtained from Bjerrum (1972)'s corrected field vane strength, and OCR, in OC range. At \( \sigma'_p \) value, the \( c_u/\sigma'_{vc} \) is 0.26 (Sambhandharaksa et al., 2003). This relation can be used in stability analysis of sand embankment for both Type I and Type II clay.

\[
c_u/\sigma'_{vc} = 0.26(OCR)^{0.7}
\] (3)

The \( c_u/\sigma'_{vc} \) value decrease in NC range with the increase in \( \sigma'_{vc}/\sigma'_p \) in Type II clay, shown by Jeerangsuan et al (2003)

**\( c_u/\sigma'_{vc} \) in Normally Consolidated Range**

Jeerangsuan et al. (2003) made the analyses of the failures of the reconstructed embankments where the initial state of stress has \( \sigma'_{vc}/\sigma'_p \) more than 1.0. Two embankments were studies, located in Type II clay. Data were from Thamasiri (1985) and Boontharaksa (1992). Equation (4) shows the equation for the reduction of \( c_u/\sigma'_{vc} \) with the increase in \( \sigma'_{vc}/\sigma'_p \) for Type II clay. It is expected that more reduction will be in Type I clay.

\[
c_u/\sigma'_{vc} = 0.23 (\sigma'_{vc}/\sigma'_p)^{0.19}; R^2 = 0.07
\] (4)

The minimum analysis value of \( c_u/\sigma'_{vc} \) is 0.18. This leads the reduction from the value of 0.26 of about 40%. Upon comparing with \( c_u/\sigma'_{vc} \) of 0.23 the reduction is about 30%
Due to the different time of measured in situ vane shear strength, the influence of thixotropy is, therefore, included. Equation (4) leads to the consideration of the factor safety of 2.0 required to perform stability analysis in NC state, if \( c_u/\sigma'^{vc} \) of 0.26 was used. Upon using the lower factor of safety, serious time dependency undrained settlements and outward time dependent lateral movements will occur and can cause damages to the embankment, if failure does not occur.

**Data From TC Tests**

Laboratory results from TC tests were also confirmed the above behaviour. The undrained strength from TC can be used in tunneling and sheet pile excavation problems, as the value of \( c_u/\sigma'^{vc} \) in OC range is independent of total stress path directions (Samfhandharaksa et al. 2003), Kurojjanawong (2002) provide \( c_u/\sigma'^{vc} \) and OCR relationships for both Type I and Type II clay in equations (5) and (6) respectively.

\[
c_u/\sigma'^{vc} = 0.35(OCR)^{0.6}
\]

\[
c_u/\sigma'^{vc} = 0.37(OCR)^{0.7}
\]

The triaxial extension strength is about 80% of the compression strength.

The effective stress parameters has the effective cohesion intercept \( (c') \) at \( (\sigma'$\nu/\sigma'$s)_{max} \) depending upon the magnitude of \( \sigma'$p \) and hence the depth. The values of \( \phi' \) and \( c'/\sigma'$p \) at \( (\sigma'$\nu/\sigma'$s)_{max} \) are shown in Table 1. As the significantly changes in \( c_u/\sigma'^{vc} \) with the increase in the value \( \sigma'^{vc}/\sigma'$p \) in NC range, occur in Type I clay, it leads to the change in normalized effective stress \( (\phi', c'/\sigma'$p) \) envelope in NC range. But for Type II clay, due to the smaller changes, the values of \( \phi' \) and \( c'/\sigma'$p \) do not change.

**Table 1 Summary of Normalized Effective Stress Envelope (NESE)**

<table>
<thead>
<tr>
<th>Clay Type</th>
<th>( \phi' ) (degrees)</th>
<th>( c'/\sigma'$p )</th>
<th>Stress History</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type I Clay</td>
<td>34 - 36</td>
<td>0.02 - 0.025</td>
<td>OC</td>
</tr>
<tr>
<td>( I_p \sim 45%; w_n \sim 70-80% )</td>
<td></td>
<td></td>
<td>( p'/\sigma'$p &lt; 0.4 )</td>
</tr>
<tr>
<td>( I_L \sim 0.8 )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type I Clay (NC range)</td>
<td>11 to 12</td>
<td>0.16</td>
<td>NC, ( p'/\sigma'$p &gt; 0.4 )</td>
</tr>
<tr>
<td>Type II Clay</td>
<td>23 (^\circ) to 26 (^\circ)</td>
<td>0.12 - 0.13</td>
<td>both NC and OC</td>
</tr>
<tr>
<td>( I_p \sim 70-80% )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( w_n \sim 90-120% )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( I_L \sim 1.0 )</td>
<td></td>
<td></td>
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</tbody>
</table>

The reduction in \( c_u/\sigma'^{vc} \) with the increase in \( \sigma'^{vc}/\sigma'$p \) from TC are shown in Table 2

**Table 2 Reduction in \( c_u/\sigma'^{vc} \), in NC range with the increase in \( \sigma'^{vc}/\sigma'$p \)**

<table>
<thead>
<tr>
<th>Clay Type</th>
<th>( \sigma'^{vc}/\sigma'$p )</th>
<th>( c_u/\sigma'^{vc} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type I Clay</td>
<td>1.02</td>
<td>0.34</td>
</tr>
<tr>
<td></td>
<td>1.40</td>
<td>0.29</td>
</tr>
<tr>
<td></td>
<td>3.0</td>
<td>0.26</td>
</tr>
<tr>
<td>Type II Clay</td>
<td>1.15</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>1.40</td>
<td>0.33</td>
</tr>
<tr>
<td></td>
<td>3.0</td>
<td>0.30</td>
</tr>
</tbody>
</table>

37
CONCLUSIONS

The engineering properties of soft Bangkok clays were summarized. The influences of index properties, of aging effects, of leaching effects, of chemical cementations from deep well pumping, of thixotropy, and of viscous effects, were presented. The sudden changes of soil structure and undrained viscous effect were considered to cause time dependent undrained movements. Discussion were made on deformations and strength characteristics, based on recent research results obtained after the construction of the airport has begun.

Problems from viscous effects, from leaching effects, from aging effects, and from the breaking down of bonds and chemical cementations, lead to the following detrimental effects.

(1) Large deformations in NC range from consolidation, and from time dependent undrained deformation are expected. Undrained creep, sudden breaking down of soil structure, undrained plastic deformation and plastic consolidation will lead to large time dependent (outward) lateral movement, and no significant changes in pore pressure.

(2) In NC range, post construction settlement will include time dependent undrained movements, consolidation, and drained creep where $C_{ac}$ significantly increases with time, in Type II clay

At the airport, where the nonuniform soil condition and the very difficult soil conditions occur, upon even using vertical drains may lead to exceptionally long time required for settlement to be minimized. Settlement from deep well pumping may not be uniform. The localized deep well pumping in nearby areas will lead to the difference in piezometric elevations in the first sand located at different depths. Moreover, the declinations of pore pressure in the soft an stiff clay including those deep seated layers, which are underlying the first sand layer, are still in progress.

The ratios of $\sigma_{v}'/\sigma_{p}'$ in NC range, which cause most detrimental effects are about 1.1 to 1.4. Problems are expected to be less when $\sigma_{v}'/\sigma_{p}'$ is greater than 3.0. Unloading and reloading process also make the problem be less, provided that excess pore pressure is completely dissipated before unloading.

REFERENCES:


SPECIAL LECTURES
STRENGTH, COMPRESSIBILITY AND FLOW PARAMETERS FOR PVD IMPROVEMENT OF SOFT BANGKOK CLAY AT SBIA PROJECT

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ABSTRACT

This paper presents the back-analyzed compressibility and flow parameters obtained from 3 full scale test embankments, namely: TS1, TS2 and TS3 constructed on soft Bangkok clay with Prefabricated Vertical Drains (PVD) at the site of the Second Bangkok International Airport (SBIA) in Thailand. The PVD spacings ranged from 1.0 m, 1.2 m and 1.5 m in square pattern for TS3, TS2 and TS1, respectively. The back-calculated discharge capacities vary from 30 m³/year to 90 m³/year for the 3 test embankments. The measured increases in undrained shear strengths with depth are in agreement with the values calculated from the SHANSEP technique. The secondary compression ratio, $C_a$, was found to be 0.018 or within the normal values for marine clays. The $C_h$ (field) values at 4 m and 10 m depths are higher than the weakest soil at 6 m depth. The back-calculated $C_h$ (field) values range from 3 to 8 m²/yr and the $C_h$ (field)/$C_h$ (lab) ranges from 4 to 5. The degree of consolidation estimated from the pore pressure dissipation measurements agreed with those obtained from settlement measurements. The water content reductions from field measurements were also in good agreement with the computed values from the consolidation settlements. The full scale study confirmed that the magnitudes of consolidation settlements increased with the corresponding decrease of PVD spacing at a particular time period. Finally, the results of the full scale study has proven the effectiveness of PVD for the improvement of soft Bangkok clay.

INTRODUCTION

To reduce the time for consolidation settlement of fine-grained soils, prefabricated vertical drains (PVD) can be used together with preloading to accelerate consolidation settlements. The PVD is a slender, synthetic drainage element comprised of a drainage core wrapped in a geotextile filter. Excess pore water pressures, created by preloading, leads to preferential flow in the horizontal direction towards the PVD and along it vertically into the permeable drainage layers. Therefore, the PVD installation reduces the length of drainage paths, and, consequently, shortens the time required to complete the consolidation process.

The Asian Institute of Technology (AIT) was engaged by the Airport Authority of Thailand to undertake the full scale test of PVD alternatives for the Second Bangkok International Airport Project (SBIA). Besides identifying the PVD types suitable for the
project and verifying the actual field performance of PVD, the full scale study also examined the effects of using different PVD types and spacings. The full scale study also confirm the effectiveness of the prediction and monitoring methods such as the comparison of the degree of consolidation using pore pressure measurement versus settlement measurement, comparison of the actual water content reduction with computed values, and comparison of the actual increase in shear strength with the predicted values. Three full scale test embankments (TS1 to TS3) were constructed in stages on PVD improved soft Bangkok clay at Nong Ngu Hao, Thailand with PVD spacings of 1.5, 1.2 and 1.0 m, respectively, in a square pattern (Table 1). All PVDs were installed to 12 m depth. This study is a continuation of earlier publications (Bergado et al. 1996a; Bergado et al. 1997) concerning the results and analyses of the full scale study.

SITE AND SOIL CONDITIONS

The test site is located approximately 30 km east of the City of Bangkok at the SBIA land. The generalized soil profile and soil properties are shown in Figure 1. The soil profile is relatively uniform consisting of a 2 m thick weathered crust overlying very soft to soft clay approximately 10 m thick. Underlying the soft clay is a medium stiff clay layer about 4 m thick followed by a stiff clay layer extending down to 24 m depth which is in turn underlain by a layer of dense sand (30 to 50 SPT N-values). The profiles of soil strength and compressibility parameters determined by laboratory and field tests are also shown in Figure 1. The natural water contents are reasonably uniform across the site, and lie close to the liquid limit between depths of 2 and 16 m. Most of the Atterberg limit values lie above the A-line on the plasticity chart, confirming the high plasticity of the Bangkok clay. The groundwater table varies between 0.5 to 1.0 m below ground.

SELECTION AND INSTALLATION OF PREFABRICATED VERTICAL DRAIN

Three factors are generally used to describe the effects on the ideal required discharge capacity of PVD as calculated from the theory of consolidation (Bergado et al., 1996b). These factors are the influence of time, the influence of deformation, and the influence of filtration and clogging. Based on the results of the tests on PVD types, three PVD types were installed in the full scale test embankments, namely: Flodrain in TS1, Castle Board drain in TS2, and Mebra drain in TS3 as tabulated in Table 1. The installation of PVD is implemented by means of the anchor plate and mandrel in which PVD is driven into the desired depth. Subsequently, the mandrel is pulled out and the PVD remained anchored in place after cutting it 0.15 m above the surface of the sand blanket.

Table 1. PVD Types Installed in the Test Embankments

<table>
<thead>
<tr>
<th>Test Embankments</th>
<th>PVD Pattern</th>
<th>PVD Spacings (m)</th>
<th>PVD Types</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS1</td>
<td>Square</td>
<td>1.5</td>
<td>Flodrain (FD4-EX) (Studded core not fixed to filter)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Castle Board (CS1) (Groove core fixed to filter)</td>
</tr>
<tr>
<td>TS3</td>
<td>Square</td>
<td>1.0</td>
<td>Mebra (MD-7007) (Grooved core not fixed to filter)</td>
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</tbody>
</table>
Table 2. Sand Blanket Specification

<table>
<thead>
<tr>
<th>Sieve Size No.</th>
<th>Percent Finer</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>95-100</td>
</tr>
<tr>
<td>10</td>
<td>70-100</td>
</tr>
<tr>
<td>40</td>
<td>20-60</td>
</tr>
<tr>
<td>200</td>
<td>0-10</td>
</tr>
</tbody>
</table>

Permeability (mm/sec) 5 x 10^{-2} to 5 x 10^{-1}

Fig. 1 General Soil Profile and Properties
TEST EMBANKMENT CONSTRUCTION AND STAGE LOADING

In the area of the 3 test embankments, the original ground was cleared of grass roots and excavated to −0.3m (Mean Sea Level). Each test embankment is 40 m x 40 m in plan dimensions with 3H:1V side slopes and a final height of 4.2 m (Figure 2). A sand drainage blanket of 1.0 m thickness was laid on the excavated ground. The specifications for sand blanket is given in Table 2. After the PVD installation, the thickness of the sand drainage blanket was increased to 1.5 m. Then, clayey sand was used to raise the embankment to 4.2 m (i.e., 75 kPa of surcharge) in stages. As shown in Figure 3 for TS3, Stage 1 loading was up to 18 kPa, Stage 2 was taken to 45 kPa, followed by Stage 3 to 54 kPa and Stage 4 to 75 kPa (4.2 m fill height). A berm width of 5 m and 1.5 m high was included when the surcharge increased from 45 to 54 kPa (see Figure 3). After the test embankment was preloaded to a certain height in the stage loading, a waiting period is needed in order for the underlying soft clay foundation to gain additional shear strength in order to achieve a minimum factor of safety of 1.3 against slope failure without machinery live load. The waiting period was 30 days for TS3 with 45 kPa surcharge which was subsequently increased to 54 kPa. The design waiting period was 105 days for the embankment when the surcharge increased from 54 to 75 kPa. The factor of safety without considering short term condition of machinery live load (5 kPa live load) was generally in the range higher than 1.30 as shown in Figure 3. The long term factor of safety should be higher due to excess pore pressure dissipation in the underlying clay layers.

A section of the test embankment, TS3, is shown in Figure 4. The PVDs were installed to a depth of 12 m on a square pattern with 1.0 m spacing for TS3. The mandrel was rectangular in cross section with a thickness of 6 mm and outside dimensions of 150 mm by 45 mm. Rectangular shaped anchoring shoes with dimensions of 150 mm by 45 mm were utilized. Construction commenced in April 1994 and was completed 9 months later. The fill material was compacted to an average bulk unit weight of 18 kN/m³. The same dimensions and procedures were followed for the other test embankments, TS1 and TS2.

Fig. 2 Locations of Boreholes and Test Embankments TS, TS2 and TS3
SUMMARY OF THE STABILITY ANALYSIS

<table>
<thead>
<tr>
<th>POINT</th>
<th>THICKNESS (m)</th>
<th>LOAD (kPa)</th>
<th>DURATION (Days)</th>
<th>FACTOR OF SAFETY</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>with 5 kPa Load</td>
</tr>
<tr>
<td>C</td>
<td>2.5</td>
<td>45</td>
<td>30</td>
<td>1.18</td>
</tr>
<tr>
<td>D</td>
<td>2.5</td>
<td>45</td>
<td>30</td>
<td>1.33</td>
</tr>
<tr>
<td>E</td>
<td>3.0</td>
<td>54</td>
<td>105</td>
<td>1.23</td>
</tr>
<tr>
<td>F</td>
<td>3.0</td>
<td>54</td>
<td>105</td>
<td>1.54</td>
</tr>
<tr>
<td>G</td>
<td>4.0</td>
<td>72</td>
<td></td>
<td>1.56</td>
</tr>
</tbody>
</table>

Calculated Strength Gain and Settlement at the End of Each Loading Stage

<table>
<thead>
<tr>
<th>POINT</th>
<th>$\Delta \sigma_v$ (kPa)</th>
<th>$\Delta \sigma_{v}/\Delta q_e$</th>
<th>$\Delta \sigma_{v}/\Delta u_p$</th>
<th>$S_c$ (cm)</th>
<th>$S_c/S_{cf}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>D</td>
<td>11.4</td>
<td>0.25</td>
<td>0.25</td>
<td>22</td>
<td>0.17</td>
</tr>
<tr>
<td>F</td>
<td>35.1</td>
<td>0.65</td>
<td>0.83</td>
<td>65</td>
<td>0.50</td>
</tr>
</tbody>
</table>

$\Delta \sigma_v$: Increase of effective stress at calculated time
$q_e$: Embankment load at calculated time
$\Delta u_p$: Excess pore pressure just after adding the additional load including the remaining pore pressure from the previous stage
$S_c$: Consolidation settlement at calculated time
$S_{cf}$: Final consolidation settlement at 72 kPa load = 130 cm

Fig. 3 Summary of Stability and Settlement Analyses for Embankment TS3 (with 1.0 m Drain Spacing)

Fig. 4 Test Embankment TS3 (4.2 m Height)
FIELD PERFORMANCE OF TEST EMBANKMENT

A field monitoring program was established to monitor surface and subsurface settlements, lateral movements, and excess pore pressures. In total, 20 settlement plates, 2 inclinometers and 21 piezometers were installed in each test embankment. The surface and subsurface settlement gauges were installed near the center of the test embankment. The subsurface settlement gauges and the piezometers were installed at 2 m vertical interval. The piezometers were installed at locations between PVDs. The plan and section views showing the embankment instrumentation are given in Figure 5.

Similar trends of settlements were observed in all 3 test embankments. A typical settlement characteristics is shown in Figure 6 taken from TS2 at the surface and at depth intervals of 2m for subsurface settlements. Most of the compression occurred at 2 m to 8 m depth corresponding to the very soft clay layer. The magnitude of surface settlements increased from TS1 to TS3, which corresponds to decreasing PVD spacing of 1.5 m, 1.2 m and 1.0 m as shown in Figure 7. Thus, the test embankment with closest PVD spacing yielded the fastest settlement. A comparison of the surface settlement profiles was made for the three areas, and all were observed to be quite symmetrical.
In each embankment, two slope indicators labeled I1 and I2 were installed. The inclinometer I1 was located at the outermost edge of the embankment at a distance of 20 m from the center while the inclinometer I2 was installed at the shoulder at maximum fill height. The results of inclinometer I2 measurements in TS1 are shown in Figure 8. Most of the lateral movements occurred at 2 to 8 m depth in the very soft to soft clay layer.

The total pore pressures were measured redundantly by pneumatic, hydraulic, and open standpipe piezometers. The readings were corrected for settlements at piezometer tips. Figure 9 shows the data measured by pneumatic piezometers in TS1. The pneumatic piezometers PP2, PPS4, PPS6 and PP8 were installed at depths of 2 m, 4 m, 6 m and 8 m, respectively. Although the rate was slow and delayed, the dissipation of excess pore pressures definitely occurred.
ESTIMATION OF TOTAL SETTLEMENTS

Terzaghi's one-dimensional consolidation theory was used to estimate the one-dimensional primary consolidation settlements. In Figure 7, the calculated total settlements using one-dimensional consolidation theory for TS1, TS2 and TS3 are shown as solid lines in the time-settlement plots which are similar to the corresponding measured values plotted in dotted lines. The data used in the 1-D settlement calculation of test embankment TS3 are tabulated in Table 3.

Fig. 8 Lateral Deformation with Depth (TS1-I2)

Fig. 9 Pore Pressures from Pneumatic Peizometers Corrected for Settlements (TS1)
Table 3. CR, RR and \( \sigma'_{p} \) Values used in 1-D Consolidation Settlement Analysis

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>CR</th>
<th>RR</th>
<th>( \sigma'_{p} ) (kPa)</th>
<th>Settlement (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-2</td>
<td>0.30</td>
<td>0.06</td>
<td>60</td>
<td>0.22</td>
</tr>
<tr>
<td>2-4</td>
<td>0.40</td>
<td>0.08</td>
<td>55</td>
<td>0.29</td>
</tr>
<tr>
<td>4-6</td>
<td>0.50</td>
<td>0.10</td>
<td>50</td>
<td>0.40</td>
</tr>
<tr>
<td>6-8</td>
<td>0.50</td>
<td>0.10</td>
<td>55</td>
<td>0.35</td>
</tr>
<tr>
<td>8-10</td>
<td>0.45</td>
<td>0.09</td>
<td>75</td>
<td>0.21</td>
</tr>
<tr>
<td>10-12</td>
<td>0.35</td>
<td>0.07</td>
<td>90</td>
<td>0.12</td>
</tr>
<tr>
<td>12-16</td>
<td>0.30</td>
<td>0.06</td>
<td>124</td>
<td>0.15</td>
</tr>
</tbody>
</table>

Note: CR = Compression ratio
RR = Recompression ratio

PORE PRESSURE PROFILE

The measured pore pressure profile for TS3 indicated that the excess pore pressure has fully dissipated. Figure 10 illustrates the measured and assumed pore pressure profiles with depth for TS3. In this figure, line ABC is the dummy pore pressures and line DEF represent the theoretical pore pressure immediately after loading with 75 kPa surcharge. The dummy pore pressures were obtained at the unloaded part of the site and were located away from the influences of the test embankments. The dummy readings represent the initial pore pressure conditions before preloading which indicated the pore pressure drawdown due to excessive withdrawal of groundwater from the underlying aquifers. In Figure 10, the theoretical total pore pressure profile immediately after 75 kPa loading is indicated by curve DEF. The measured pore pressures after one year of preloading in June, 1995 are indicated by curve AGHIJ. Curve MNPQ represent the average total pore pressure readings in February, 1996. In comparison with the dummy readings (curve ABC) which represented the initial piezometric pressures, it can be observed that there is recharge from 8 to 12 m depth. Thus, the recharge extends to the PVD zone down to 12 m depth. The hydrostatic pore pressure without any drawdown is indicated by line AKL. The measured excess pore pressure profile as of February 1996 corresponding to curve MNPQ which indicate full dissipation of excess pore pressures.

CONSOLIDATION WITH PVD

Barron (1948) first presented the solution to the problem of consolidation of a soil cylinder containing a central sand drain. Hansbo (1979) recommended modifications to the solution which dealt mainly with simplifying assumptions due to the physical dimensions, characteristics of PVD, and the effects of PVD installation. The compressibility and flow parameters have been discussed by Bergado et al (1996a) which include: \( D_{e} \), the equivalent diameter of a unit PVD influence zone; \( d_{w} \), the equivalent diameter of PVD; \( k_{h} \), the horizontal permeability in the undisturbed soil; \( k_{s} \), the horizontal permeability of smeared zone; \( d_{s} \), the diameter of the smeared zone; \( z \), the distance from the drainage end of the PVD; \( L \), the length of PVD for one way drainage and half of the PVD length for double drainage; and \( q_{w} \), the discharge capacity of the PVD at hydraulic gradient of 1.
It has been shown from back-analysis of an instrumented test embankment on a similar clay in Canada (Crawford et al., 1992) that the calculated rates of consolidation are not sensitive to common methods of establishing $D_e$, nor to the value of $q_{uw}$, and that the most significant design parameter is the coefficient of consolidation in radial drainage, $C_h$. In this study, on the other hand, the aforementioned compressibility parameters played a vital role in the successful prediction of consolidation settlements. The calculated surface settlements using one-dimensional consolidation with radial drainage (Hansbo, 1979) are also plotted in Figure 7. The predictions agreed with the measured values.

DEGREE OF CONSOLIDATION

The degree of consolidation of the clay layers below the test embankments were calculated both from pore pressure dissipation and from the settlements of the test embankments. The pore pressure dissipation with depth is presented in Figure 10. If the compression ratio is assumed to be constant, then the degree of consolidation can be obtained from the measured pore pressures. The corresponding values of the degree of consolidation can also be obtained from the measured settlements. Figure 11 compares the calculated degrees of consolidation. The degrees of consolidation obtained from settlement measurements were confirmed by the corresponding values from excess pore pressure measurements.

The degree of consolidation obtained from pore pressures ($U_p$) is consistently less than that from settlements ($U_s$). Similar observation was reported earlier by Holtz and Broms (1972). The delay in calculated degree of consolidation from pore pressure observations obtained here is in accordance with Mikasa consolidation theory (Mikasa, 1965). During the compression and rearrangement of the soil structure, the excess pore pressures were maintained at higher levels similar to the observations of Holtz and Broms (1972) and Crawford et al. (1992).

Fig. 10  Pore Pressure Profiles for Settlement Computations (TS-3)
SECONDARY COMPRESSION

Further insight into the settlement characteristics can be obtained by plotting the rate of settlement versus inverse of time (Figure 12). Over a period of approximately 20 months, the rate of settlement reduced very quickly with time in an exponential manner. However, beyond a period of 20 months, the rate seemed to reach an asymptotic decay with a slope of 25:1 which could be attributed to the secondary compression. Assuming that the secondary compression is a linear function of log time, then the slope of this line is the product of the soil thickness, H, and the secondary compression ratio, \( C_\alpha \). With \( H = 14 \) m, \( C_\alpha \) can be obtained as 0.018. This value of \( C_\alpha \) is within the range of 0.01 to 0.02 as suggested by Mesri (1973) for marine clays. The \( C_\alpha \) values for soft Bangkok clay ranges from 0.011 to 0.021 (Kulatilake, 1978; Ong, 1983) with \( C_\alpha/C_c \) value of 0.04 to 0.05 (Cox, 1981; Wongprasert, 1990).

The consolidation settlement-log time plots were plotted by Bergado et al (1997) to estimate the secondary consolidation parameter, \( C_\alpha \). The final linear portion is almost achieved and based on the slope of this curve of 25:1 (or \( C_\alpha = 0.018 \)), as obtained from the rate of settlement-inverse time plot, the observations seems to approach a linear portion (Bergado et al, 1997). As plotted in curve MNPQ in Fig. 10, the excess pore pressures have fully dissipated.

REDUCTION OF WATER CONTENT

Figure 13 illustrates the reduction of water content with depth for TS3 in February, 1996 after 660 days of preloading compared to the mean values of the initial water contents measured in February, 1994. The back-calculated values of water content from settlements in March, 1996 are also plotted in Figure 13 for TS3 which is in agreement with the measured water content data.

![Figure 13](image)

Fig. 11 Comparison of Degrees of Consolidation from Settlement \( (U_s) \) and Pore Pressures \( (U_p) \)
Under the test embankment, substantial reduction in water content is noted. For the very soft clay from 2 to 6 m depth, the water content reduction is consistent, i.e., higher reduction corresponding to smaller PVD spacing in TS3. The reduction in water content in TS3 is more than 20%.

INCREASE OF SHEAR STRENGTH

The increase in undrained shear strength was predicted by the SHANSEP technique (Ladd, 1991) as follows:

\[
\left( \frac{S_u}{\sigma_{vo}} \right)_{OC} = \left( \frac{S_u}{\sigma_{vo}} \right)_{NC} \text{OCR}^m
\]

For soft Bangkok clay:

\[
\left( \frac{S_u}{\sigma_{vo}} \right)_{NC} = 0.22 \quad \text{and} \quad m = 0.8
\]

Therefore,

\[
\left( \frac{S_u}{\sigma_{vo}} \right)_{OC} = 0.22 (\text{OCR})^{0.8}
\]

The predicted increases in undrained shear strengths are indicated by solid lines in Figure 14. The corrected undrained shear strengths measured by field vane shear tests in March 1996 are also plotted for comparison. As shown in Figure 14, there is excellent agreement between the measured and predicted data with regards to the increase in undrained shear strength due to preconsolidation and drainage. The SHANSEP technique was also used in the calculation of the increase in undrained shear strength for the stability analyses as demonstrated in Figure 3.

Fig. 12 Rate of Settlement Versus Inverse Time Plot-TS3
BACK CALCULATION $C_h$ VALUES FROM PORE PRESSURE MEASUREMENTS

From the equation of Hansbo (1979) for consolidation of PVD, the following equation can be derived:

$$1 - \frac{\Delta u_t}{\Delta u_0} = 1 - \exp(-\alpha t)$$  \hspace{1cm} (4)

$$\alpha = \frac{8 C_h}{D_e^2 F}$$  \hspace{1cm} (5)

$$F = F_n + F_t + F_s$$  \hspace{1cm} (6)

$$F_n = \log_e (D_e/d_w) - 0.75$$  \hspace{1cm} (7)

$$F_s = (k_h/k_x - 1) \log_e (d_e/d_w)$$  \hspace{1cm} (8)

$$F_t = \pi z (2L-z) k_h/q_w$$  \hspace{1cm} (9)

where $\Delta u_0$ is the excess pore pressure at reference time of $t = 0$; $\Delta u_t$ is the excess pore pressure at time $t$; $D_e$ is the effective diameter of unit cell of drain; $F$ is the resistance factor for the effects of spacing ($F_n$), smear ($F_s$), and well-resistance ($F_t$); $d_w$ is the equivalent diameter of PVD; $k_h$ is the horizontal permeability in the undisturbed soil; $k_x$ is the horizontal permeability of the smeared zone; $z$ is the distance from the drainage end of the drain; $L$ is the length of the PVD for one way drainage and is half of drain length for drainage boundary at both ends; and $q_w$ is the discharge capacity of the PVD at hydraulic gradient of 1.

Fig. 13 Back-Calculated Water Contents from Settlements-TS3

55
From Equation 4 we can get,

$$\ln \frac{\Delta u_0}{\Delta u_1} = \alpha t$$  \hspace{1cm} (10)

Therefore, the values of $\alpha$ can be obtained as the slope of the plot $\ln (\Delta u_0/ \Delta u_1)$ vs $t$. Having the values of $\alpha$, the coefficient of horizontal consolidation $C_h$ can be calculated from Equation 5.

The back-calculated $C_h$ values from the hydraulic piezometer data are plotted in Figure 15 against the increase in effective stress. It can be seen that the $C_h$ values decreased consistently with the increase in effective stress (with the progress of consolidation) for all depths. The weathered crust (2 m depth) has the highest $C_h$ value and the weakest soil at 6 m depth has the lowest $C_h$ value. The $C_h$ values at 4 m and 10 m are higher than at 6 m depth.

![Graph showing field vane shear strength measured below in embankment-TS3](Image)

**Fig. 14** Field Vane Shear Strength Measured Below in Embankment–TS3
BACK CALCULATION OF $C_h$ VALUES FROM SETTLEMENT MEASUREMENTS

Equation 4 can be written in terms of settlement as follows:

$$\frac{S_t}{S_f} = 1 - \exp(-\alpha t)$$

(11)

where $S_t$ is the consolidation settlement, $S_f$ is the final settlement. The other terms have been defined previously. From Equation 7, the following equation can be derived:

$$\ln \frac{S_f}{S_f - S_t} = \alpha t$$

(12)

Therefore, the $C_h$ values can be obtained from the slope, $\alpha$, of the $\ln \frac{S_f}{S_f - S_t}$ vs $t$ plot. The average values of $C_h$ (at 12 m depth PVD improved zone) calculated from embankment TS3 were obtained as a function of the increased effective stress. The average $C_h$ value at the end of construction is about 3 m$^2$/year and tend to reduce to less than 0.5 m$^2$/year at the full increase of effective stress of 75 kPa. Thus, the $C_h$ values obtained from settlement are slightly higher than those estimated from pore pressure back-calculations.

Fig. 15 Plot of Back-Calculated $c_h$ Values from Pore Pressure versus Effective Stress
FINAL SETTLEMENT AND COEFFICIENT OF COMPRESSIBILITY

Asaoka (1978) proposed a graphical method to determine the final settlements based on observational procedures. The observed time-settlement curves plotted to an arithmetic scale were divided into equal time interval, Δt. The settlements $S_i$ corresponding to $t_i$ are read off and their relation of $(S_i - S_{i-1})$ is plotted in the coordinate system as shown in Figure 16 for test embankment TS-3 where Δt is taken as 30 days. A straight line is fitted through the points. The slope of this line is $β$, and its intercept with the ordinate axis is $β_0$. The 45° line ($S_i = S_{i-1}$) is also plotted. The point where the plotted line intersect the 45° line yields final settlement, $S_f$. Subsequently, the final settlements for test embankments TS-1, TS-2, and TS-3 were obtained as 1.70 m, 1.70 m, and 1.72 m, respectively. The same magnitude of final settlements of three test embankments indicated the uniform soil profile at the test site. Similar conclusions on the uniformity of soft soils in Nong Ngú Hao site were made by NGI (1992).

The compression coefficient, $M_v$, can be back-calculated from the final settlement as follows:

$$M_v = \frac{\Delta H}{H(\Delta \sigma')}$$  \hspace{1cm} (13)

where $\Delta H$ is the final settlement of the considered soil later having thickness of $H$ and $\Delta \sigma'$ is the increase of final effective stress. The values of $\Delta H$ can be calculated from Asaoka’s method and then the corresponding values of $M_v$ for the subsoils under test embankment TS-2 are computed and tabulated in Table 4.
Table 4 Calculated Values of Final Settlement and Coefficient of Compressibility

<table>
<thead>
<tr>
<th>Soil layers</th>
<th>Weather crust</th>
<th>Soft clay</th>
<th>Very soft to soft</th>
<th>Soft to medium clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m)</td>
<td>0-2</td>
<td>2-4</td>
<td>4-8</td>
<td>8-12</td>
</tr>
<tr>
<td>ΔH (m)</td>
<td>0.22</td>
<td>0.24</td>
<td>0.72</td>
<td>0.30</td>
</tr>
<tr>
<td>$M_v$ ($10^3$m²/kN)</td>
<td>1.47</td>
<td>1.60</td>
<td>2.40</td>
<td>1.00</td>
</tr>
</tbody>
</table>

THE $C_h \sim q_w$ RELATIONSHIP

Using Asaoka’s approach for radial consolidation with the use of PVD, the horizontal coefficient of consolidation, $C_h$, can be derived as follows:

$$C_h = \frac{(1-\beta)D_e^2F}{8\beta\Delta t}$$  \hspace{1cm} (14)

where the terms have been defined previously. The value of $C_h$ cannot be obtained directly from Equation 14 because of the unknown value of $k_b$ existing in the factor $F$ as seen in Equations 6 and 9.

Assuming the compression coefficient in vertical direction, $M_v$ as equal to that in horizontal direction, $C_h$, the following expression can be written:

$$k_s = M_vC_h \gamma_w$$  \hspace{1cm} (15)

Substituting for $k_b$ from Equation 15, for $F$ using Equations 6 to 9 into Equation 14, the following equation can be derived:

$$C_h = \frac{F_e + F_i}{C_i - C_2}$$  \hspace{1cm} (16)

$$C_i = \frac{8\beta\Delta t}{(1-\beta)D_e^2}$$  \hspace{1cm} (17)

$$C_2 = \pi z(2L-z)M_v\gamma_w$$  \hspace{1cm} (18)

Equation 16 consists of four unknowns: $k_b/k_s$, $d_s/d_a$, $q_w$, and $C_h$. Hence, the back-calculated values of $C_h$ will be dependent on the assumed values of the other three unknowns. Furthermore, by assuming the diameter of the smeared zone, $d_s$, as twice as equivalent diameter of the mandrel ($d_a$) as suggested by Hansbo (1987) and confirmed for soft Bangkok clay by Bergado et al (1991), the relationship between $C_h$ and $q_w$ can be obtained for different values of the smear ratio, $k_b/k_s$.

From the measured settlements, the $\beta$ values together with final settlements for the soft clay later at depth interval from 4 m to 8 m under test embankments TS-1, TS-2, and TS-3 were obtained as illustrated in Figures 17 to 19, respectively, in which the time interval, $\Delta t$, of
30 days were used. Using the aforementioned assumptions of \(d_s\) and \(k_b/k_s\) and the other parameters as tabulated in Table 5, the \((C_h \sim q_w)\) relationships for different ratios of \(k_b/k_s\) are plotted in Figures 20 to 22 for the corresponding embankments TS-1, TS-2, and TS-3, respectively. The calculated results are also tabulated in Table 6.

**Table 5 Data for PVD Used in Three Test Embankments**

<table>
<thead>
<tr>
<th>1. Dimensions of PVD</th>
<th>(a = 0.004 \text{ m; } b = 0.100 \text{ m; } d_w = 0.052 \text{ m})</th>
</tr>
</thead>
<tbody>
<tr>
<td>2. Dimensions of Mandrel</td>
<td>(a_m = 0.045 \text{ m; } b_m = 0.150 \text{ m; } d_m = 0.046 \text{ m})</td>
</tr>
<tr>
<td>3. Assumed Diameter of Smear Zone</td>
<td>(d_s = 2d_m = 0.093 \text{ m})</td>
</tr>
</tbody>
</table>

**Table 6 Back-Calculated Results of \(C_h\sim q_w\) Relationship**

<table>
<thead>
<tr>
<th>(q_w) ((\text{m}^2/\text{yr}))</th>
<th>(C_h) ((\text{m}^2/\text{yr})) for Embankment TS1</th>
<th>(C_h) ((\text{m}^2/\text{yr})) for Embankment TS2</th>
<th>(C_h) ((\text{m}^2/\text{yr})) for Embankment TS3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(k_s/k_s = 3)</td>
<td>(k_s/k_s = 5)</td>
<td>(k_s/k_s = 7)</td>
</tr>
<tr>
<td>4</td>
<td>-66.9</td>
<td>-87.0</td>
<td>-107.1</td>
</tr>
<tr>
<td>5</td>
<td>11.2</td>
<td>14.6</td>
<td>18.0</td>
</tr>
<tr>
<td>6</td>
<td>6.3</td>
<td>8.2</td>
<td>10.1</td>
</tr>
<tr>
<td>7</td>
<td>4.8</td>
<td>6.3</td>
<td>7.7</td>
</tr>
<tr>
<td>8</td>
<td>4.1</td>
<td>5.3</td>
<td>6.5</td>
</tr>
<tr>
<td>15</td>
<td>2.7</td>
<td>3.5</td>
<td>4.4</td>
</tr>
<tr>
<td>30</td>
<td>2.3</td>
<td>3.0</td>
<td>3.7</td>
</tr>
<tr>
<td>45</td>
<td>2.2</td>
<td>2.8</td>
<td>3.5</td>
</tr>
<tr>
<td>60</td>
<td>2.1</td>
<td>2.8</td>
<td>3.4</td>
</tr>
<tr>
<td>90</td>
<td>2.1</td>
<td>2.7</td>
<td>3.3</td>
</tr>
<tr>
<td>180</td>
<td>2.0</td>
<td>2.6</td>
<td>3.2</td>
</tr>
<tr>
<td>360</td>
<td>2.0</td>
<td>2.6</td>
<td>3.2</td>
</tr>
</tbody>
</table>
Fig. 17 Settlement Plot for 4-8 Depth Interval of TS1

\[ \beta = 0.895 \]
\[ S_f = 71 \text{ cm} \]

Fig. 18 Settlement Plot for 4-8 m Depth Interval of TS2

\[ \beta = 0.833 \]
\[ S_f = 72 \text{ cm} \]

Fig. 19 Settlement Plot for 4-8 Depth Interval of TS3

\[ \beta = 0.725 \]
\[ S_f = 71 \text{ cm} \]
Based on physical condition that the coefficient of consolidation, $C_h$, cannot be negative, the minimum field values of the discharge capacity, $q_w$, must be greater than 4 m$^3$/year as seen in Table 6. Moreover, Figures 20 to 22 indicated that the calculated $C_h$ values become little affected by the value of discharge capacity when $q_w$ is greater than 30 m$^3$/year. Also seen in these figures is the minimum value of $C_h$ cannot be smaller than 2 m$^2$/year if the ratio $k_h/k_s$ is greater or equal to 3.

Table 7 Calculated Values of $\beta$, $F$, and $C_h$ for Depth Interval of 0-12 m

<table>
<thead>
<tr>
<th>Embankment</th>
<th>$\beta$</th>
<th>$F$</th>
<th>$C_h$ (m$^3$/year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS1</td>
<td>0.865</td>
<td>6.24</td>
<td>4.2</td>
</tr>
<tr>
<td>TS2</td>
<td>0.800</td>
<td>5.98</td>
<td>4.1</td>
</tr>
<tr>
<td>TS3</td>
<td>0.725</td>
<td>5.77</td>
<td>4.2</td>
</tr>
</tbody>
</table>

Table 8 Back-Calculated Values of $C_h$ for Subsoils Under TS2 Embankments

<table>
<thead>
<tr>
<th>Subsoil Layer</th>
<th>Weathered crust</th>
<th>Soft clay</th>
<th>Very soft to soft clay</th>
<th>Soft to medium clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0-2</td>
<td>4.0</td>
<td>3.1</td>
<td>3.0</td>
<td>7.8</td>
</tr>
<tr>
<td>$C_h$ (m$^3$/year)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 9 Values of $C_h$ from Piezocone Tests (m$^2$/year)

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>PC-1</th>
<th>PC-2</th>
<th>PC-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>3.3</td>
<td>4.4</td>
<td>3.8</td>
</tr>
<tr>
<td>8</td>
<td>4.3</td>
<td>4.7</td>
<td>4.2</td>
</tr>
<tr>
<td>12</td>
<td>8.8</td>
<td>7.9</td>
<td>7.0</td>
</tr>
</tbody>
</table>

Fig. 20 Back-Calculated $C_h$-$q_w$ Relations for TS1 Test Embankment

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Fig. 21 Back-Calculated $C_h$-q$_w$ Relations for TS2 Test Embankment

Fig. 22 Back-Calculated $C_h$-q$_w$ Relations for TS3 Test Embankment

Fig. 23 Comparison of $C_h$ Values Computed from TS2 with the values obtained from 1983 Test Embankment with Sand Drains and the 1994 Peizocone Tests
Fig. 24 Comparison of $C_h$-$q_w$ Relations of Three Test Embankments TS1, TS2, and TS3 Using the Same Value of Smear Ratio, $k_b/k_s = 5$.

The back-calculated value of $C_h$ are dependent significantly on the effects of smear. Assuming $k_b/k_s = 5$ and $d_s/d_m = 2$, the $\beta, F$, and the average $C_h$ values for a depth interval of 0 to 12 m, have been calculated and tabulated in Table 7 for a certain assumed value of $q_w$ of 30 m$^3$/year. The corresponding $C_h$ values for the soil profile are given in Table 8. These calculated values can be compared directly with the field values measured by piezocone and piezoprobe tests as presented in Table 9 and Figure 3. There is an excellent agreement between back-calculated and field values of $C_h$.

The final settlements for the very soft clay layer from 4 to 8 m depth were obtained to be 0.71 m, 0.72 m and 0.71 m for TS-1, TS-2 and TS-3, respectively, as demonstrated in Figs. 17, 18, and 19. These values again confirmed the uniformity of the soil profiles under these test embankment. Thus, the same values of $C_h$ can be assumed for all three sites. If the $k_b/k_s = 5$ is assumed, and if taking $C_h = 3$ m$^2$/year as obtained from the piezoprobe tests by Moh and Woo (1987) for the subsoil at the depth interval of 4 m to 8 m, then the discharge capacities of 30, 45, and 90 m$^3$/year can be obtained by Figure 24 for PVDs at TS-1, TS-2, and TS-3, respectively. The back-calculated $C_h$ (field) values using the method of Asaoka (1978) range from 3 to 8 m$^2$/yr. The corresponding laboratory values range from 0.5 to 1.5 m$^2$/yr. Bergado et al (1992) obtained $C_h$ (field)/$C_h$ (lab) ranging from 4 to 5.

CONCLUSIONS

The performance of a full scale test embankment constructed on soft Bangkok clay with Prefabricated Vertical Drains (PVD) at the site of the Second Bangkok International Airport (SBIA) Project are presented in this paper. From the field measurements and the subsequent analyses, the following conclusions can be made:

1. For test embankments TS1, TS2 and TS3, the magnitudes of consolidation settlements correspondingly increased with the decrease of PVD spacing at a particular time period
and the effectiveness of PVD for improvement of soft Bangkok clay has been demonstrated.

2. The degree of consolidation obtained from pore pressure measurements with the corresponding values obtained from settlement measurements.

3. The water content reduction from the field measurements are in agreement with the computed values from the consolidation settlements.

4. There is excellent agreement between measured and predicted increase in undrained shear strengths. The prediction employed SHANSEP technique.

5. The back-calculated $C_h$ values reduce substantially with the increase in the degree of consolidation and the weakest soil layer at 6 m depth has lower $C_h$ value compared to those at 4 m and 10 m depth. The field $C_h$ values range from 3 to 8 m$^2$/yr and the ratio of $C_h$ (field)/$C_h$ (lab) ranges from 4 to 5. The corresponding laboratory $C_h$ values range from 0.5 to 1.5 m$^2$/year.

6. The results of the full scale study indicate that the discharge capacities back-calculated from test embankments TS1, TS2 and TS3 range from 30 to 90 m$^3$/year corresponding to $C_h$ value of 3 m$^2$/year.

7. The rate of settlement versus time plots indicate a secondary compression ratio, $C_o$, value of 0.018 which lie within the range of values for marine clays.

REFERENCES


NUMERICAL ANALYSES OF PVD IMPROVED GROUND AT REFERENCE SECTION OF SECOND BANGKOK INTERNATIONAL AIRPORT

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Department of Soil and Water Conservation, National Chung-Hsing University, Taichung, Taiwan

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P.C. Lin
MAA Engineering Consultants International Ltd., Taipei, Taiwan

ABSTRACT

A simulation scheme for the smear effect and well resistance of Prefabricated Vertical Drain (PVD) improved ground was proposed. Incorporating with the proposed simulation scheme a finite element analysis was performed on a unit cell and full-scale PVD improved ground at Reference Section of SBIA (Second Bangkok International Airport or Suvarnabhumi Airport) to validate the effectiveness of the numerical computation scheme. The numerical results encompassed consolidation rate, settlement and lateral movement profiles and excess pore pressure variation of PVD improved ground exhibit good coincidence with those from the Hansbo’s theoretical solution of unit cell and the field measurement of Reference Section. In addition, according to the proposed numerical procedure, a series of parametric studies were carried out on PVD installation parameters comprised of installation spacing and discharge capacity under specific surcharge fill height and drainage length.

INTRODUCTION

The Second Bangkok International Airport (SBIA) at Nong Ngu Hao (NNH) has been foreseen for more than 20 years to meet the needs of a growing demand for air travel in the region. The project site, NNH, is located about 30 km to the east of Bangkok, between Khlong Prawetbureerom and Highway No.34. It occupies an area of approximately 4 km × 8 km. The project site is associated with the problems of low strength and high compressibility of the soft clay strata, and ground improvement method by using PVD and surcharge is adopted in the design to improve the soft, compressible subsoil before the development of pavement and other facilities on top of the site soils. As required in the Technical Specification, the initial section of ground improvement, as called “Reference Section”, should have a length up to 500 m and be located along one of the runways with one Khlong crossing. However, Reference
Section was constructed on the Taxiway (W4) according to the design drawing. In connection with the PVD project, the full scale testing embankment was constructed. The PVD was installed at spacing of 1.0 m in square pattern and penetrated to a depth of 10 m. Construction of the Reference Section was commenced in January 1998 and was completed in December 1998. After fulfilling the required criteria, removing of surcharge was executed on June 14, 1999, the six-month waiting period at the Reference Section ended on June 15, 1999. The analyses of various monitoring data include surface settlement, lateral movement, excess pore water pressure dissipation, subsoil condition and properties.

In this study, the proposed matching scheme for smear effect and well resistance was verified initially by comparing the numerical results of PVD unit cell with Hansbo's solution (1981) in terms of consolidation rate. Subsequently, the PVD improved ground under embankment surcharge loading at Reference Section Project in SBIA was analyzed using the proposed matching scheme. A two dimensional (2-D) plane strain finite element analysis method was mainly performed for the present study and the consolidation behavior of soft clay layer was modeled by critical state soil model. According to the calculation results of parametric studies, a quick design table is furnished for PVD design in typical Bangkok subsoil.

**SITE INVESTIGATIONS AND FIELD CONSTRUCTIONS**

In order to confirm the subsoil condition before PVD installation and as required in the Technical Specification, a total of 11 soil borings for the undisturbed sampling, 19 field vane shear tests and 41 piezocone penetration tests were carried out on site. Laboratory testing was also carried out during the investigation period (NBIA, 1999). As concluded from the soil investigation program and laboratory tests, the subsoil condition at Reference Section can be classified into four strata to the maximum drilled depth of 20 m with the average soil parameters as shown in Table 1.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Depth (m)</th>
<th>$\gamma_m$ (kN/m$^3$)</th>
<th>$w_c$ (%)</th>
<th>$LL$ (%)</th>
<th>$PL$ (%)</th>
<th>$S_u$ (kPa)</th>
<th>$e_o$</th>
<th>$C_v$</th>
<th>$C_s$</th>
<th>OCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weathed crust</td>
<td>0–2</td>
<td>15.7</td>
<td>70</td>
<td>100</td>
<td>30</td>
<td>26.5</td>
<td>1.68</td>
<td>0.98</td>
<td>0.16</td>
<td>2.5</td>
</tr>
<tr>
<td>Very soft to soft clay</td>
<td>2–10</td>
<td>13.7</td>
<td>110</td>
<td>115</td>
<td>40</td>
<td>19.6</td>
<td>2.72</td>
<td>1.64</td>
<td>0.29</td>
<td>1.5</td>
</tr>
<tr>
<td>Medium stiff clay</td>
<td>10–15</td>
<td>16.0</td>
<td>59</td>
<td>70</td>
<td>30</td>
<td>34.3</td>
<td>2.02</td>
<td>0.99</td>
<td>0.17</td>
<td>1.4</td>
</tr>
<tr>
<td>Stiff clay</td>
<td>15–20</td>
<td>17.3</td>
<td>40</td>
<td>70</td>
<td>30</td>
<td>78.5</td>
<td>2.01</td>
<td>0.88</td>
<td>0.20</td>
<td>1.08</td>
</tr>
</tbody>
</table>
As displayed in Figure 1(a), after the installation of PVD and associated drainage system, the 1\textsuperscript{st} stage loading (crush rock in thickness of 1.3 m), the 2\textsuperscript{nd} stage loading (crush rock in thickness of 1.0 m) and the drain sand layer (1.5 m in thickness includes the sand blanket) had resulted in a total fill height of 3.8 m equivalent to an average surcharge load of 81 kPa. The crushed rock was used to as the embankment materials to provide overburden surcharge for initiating consolidation process. The embankment was placed in two stages with 3-month waiting period in-between to ensure the embankment stability as specified in the design. The construction detail is summarized in Table 2.

Monitoring of instruments, including surface settlement plates, deep settlement gauge, electric piezometers, inclinometers, observation well and surface settlement monuments (after the completion of full fill height) are being carried out on weekly basis after the installation as shown in Figure 1(b).

![Diagram of PVD Improvement Procedure and Construction Sequence of Embankment Surcharge at Reference Section](image)

**Fig. 1(a) PVD Improvement Procedure and Construction Sequence of Embankment Surcharge at Reference Section**

| Table 2 Construction Detail of PVD Improvement and Embankment Surcharge |
|--------------------------|--------------------------|-----|
| **Stage** | **Construction Type** | **Duration (day)** | **Activities** |
| 1 | 1\textsuperscript{st} layer of sand blanket | 7 | Place sand blanket with thickness of 0.5 m and used as the working platform for the installation of drainage instrumentation |
| 2 | 2\textsuperscript{nd} layer of sand blanket | 6 | Place sand blanket with thickness of 0.5 m and used as the working platform for PVD installation |
| 3 | Drainage layer | 10 | Provide the drainage path with thickness of 0.5 m |
| 4 | 1\textsuperscript{st} stage loading | 34 | Place crush rock surcharge with thickness of 1.3 m |
| 5 | 2\textsuperscript{nd} stage loading | 12 | Place crush rock surcharge with thickness of 1.0 m |
NUMERICAL MODELING OF WELL RESISTANCE AND SMEAR EFFECT OF VERTICAL DRAIN

The geometry parameters and associated drainage parameters with the well resistance and smear effect in PVD design are illustrated in Figure 2.

Fig. 2 Approximations of the Well Resistance and Smeared Zone of PVD
(Modified from Rixner et al, 1986)
WELL RESISTANCE

The well resistance of vertical drain can retard the water flow in drainage channel and alternately reduce the consolidation rate of soft clay layer. The influence factors that may cause significant well resistance includes increase of drain length, deterioration of the drain filter (reduction of drain cross-section), silt intrusion into the filter (reduction of pore space), and folding of the drain due to lateral movement. In general, the well resistance can be expressed in term of the discharge capacity of vertical drain, \( q_w \).

Based on the derivation of Mesri (1991), a well resistance factor \( R \) can be defined as:

\[
R = \pi \frac{k_w}{k_h} \left( \frac{r_w}{l_d} \right)^2
\]  

(1a)

Where, \( k_h \) = horizontal permeability of the undisturbed soils and \( l_d \) = drainage length of PVD. In numerical analysis, the well resistance of PVD can be modeled by imposing a specific longitudinal permeability \( k_w = q_w/i_{eq}r_w^2 \), \( i_{eq} \) = hydraulic gradient=1.0 and \( r_w \) = the equivalent radius of PVD) to PVD drainage elements which represents the drainage channel of PVD. Eventually the well resistance factor \( R \) can be given as:

\[
R = \frac{q_w}{k_w l_d^2}
\]  

(1b)

The analysis of field performance of vertical drains in the soft clay deposits indicates that well resistance is negligible when the \( R \) value is greater than 5. This also implies the minimum discharge capacity \( q_w(min) \) of vertical drains required for a negligible well resistance is \( q_w(min) = 5k_w l_d^2 \). The most typical value of \( q_w(min) \) may range from 2 to 80 m\(^3\)/year. Lin et al. (2000) introduced an interface element to simulate the vertical drain with finite permeability to take the well resistance of PVD into account. Well resistance was automatically considered by the discharge of interface element, which possesses equivalent discharge capacity to actual vertical drain.

A wide range of discharge capacity values has been suggested for a proper function of vertical drains. Hansbo (1987) suggested that proper value of \( q_w \) must be 50~100 m\(^3\)/year. Bergado et al. (1996) indicated that back-calculated the discharge capacity in the field to vary from 30~100 m\(^3\)/year with PVD length of 12 m in soft Bangkok Clay. A summary of previous researches on discharge capacity of PVD are summarized in Table 3.
Table 3 Discharge Capacity of PVD by Previous Researches (Chai et al., 1999)

<table>
<thead>
<tr>
<th>Researcher</th>
<th>Discharge Capacity (m³/year)</th>
<th>Confining Pressure (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hansbo (1979)</td>
<td>50–100</td>
<td>--</td>
</tr>
<tr>
<td>Den Hoedt (1981)</td>
<td>95</td>
<td>50–300</td>
</tr>
<tr>
<td>Kremer et al. (1982)</td>
<td>256</td>
<td>100</td>
</tr>
<tr>
<td>Jamiołkowski et al. (1983)</td>
<td>10–15</td>
<td>300–500</td>
</tr>
<tr>
<td>Kremer (1983)</td>
<td>790</td>
<td>15</td>
</tr>
<tr>
<td>Koda et al. (1984)</td>
<td>100</td>
<td>50</td>
</tr>
<tr>
<td>Ricner et al. (1986)</td>
<td>100</td>
<td>--</td>
</tr>
<tr>
<td>Van Zanten (1986)</td>
<td>790–1580</td>
<td>150–300</td>
</tr>
<tr>
<td>Lawrence and Koerner (1988)</td>
<td>150</td>
<td>--</td>
</tr>
<tr>
<td>Holtz et al. (1989)</td>
<td>100–150</td>
<td>300–500</td>
</tr>
<tr>
<td>De Jager et al. (1990)</td>
<td>315–1580</td>
<td>150–300</td>
</tr>
</tbody>
</table>

SMEAR EFFECTS

The installation of vertical drains is usually carried out by the aid of a special mandrel designed so as to cause a minimum of disturbance on the surrounding soils of vertical drains. The diameter of smeared zone is varied with the shape of mandrel and anchor plate and with installation method. For design purposes, it has been evaluated by Jamiołkowski et al (1981) that the diameter of smeared zone, $d_s (=2r_s)$, can be related to the cross-sectional dimension of the mandrel as follows:

$$d_s = \frac{(5 \sim 6)d_m}{2}$$ \hspace{1cm} \text{(2a)}

where, $d_m (=2r_m)$ is the diameter of a circle with an area equal to the cross-sectional area of the mandrel. Hansbo (1987) recommended the following expression based on the results of Holtz and Holm (1973) and Akagi (1976):

$$d_s = 2d_m$$ \hspace{1cm} \text{(2b)}

The radius of smeared zone $r_s = 2r_m$, for the reconstituted soft Bangkok clay has been verified by Bergado et al (1991).

Moreover, according to the oedometer tests performed on samples taken shortly after drain installation on reconstituted soft Bangkok clay at different distances from the drain, it can be found that the horizontal permeability of smeared zone, $k_s$, can be equivalent to vertical
permeability of the undisturbed soils, \( k_w \), determined by oedometer tests. The aforementioned experimental results have been presented by Hansbo (1987).

Bergado et al (1993) indicated the effects of smear can be simulated in finite element analysis by an equivalent horizontal permeability of surrounding soils, \( k_e \), for a radial consolidation as follow:

\[
k_e = \frac{k_s k_i \ln \left( \frac{r_e}{r_w} \right)}{k_s \ln \left( \frac{r_e}{r_s} \right) + k_h \ln \left( \frac{r_e}{r_w} \right)}
\]  

(3)

where, \( r_e = \frac{d_e}{2} \) is the equivalent radius of influence zone of vertical drain.

From a macro point of view, vertical drain increases the mass permeability in vertical direction. Therefore, it is possible to establish an equivalent vertical permeability, \( k_{ve} \), approximately represents the effect of both the vertical permeability of natural subsoil and radial consolidation by vertical drain. By assuming the settlement of vertical drain improved ground is close to one dimensional (1-D) deformation mode, the \( k_{ve} \) value can be derived based on 1-D consolidation theory represents the vertical consolidation and Hansbo’s unit cell theory (1981) depicts the radial consolidation. The total degree of consolidation of vertical drain improved subsoil is the combination of vertical and radial consolidation by using the equation proposed by Scott (1963). Eventually, the equivalent vertical permeability, \( k_{ve} \), proposed by Chai and Miura (1997a) can be expressed as:

\[
k_{ve} = \left( 1 + \frac{2.26L^2}{d_e^2} \frac{k_h}{k_v} \right) k_v \quad \text{and} \quad \mu = \ln \frac{n}{s} + \frac{k_h}{k_i} \ln s - \frac{3}{4} + \frac{2L^2 \frac{k_h}{3q_w}}{3q_w}
\]  

(4)

where, \( d_e = 2r_e \), diameter of the influence zone of PVD \( (d_e = 1.13S \) for square configuration and \( d_e = 1.05S \) for triangular configuration; \( S \) = installation spacing of PVD); \( n = d_e/d_w = r_e/r_w \), spacing influence factor (or spacing ratio) of PVD; \( d_w = 2r_w \), equivalent diameter of PVD; \( s = d_e/d_w = r_s/r_w \), smear or disturbance ratio of PVD.

**NUMERICAL ANALYSIS OF PVD IMPROVED UNIT CELL**

For the numerical analysis of PVD improved unit cell, an axis-symmetric condition with well resistance and smear effect were considered. A drainage element in finite element mesh was specified to simulate the drainage of vertical drain. Through the input of a specific cross sectional area and discharge capacity, \( q_w \), or permeability of \( k_w \) for the drainage element, the well resistance can be introduced to the analysis. In addition, the smear effect is considered simultaneously by the equivalent vertical permeability, \( k_e \), for radial consolidation
and the numerical solutions are illustrated by $k_e$-solution. Eventually, the numerical procedures are verified by comparing the $k_e$-solution with the theoretical solution of Hansbo (1981).

**GEOMETRY MODEL**

Both vertical and horizontal movements of the bottom boundary of unit cell is restrained, while the side boundaries are free to move vertically but restrained horizontally. A uniform pressure loading is applied at the upper boundary and which is the only drainage boundary during the consolidation analysis. The geometry model and the corresponding finite element mesh for PVD unit cell are displayed in Figures 3(a) and (b).

![Geometry Model](image)

**INPUT MODEL PARAMETERS**

The numerical model includes the geometry dimension; the permeability of soils and the discharge capacity of vertical drain are coincided with those of Hansbo’s mathematical model. In addition to the linear elastic model employed for the soil material, the permeability of soils was kept constant during the consolidation. The PVD parameters and soil parameters used for analysis are shown in Tables 4 and 5.

**Table 4 Vertical Drain Parameters for PVD Improved Unit Cell**

<table>
<thead>
<tr>
<th>Drainage length $l_d$ (m)</th>
<th>10</th>
<th>$d_w$ (m)</th>
<th>0.4</th>
<th>$q_w$ (m$^3$/sec)</th>
<th>5.10E-6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing ratio $n$</td>
<td>25</td>
<td>$d_e$ (m)</td>
<td>10</td>
<td>$k_e$ (m/sec)</td>
<td>4.06E-5</td>
</tr>
<tr>
<td>Disturbance ratio (or Smear ratio) $s$</td>
<td>5</td>
<td>$d_f$ (m)</td>
<td>2</td>
<td></td>
<td>$n = d_e/d_w$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$s = d_f/d_w$</td>
</tr>
</tbody>
</table>
Table 5 Soil Model Parameters for PVD Improved Unit Cell

<table>
<thead>
<tr>
<th>Material model</th>
<th>Loading (kPa)</th>
<th>$E$ (kPa)</th>
<th>$\nu$</th>
<th>$k_h$ (m/sec)</th>
<th>$k_e$ (m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Isotropic Elastic</td>
<td>10</td>
<td>10000</td>
<td>0.3</td>
<td>1.0E-8</td>
<td>1.0E-16</td>
</tr>
</tbody>
</table>

$PVD$ Improved Unit Cell (disturbed zone)

<table>
<thead>
<tr>
<th>Height (m)</th>
<th>$L = \frac{8k_d l_d^2}{\pi q_w}$</th>
<th>$K$ (kPa)</th>
<th>$C_h$ (m$^2$/sec)</th>
<th>$k_e$ (m/sec)</th>
<th>$k_e$ (m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>0.5</td>
<td>13462</td>
<td>1.346E-5</td>
<td>2.0E-9</td>
<td>3.3E-9</td>
</tr>
</tbody>
</table>

NUMERICAL ANALYSIS OF FULL SCALE PVD IMPROVED GROUND AT REFERENCE SECTION

In stead of using the equivalent vertical permeability, $k_e$, for radial consolidation in PVD unit cell, the equivalent vertical permeability, $k_{ve}$, which incorporates the vertical consolidation of natural soil into the radial consolidation of PVD was applied for the analysis of PVD improved ground. In addition, the $k_{ve}$ coefficient enables to take the smear effect and well resistance into account simultaneously in the formulation. However, it should be indicated that a plenty of thin drainage element which supposed to consider the well resistance in analysis need to be specified in finite element mesh when using the $k_e$ coefficient to cover the smear effect. As a consequence, this may generates exceedingly large number of nodal point in finite element mesh and causes numerical difficulties during the computation.

GEOMETRY AND NUMERICAL MODEL OF REFERENCE SECTION

The Reference Section of PVD improved ground located at coordinates of $X=13639.00$ and $Y=11845.50$~11942.50 was selected for the numerical analysis and the dimension of full scale PVD improved ground with embankment surcharge loading is shown as Figure 4.

Fig. 4 Geometry Model for Numerical Analysis of PVD Improved Ground of Reference Section at SBIA
Considering the symmetric condition and the induced deformation of the stiff clay layer immediately underlain the medium stiff clay layer is negligible, the central and bottom boundary can be determined. The side boundary is specified at a distance equal to three times of compressible depth, from the toe of embankment. In addition, the drainage is merely allowed at the top surface of PVD improved ground. The numerical model for finite element analysis is presented as Figure 5.

![Finite Element Mesh for Numerical Analysis of PVD Improved Ground of Reference Section at SBIA](image)

**CONSTRUCTION SEQUENCE**

The fill height time history of the embankment construction as shown in Figure 6 was employed for numerical simulation. The crushed rock was used as the construction material of embankment. The embankment surcharge fill provides overburden load for initiating consolidation process was placed in two stages with 3-month waiting period in-between to ensure the embankment stability as specified in the design. The 1\(^{st}\) stage loading (1.3 m in thickness), the 2\(^{nd}\) stage loading (1.0 m in thickness) and the drain sand layer (1.5 m in thickness include the sand blanket) had resulted in a total fill height of 3.8 m equivalent to an average surcharge load of 81 kPa. The embankment was constructed by several layers of element to stimulate different stages of construction.
SOIL MODEL PARAMETERS

For the uppermost 0 to 2 m depth of weathered crust and the stiff clay which extends below 10 to 15 m depth, Mohr-Coulomb plasticity model was used for numerical simulation. On the other hand, Modified Cam-Clay model was adopted to describe the behavior of the soft clay layer ranges from 2 to 10 m depth. For embankment fill, the deformation behavior was simply simulated by linear elastic model. The required input parameters for various soil models are listed in Table 6.

Table 6 Input Parameters of Various Soil Models for 2-D Numerical Analysis of Full-Scale PVD Improved Ground at SBIA

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Soil Type</th>
<th>Soil Model</th>
<th>$\gamma_m$ (kN/m$^3$)</th>
<th>$E_o$ (kPa)</th>
<th>$\nu$</th>
<th>$C$ (kPa)</th>
<th>$\phi$ (deg.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0–2</td>
<td>Weathered crust</td>
<td>Mohr-Coulomb</td>
<td>15.7</td>
<td>1.324 E4</td>
<td>0.30</td>
<td>30</td>
<td>28</td>
</tr>
<tr>
<td>10–15</td>
<td>Medium stiff clay</td>
<td>Mohr-Coulomb</td>
<td>16.0</td>
<td>1.717 E4</td>
<td>0.30</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>2–10</td>
<td>Very soft to soft clay</td>
<td>Modified Cam-Clay</td>
<td>13.73</td>
<td>0.13</td>
<td>0.71 (0.31)</td>
<td>M</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Thickness (m)</th>
<th>Fill Type</th>
<th>Soil Model</th>
<th>$\gamma_m$ (kN/m$^3$)</th>
<th>$E_o$ (kPa)</th>
<th>$\nu$</th>
<th>$k_h$ (m/sec)</th>
<th>$k_v$ (m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0 (0.5+0.5)</td>
<td>Sand Blanket</td>
<td>Linear Elastic</td>
<td>18.4</td>
<td>1.000 E4</td>
<td>0.30</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0.5</td>
<td>Sand Drainage</td>
<td>Linear Elastic</td>
<td>18.3</td>
<td>1.000 E4</td>
<td>0.30</td>
<td>5.0E-5</td>
<td>5.0E-5</td>
</tr>
<tr>
<td>2.3 (=1.3+1.0)</td>
<td>Crushed Rock</td>
<td>Linear Elastic</td>
<td>21.0</td>
<td>6.000 E4</td>
<td>0.28</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

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DESIGN AND DRAINAGE PARAMETERS OF PVD

In reality, the permeability ratio \((k_h / k_s)\) is a function of the structure and sensitivity of subsoil surrounding PVD. Since for most natural deposit, the permeability in horizontal direction, \(k_h\), is higher than in vertical direction, \(k_s\), and the \((k_h / k_s)\) ratio can vary from 1 to 15 (Jamiokowski et al., 1983). Meanwhile, Hansbo (1987) indicated that \(k_s\) can be in the same level with vertical permeability of natural soil.

In addition, Bergado et al. (1991) conducted a thorough laboratory study on the development of the smear zone in soft Bangkok clay and reported that the ratio of the horizontal permeability coefficient of the undisturbed zone to that of the smear zone, \((k_h / k_s)_{\text{laboratory}}\), varied between 1.5 and 2, with an average of 1.75. Laboratory test may be a correct way for determining \(k_s\) value, but it generally under-estimates the permeability of field deposit because of sample disturbance and sample size effect. Considering this fact, the values of \((k_h / k_s)\) and \((k_h / k_s)_{\text{fields}}\) ratios in field condition are proposed by Bergado (1991) and Indraratna (1995) and the ratio in the fields, \((k_h / k_s)_{\text{fields}}\), can be obtained from the identical ratio in the laboratory \((k_h / k_s)_{\text{laboratory}}\) by multiplying a modification coefficient \(C_f\). The drainage parameters of PVD required for numerical analysis is tabulated in Table 7(a).

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Soil Type</th>
<th>(k_h) (m/sec)</th>
<th>(k_v) (m/sec)</th>
<th>(k_s) (m/sec)</th>
<th>(k_{sw}) (m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0–2</td>
<td>Weathered crust</td>
<td>2.46E-9</td>
<td>1.51E-9</td>
<td>3.51E-10</td>
<td>4.72E-8</td>
</tr>
<tr>
<td>2–10</td>
<td>Very soft to soft clay</td>
<td>7.99E-9</td>
<td>4.90E-9</td>
<td>1.14E-9</td>
<td>1.53E-7</td>
</tr>
<tr>
<td>10–15</td>
<td>Medium stiff clay</td>
<td>2.50E-9</td>
<td>1.53E-9</td>
<td>3.57E-10</td>
<td>1.53E-9</td>
</tr>
</tbody>
</table>

\((k_h / k_s)_{\text{fields}} = C_f \cdot (k_h / k_s)_{\text{laboratory}}\); \((k_h / k_s)_{\text{laboratory}} = 1.75\); \(C_f = 4.0\); \((k_h / k_s)_{\text{fields}} = 7.0\)

According to the design specification, PVD was installed by square configuration with spacing of 1 m x 1 m and penetrated to 10 m depth from the ground surface. The PVD used at Reference Section should satisfy various requirements specified in the Specification. During the installation, three PVD samples were selected for the routine tests to ensure the requirement of quality control. The PVD geometry and the design parameters associated with numerical analysis are summarized in Table 7(b).
Table 7 (b) PVD Geometry and Design Parameter for Numerical Analysis

<table>
<thead>
<tr>
<th>Configuration of PVD Installation:</th>
<th>Square Pattern</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing of PVD Installation: S x S</td>
<td>1 m x 1 m</td>
</tr>
<tr>
<td>Drain Length of PVD: L_d</td>
<td>10 m</td>
</tr>
<tr>
<td>Cross Section of PVD: a x b</td>
<td>4 mm x 98 mm</td>
</tr>
<tr>
<td>Mandrel Dimension: l x w</td>
<td>125 mm x 45 mm</td>
</tr>
<tr>
<td>Equivalent Diameter of the Mandrel: d_m (= 2 r_m)</td>
<td>84.6 mm</td>
</tr>
<tr>
<td>Diameter of Smeared Zone: d_r (= 2 r, and r_r=2 r_m = d_m)</td>
<td>169.2 mm</td>
</tr>
<tr>
<td>Diameter of Influence Zone of PVD : d_r=2 r_s=1.13S</td>
<td>1130 mm</td>
</tr>
<tr>
<td>Equivalent Diameter of PVD: d_a=(a+b)/2= 2 r_w</td>
<td>51 mm</td>
</tr>
<tr>
<td>Spacing Ratio of PVD: n = d_c / d_w =1.13S/ d_w</td>
<td>22.2</td>
</tr>
<tr>
<td>Disturbance Ratio of Subsoil, s = d_i / d_w</td>
<td>3.3</td>
</tr>
<tr>
<td>Discharge Capacity of PVD (Laboratory): q_w=k_w A</td>
<td>940.83 m³/yr</td>
</tr>
</tbody>
</table>

PARAMETRIC STUDIES

IMPLEMENTATION OF CALCULATION

Employing the numerical procedure established for the full-scale PVD improved ground at Reference Section in SBIA, a series of parametric studies was performed on PVD design parameters. The effect of design parameters on settlement behavior of PVD improved ground includes installation spacing $S$, embankment surcharge height $H$ and discharge capacity $q_w$ were evaluated in terms of consolidation rates, $U(\%)=[S_r(t)/S_r(\infty)]x100(\%)$. In which $S_r(t)$ and $S_r(\infty)$ denote the settlement of ground surface of PVD treated ground at the centerline of embankment for a specific elapsed consolidation time $t$ and ultimate time ($t=\infty$).

NUMERICAL VARIABLE

From practical view point, spacing $S$ of 1, 1.5 and 2 m were considered for PVD installation in typical Bangkok subsoil condition and it alternately be evaluated in term of spacing ratio, $n=d_c/d_w$ (=22.2, 33.3 and 44.4) for $d_c=1.13S$ for a square configuration. Moreover, the embankment surcharge height $H$ of 1, 2, 3 and 4 m and the PVD discharge capacity $q_w$ of 10, 100 and 1000 m³/year which are commonly encountered in engineering practice were adopted for numerical studies. Table 8 summarizes the numerical variables used for parametric study.

Table 8 Numerical Variables for Parametric Study on PVD Improved Ground in Bangkok

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### Subsoil

<table>
<thead>
<tr>
<th>$S$ (m)</th>
<th>$d_e$ (m) (=1.13S)</th>
<th>$n$ ($=d_e/d_w$)</th>
<th>$q_w$ (m³/year)</th>
<th>$k_w$ (m/sec)</th>
<th>$k_w$ (m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>1.130</td>
<td>22.2</td>
<td>10</td>
<td>4.06E-8</td>
<td>1.00E-7</td>
</tr>
<tr>
<td>1.0</td>
<td>1.130</td>
<td>22.2</td>
<td>100</td>
<td>4.65E-8</td>
<td>1.46E-7</td>
</tr>
<tr>
<td>1.0</td>
<td>1.130</td>
<td>22.2</td>
<td>1000</td>
<td>4.72E-8</td>
<td>1.53E-7</td>
</tr>
<tr>
<td>1.5</td>
<td>1.695</td>
<td>33.3</td>
<td>10</td>
<td>1.83E-8</td>
<td>4.62E-8</td>
</tr>
<tr>
<td>1.5</td>
<td>1.695</td>
<td>33.3</td>
<td>100</td>
<td>2.00E-8</td>
<td>6.50E-8</td>
</tr>
<tr>
<td>1.5</td>
<td>1.695</td>
<td>33.3</td>
<td>1000</td>
<td>2.10E-8</td>
<td>6.80E-8</td>
</tr>
<tr>
<td>2.0</td>
<td>2.260</td>
<td>44.4</td>
<td>10</td>
<td>1.07E-8</td>
<td>2.80E-8</td>
</tr>
<tr>
<td>2.0</td>
<td>2.260</td>
<td>44.4</td>
<td>100</td>
<td>1.20E-8</td>
<td>3.78E-8</td>
</tr>
<tr>
<td>2.0</td>
<td>2.260</td>
<td>44.4</td>
<td>1000</td>
<td>1.22E-8</td>
<td>3.93E-8</td>
</tr>
</tbody>
</table>

$d_e=0.051$ m; $d_w=0.0846$ m; $d_s=2d_w=0.1692$ m; $s=d_e/d_w=3.3$; $L=10$ m; Fill height $H=1, 2, 3, 4$ m

### RESULTS AND DISCUSSION

In order to verify the numerical procedures, the numerical results were compared with the theoretical solutions of PVD unit cell (Hansbo, 1981) in term of degree of radial consolidation. Further, the settlement, lateral movement and excess pore pressure from full-scale numerical simulation of PVD improved ground at the Reference Section of SBIA were also compared with those from observations.

### PVD UNIT CELL

In PVD unit cell the smear effect was considered by the equivalent horizontal permeability, $k_e$, while the well resistance was simulated by specifying a finite permeability, $k_w$, to vertical drain (drainage element) which is alternately determined from the discharge capability, $q_w$, of PVD. As shown in Figure 7, the radial consolidation rate of $k_e$-solution, $U_h$, is slightly overestimated in the range of $U_h < 78\%$ and underestimated of $U_h > 78\%$. 

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Fig. 7 Comparison between Numerical Results and Hansbo’s Solution of Average Consolidation Rate of Unit Cell

Conclusively, the finite element solution exhibits a very similar trend with the theoretical solutions of Hansbo (1981) and this preliminary verifies the validity of the simulation schemes of finite element method in this study.

**PVD Improved Ground at the Reference Section of SBIA**

The numerical results of full-scale PVD improved ground at Reference Section were repeatedly compared with the measurements of settlement rate, ground movement, and variation of excess pore pressure to demonstrate the effectiveness of the numerical scheme.

**SETTLEMENT RATE AT VARIOUS DEPTHS**

The surface settlement plate SP-RF-012 (at z=0 m), deep settlement gauges DG-RF-006 (at z=-2 m), and DG-RF-009 (at z=-12 m) installed at the interfaces of each sequential subsoil layers of PVD improved ground along the center line of embankment were selected for comparisons. Figure 8 merely presents the settlement rate at depth of 2 m which displays an excellent coincidence of the prediction with the measurement.
GROUND SETTLEMENT AND LATERAL MOVEMENT

The ground settlement for three consolidation phases during the embankment construction included: (phase 1) 3 months after 1\textsuperscript{st} stage loading applied, (phase 2) 2\textsuperscript{nd} stage loading completed and (phase 3) 6 months after 2\textsuperscript{nd} stage loading applied (end of measurement). In this paper, only the result of phase 3 was presented. As shown in Figure 9(a), the predicted ground settlement profile of the central area of embankment at the end of measurement ($t=510$ days) is largely close to the measurement while it was over predicted at the toe area of embankment. The over prediction may be resulted from applying the equivalent permeability, $k_v$, thoroughly to the clay layer and this is due to the limitation of numerical tool. However, it should be pointed out that the numerical scheme ignores the fact that the PVD is absent in the toe area of embankment.

The inclinometers IM-001 and IM-002 were installed at the edge of the 1\textsuperscript{st} stage fill and possess a distance of 36.5 m away from the center line of embankment. The corresponding lateral movement of PVD improved ground at the end of measurement is illustrated in Figure 9(b). As expectation, the overestimated lateral movement was accompanied by an overestimation of ground settlement.
Fig. 9(a) Comparisons of Ground Settlement between Numerical Prediction and Field Measurement at 6 Months after 2nd Stage Loading Applied (End of Measurement)

Fig. 9(b) Comparisons of Lateral Movement between Numerical Prediction and Field Measurement of Inclinometer IM-002 at 6 Months after 2nd Stage Loading Applied
EXCESS PORE WATER PRESSURE

The monitoring points at the depth of 2 m, 5 m and 8 m along the center line of embankment were selected for comparison and the corresponding electric piezometer were numbered as PE-RF-006, PE-RF-007 and PE-RF-008 respectively. As presented in Figure 10, in numerical analysis the initial pore pressure at the depth of 5 m was specified as hydrostatic and this consequently leads to an underestimation of initial excess pore water pressure due to the existence of non-hydrostatic condition at jobsite.

Fig. 10 Comparisons of Excess Pore Water Pressure between Numerical Prediction and Field Measurement at the Depth of z=5 m along the Center Line of Embankment

It should be indicated that the underestimation of excess pore water pressure after the 1st stage loading may be partially due to the raise of groundwater table caused by rainfall.

PARAMETRIC STUDIES AND DESIGN TABLES OF PVD IMPROVED GROUND

For a specific fill height and discharge capacity \((H=4m\) and \(q_w=100 \text{ m}^3/\text{year}\) for instance), the average consolidation rate \(U\) (%) of PVD improved ground for various PVD spacing \(S\) (=1 m, 1.5 m and 2 m) or spacing influence factor \(n\) (=\(d_p/d_w\) =22.2, 33.3 and 44.4) are shown in Figure 11. It is obvious the spacing influence factor \(n\) has the most significant effect on the consolidation rate of PVD improved ground. The \(U\) (%) value for elapsed time \(t=23\) month increases from 79.8% to 97.7% as the \(n\) value decreases from 44.4 to 22.2 (or installation spacing \(S\) decreases from 2 m to 1 m).
Similarly, the average consolidation rates $U(\%)$ of PVD improved ground at different spacing influence factor, $n$, for elapsed time $t=1, 2, 3, 6$ and 23 month are shown in Figure 12. It can be seen the elapsed time required to achieve a higher degree of consolidation of typical PVD improved ground in Bangkok subsoil should not less than 1 year (for $H=4$ m, $S=1$ m~$2$ m and $q_w=100$ m$^3$/year).

![Graph showing consolidation rate](image1.png)

Fig. 11 Consolidation Rate with and without PVD Improvement for Various Spacing Influence Factor $n=d/d_w=1.13S/d_w$ (for $H=4$ m and $q_w=100$ m$^3$/yr)

![Graph showing consolidation rate](image2.png)

Fig. 12 Consolidation Rate of PVD Improved Ground for Various PVD Spacing Influence Factors ($n=1.13S/d_w$) and Different Elapsed Times $t$ for a Specific $H=4$ m and $q_w=100$ m$^3$/yr

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Finally, to provide a quick reference for PVD design in engineering practice, the calculation results of various PVD design parameters are readily furnished in Table 9. It can be found in numerical calculation the discharge capacity of PVD, \( q_w \), seems not as crucial as expected in reality. An increase of \( q_w \) value from 10 to 1000 m\(^3\)/year merely causes an increase of \( U \) (%) value for elapsed time \( t=6 \) month from 74.2 to 79.2.

Table 9 Summary of Degree of Consolidation for Various Design Parameters in PVD Improved Ground of Typical Bangkok Subsoil

<table>
<thead>
<tr>
<th>( q_w ) (m(^3)/year)</th>
<th>10</th>
<th>100</th>
<th>1000</th>
</tr>
</thead>
<tbody>
<tr>
<td>( n ) ((=d_v/d_o))</td>
<td>22.2</td>
<td>33.3</td>
<td>44.4</td>
</tr>
<tr>
<td>( U ) (%) ( t=6 ) month</td>
<td>74.4</td>
<td>50.8</td>
<td>36.8</td>
</tr>
<tr>
<td>( t=23 ) month</td>
<td>97.0</td>
<td>86.9</td>
<td>74.8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>( q_w ) (m(^3)/year)</th>
<th>10</th>
<th>100</th>
<th>1000</th>
</tr>
</thead>
<tbody>
<tr>
<td>( n ) ((=d_v/d_o))</td>
<td>22.2</td>
<td>33.3</td>
<td>44.4</td>
</tr>
<tr>
<td>( U ) (%) ( t=6 ) month</td>
<td>68.5</td>
<td>48.4</td>
<td>36.4</td>
</tr>
<tr>
<td>( t=23 ) month</td>
<td>94.9</td>
<td>83.2</td>
<td>71.3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>( q_w ) (m(^3)/year)</th>
<th>10</th>
<th>100</th>
<th>1000</th>
</tr>
</thead>
<tbody>
<tr>
<td>( n ) ((=d_v/d_o))</td>
<td>22.2</td>
<td>33.3</td>
<td>44.4</td>
</tr>
<tr>
<td>( U ) (%) ( t=6 ) month</td>
<td>71.3</td>
<td>51.4</td>
<td>38.8</td>
</tr>
<tr>
<td>( t=23 ) month</td>
<td>95.8</td>
<td>85.1</td>
<td>73.7</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>( q_w ) (m(^3)/year)</th>
<th>10</th>
<th>100</th>
<th>1000</th>
</tr>
</thead>
<tbody>
<tr>
<td>( n ) ((=d_v/d_o))</td>
<td>22.2</td>
<td>33.3</td>
<td>44.4</td>
</tr>
<tr>
<td>( U ) (%) ( t=6 ) month</td>
<td>74.2</td>
<td>54.4</td>
<td>43.2</td>
</tr>
<tr>
<td>( t=23 ) month</td>
<td>96.5</td>
<td>87.0</td>
<td>76.6</td>
</tr>
</tbody>
</table>

\( U \) (%)=[\( S_{tr}(t)/S_{tr}(\infty)\)]×100% and \( S_{tr}(t)=\left[U \right]×S_{tr}(\infty))/100\%

**CONCLUSIONS**

In this study, the numerical procedure and calculation scheme for PVD improved soil were verified through the numerical results of PVD unit cell and of the full-scale PVD improved ground at the Reference Section of SBIA. Meanwhile, part of calculation results
were tabulated into a quick design table encompassed various design parameters of PVD for
typical Bangkok subsoil. In conclusion, the spacing influence factor, \( n = d_e / d_w \), plays a very
crucial role in PVD design while the effect of discharge capacity, \( q_w \), on consolidation rate is
not as apparent as expected in numerical analysis. On the other hand, for typical Bangkok
subsoil, a PVD improved ground with 1 m \( \times \) 1 m square configuration (or \( n \approx 22.2 \)) and 10 m
drainage length, the elapsed time required to achieve high degree of consolidation (for an
\( U(\%) > 90\% \)) is suggested not less than 1 year under embankment surcharge fill of 4 m height.

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DESIGN AND CONSTRUCTION OF GROUND IMPROVEMENT WORKS AT SUVARNABHUMI AIRPORT

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ABSTRACT

The Suvarnabhumi airport construction began in mid 1990s, and the airport is expected to be in operation by 2006, which involved extensive ground improvements. Because of the soft ground condition, the subsoils supporting the pavements in the airside and landside systems were improved by various ground improvement methods, including conventional PVD preloading, soil cement columns, pile foundation, vacuum consolidation etc. This paper summarizes the GI methods adopted with some background information on the design methodology and construction related issues. Some back analysis was performed to evaluate the performance of the ground improvement, verifying the soil parameters used in the design. Most of the ground improvement works performed were executed successfully with relatively few technical obstacles, and some important construction issues are highlighted.

BACKGROUND

The Suvarnabhumi Airport site located 25 km to the west of Bangkok, occupies an area of about 32 km². Due to the soft nature of the upper soil layers, the authorities involved and the designers had considered the necessity of improving the soft soils to reduce post-construction settlements and to improve the ground stability.

Fig. 1 Zones of Ground Improvement
Over the last decade of construction, various methods of ground improvement were introduced and adopted in different areas within the airport site, depending on the site conditions and design requirements etc. The ground improvement works performed include prefabricated vertical drain (PVD) with preloading, soil cement columns, pile foundation (or bearing unit) and vacuum consolidation methods.

The polder system for the airport was the first earthwork project started in early 1990s, which involved construction of earth dike along the perimeter without any ground improvement. Large settlement was allowed in the design of the polder, and the stability was achieved by having wide counterweight berms. The flood protection system of the airport is controlled by the pumping stations located next to the perimeter canal.

In mid-1990s, trial embankments with PVD ground improvement under various configurations were constructed to evaluate the PVD performance. The results of the trial embankments were adopted by the ground improvement designers at later stage for calibrating the model used in the design.

First phase of the GI works involving PVD method was applied to two runways/parallel taxiways and the main apron as shown in Figure 1. The earthwork included placing of surcharge in 2 to 3 stages to control the stability. The work was executed with great success, and the similar design was later adopted in the cargo apron and maintenance apron areas.

Following the PVD work for the airside system, the landside (road) system had also used the PVD technique for improving the ground. Over 30 km of roadway within the airport site was also improved by PVD method.

In early 2000s, the Thai Government had decided to accelerate the construction of the airport due to significant increase in the air traffic, other parties, such as Thai International, Bangkok Airways etc., began to construct their airport supporting facilities. Since the profile grades of airside and landside systems are only around 1 to 1.8 m above the original ground level, it would be necessary to improve the ground for supporting the pavement structures around the facilities. These parties were faced with the problems of ground interference from adjacent ongoing PVD ground improvement works. For example, in the international cargo apron zone, the PVD work started at the same time as the piling work of the international cargo building. The counterweight berm of PVD work was placed in the building zone with possibility of large lateral ground movement during surcharging. Soil cement column method was introduced as a protection barrier to reduce the influence from the PVD, it also served as supporting unit for the pavement in that area.

Following the introduction of soil cement column at the apron zone, this method was adopted in various roads and car parks around the airport supporting facilities, including fire rescue stations, TG ground support and equipment buildings, airport fuel station etc.

Pile foundation or bearing unit was also introduced as an alternative to the soil cement column method during the same period. Several road extensions built with relatively short duration were supported by short small piles and concrete slab. The primarily reason for using the piles instead of soil cement columns was short construction required since the piles were prefabricated and could be installed easily.
In mid 2000, the Thai Government had moved forwards with the airport expansion by constructing the third runway and midfield satellite apron, involving several new taxiway connections to the existing completed taxiways and runways. The subsoils of the new connections (known as Enabling Works) were not improved in previous GI program; therefore there was a great technical challenge to improve the subsoils with minimum impact on the adjacent completed airfield pavements. Pile foundation and soil cement columns were not considered due to the possibility of significant differential settlement at the interfacing areas during operation. Conventional PVD method was not suitable due to the possibility of large lateral movement during construction, affecting the existing pavement. Vacuum consolidation was selected which was considered to be the best option available. Over 400,000 m$^2$ of area was improved by vacuum consolidation method with minimum impact to the adjacent pavements.

**SUBSOIL AND PIEZOMETRIC CONDITIONS**

Based on the soil investigation, the subsoils down to depth of 25 meters for the design of ground improvement are relatively uniform, and it can be divided into five major layers:

- **Weathered crust:** The weathered crust consists of moderate olive brown to grayish black silty clay. The natural water content of this layer is relatively low with value of 10 to 47%.
- **Soft to very soft CLAY:** A layer of soft to very soft, dark gray to greenish gray CLAY (CH) is found below the layer of weathered crust down to elevation of about -10 to -11 m (MSL) with average thickness of 8 to 10 m. The natural water content and plasticity are generally high. The undrained shear strength ranges from 1 to 2.6 ton/m$^2$. Most settlements would result from the consolidation of this clay layer when loaded; therefore the properties of this clay layer are of utmost importance in the PVD ground improvement work.

![Fig. 2 Typical Soil Profile at Airport Site](image-url)

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- **Medium stiff CLAY:** This layer is present at elevations of -10 to -15 m MSL, consisting of medium dark gray to light greenish gray medium stiff clay. This clay has higher the undrained shear strength than the upper soft to very soft clay with value of 2.5 to 5.3 ton/m², having water content of 46% to 82%. The thickness of this layer is about 4 to 5 m.

- **Stiff to very stiff CLAY:** A layer of light greenish gray to grayish brown, stiff to very stiff silty clay is found directly below the medium stiff clay layer with thickness varying from 6 to 9 m. The natural water content ranges from 28 to 42% with SPT N value of 13 to 29 and undrained shear strength of 5.3 to 12.4 ton/m².

- **Medium dense to dense first SAND:** This layer is found below the stiff to very stiff clay, consisting of yellowish gray to grayish brown, medium dense to dense silty and clayey sand, with SPT N value of 14 to 43. The natural water content of this layer varies from 12 to 22%. Due to deep pumping, the piezometric pressure of this layer is close to zero at the interface with the stiff clay, creating a downward flow condition from the upper 20 m of clay.

Since the soil profiles within the site are relatively uniform with some small variations in the soil thickness, therefore typical soil properties along with soil parameters are used as summarized in Figure 3.

Extraction of water from the ground causes settlements in the ground due to the increase in effective stress of the soil. The settlements due to land subsidence or deep pumping have been a major environmental concern in Bangkok since the 1970s.

![Fig. 3 Soil Parameters used in Most GI Projects at Airport Site](image-url)
Fig. 4 Piezometric Pressure Profiles

Various agencies and research institutes had conducted research and monitoring programs to study the effects of deep pumping in the ground. Figure 4 shows a typical pore water pressure profile at the site measured during the ground improvement for the airside pavement, indicating severe pumping in the sand layer. This lowering of pore water pressure causes compression in the subsoils because of the increase in effective stress of soil. At some locations within the site, the pore water pressure at depth of about 20 m is almost zero, indicating that there is downward flow of water from the upper stiff and soft clay layers into the high permeability sand layer. It should be noted that the rate of settlement at this stage seems to be decreasing with time, unless there is new pumping in the vicinity close to the area. Figure 4 also shows the pore water pressure profile measured in the ground improvement by vertical drain (Seah et al.) at this project site. It should be noted that the piezometric pressure down to depth of 10 m (full length of vertical drain) had recovered to hydrostatic condition after installation of vertical drain since the discharge capacity of the drains was relatively high, acting as small pipe with recharging from the surface. This phenomenon had reduced the effective stress gain of the lower soft clay (8-10 m) in the PVD preloading process.

DESIGN OF GROUND IMPROVEMENT

Design Criteria

The main design criteria proposed and adopted by the Airside Design Group (ADG) in 1995, which were adopted for most of the airside system works, including:

- **Runways**: The differential settlement criterion is specified as 25-30 mm over 45 m. The post construction settlements during the first 10 years after pavement construction should not exceed 300 mm (Criterion 1).
- **Taxiways**: The differential settlement criterion is that the slope change should not exceed 1%, corresponding to 300 mm in 30 m. The surface of a taxiway should not have irregularities that cause damage to aircraft structures. But for drainage concerns, the criterion is specified as 300 mm total settlement during the first 10 years.
- **Aprons**: The design slopes of the apron stands are generally 0.7% while the minimum slope for drainage should be about 0.5%. The total settlement criterion is specified as 450 mm during the first 10 years (Criterion 2) for areas at distance away from buildings. But in areas close to the building, the limit is set at 300 mm in 10 years (Criterion 1).
For runway and taxiway with preloading method, such as PVD with preloading, it had been suggested by ADG to preload the ground to 1.2 times the future effective stresses, for the permanent loads as proposed by ISES. In other words, the required gain in effective stress would have to be 1.2 times higher than the permanent loads. For apron under Criterion 2, the required gain in effective stress will be equal to permanent loads, while additional 0.5 ton/m² has been included for design.

In general, with the flood protection control by the polder system, the final elevations of the pavement were kept between +1m and +1.8 m MSL, with equivalent load of 2.2 to 4 ton/m² excluding the live loads (of 2 ton/m²). Therefore for preloading work, the required increase in vertical effective stress of the soft clay should be at least 4.2 to 6 ton/m².

Table 1  Methods of Ground Improvement for Airside Pavements

<table>
<thead>
<tr>
<th>Technical Issues</th>
<th>Method</th>
<th>Prefabricated Vertical Drain (PVD) with preloading</th>
<th>Soil Cement column + Cement Stabilized Mat</th>
<th>Pile Foundation (Bearing Unit)</th>
<th>Vacuum Consolidation with preloading</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Settlement</td>
<td>Low if factor of safety is high</td>
<td>Very low due to load transferred to lower stiffer layer</td>
<td>Very low due to load transferred to lower stiffer layer</td>
<td>Low because of low plastic flow to the side</td>
<td></td>
</tr>
<tr>
<td>Consolidation Settlement</td>
<td>High</td>
<td>Very Low</td>
<td>Very Low</td>
<td>High</td>
<td></td>
</tr>
<tr>
<td>Residual Settlement</td>
<td>Can be controlled through proper application of surcharge</td>
<td>Very low due to load transferred to lower stiffer layer</td>
<td>Very low due to load transferred to lower stiffer layer</td>
<td>Can be controlled through proper application of surcharge</td>
<td></td>
</tr>
<tr>
<td>Stability</td>
<td>Increase in Factor of Safety due to increase in soil strength during consolidation</td>
<td>High Factor of Safety initially, but may reduce with time</td>
<td>High Factor of Safety initially, but may reduce with time</td>
<td>Very stable for vacuum alone, but lower factor of safety when surcharge is placed.</td>
<td></td>
</tr>
</tbody>
</table>

| Financial Issues | Maintenance Cost | Low | Low | Low | Low |
| Construction Cost | Moderate | High for large loading | High | High |

| Other Related Issues | Construction Period | Long – need 1.5 years | Short | Short | Moderate |
| Long Term Performance | Less differential settlement | Less differential settlement | Less differential settlement | Less differential settlement |
| Right of Way | Require significant area for counterweight berm | No ROW problem | No ROW problem | Require some ROW (3-5m) |
| Local Experience in Construction | Good – less operator dependent | Good – High operator dependent | Good | Lack of Local experience |
| Market Supply | No problem | No problem | No problem | Some import of material and equipment required |
| Likelihood of Usage | Most cost effective method but requiring long construction period | More expensive with short construction period | Most expensive with short construction period | Most suitable if construction time and right of way are limited |

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GROUND IMPROVEMENT METHODS

There are several methods (Table 1) of improving the properties of soft clay to reduce the post-construction settlement or to improve the stability of the pavement structures. Without any improvement, the pavement would suffer significant settlement with time, and the pavement structure itself needs to be stronger due to poor subgrade condition.

The following sections briefly describe the ground improvement methods used at this airport during the last decade.

PREFABRICATED VERTICAL DRAIN WITH PRELOADING

The PVD with preloading method (Figure 5) has extensively been used for treating the subsoils of the airside and landside areas. The improvement via this method had proven to be very effective, but it requires long construction period and counterweight berm for stability purpose. It is usually more economical to construct the embankment in stages, benefiting from the gain in strength through consolidation in the previous stage. The number of loading stages varied from 2 to 3 stages depending on the surcharge loads, the dimensions of counterweight berm and the construction time. This method was adopted in areas where there was sufficient right of way for counterweight berms, having relatively long improvement period of over one year. Method of estimating the consolidation settlement is given in later section.

![Fig. 5 Typical PVD Section of Runway](image)

The major components in the PVD with preloading method in the airside system were:

- **Filter fabric**: Two layers of filter fabric were laid to isolate the sand blanket from any contamination.
- **Sand Blanket**: Low fine content (< 5%) sand was specified as the drainage blanket with thickness of 1.5 m.
- **PVD**: PVDs installed down to elevation of -10 m MSL with 1 m spacing in square grid pattern were used.
- **Drainage hose and pumping manhole**: Drainage hoses and pumping through manhole were adopted to keep the water level in the sand blanket as low as possible, creating a maximum possible hydraulic gradient in the system.
- **Counterweight berm**: The berm was constructed beyond the PVD installed zone on both sides for stability purpose.
- **Surcharge fill and counterweight berm**: The surcharge fill was placed as loads in the improved zone.
The PVD designs adopted for the airside and landside systems differed in many aspects, including the PVD arrangement and spacing, use of filter fabric, thickness of sand blanket, types of drainage pipes, outlet drainage system, use of counterweight berms and types of surcharge materials etc.

The maximum embankment height varied from 3.8 m to 5.2 m with loading in 2 to 3 stages. The designed embankment height depends greatly on the available construction period and the right of way. Typically, a 2:1 side slope was used for low embankment (h< 2.5m), but a 4:1 side slope was adopted for high embankment to reduce the effect from erosion. It should be kept in mind that high embankment is usually constructed along with counterweight berm for stability purpose; therefore the gradient of side slope is integrated with the berm.

INSTRUMENTATION AND MONITORING

For verifying the performance as well as controlling the construction work, several types of monitoring instruments were used, including settlement plates, settlement monuments, deep settlement gauges, piezometers, inclinometers, observation wells etc. Each type of instrument as shown in Figure 6, serves with the following purposes:

- *Settlement plates:* The settlement plates were installed to monitor the vertical settlement of the original ground. The measurements were used for determining the degree of consolidation during waiting period, which are the most important readings in the monitoring works.
- *Settlement monument:* The settlement monuments were used for checking the compression of the surcharge fill when the measurements were combined with the settlement plates. Since the degree of compaction on the surcharge was relatively high, therefore the measurements of the monuments are relatively unimportant.
- *Deep settlement gauges:* The deep settlement gauges were installed at various depths from 2 to 16 m, to quantify the compression of each soil layer. They were also used to correct the vertical locations of the piezometers installed within the compressible clay layer during consolidation.
- *Piezometers:* Several types of piezometers were specified in the work, including electrical piezometers, pneumatic piezometers and standpipe piezometers. The electrical and pneumatic piezometers were primarily installed in the clay layers because of their quick responses. Standpipe piezometers were used to check the piezometric pressure in the lower sand at depths of 25 to 45 m.
- *Inclinometers:* Inclinometers were installed at critical locations of the surcharge embankments. The ratios of maximum lateral movement and vertical settlement were used to check the stability during stage loading.
- *Observation wells.* Observation wells were installed in the sand blanket of the PVD system to check the water heads at the outlets of the system, ensuring that the water was properly drained out of the system with minimum build-up of pressure.
Fig. 6 Monitoring Instruments used in PVD System

Fig. 7 Typical Instrumentation Arrangement for Runway
CRITERIA FOR ACCEPTANCE

The criteria for acceptance of the PVD preloading, i.e., the earliest time when the surcharge could be removed, are based on the degree of settlement (S) for the preloading estimated from the Asaoka method. In addition, the settlement ratio (defined as the change in settlement over the past month to the present settlement) would have to be less than 3-4% with degree of consolidation of over 80%.

Asaoka method (1978) is commonly used to estimate the magnitude of final settlement as well as the horizontal coefficient of consolidation from the measured settlement data. This method adopts a curve fitting procedure based on the consolidation theory, and some essence of the method is explained below.

The solution of the consolidation equation under radial drainage condition takes the following form:

\[ \rho(t) = \rho_f - \rho_f \exp \left[ -\frac{8c_h}{d_e^2 F} \right] \] or \[ \rho(t) = \rho_f \left[ 1 - \exp(\lambda \Delta t) \right] \] where \( \lambda = -\frac{8}{d_e^2 F} c_h \)

By expressing the above equation in the form of an ordinary differential equation, and dividing the time evenly into \( \Delta t \) interval, the constants, \( \beta_0 \) and \( \beta_1 \), are obtained:

\[ \beta_1 = \frac{1}{1 - \lambda \Delta t} \] and \[ \beta_0 = \frac{\Delta t}{1 - \lambda \Delta t} \]

The constants (\( \beta_0 \) and \( \beta_1 \)) are the intercept and the slope of the fitted straight line in \( \rho_i \) vs. \( \rho_f \) axes, which can be obtained graphically. The final settlement and the horizontal coefficient of consolidation can also be obtained from the following expressions:

\[ \rho_f = \frac{\beta_0}{1 - \beta_1} \] (based on \( \rho_i = \rho_{i-1} = \rho_f \) as time approaches infinity)

and \[ c_h = \frac{(1 - \beta_1) d_e^2 F}{8 \beta_1 \Delta t} \]

It should be emphasized that some experience in data interpretation and analysis would be needed; otherwise the results might be misleading.

SOIL CEMENT COLUMNS

Soil cement column method (Figure 8) was primarily used for supporting road structures due to limited construction time. The primarily use of this method is to transfer the surface loading (including embankment, pavement and traffic load) to soil cement columns in the same way as pile foundation. As a lateral retaining member, it is commonly used for slope protection in irrigation work.
Fig. 8 Typical Soil Cement Column Design

The cement column method has been evolved from the lime column method, starting in 1960s and it was introduced into practice in 1974 in Asia. The cement column method by means of cement slurry was brought into practice in 1975 and the dry cement powder mixing was introduced in 1980. These two methods are sometimes referred to as wet method and dry method, respectively. Presently in Thailand, the wet method is extensively used in highway projects, and the dry method has also been used on a number of projects.

Soil cement column can be formed by injecting grout at high pressure (known as jet grouting) to the soil, creating a soil cement mix column as the injection rod rotates. Once the mixture is cured, a column is formed. Jet grouting will not produce a uniform cross section without a guiding blade; therefore, it is not very popular because higher amount of cement is needed to overcome the non-uniformity.

Typical, the soil cement column zone is modeled as a block in the stability analysis by substituting weighted shear strength of improved and unimproved areas to the value of the original soil. With a ratio of column spacing to column diameter of 2.5 and undrained shear strengths of improved and unimproved soils of 30 and 1 ton/m², the average shear strength of the stabilized soil mass will be 4.7 ton/m².

Because of low embankment and pavement, a sand cement stabilized mat of 0.6 to 1 m was introduced to improve the load transfer mechanism. The unconfined compressive strength of the mat was specified at 200 ton/m².

PILE FOUNDATION (BEARING UNIT)

The pile foundation is typically used for supporting embankment structure by transferring of loads to deeper stronger soil strata. In Thailand, the type of structure as vertical support member has been adopted in the west Outer Bangkok ring road as well as km 15 of Bangna-Trad Highway. The post construction settlement of the road supported by piles can be controlled by pile dimensions (size, length and spacing) and pile cap structures, depending on the requirements. As for lateral retaining system, the piles will have to withstand the forces acting, additional reinforcement may have to be used to increase the lateral resistance of the piles. The method is relatively simple and fast, but it is more costly than other methods.
Fig. 9 Pile Foundations in Bridge Approaches

VACUUM CONSOLIDATION WITH PRELOADING

This method was introduced to Thailand several years ago to improve the soft clay of a 2 km road in Samut Prakan with subsoil condition softer than this airport site. It was adopted primarily because of time constraint. The main advantage of this method is that it accelerates consolidation of the soils with installed vertical drains in a relatively stable manner. In conventional PVD method, stage loading is required to control the stability through strength gain via consolidation, but the vacuum consolidation method increases the effective stress with very small change in the shear stress, creating a gain in effective stress with better stability. Typically, a 6 ton/m² or 60 kPa of vacuum can be applied to the depressurized improved zone with installed vertical drains as illustrated in Figure 10. The effectiveness of the method depends greatly on the sealing or isolation of vacuum within the depressurized zone and the distribution of vacuum in the drains. Therefore, the drains have to be designed to withstand the vacuum pressure; any collapse of flow channel within the drains would result in catastrophic consequences, such as embankment failure or unacceptable degree of consolidation. As a result, this type of work is normally executed by ground improvement specialists. Each specialist firm would adopt his own vacuum application system ranging from the type of drains to connections and vacuum pumps. Therefore, the work was executed as a package based on performance basis with guidelines specified in the design.

Apart from applying the vacuum pressure, it is also common to place additional surcharge fill on top of the depressurized zone to increase the total stress of the soil, resulting in acceleration of consolidation and reduction in the consolidation time. But it should be
noted that there is also a limit in placing the surcharge due to stability as in PVD preloading method. Therefore for high surcharge load, there will be a need to perform staged loading or to introduce counterweight berm for improving stability during consolidation.

![Diagram](image.png)

Fig. 10 Typical Vacuum Consolidation System

For this site, a maximum surcharge height of 2.8 m was adopted with vacuum pressure of less than -6 ton/m², giving an equivalent vertical load of 11.6 ton/m².

**Consolidation Analysis**

The induced settlement by the vacuum method is somewhat similar to the PVD preloading system, except less lateral deformation is expected in the vacuum system. The method of estimating the rate of settlement in both system is the same, but the surcharge of the PVD system is placed in stages to minimize any instability of the embankment.

For the vacuum consolidation method, once the vacuum is applied to the improved zone through the vertical drains, the piezometric pressure in the drains will reduce. Consolidation will occur due to the withdrawn of water from the soil by the suction pressure with time until the piezometric pressure is stabilized. With additional surcharge, greater hydraulic gradient between the soils and the drains is created, accelerating the flow of water from the soils; hence higher settlement rate can be achieved.

In the design of vacuum consolidation, the procedure is relatively simple. The calculations consist of estimation of total consolidation settlement and the rate of settlement. From the design load ($\sigma_d$), one could estimate the corresponding settlement ($\rho_d$) at the end of primary consolidation. For a given vacuum period ($t$), the degree of consolidation ($U_t$) can be obtained from the established graph of $U$ versus time for a particular drain arrangement, giving the total surcharge load ($\sigma_s = \sigma_d/U_t$). Since the soil is a non-linear material and the criterion of acceptance in loading is often based on measured settlement, therefore the degree of settlement has to be adopted instead. For a given stage in the consolidation process, the ratio of the settlement at $\sigma_s$ to the total settlement at $\sigma_s$ is defined as the degree of settlement ($S$), which is used as the criterion of acceptance in the GI work. The total settlement ($\rho_s$) at
\( \sigma_n \) is estimated from the Asaoka method; hence if the measured settlement reaches a value equal to \( S \times \rho_n \), then the criterion is satisfied.

The total consolidation settlement is estimated based on one-dimensional consolidation equation with soil parameters given in Figure 3. For estimation of settlement, the analysis divides the soil into two layers: upper layer of soft clay at depth of 0 to 10 m and lower layer of medium to stiff clay at depth of 10 to 20 m. The estimated settlement versus surcharge load is presented in Figures 11 and 12 for the upper and lower clay layers, respectively.

The rate of settlement is estimated based on a vertical drain spacing of 0.85 m arranged in triangular grid pattern with the installation length of 10 m (-10 m MSL) under radial flow condition. The settlement rate of the lower clay layer is calculated by assuming a double vertical drainage condition with drainage boundaries at 10 and 20 m depth.

For radial flow as in vertical drains, Barron (1948) proposed a solution for consolidation by radial drainage only as follows,

\[ U_h = 1 - e^{-\frac{c_h t}{T_h}} \]

where \( T_h = \frac{c_h t}{d_e^2} \)

- \( c_h \) = horizontal coefficient of consolidation
- \( d_e \) = diameter of equivalent soil cylinder = 0.89 m

Hansbo (1979) suggested that the factor, \( F \), might consist of the following components with consideration of the effect of smear zone and well resistance:

\[ F = F(n) + F_x + F_r \]

\[ F(n) = \ln \frac{d_x}{d_w} - 0.75 \]

\[ = 1.84 \text{ for Band drain with } d_w = 0.067 \text{ m (} d_w \text{ = equivalent diameter of the drain)} \]

\[ F_x = \left[ \frac{k_h}{k_s} - 1 \right] \ln \frac{d_s}{d_w} \]

(\( \frac{k_h}{k_s} \approx 1.4 \text{ from consolidation test data at final vertical stress, and } \frac{d_s}{d_w} = 2 \text{ based on recommendation by Hansbo, 1979) } \]

\[ F_r = \pi z (L - z) \frac{k_h}{q_w} = 0.001 \text{ (} k_h = 0.014 \text{ m/year, } q_w = 1,000 \text{ m}^3/\text{year) } \]

Giving, \( F = 1.84 + 0.28 + 0.001 = 2.12 \text{ for Band drain} \)

It is assumed that the coefficient of permeability of the disturbed clay is similar to the vertical coefficient of permeability, giving rise to low \( F_x \) value. For the third component, since the discharge capacity of the PVD is over 1,000 m\(^3\)/year at hydraulic gradient of 1, the \( F_r \) value is therefore negligible. In summary, the \( F \) value is dominated by the first component; the other two components have small contribution to the degree of consolidation, hence the effect of smear zone is not of major concern. Based on the above equations, the relationship between the degree of consolidation and time is presented in Figure 11. For a consolidation time of 3 months, the degree of consolidation would be 60%, that is, the effective stress increase would be 0.6 of the applied surcharge load.
The rate of settlement for the lower clay is presented in Figure 12a, which is about 20 times slower than the upper clay with assistance from the vertical drains. The settlement of the lower clay dominates the post construction settlement with 80% consolidation occurred during the first 10 years.

To achieve a degree of consolidation of 60%, the estimated vacuum period would be 3 months with a gain in the effective stress of about 7 ton/m² (=0.6 x 11.6 ton/m²). It should be emphasized that the required total load is set at 11.6 ton/m² because of stability reason. Higher surcharge may be applied, but stability should be checked accordingly.

**Settlement Profile and Influence Zone Under Vacuum Method**

Apart from settlement in the depressurized zone, the area adjacent to the vacuum application zone will undergo some settlement as well. For improvement close to the existing pavements, the influence zone due to vacuuming has to be determined. The finite difference analysis has been adopted for determining the extent of affected area. Assuming a plane strain condition where one side of the soil block is subjected to vacuum or suction...
pressure, the effective stress along with the influence area will increase gradually with time. The governing equation used is based on one-dimensional consolidation theory:

\[
\frac{\partial u}{\partial t} = c_h \frac{\partial^2 u}{\partial x^2}
\]

where \( u \) = pore water pressure, \( t \) is the time and \( x \) is the distance.

The equation can be rewritten for finite interval of time (\( \Delta t \)) and distance (\( \Delta x \)) as follows:

\[
\frac{c_h \Delta t}{(\Delta x)^2} \left[ u_{x+1,t} + u_{x-1,t} - 2u_{x,t} \right] + u_{x,t}
\]

From the above equation, the pore water pressure at a given location in the next time step can be estimated from the current pore water pressure values, providing a way to project the pore water pressure in subsequent time. By applying this method to the problem with vacuum pressure of 6 ton/m² and a \( c_h \) value of 0.8 m²/year, the gain in vertical effective stress adjacent to the vacuum zone can be estimated. Based on the increase in effective stress, the settlement profile can be generated as shown in Figure 13. The results indicated that the vacuum would only affect the area of about 1-2 m beyond the boundary. The impact to the adjacent existing pavement would therefore be minimum. Nevertheless, inclinometer was installed close to the boundary for monitoring the lateral movement during vacuum application as well as surcharge loading.

![Graph showing settlement profiles](image)

Fig. 13 Surface Settlement Profiles of Vacuum Consolidation

**INTERFACING ZONES**

With different methods of ground improvement, the design of the interfacing zone becomes critical. For example, the rate of residual settlement or post-construction settlement between the pavements supported soil cement columns and improved soil by PVD will not be the same due to the difference in stress distribution and post-improvement properties of the subsoils. The adopted solution in the zone is to extend the soil cement columns or pile foundation into the PVD improved zone by around 6-10m depending on the estimated...
magnitude of differential settlements, forming a transiting zone. The length of the columns or piles may vary as illustrated in Figure 9.

**BACK ANALYSIS OF REFERENCE SECTION OF PVD SYSTEM**

**Finite Element Analysis**

In the contract of the Ground Improvement for the Airside Pavement (Phase 1), it was required to first construct a 500m “Reference Section” in the west runway close to the West Support Zone. The reference section was intended to be used for evaluating the performance of ground improvement work. The available data include soil investigation before and after the improvement, monitoring data of field instrumentation during construction period etc. The information gathered from the reference section can ultimately be used as the inputs for redesign of the ground improvement work. The embankment was loaded in two (2) stages, first at a height of 2.8m, followed by an additional fill of 1 m of fill. The measured unit weight of the sand blanket and surcharge fill were 2 ton/m³ and 2.3 ton/m³, respectively, giving a surcharge load of 8.3 ton/m², and this value is close to the range of 7.9 to 8.7 ton/m² in this project.

Some evaluation of the reference section has been carried out by the Construction Supervision team, TMSUM, which was summarized in “*Evaluation Report of Ground Improvement at Reference Section*”.

To obtain other important design parameters, further evaluation has to be made with emphasis on the deformation behaviour of the reference section in vertical and lateral directions. This task was achieved through calibration of input soil parameters in the finite element analysis FEM) having suitable soil model.

Finite element method (FEM) has been used to analyze the behaviour of the reference section embankment with PVD ground improvement. The commercial program used is PLAXIS Version 7, which is widely used for modeling geotechnical problems. This software has the capability of generating the mesh automatically once the dimensions and boundary conditions are defined. It also incorporates various soil models in the package, including Mohr-Coulomb, Cam-Clay etc. The interface behavior between different materials, such as soil and structure, can also be assigned in the software. Therefore, this program has been selected for modeling of the embankment deformation. For this problem, the selected soil model is “soft soil” model, which is more realistic and suitable for modeling the soil deformation problem under plane strain condition. In addition, to simulate the PVD drainage condition, the flow model has to be converted into 2-dimensional (2-D) condition, the equivalent 2-D parameters are demonstrated in Figure 14.
The back analysis was done to calibrate the soil model used for the later ground improvement design work. The input parameters after a series of calibration are presented in Figure 3, along with the boundary conditions shown in Figure 14.

**Measurement Versus Prediction**

The results of the finite element analysis executed with calibrated input data are compared with the measured data, including vertical and lateral deformation. Figure 15a shows the vertical settlement along the centerline versus time of the embankment; the predicted curve is fairly close to the measured curves. The final settlements in Figure 15b, are estimated based on Asaoka method, giving values ranging 134 cm to 157 cm for measurement and 156 cm for prediction. The surface settlement profiles across the embankment at different stages of loading were compared as shown in Figure 15c, showing very good match. The lateral movements measured by inclinometers at 37 m away from the centerline of the embankment were superimposed with predicted profile from FEM at different stages of loading as presented in Figure 16, giving relatively good match.
Fig. 15 Settlement-Time Curves along Centerline of Reference Section
Fig. 16 Lateral Movement at 37m from Centerline of Reference Section

The above example has demonstrated the usefulness of the finite element analysis in analyzing 2-dimensional problem of PVD system. The results had verified the soil parameters used, which were adopted in later GI design, such as the vacuum consolidation etc.

CONSTRUCTION RELATED ISSUES

For various ground improvement projects executed, the construction related issues for each method are somewhat different. Some important issues encountered are briefly described in the following sections.

Initial Ground Elevations and Flooding Control

The first technical issue at the start of the work was related to the average ground elevation. During design stage, topographical survey was conducted to map the ground elevations. Measurements of ground elevations made during construction were often different from those on the design drawings. This discrepancy was often due to unaccounted settlement caused by land subsidence occurred between design and construction stages, volume reduction due to drying of upper soft soil after site clearing. This factor had affected the quantities of fill materials required to certain extent.

Before the completion of pumping stations, several small dikes were constructed by the contractors to control the water level within the ground improvement zone. From the monitoring records of the PVD system, it was found that the level of water table within the
improvement area could affect the rate of settlement, hence temporary dikes were introduced to isolate and to control the water level in the improvement area.

Conventional PVD Preloading System

Very few technical problems were encountered in the PVD preloading system, and the common issues were related to the quality of construction materials, managing the schedule of recycled materials. In the first PVD work, there were some problems related to the PVD specifications, such as material orientation for strength test, but the issue was later resolved with supporting technical information from independent parties. The PVDs were primarily manufactured in Thailand with reasonable quality control as indicated in routine quality control checks.

The quality of the sand blanket was a main concern, it appeared that the sand quality deteriorated over the years due to the shortage of material in nearby areas, hence the contractors had to haul the sand from a greater distance, increasing the cost. River sand was initially used, but in recent years, sand recovered from the residual soils through sieve washing in Chonburi area had replaced the river sand. The variation in the quality of sand was much greater than before, leading to tighter inspection and screening.

The main challenge to the supervision engineers involved in the work was how to manage the recycling of surcharge materials, so that the use of surcharge could be optimized. In the first phase of PVD ground improvement work, crushed rock was used as the surcharge. Since the entire surcharge would eventually be removed after completion of consolidation to allow for the pavement work. Therefore the crushed rock to be used in the pavement construction was imported during the GI stage as surcharge, resulting in significant cost savings. Using the crushed rock as surcharge had several advantages, including low degree of compaction required, high unit weight compared with compacted sand, and no dust problem. Due to the demand of general fill within the site, sand was imported to be used as surcharge fill in later projects rather than using crushed rock as surcharge.

In a number of PVD projects, the waiting period of the first stage loading with surcharge of 2.8m was shortened during to tight construction schedule. Close monitoring on the lateral movement was made during second stage loading to prevent any possible failure. Fortunately, no failure was resulted from this variation.

For all PVD preloading works over the decade, no failure had encountered.

Soil Cement Column Method

In the soil cement column method, it should be emphasized that the quality control plays a very important role in the construction. There are a number of soil cement column installation contractors in Thailand. The mixing techniques include dry mixing, wet mixing and jet grouting. Each contractor had adopted different types of equipment, mixing techniques etc., leading to large variation in the quality of the soil cement columns. Special attention should be placed in the following areas:

- Variability of Shear Strength of Soil Cement Column: Columns can be easily formed with the same diameter by means of mixing blades or auger. However, the uniformity of columns with respect to shear strength can have significant variation. The quality of the
columns is depending on not only the mechanics of installation machine and the characteristics of soft ground, but also on workmanship. In addition, even though mixing of cement with in-situ clay might have been done very carefully, the shear strength of the cement column can often be non-uniform. It should also be noted that the individually installed cement columns might have many weak zones, such as joints and fissures.

- **Verification of Soil Cement Column Quality:** Field verification tests are essential in checking the quality of the column, and the installation procedure of the tested columns should be similar to the routine procedure. During installation, automatic data logging system should be used to collect the installation information as the main tool for quality control.

- **Other Components in Soil Cement Column work:** There are many uncertainties in the installation method, especially regarding the operator's skill that will affect the quality of columns severely. Even if those columns have weak zones, settlement can be greatly reduced.

From the construction records and post-construction measurements, it can be seen that the quality of the soil cement column works performed by various contractors varied to a certain extent. Some areas had encountered a pavement settlement of over 10 cm within one year after construction, and others did not show any sign of large settlement. Therefore, the quality control is the most important issue in the soil cement column work.

Apart from the soil cement columns, a cement stabilized mat was also introduced with thickness ranging from 60 to 100 cm. Any poor connection between the soil cement columns and the sand cement stabilized mat will lead to uneven settlement of pavement structures. The mixing process also varied among the contractors, some had mixed the sand and cement in the cement mixing plant, and others had performed in-situ mixing with the use of graders.

**Pile Foundation**

Since the piles used are relatively short, there was no difficulty encountered during installation of piles. No pile load test was performed since the piles were designed as floating structures.

**Vacuum Consolidation System**

It should be noted that the vacuum consolidation system is considered to be a very specialized field, and the contractors involved are generally specialist firms. The methods of applying the vacuum and materials used in the system vary significantly from company to company. The work performed at this airport was executed by a Dutch specialist, which applied the vacuum directly to individual PVD through a network of flexible tubes as shown in Figure 17.
Initial problems encountered included the installation of PVD through compacted sand, durability of the vacuum pumps, wetting of upper soil layer etc., these problems were overcome by the contractor. Typical results of the vacuum consolidation method are shown in Figure 18, indicating a single stage loading with settlement reaching the required value within 4 months of construction.

Fig. 17 PVD and Tubing in Vacuum Consolidation

Fig. 18 Typical Results of Vacuum Consolidation Method

One of the main uncertainties in the design of the vacuum consolidation method was to determine the extent of the influence zone adjacent in the depressurized zone. Since most of the vacuum consolidation areas were connected to existing pavements, therefore there was expected to have some impact in the interfacing zone. Vertical cracks along the edge of the pavement shoulder and the improvement zone were encountered in one of the connections as illustrated in Figure 19, but the range was within the design estimates.
SUMMARY

The ground improvement works conducted at the Suvarnabhumi airport is considered to be one of the largest in the world. Various ground improvement techniques were adopted to serve their objectives with significant success. Local consulting firms and contractors had benefited greatly from these constructions in terms of experiences and capabilities.

REFERENCES


DESIGN AND CONSTRUCTION OF THE UNDERGROUND TRAIN STATION

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[formerly, Resident Project Manager with TAMS Consultants]

ABSTRACT

Just like the Second Bangkok International Airport project, the Underground Train Station has been on and off, although over a shorter time scale. The planning aspects are described, together with the progress from the inclusion of diaphragm walls in the original piling contract, through the first stage of enabling works to the completion of the station box. The objective of the enabling works has been to build a “generic” box into which a station may be designed and built by others, under the control of the State Railway of Thailand who will operate the railway.

The design includes the provision of diaphragm walls, to allow top down construction in an area sensitive to building movements, and of a base slab for which the major forces in addition to the downwards vertical loads, are the heave of the soil below formation level as a result of stress relief, and the potential for hydrostatic uplift. Consideration was given to the use of a compressible layer to minimize the effects of the base heave, but the uplift forces were so great that, when applied to the base slab nearly 50 m wide, they produced unacceptable bending moments and required an excessive amount of piles or barrettes in tension. As a result it was decided to provide a permanent underdrainage system to prevent the full hydrostatic pressure from ever being built up.

Both bored piles and barrettes were used to resist the upwards forces on the base slab due to heave, and these were the subject of static load testing to confirm design parameters. Some of the barrettes were fitted with steel stanchions to provide temporary support to the roof slab and concourse slab during the top down construction.

The Elevated Frontage Road, which gives vehicular access to the Main Terminal Building at Departure and Arrivals Levels, has provision for footings which lie on the centre line of the terminal and therefore are within the station box. The piles for these, and temporary diaphragm walls to allow construction of the pile caps at considerable depth (below station platform level), had already been constructed. However the lowering of the station platform caused considerable difficulty as there was less than 2 m of diaphragm wall penetration below the final excavation level.

INTRODUCTION

The Master Plan for the Second Bangkok International Airport, produced by the General Engineering Consultant (GEC) in about 1993, was based on a Phase I with 30 million annual passengers (MAP) and a staged increase to a final phase of 100 MAP. The site at Nong Ngu Hao had been selected and allocated for the new airport since the early 1960s, such that the planners were working from a “blank sheet of paper”. One of the first considerations has to be the modes of transport to a new city airport, and their relative importance. Within the
region both the new Hong Kong Airport at Chek Lap Kok, and the new Kuala Lumpur International Airport in Malaysia, are distant from their city centers and it was decided that rail would be a major mode of transport. Conversely, the Singapore International Airport at Changi, and the Second Bangkok International Airport (SBIA), have been based on the assumption that most passengers will travel by road. This has a major impact on the positioning of the Passenger Terminal, because railways cannot turn on sharp radii. This means that, for major rail access, the Passenger Terminal must be outside of the footprint of the runways, so that the tracks can run across the face of the terminal building at a convenient level. As a result, there is increased travel distance from the check-in desks to the aircraft, and this means that an Automated People Mover (APM) is required at Chek Lap Kok, and other airports of similar geometry.

At the SBIA the Passenger Terminal Complex (PTC) has been tucked up inside the runway footprint, keeping to a minimum the taxiing distances for aircraft on both landing and take-off, as well as the distances passengers have to travel to reach their aircraft after check-in, but the cost is that a railway line must then be parallel with the runways, which means that it must be below ground.

![Diagram of the SBIA layout](image)

Fig. 1 Master Plan Layout for 100 Million Annual Passengers Showing Railway Route

GEC therefore planned for an underground train station in the North Terminal for Phase I, with the option to extend the underground railway to the south to connect up with the future South Terminal (see Figure 1). In fact the railway will be the main, if not only, means of landside access between the North and South Terminals (airside access being provided by the future APM), since there is no space for a North/South Landside Road within the airport perimeter when the final phase is reached with 4 runways.
With this in mind, the PTC designers, MJTA, included in the initial piling contract two diaphragm walls running North/South underneath the Main Terminal Building. They were 1 m thick and 49.4 m between faces, with a depth of about 33 m, which took them into the dense sand layer. However, there were serious concerns about safety, and senior members of the team were convinced that an underground railway was a major security threat. This concern was increased just before the piling work started as a result of the disaster at the World Trade Center in New York on 11 September 2001. Nevertheless, after some uncertainty, the New Bangkok International Airport (NBIA), determined to go ahead, at least with the first two walls. These were completed within 2002, and totaled about 30,000 m².

![Fig. 2 Detail of Passenger Terminal Complex and Diaphragm Walls in Original Piling Contract](image)

Because of the uncertainty over the future of the underground train station, only the two outer parallel walls were constructed initially, as shown in Figure 2, and this was clearly not enough to allow excavation of an underground station box, most particularly because there was no end wall to retain the upper soft clay layers. Therefore, in early 2003, NBIA requested the main contractor for the PTC, Italian-Thai Takenaka Obayashi Joint Venture (ITO), to prepare a proposal for the Design and Construction of the Underground Train Station Enabling Works. These were to be the minimum necessary to allow a station to be constructed at a later date without causing damage to the Main Terminal Building, and to minimize disruption to the operating airport.

They included the design and construction of the South end diaphragm wall, the extension of the side diaphragm walls, the excavation and casting of the roof slab for a length of 150 m and a temporary diaphragm wall at the North end. ITO appointed Arup as their Designer. However, in order to do the design required, the planning for the eventual station also had to be done to ensure that the construction would allow the future station to meet the anticipated passenger flows, safety requirements, M&E plant etc. Along with the top slab it was necessary to install all of the piled foundations to the final structure, since they could not be constructed through the roof slab once in place, even though no excavation was planned at
this stage. In the design process it was found that the vertical downwards loads on the base slab at a depth of about -12.5 m MSL were relatively small, but the upwards forces from both base heave of the underlying stiff clay and the potential hydraulic uplift if ground water levels were to rise in the future to near ground level were very significant, and caused excessive bending moments in the 49.4 m wide slab. After some investigation it was decided that the best solution, for the first 150 m, was to minimize the base heave by providing a compressible layer which would collapse if a predetermined upwards pressure was exceeded.

Another feature was the inclusion of deep foundations, below the future base slab, for the Elevated Frontage Roads (see Figure 3). The piles for most of these had been installed under the PTC Piling Contract, as well as temporary diaphragm walled boxes to allow the construction of the pile caps. The first stage works had to include the excavation within the temporary diaphragm walls, the forming of the pile caps and of the pier back up to ground level, with provision for connections into the future Base and Concourse Slabs. This work was complicated by the fact that the whole station platform level had been lowered since the original MJTA design, which meant that the pile caps were also lowered and were nearer to the toe level of the existing diaphragm walls.

![Diagram of the First Stage Works](image)

Fig. 3 Plan of the First Stage Works (the Face of the MTB is on Gridline Q)

The first stage of these works started in early 2003, but in September the Cabinet agreed to build the whole airport railway, with a City Air Terminal at Makkasan and both an Airport Express and a commuter line running on the same elevated tracks into the Suvarnabhumi Airport Station. ITO were therefore requested to Design and Construct the remainder of the Enabling Works, completing the underground works where the tunnel reached ground level, at about 1,000 m from the South end wall. The main works included an additional 38,000 m$^3$ of diaphragm wall, 200 bored piles of 1 m diameter, and 122 barrettes of 3 x 1 m plan size; the excavation and casting of the Roof Slab, with openings; the excavation and casting of the Concourse Slab, with openings; the excavation and casting of the Base Slab; the forming of the box tunnel, which forms a ramp from the Base Slab up towards ground level and passing under the Airport Hotel; the construction of a tunnel portal and an open channel section up to the ground surface; a Plant Room, which was to be above ground level to allow space at Concourse Level to be used as a Retail Area, through which passengers will walk between the Airport Hotel and Main Terminal Building.

Some of the special features included allowance for details under the hotel to minimize noise and vibrations, the need to accommodate a fuel pipeline crossing the route for the
hydrant refueling system, and the requirements of the baggage handling system to take checked baggage from the Airport Express straight into the PTC baggage handling system. It was also found, during design development for the larger area, that the compressible layer was no longer the most viable option for uplift force resistance, and that it was more economical to provide a permanent underdrainage system with permeable, layer, pipes and pumps.

MAIN DESIGN ISSUES

The main design issues for the Underground Train Station Enabling Works can be divided into the following:

- Soil parameters
- Diaphragm walls and barrettes
- Bored piles
- Underdrainage system
- Elevated Frontage Road foundations

These will now be discussed in turn.

Soil parameters

The general description of the soils and soil properties in the area of the SBIA have been described by others (Moh et al (2006), Sambandharaks (2006)) and will not be repeated here. However it is worth noting that, whereas the stratigraphy within urban Bangkok which has also been described in many technical papers, is characterized by a “First Sand Layer” about 5 m in thickness at about 25 m below ground level and a “Second Sand Layer” at about 45 m below ground level, noting that is some areas such as Wireless Road the “First Sand Layer” is missing, at the SBIA site it is the intermediate clay layers which are missing or very thin and, throughout the area of the PTC, the sand seems to be generally from 25 to 45 m.

Based on the local borehole information but also using their previous experience in Bangkok, particularly their design work for the ION Joint Venture on the MRTA North Project, Arup proposed the following general stratigraphy:

Table 1 General Soil Profile

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Thickness (m)</th>
<th>Typical Depth to Top of Stratum (mbgl)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Made Ground</td>
<td>0 to 1.5</td>
<td>0</td>
</tr>
<tr>
<td>Bangkok Soft Clay</td>
<td>13 to 16</td>
<td>0 to 1.5</td>
</tr>
<tr>
<td>First Stiff Clay</td>
<td>6 to 11</td>
<td>14 to 16</td>
</tr>
<tr>
<td>Bangkok Aquifer – Upper Sand</td>
<td>18 to 24</td>
<td>21.7 to 26</td>
</tr>
<tr>
<td>Intermediate Clay Layer</td>
<td>1.1 to 2.3</td>
<td>30 to 32</td>
</tr>
<tr>
<td>Lower Clay</td>
<td>4 to 16 (proven)</td>
<td>43.5 to 46.5</td>
</tr>
</tbody>
</table>
For the detailed design the stratigraphy was modified slightly based on three local boreholes for the station, as shown in Table 2, and the soil parameters assigned to each layer are shown in Table 3.

### Table 2 Local Soil Profile

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Thickness (m)</th>
<th>Level of Top of Stratum (mPTD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bangkok Soft Clay</td>
<td>15.5</td>
<td>-1.5</td>
</tr>
<tr>
<td>First Stiff Clay</td>
<td>9</td>
<td>-17</td>
</tr>
<tr>
<td>Bangkok Aquifer – Upper Sand</td>
<td>7</td>
<td>-26</td>
</tr>
<tr>
<td>Intermediate Clay Layer</td>
<td>3</td>
<td>-33</td>
</tr>
<tr>
<td>Lower Sand</td>
<td>9.5</td>
<td>-36</td>
</tr>
<tr>
<td>Lower Clay</td>
<td>&gt; 17.2</td>
<td>-45.5</td>
</tr>
</tbody>
</table>

NB: mPTD refers to the Passenger Terminal Datum which is +1.8 mMSL.

### Table 3 Soil Parameters

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Undrained shear Strength (kPa)</th>
<th>Drained shear Strength (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bangkok Soft Clay – 0 to -7.5 mPTD</td>
<td>$c_u = 10$</td>
<td>$c' = 0; \phi' = 20^\circ$</td>
</tr>
<tr>
<td>-7.5 to -12.5 mPTD</td>
<td>$c_u = 0.35\sigma'$</td>
<td>$c' = 0; \phi' = 26^\circ$</td>
</tr>
<tr>
<td>-12.5 to -17 mPTD</td>
<td>$c_u = 21 + 6z$</td>
<td>$z$ measured from PTD</td>
</tr>
<tr>
<td>First Stiff Clay</td>
<td>$c_u = 50 + 5z$</td>
<td>$c' = 0; \phi' = 26^\circ$</td>
</tr>
<tr>
<td>Bangkok Aquifer Sand</td>
<td>$N_{60} = 26 + 0.5z$</td>
<td>$\phi_{\text{peak}} = 36^\circ$ $\phi_{cv} = 30^\circ$ $z$ measured from -26 mPTD</td>
</tr>
</tbody>
</table>

### Diaphragm walls and Barrettes

The initial design by MJTA had assumed that the base slab was at about -12 m relative to the PTC datum at +1.8 m MSL, but had no slab at roof level to provide a propping force. The upper part of the diaphragm walls, above Concourse Slab level, was therefore an unpropped cantilever. It was later found that the whole tunnel had to be lowered about 2 m to avoid conflict with a near surface utility corridor, but the effect on the upper part of the diaphragm walls of the increased lever arm was compensated by introducing a roof slab which provided a significant prop just below ground level. However the toe-in of the diaphragm walls beneath the formation level was minimized. The key design requirements were therefore as follows:

- Formation level: -17 mPTD
- Top slab (roof): 500 mm thick at +0.35 mPTD
Concourse slab: 700 mm thick at -6.95 mPTD
Base slab: 2000 mm thick at -14.5 mPTD
Width: 49.4 m
Cut-off level: -1.35 mPTD
Panel length: 3 to 4.5 m

The main purposes of the diaphragm walls, and barrettes, were to:

- provide lateral support to the excavation – short term and long term
- provide vertical support to the station roof and concourse slabs prior to installation of base slab
- limit groundwater ingress
- ensure adequate factor of safety against uplift due to water pressure and soil heave
- limit deflections outside the excavation which might harm adjacent structures

The key design assumptions were as follows:

- Groundwater
  - would be as the design profile, neither higher during construction nor lower in the future
  - in the long term would be hydrostatic from ground level, or from +1 m in the extreme case

- Structural
  - The prescribed construction sequence would be followed by the Contractor
  - An over-excavation allowance 0.5 m was made for the upper slabs but 0 for the base as the maximum depth was already critical
  - The slab connections to the walls would be pinned
  - 25% slab stiffness reduction would be allowed for openings
  - Wall tensile strength and stiffness would come from reinforcement only
  - All vertical loads on side walls before casting of the base would be carried on side walls
  - An ULS design would be carried out using \( c_u/1.2 \) for soft clay, \( c_u/1.5 \) for other clays and \( \phi = \tan^{-1}(\tan \phi/1.2) \)
  - At ULS the cracked stiffness would be taken as 0.7EI
  - Wall creep occurs before the soil reaches its at rest pressure

- Uplift resistance
  - The diaphragm wall resists uplift by weight of walls and slabs; if this is not enough then skin friction or the weight of soil wedges will be added, whichever is less
  - Skin friction in tension and compression are equal
  - The active soil wedge rises at 1:4 up to 3m maximum distance from the wall; passive soil wedge at 1:4.
Surcharges

- No backfilling to 0mPTD till base cast and up to strength
- Traffic load 20 kPa
- No permanent development loads allowed

The Designer for the Contractor, Arup, had recently gained experience in the design and construction of diaphragm walls for underground train stations on the MRTA North Project. Some of their findings have been published (Davies et al 2001), and these were used to optimize the design of the additional diaphragm walls. As a result the stiffness parameters shown in Table 4 were utilized.

Table 4 Soil Stiffnesses

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Undrained soil stiffness $E_{u1h}$ (kPa)</th>
<th>Drained soil stiffness $E_{kh}$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bangkok Soft Clay</td>
<td>$800 \times c_u$</td>
<td>$660 \times c_u$</td>
</tr>
<tr>
<td>First Stiff Clay –</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Above -20.5 mPTD</td>
<td>$1000 \times c_u$</td>
<td>$830 \times c_u$</td>
</tr>
<tr>
<td>Below -20.5 mPTD</td>
<td>$1500 \times c_u$</td>
<td>$1240 \times c_u$</td>
</tr>
<tr>
<td>Bangkok Aquifer Clay</td>
<td>$1000 \times c_u$</td>
<td>$830 \times c_u$</td>
</tr>
</tbody>
</table>

Initially during the top down construction, the vertical loads are carried by the diaphragm walls and barrettes. However, once the base slab is installed any excess vertical loads can be carried by the base against the First Stiff Clay. In order to model this a stiffness of 1,500 kN/m/m² was used, based on laboratory test results. However, since there were to be unload/reload cycles and their effects on the stiffness were unknown, a sensitivity analysis was carried out using a stiffness of 15,000 kN/m/m². It was also noted that the First Stiff Clay will heave during the excavation and after casting the base slab, as a result of stress relief, and that this heave would continue until the reducing upward heave pressure was balanced by the downward pressure of the slab. This was modeled by assuming a heave pressure of 182 kPa, but with a spring stiffness of -2,400 kN/m/m², which would reduce the heave pressure with increasing displacement.

The vertical loads are summarized below, separated into the zones between gridlines X1 – X4 and X4 – X17, because of changes in roof slab level:

Table 5 Vertical Loads on Diaphragm Walls and Barrettes

<table>
<thead>
<tr>
<th>Load case</th>
<th>Between X1 – X4</th>
<th>Between X4 – X17</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Diaphragm wall (kN/m)</td>
<td>Outer Barrette (kN)</td>
</tr>
<tr>
<td>Construction</td>
<td>631</td>
<td>9159</td>
</tr>
<tr>
<td>Operation</td>
<td>970</td>
<td>13748</td>
</tr>
<tr>
<td>Uplift</td>
<td>-12</td>
<td>-5909</td>
</tr>
</tbody>
</table>
For the design of the diaphragm walls and barrettes, skin friction values were back analysed from a test barrette called NPB1. They are listed in Table 6, and from these the vertical capacities were calculated as shown in Table 7, and compared with the applied loads from Table 5.

Table 6 Back Analyzed Skin Friction for Diaphragm Walls and Barrettes

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Level of top of stratum (mPTD)</th>
<th>Back-analysed skin friction (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bangkok Soft Clay</td>
<td>-1.5</td>
<td>15</td>
</tr>
<tr>
<td>First Stiff Clay</td>
<td>-17</td>
<td>60</td>
</tr>
<tr>
<td>Bangkok Aquifer – Upper Sand</td>
<td>-26</td>
<td>60</td>
</tr>
<tr>
<td>Intermediate Clay Layer</td>
<td>-33</td>
<td>80</td>
</tr>
<tr>
<td>Lower Sand</td>
<td>-36</td>
<td>100</td>
</tr>
<tr>
<td>Lower Clay</td>
<td>-45.5</td>
<td>80</td>
</tr>
</tbody>
</table>

Table 7 Vertical Capacities and Loads

<table>
<thead>
<tr>
<th>Load case</th>
<th>Between X1 – X4</th>
<th>Diaphragm wall (kN/m) – toe level -29mPTD</th>
<th>Outer Barrette (kN) – Toe level -47mPTD</th>
<th>Centre Barrette (kN) – Toe level -47mPTD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Construction</td>
<td>690 (631)</td>
<td>9895 (9159)</td>
<td>9895 (9659)</td>
<td></td>
</tr>
<tr>
<td>Operation</td>
<td>824 (970)</td>
<td>14902 (13748)</td>
<td>14902 (14428)</td>
<td></td>
</tr>
<tr>
<td>Uplift</td>
<td>-548 (-12)</td>
<td>-7460 (-5909)</td>
<td>-7460 (-6249)</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Load case</th>
<th>Between X4 – X17</th>
<th>Diaphragm wall (kN/m)</th>
<th>Outer Barrette (kN)</th>
<th>Centre Barrette (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Construction</td>
<td>770 (680)</td>
<td>9379 (8970)</td>
<td>8845 (8475)</td>
<td></td>
</tr>
<tr>
<td>Operation</td>
<td>935 (1125)</td>
<td>12757 (15330)</td>
<td>11957 (16670)</td>
<td></td>
</tr>
<tr>
<td>Uplift</td>
<td>-603 (-275)</td>
<td>-6388 (-4787)</td>
<td>-5988 (-9029)</td>
<td></td>
</tr>
</tbody>
</table>

It will be seen that, particularly in the operation stage, between gridlines X4 and X17 there is an excess load over capacity for all elements. Considering a strip 10.8 m wide, the excess load will be \((2 \times (1125 - 935)) \times 10.8 = 4104 \text{ kN}\), \((2 \times 15330 - 2 \times 12757) = 5146 \text{ kN}\), and \((16670 - 11957) = 4713 \text{ kN}\). The full base slab area is \(49.4 \times 10.8 = 486 \text{ m}^2\), so additional downward pressure is \(13963/486 = 28.7 \text{ kPa}\). In the elastic analysis spring stiffnesses were chosen to give 5 and 25 mm settlement. A later static load test on an instrumented barrette found that the maximum skin friction in the First Stiff Clay was only 25 kPa. The design was therefore re-examined and the capacities downgraded. The additional excess load was carried on the base.
The following external loads were considered to apply horizontal pressures to the wall:

- Temporary construction load surcharge of UDL 10 kN/m²
- Backfill from existing ground level up to -0.3 mPTD = 1.5 m x 18 kN/m³ = 27 kN/m²
- Traffic surcharge of UDL 20 kN/m²

As a result of the vertical and horizontal loads the design bending moments and shears in Table 8 were derived.

Table 8 Bending Moments and Shear Forces in Diaphragm Walls

<table>
<thead>
<tr>
<th>Between gridlines X1 – X4</th>
<th>Bending Moment (kNm/m)</th>
<th>Shear Force (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Retained side</td>
<td>Excavation side</td>
</tr>
<tr>
<td>Construction</td>
<td>-2229</td>
<td>3349</td>
</tr>
<tr>
<td>Operation 1</td>
<td>-1346</td>
<td>2015</td>
</tr>
<tr>
<td>Operation 2</td>
<td>-1303</td>
<td>2129</td>
</tr>
<tr>
<td>Extreme</td>
<td>-1319</td>
<td>2132</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Between gridlines X4 – X17</th>
<th>Bending Moment (kNm/m)</th>
<th>Shear Force (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Retained side</td>
<td>Excavation side</td>
</tr>
<tr>
<td>Construction</td>
<td>-2006</td>
<td>3355</td>
</tr>
<tr>
<td>Operation 1</td>
<td>-1333</td>
<td>2164</td>
</tr>
<tr>
<td>Operation 2</td>
<td>-1320</td>
<td>2285</td>
</tr>
<tr>
<td>Extreme</td>
<td>-1345</td>
<td>2288</td>
</tr>
</tbody>
</table>

The wall reinforcement was designed to resist these moments and shears.

Bored piles

Bored piles were used to support the Elevated Frontage Road foundation, at gridline X18, seen at the left of Figure 3. The Designer produced a comprehensive Geotechnical Interpretative Report, based on many previous soil investigations carried out on the Nong Ngu Hao site, the specific boreholes drilled for this project, and their experience in Bangkok. This identified design parameters relevant to each soil layer. However when the undrained shear strengths for the First Stiff Clay were applied to the pile design, it was found that the result was significantly more conservative than that which had been proposed in Littlechild et al (1998). A back analysis was therefore undertaken of the static load tests carried out as part of the Piling Work for the PTC, and this is shown in the graph in Figure 4.

The design line based on the pile tests, with a limiting value of 100 kPa, was therefore adopted for the First Stiff Clay layer. Later a pile test was carried out on an instrumented working pile. This showed that the maximum skin friction values in all layers were less than had been expected, with a value of only 34 kPa in the First Stiff Clay. In a similar instrumented static load test on a barrette at the same location, a maximum skin friction value of 25 kPa was obtained. Both of these results were considered to be the result of the build up of excessive filter cake from the bentonite drilling fluid during the construction phase, and the subsequent inability of the flowing concrete to scour the surface clean. As a result the skin
friction of all piles already constructed was downgraded, and additional piles were constructed to meet the design requirements.

![Skin friction (kPa)](image)

Fig. 4 Design Skin Friction Values in First Stiff Clay

**Underdrainage system**

The underside of the permanent station base slab was designed to be at -16.5 mPTD which, with a slab thickness of 2000 mm and with an allowance for the rail track slab gave a finished rail level acceptable to the NBIA and the State Railway of Thailand. Although the groundwater level at present is about -20 mPTD, there is a significant risk that it will rise within the 120 year design life of the station. This would lead to uplift pressures due to both the water pressure and heave pressures from swelling of the clay as a result of decreased vertical effective stress. In order to resist these upward pressures three options were considered:

- Full restraint, by tying the slab to the diaphragm walls, barrettes and piles and installing any further barrettes/piles necessary
o Accepting the heave pressures but avoiding the water pressures by providing permanent underdrainage
o Limiting the heave pressures and designing for the water pressure

As mentioned above, the heave will start during the excavation process, but once the slab has been cast any further heave must be resisted by the weight of the slab, and the uplift capacity of the piles, barrettes and diaphragm walls. Additionally, if groundwater does rise, the reduction ineffective stress will itself result in further heave. The hydrostatic pressure alone accounts for 147 kPa of uplift, and if a zero slab deflection criterion is set this increases to 259 kPa.

When the full restraint option was considered for the portion under the terminal between gridlines L and R, if 10 mm upward movement of the slab were allowed there was still a requirement for 47 additional barrettes. This concept was therefore abandoned as impractical. However, outside the terminal between gridlines R and X17 the additional loads of the Elevated Frontage Road could be used to counteract heave, and only 12 additional barrettes were required.

The under-drainage option had been previously used at 6 high rise buildings in Hong Kong, the oldest of which was built about 23 years ago. If it was used at the station and assuming 5 mm of upward movement of the slab 8 extra piles would be needed under the terminal building between gridlines L and R. Outside the building, between R and X17, again the extra load of the Elevated Frontage Road meant that no extra piles were required. The under-drainage system would comprise a free-draining layer on a geotextile, and drains within the drainage layer would convey the water to sumps. From there pumps would discharge the water to the public drainage system. Monitoring devices would be provided to control the operation of the system, and a fail safe pressure relief system would also be needed. This method would deal with any leakage through the diaphragm wall without difficulty.

The permeability of the strata beneath the station are as shown in Table 9.

Table 9 Permeability of Soil Layers

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Thickness (m)</th>
<th>Level of top of stratum (mPTD)</th>
<th>Permeability (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bangkok Soft Clay</td>
<td>15</td>
<td>-2.45</td>
<td>$3 \times 10^{-9}$</td>
</tr>
<tr>
<td>First Stiff Clay</td>
<td>8.5</td>
<td>-17.45</td>
<td>$1 \times 10^{-9}$</td>
</tr>
<tr>
<td>Bangkok Aquifer – Upper Sand</td>
<td>23</td>
<td>-25.95</td>
<td>$1 \times 10^{-7}$</td>
</tr>
<tr>
<td>Upper Clay</td>
<td>11</td>
<td>-48.95</td>
<td>$1 \times 10^{-9}$</td>
</tr>
<tr>
<td>Lower Sand</td>
<td>&gt;2.7</td>
<td>-59.95</td>
<td>$1 \times 10^{-7}$</td>
</tr>
</tbody>
</table>

Using this data the inflow was calculated to be 2.3 m$^3$/day, and a Factor of safety of 5 was applied to this value. The drainage blanket was therefore selected to be a 500 mm thick layer of free draining sand, with a permeability of $1 \times 10^{-4}$ m/s. Since the First Stiff Clay with permeability $1 \times 10^{-9}$ was considered to be impermeable, the water would be held within the drainage layer and the uplift pressure on the slab would be zero. The standing water within
the drainage layer would create a pressure of not more than 10 kPa, which would reduce the water pressure in the First Stiff Clay and minimize heave pressure.

The drainage layer would be fully compacted to minimize future settlement, and a filter fabric would be placed between the drainage layer and the clay formation to prevent contamination. On top of the drainage layer a plastic sheet would be used to prevent grout from the concrete from contaminating the sand. The drainage pipes within the drainage layer would be 150 mm diameter slotted PVC, and these would lead the water to sumps with adequate storage capacity from which the water would be pumped to storm drains.

Regarding instrumentation, piezometers will be provided at 40 m centres to detect groundwater pressure rise, and flowmeters will be fitted to all pumps to record actual flows. Nevertheless the whole system is reliant on electromechanical processes and is therefore not fail safe. Standby pumps and power could be provided, but could also fail at the critical moment. A flow rate of 2.3 m³/day is equivalent to a groundwater level rise within the station box of only 0.14 mm/day, which means that there should be enough time to repair pumps etc. in case of a catastrophic failure. In addition relief wells are provided which will drain any excess water pressure into the under platform void in case of a complete breakdown of all operating systems.

When considering allowing a “heave space” to limit the uplift pressures and designing for the full hydrostatic uplift, it was found that the foundations in the zone under the terminal building, between gridlines L and R, were adequate to limit movement to about 5 mm with a 2000 mm thick base slab. This technique has been developed in London where rising groundwater levels are also a problem, and has also been used at Osaka in Japan. A proprietary expanded polystyrene material called “Cellcore” is used, which is strong enough to withstand the weight of wet concrete, but will collapse if subjected to a higher stress. However in the zone between gridlines R and X17 the Operation Stage 1 became the critical load case with a void under the slab, and both more and longer barrettes were required. This solution was neither economic nor practical.

The conclusions of the special design study carried out were that:

- Upward pressures could be as high as 260 kPa which would exceed the design capacity
- In the zone under the terminal building Cellcore could be used
- In the zone outside the building underdrainage was recommended

**Elevated Frontage Road foundations**

As seen in Figure 3, four footings for the Elevated Frontage Road were required in the station area. One near gridline X25; two more near gridlines X23 and X20; and a new one near gridline X18. The elevated Frontage Road was to be supported on portal frames, which straddled between X25 and X23, and between X23 and X20. Provision for a third (future) portal between X20 and X18 was also included. This explains why X23 and X20 are much larger than X25 and X18.

The excavations required were deep, more than 14 m, and in area where ground movements were critical. All excavations were therefore carried out within temporary diaphragm walls. The diaphragm walls for X18 were designed and constructed by the station
contractor to provide safe excavation to a formation level of -17 mPTD. They had internal dimensions of 14 x 11 m with twenty 1 m diameter bored piles inside, and were designed to be 1 m thick with a toe level of -24.5 mPTD, to be braced with 4 levels of struts. The construction sequence was as follows:

- Install diaphragm wall with cut-off level of -2.4 mPTD
- Excavate to -4 m PTD
- Install struts at -3 mPTD
- Excavate to -9.5 mPTD
- Install struts at -8.5 mPTD
- Excavate to -12.5 mPTD
- Install struts at -11.5 mPTD
- Excavate to -14.5 mPTD
- Install struts at -13.5 mPTD
- Excavate to -17.15 mPTD
- Construct 2.5 m thick pile cap and pier to surface
- Backfill

Calculations showed that the lateral stability of the walls met the Serviceability and Ultimate Limit States of the design codes, and that the factor of safety against base heave was greater than 1.5.

The diaphragm walls for X20, X23 and X25 had already been installed under the Piling Works contract. Because the platform level had been higher when they were designed, the toe level was only at -19.4 mPTD compared with the -24.5 mPTD for X18. The two larger boxes were 20 x 17 m internally, with forty two 1 m diameter bored piles in each. The pile caps were to be 3 m thick, requiring a formation level of -17.5 mPTD. The ground surface was therefore lowered to -3.35 mPTD before excavation started, limiting the excavation depth to 14.15 m. X25 was slightly easier since the pile cap was smaller and only 2.5 m thick. Nevertheless the toe of the diaphragm wall was less than 2 m below formation level and special support measures were required. It was decided to use a system of buried props, installed in narrow slit trenches, excavated one at a time across the narrower dimension of the box. In the longer direction the small spacing between boxes significantly reduced the horizontal soil pressures acting. The upper buried props were to be made of steel and removable, while the lower ones were reinforced concrete and permanent. Since the diaphragm walls were already in place the construction sequence was as follows:

- Excavate to -4.3 mPTD
- Install struts at -3.5 mPTD
- Excavate to -8 mPTD
- Install struts at -7.5 mPTD
- Excavate to -10.3 mPTD
- Install struts at -9.8 mPTD
- Excavate to -14 mPTD
- Install struts at -13.7 mPTD
- Excavate to -15 mPTD
- Dig one slit trench to -16.7 mPTD
- Install heavy steel section with concrete end connections to diaphragm walls
- Backfill and install next strut until all struts in place
• Excavate to -16.7 mPTD
• Dig one slit trench to -18.1 mPTD
• Construct in-situ concrete strut 500 x 600 mm deep
• Backfill and install next strut until all struts in place
• Remove heavy steel struts
• Excavate to -17 mPTD (X25) or -17.5 mPTD (X23 and X20)
• Construct pile cap and pier to surface
• Backfill

As with the new box at X18, calculations showed that with this construction method the lateral stability of the walls met the Serviceability and Ultimate Limit States of the design codes, and that the factor of safety against base heave was greater than 1.5.

CONSTRUCTION

In top down construction each slab is cast on the ground, and requires temporary support until the excavation is complete, the pile caps or base slab have been cast, and columns built back upwards to carry the slabs. One way that this is commonly achieved is to use very heavy steel stanchions cast into high capacity foundations such as barrettes. These act as temporary columns during the construction phase, and can be cast into concrete columns in the permanent phase. At the SBIA station the stanchions were fabricated box sections made form heavy steel plate. The overall construction sequence was therefore as follows:

• Install 1m thick diaphragm walls and barrettes, some of which have top down stanchions from ground level at -1.5mPTD
• Excavate, trim, place blinding concrete and cast 500 mm thick roof slab with a top level of -1.35 mPTD from X1 – X4; -0.30mPTD from X4 – X8; and +0.35mPTD from X8 – X17
• Excavate to temporary formation at -7.725mPTD
• Cast 700 mm thick concourse slab with a top level of -6.95mPTD
• Excavate to final formation level at -17mPTD
• Install 500 mm thick underdrainage system with geotextile, drainage layer, drainage pipes, and plastic sheet
• Cast 2000 mm thick base slab with a top level of -14.5mPTD
• Backfill to 0.00 mPTD

All construction was carried out as planned, with monitoring of groundwater levels and wall movements. No adverse situations arose and the works were completed ahead of schedule, the last concrete being poured in June 2005.

REFERENCES

EFFECT OF LAND SUBSIDENCE ON THE DEVELOPMENT OF SUVARNABHUMI AIRPORT

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School of Engineering and Technology, Asian Institute of Technology

ABSTRACT

Land subsidence from deep well pumping has been affecting Bangkok for the past 35 years. Its impact is particularly critical because of the flat low-lying topography and the presence of a thick soft clay layer at the ground surface that augment flood risk and foundation engineering problems, respectively. The subsidence reached its most critical state in the early 1980s when it occurred at a rate as high as 120 mm/year. The rate decreased in the subsequent period but the subsidence-affected area expanded following the growth of the city. Despite various attempts implemented to remedy the crisis, groundwater pumping from the thick aquifer system underneath the city continued to increase. The strict mitigation measures adopted recently, comprising a pricing policy for groundwater management, an expansion of tap water supply, and strict enforcement of groundwater laws, have resulted in a marked drop in groundwater use. However, the land subsidence will continue for a while owing to the time-dependent response of the soft clay layer and clay aquitards. The phenomenon affects the design development of major infrastructure projects of the city including the Suvarnabhumi Airport. The effects are outlined.

INTRODUCTION

Land subsidence from excessive deep well pumping has been the human-induced hazard affecting Bangkok since 1960s. Up till now, the situation has not been fully under control and it is rather critical compared with other affected cities in Asia as the Bangkok ground has already sunk almost 2.00 m since the onset of the subsidence. Its impacts on the city’s environment have been serious due to the unfavorable topographic, geologic and hydrological settings of the city. This includes the development of the Suvarnabhumi International Airport, the site of which is situated in the low-lying swamplike area in the eastern suburb of Bangkok (Figure 1). The area, which is locally known as Nong Ngu Hao, was once designated as a part of the main flood diversion route of the city.

Bangkok is situated on a flat low land in the southern part of the Lower Central Plain of the country. The Chao Phraya River flows through the city from the northern highland and discharges into the Gulf of Thailand, 25 km south of the city. Bangkok Metropolis has expanded rapidly during the past 40 years on both banks of the river. Due to its low-lying topography and close proximity to the sea, flooding threatens the city every year. Urbanization has resulted in drastic deficiency of natural drains increasing the flood risk. The city has to rely on flood protection systems to prevent inundation in the monsoon season. However, their effectiveness is only temporary because of the on-going problem of land subsidence from deep well pumping. The pronounced land subsidence of the city started in the 1970s (AIT, 1981). It intensifies the flood risk and creates numerous problems in foundation engineering practice. Ground surface in some of the city areas has sunk below the mean sea level, which makes flood drainage more difficult and costly.
Due to the thick soft clay layer underlying the ground surface, land subsidence results in complication in foundation design of buildings and underground utilities. The decrease in porewater pressure in the shallow clay layers brought about by the deep well pumping leads to a large consolidation of the clay and thus problems of differential settlements between adjoining buildings and negative skin frictions on piled foundations of buildings.

After having recognized the problem in the late 1970s, the situation has been monitored annually ever since. Surface and subsurface subsidence, groundwater levels in aquifer layers and rate of groundwater pumping in the metropolis are monitored by the Royal Thai Survey Department and the Department of Mineral Resources. In addition, there are a number of comprehensive investigations on the issue (e.g. AIT, 1981; JICA, 1995; Giao, 1997; Duc, 1996; DMR, 2000; and DGW, 2004; etc.). This paper outlines past and present situations of the Bangkok land subsidence. The impact of the subsidence on flood risk and foundation engineering practice in Bangkok with the emphasis on the design and development of the Suvarnabhumi Airport is presented.

**GEOLOGIC SETTINGS AND AQUIFER SYSTEM**

The Lower Central Plain (Bangkok Plain) is a large flat plain consisting of fluvial and marine deposits of the Chao Phraya delta. It is bounded by a mountain range on the west, an Upper Plain on the north, and Khorat Plateau on the east. Fans and terraces occupy the west and east marginal zones of the plain. In the central region of the delta where Bangkok Metropolis is situated, the plain is mantled seaward by a tidal flat of brackish clays, a tidal flat of marine clays and a tidal zone. According to Rau and Notalaya (1983) and Notalaya and Rau (1987) who presented a detailed description of the geomorphology and geology of the
Lower Central Plain, the topography and soils of the plain reflected the fact that it was covered by a shallow marine sea in the Holocene, during which a soft clay was deposited in shallow nearshore waters. During the past 3,000 years the sea has retreated from the plain, leaving behind a veneer of soft marine clay that now mantles 13,800 sq km. of the Plain. The thickness of the soft clay in the city area is generally about 12-16 m, but in few areas it may exceed 20 m.

In Bangkok area, data from boreholes drilled for water and oil explorataion and seismic reflection probing show that the depth of the basement rocks was large and variable. To the south of the metropolis, the base is found at a depth of 2100 m at the mouth of the Chao Phraya River, but at a depth of 450 m at a location on the coastline just to the east of it. Only the upper 450-m depth of the sediments is well defined. It is referred to as the Bangkok aquifer system (AIT, 1981 and Ramnarong, 1983). It comprises 8 main confined aquifers composed of sandy and gravely sediments interbedded with stiff to hard clays (Figure 2). Layers of clay aquitard separate the aquifers. However, leakages between them are expected over a large area as the separating aquitard may discontinue or thin out at places. Thus, extensive lowering of groundwater level from well pumping in one aquifer may induce drawdown in the adjacent aquifers. The hydrogeological properties of the aquifers and aquitards were investigated and interpreted by AIT (1981), JICA (1995), Ramnarong et al (1997), and DGW (2004).

Fig. 2 Bangkok Aquifer System

The uppermost aquifer, Bangkok aquifer (50-m depth zone), is 20-30 m thick. It is subdivided by an interbedding layer of stiff to hard clay into two sub-layers, i.e., the Upper Bangkok aquifer existing from about 16 m to 30 m and the Lower Bangkok aquifer from about 35 m to 55 m. In foundation engineering context, they are referred to as the First and Second Sand layers, respectively. Groundwater of the Bangkok aquifer in the metropolitan area is not potable due to high salinity and partly contaminated from exposure in borrow pits excavated for landfill material used in development projects in the city to raise the low lying ground level. The productive aquifers are mainly the second aquifer (Phra Pradaeng aquifer, PD, 100-m zone), the third aquifer (Nakorn Luang aquifer, NL, 150-m zone) and the fourth aquifer (Nonthaburi, NB, 200-m zone). The NL aquifer, 50-70 m in thickness, is the most pumped aquifer with about 50% of the total extraction (Ramnarong et al., 1997). Groundwater
extraction from deeper aquifers has recently become necessary in some areas of the city (especially the southern outskirt) due to excessive piezometric drawdown, which has induced saltwater intrusion in these three aquifers (Ramnarong et al., 1997).

GROUNDWATER EXTRACTION AND LAND SUBSIDENCE

![Groundwater Usage (1955-2004)](image)

Fig. 3 Chronicle of Groundwater Usage in Bangkok Metropolis

Large-scale groundwater extraction from the aquifer system of Bangkok began in the early 1950s (Ramnarong et al., 1997). The chronicle of the groundwater extraction in Bangkok and surrounding provincial suburbs during the past 50 years is shown in Figure 3. Daily pumping rates were recorded at 0.65 million m³ in 1975 and rapidly jumped to 1.2 million m³ in 1980. Following the conclusion of the fact-finding investigation on the issue initiated by the Thai government (AIT, 1981) confirming that land subsidence is mainly the result of rash groundwater pumping, a resolution on "the Mitigation of Groundwater Crisis and Land Subsidence in Bangkok Metropolis" was issued by the government. Several measures were implemented to regulate and reduce groundwater use. Consequently, the pumping rate was somewhat reduced and stabilized during a period from the mid-1980s to the early 1990s. The pumping was much reduced in the inner city areas where tap water supply had been made available. However, the rate again increased rapidly after 1993 following a new wave of economic growth. From that year, the metropolis expanded rapidly into the outskirt areas where surface water supply was mostly not available. Therefore, almost all of the developments in the areas had to rely on groundwater use. The registered pumping rate reached 2.2 million m³ in 2000.

Groundwater over-pumping in the 1970-1980s led to a drastic drawdown in piezometric head of up to 40-50 m in the exploited aquifers (NL and NB aquifers). The drawdown depression cone covering the entire city area, has its center located in the eastern district, which was the nucleus of the residential and industrial growths. The continued increase in the rate of groundwater extraction during the 1990s induced a further significant drawdown. The piezometric surface of the NL aquifer in 2002 in various areas of the metropolis is shown in Figure 4. The deepest drawdown occurred in the east suburban area where the groundwater level was as low as 65 m below the ground surface (15 m deeper than that existed in 1981). The size of the main depression cone had also expanded following the growth of the city. A
secondary cone of a smaller size developed in the industrial southwest suburb where surface water supply was not available.

Fig. 4 Piezometric Level in NL Aquifer in 2002

Fig. 5 Correlation Between Land Subsidence and Declines of Piezometric Levels in the Pumped Aquifers

The Bangkok land subsidence was first reported by Cox (1968). Nowadays, evidences of its occurrence can be seen throughout the city in the form of settlement and separation of ground and footpath from adjoining buildings, buckling of pavement over footings, differential settlement between buildings of different foundation depths. Although other causes, such as consolidation and lateral flow of the soft clay from surface loads of land fill or buildings, lowering of the perched water table near the ground surface, and erosion of sand fill below pavements or around drain pipes, etc., could have induced land subsidence, the prime cause was the groundwater overdraft (AIT, 1981). The consequent piezometric decline led to
aquitard compression, which manifested as widespread land subsidence. The recorded ground subsidence and piezometric drawdown in the pumped aquifers at a monitoring station in the southeastern area of the metropolis, shown in Figure 5, clearly show the close correlation between the two processes. Nutilaya et al (1989) reported that the largest subsidence in the city area over the 54-year period from 1933 to 1987 was 1.60 m. Subsequent data showed that it increased to 2.05 m in 2002 (Phienwej et al, 2006).

The annual subsidence rate of the city is at present based on the result of the levelling survey including more than 220 subsidence benchmarks installed throughout the metropolis. In Figure 6b the 2002 subsidence rate is shown and compared with the 1981 one (Figure 6a). Even though the rate has significantly decreased the affected area has spread out to the outlying city areas and provincial suburbs. The maximum subsidence rate, equal to 30mm/year, is now occurring in the outlying southeast and southwest industrial zones, while in 1981 the highest subsidence rate, up to 120 mm/year, had been recorded in the eastern area. Actually, the latter area still continues to subside at a rate of 20 mm/year. In the inner Bangkok areas, where groundwater pumping had ceased since the late 1980s, the subsidence rate has decreased to 5 to 10 mm/year. The phenomenon of continuing subsidence is the result of the time-dependent consolidation behavior of the aquitards and the soft clay layer at the surface induced by the past piezometric drawdowns in the aquifers. Drawdowns in the inner city areas have not yet been recovered since the influence of the increased groundwater extraction in the outlying areas is strongly felt there. Consequently, the entire metropolis still subsides. Figure 7 shows the time-rate land subsidence in various zones of the city during the last 20 years. The subsidence in the inner city area has been occurring at a relatively uniform but small rate over the last 10 years.
IMPACTS OF LAND SUBSIDENCE

Both the land subsidence and piezometric drawdown of the Bangkok aquifer system have many negative impacts on the city. They are particularly critical because of the very low lying topography of the city, the close proximity to the sea, and the presence of a thick soft clay layer underlying the ground surface.

The most serious impact is the intensification of city flood risk. The last very severe flood occurred in 1995, when widespread inundation of the unprotected areas in the northern, western and eastern suburban areas lasted for 1-2 months. Flood protection dikes in few residential areas were overtopped and breached out by the high flood level. Due to continuing subsidence, the flood protection systems of the city have become less efficient, because the original crest of flood protection dikes and walls subsides consistently with the ground. Now the city has to resort to the construction of large capacity flood drain tunnels and dug channels to assist the drainage through existing canals, which have become less efficient over the years. Because the ground surface is very low (ground surface elevation in the eastern area of the city is already 1.00 m below msl), a further lowering would be critical for the city flood risk.

Apart from intensification of flood risk, land subsidence causes numerous problems in foundation engineering practice. It is critical for Bangkok because of the existence of the 12-16 m thick layer of highly compressible soft clay at the ground surface. Although the wells extract groundwater from deep aquifers, the piezometric drawdown is felt up to the bottom of the soft clay layer. At present, the piezometric head in the First Sand layer (Upper Bangkok aquifer) underlying the soft clay and the First Stiff clay layers is mostly at 22-24 m depth below the ground surface in the inner city area (Phienwej, 1999). The reduction in the porewater pressure in the clays, which is time-dependent, leads to their consolidation (delayed compression). The compression of these shallow clay layers constitutes a large portion of the total subsidence observed at the ground surface. The recorded data on shallow ground subsidence suggest that about 30-50% of the total land subsidence would have resulted from the consolidation of both shallow soft and stiff clay layers (within the first 50 m depth) above the Second Sand layer (Lower Bangkok aquifer), brought about by the drawdown effect of the deep well pumping.
The FEM one-dimensional coupling flow-consolidation analysis made by Giao (1997) to determine consolidation of aquitards, shallow soft clay, and shallow stiff clays, in response to drawdowns induced in various aquifers at Lad Krabang site is shown in Figure 8. The site is located at the close proximity to the Suvarnabhumi Airport. The figure shows the predicted value of cumulative compression of the clays with depth at various elapsed times after occurrence of the drawdowns in the aquifer layers. The predicted subsurface settlements are comparable with the measurement data taken after 1995. The calculation suggests that the compression of the shallow clay layers (within the first 50 m depth) in long term is up to 25% of the total subsidence at the surface. The shallower the depth is of clay layers, the larger is the magnitude of ground subsidence. This leads to differential settlement between adjoining structures if their foundations are not at the same levels. Figure 9 schematically illustrates differential settlement conditions of structures built on the subsiding Bangkok ground.

Because of the existence of the soft clay layer below the surface, pile foundation is necessary for buildings constructed in Bangkok. Various pile depths are used depending on the type and size of structures. Small or old buildings are typically founded on short piles driven into the soft clay layer (depth 6-12 m). Medium sized buildings are mostly founded on long piles having tips extending to the First Stiff clay or First Sand layers (depth 18-30 m). Tall buildings or large-sized structures usually have pile tips extending in the Second Sand layer (depth 40-60 m). Buildings on deep piles would experience less subsidence than adjoining structures founded at shallower depths. Distortion and cracks of structures from the differential settlements may jeopardize the function for which the structures were built. While the differential settlement induced by a landfill to raise the ground surface can be prevented, or minimized, by adopting a proper foundation design, the differential settlement due to land subsidence can not be avoided. Unlike land subsidence, ground settlement induced by a land fill is only confined to a small area and can be reliably predicted by common methods of soil mechanics analysis. Differential settlement caused by land subsidence will not stop as long as
the depressurizing of clay layers still continues. This phenomenon was then included as one of the major design considerations in the recent infrastructure development projects in Bangkok including Suvarnabhumi Airport project and the MRT projects.

Fig. 9 Schematic Illustrating Differential Settlement Conditions of Structures in Bangkok Induced by Land Subsidence

Fig. 10 Calculated Future Subsidence at Suvarnabhumi Airport Site (Thepparak, 2001)

Prediction of the future magnitudes of differential settlements between adjoining structures due to land subsidence is usually skeptical due to a lack of a reliable prediction tool as well as inadequacy of information on soil conditions and on past piezometric conditions at the site. Commonly, empirical predictions are made by extrapolation of available monitoring data at nearby locations. To provide a better prediction tool, a finite difference code for the one-dimensional coupled flow-consolidation analysis of the multi clay and sand layers of the Bangkok soils was developed (Thepparak, 2001). The code predicts the time-dependent differential settlements induced by piezometric drawdowns in the First and Second Sand layers from deep well pumping as well as by surcharge loads from landfill. Both the piezometric pressure drawdowns and surcharge load are input as boundary conditions. These values may be changed during the time period under consideration to simulate different
scenarios of land subsidence. The code, written in MS Visual Basic, gives promising predictions as compared with the measurement data at a number of subsidence and groundwater monitoring stations (Thepparap, 2001; Phienwej et al., 2004); an example of the prediction at Suvarnabhumi Airport Site is shown in Figure 10. The method was also used to predict long-term differential settlements between adjoining structures of the recently completed MRT subway of Bangkok (Phienwej et al., 2004).

CURRENT SITUATION

The land subsidence problem that has severely affected Bangkok for the past 35 years results from inadequate demand management of groundwater to cope with the expanding city. The problem was compounded by the lack of a land use zoning law to manage the growth of the city and the utilization of its infrastructures and utilities. Realizing the seriousness of the problem, the government at the turn of the century started to implement a rigorous plan of mitigation measures which included a pricing policy on groundwater and expansion of tap water supply from surface sources in the needy industrial suburban areas. The plan of gradually raising the price of groundwater to eventually match the consumption costs of tap water became effective in 2001. Before, the cost for groundwater users was only 1/4-1/3 of the tap water cost. During 2000-2004, surface water treatment facilities and tap water distribution systems were constructed in the northern and southwestern city suburbs. In 2005, the construction of the tap water supply distribution system is underway for the needy industrial southeast suburb (in the vicinity of the Suvarnabhumi Airport site) that is suffering the largest subsidence rate (30 mm/year) in the metropolitan area.

Mitigation measures together with the strict enforcement of the ban on groundwater use in the areas already accessible to a tap water supply has resulted in a considerable drop of groundwater use after 2001. However, the land subsidence of the city still continues, though its magnitude is less severe than in the past. This is attributed to the delayed effect of past severe pumping on the compression of the soft clay and clay aquitards. Recovering piezometric levels is more than needed to stop the land subsidence in Bangkok.

EFFECT ON SUVARNABHUMI AIRPORT DEVELOPMENT

The on-going land subsidence from deep well pumping affected the design and development of Suvarnabhumi Airport in various ways. These included the design of flood protection system, ground improvements airfield pavements and other areas, and foundation design building in term of settlement control and pile foundation.

Flood Protection System

As mentioned earlier, the site of Suvarnabhumi Airport is located in the low-lying swampy area in the eastern side of Bangkok (Figure 1). Hence, to develop the airport, flood protection and drainage systems must be established to prevent the airport from flooding all year round as well as to alleviate flooding problem in the neighboring area outside the airport. The existing ground surface at the airport site before the development was low (around + 0.5 m msl on the average). Figures 11a&b compare ground surface elevations in recorded in 1930 and 1993 (Phienwej, 1995) which shows the sinking of Bangkok ground.
The selected flood protection scheme of the airport consists of construction of the flood protection dike that encloses the entire area of the airport (4 km by 8 km, Figure 12) together with retention ponds and high capacity pumping station. The flood protection dike was constructed in the form of the clay fill embankment. The clay fill was mainly obtained from the excavation of water collection channels running along the dike. In the design of the dike, the crest height was set to allow for the settlement of the embankment due to consolidation of the soft clay foundation caused by stress increase from weight of the embankment material. The normal preloading method without any soil improvement was adopted in the dike construction because there was ample time available for the construction before the commencement of the main works of the airport construction. The designed crest elevation of the dike was set against the 1000-year flood height. However, an additional crest height of 500 mm was allowed for the anticipated future land subsidence from deep well pumping of the area. The amount was determined from extrapolation of the record of land subsidence at nearby stations.
Fig. 13 Recorded Land Subsidence at Monitoring Stations in the Vicinity of the Airport

At present, the subsidence data at monitoring stations in the vicinity of the airport shows the subsidence magnitude of around 15-30 mm/year (in 2003). Figure 13 shows the time-rate records of the land subsidence at three monitoring stations close to the airport (to the north, south and west).

Besides the construction of the flood protection dike at the airport, a number of flood drainage improvement schemes are implemented in the areas outside the airport. The construction of the flood protection dike at the airport disturbs the existing natural flood drain system of the area. In addition, previous land subsidence from deep well pumping had resulted in a sagged shape topography in the area south of the airport towards the seashore that led to deficiency in flood draining out to the sea from the airport area by gravity flow. The major improvement scheme is the construction of a large flood drain channel (100 m wide and 12 km long) with the capacity of 100 cu.m./sec from south of the airport towards the sea. A major pumping station is placed at the outlet to the sea to speed up the flood flow in the flood season.
Ground Improvement

Ground improvement methods adopted at the airport to avoid settlement and stability problems of airfield pavements and other structures consist mainly of preloading with prefabricated vertical drain (PVD) method and soil cement mixing by jet grouting or deep mixing methods. For most of the airfield pavement areas, the preloading with PVD method was mainly employed. The subsoil conditions at the airport site have been extensively investigated. The subsoils in first 30 m depth are rather uniform as shown in Figure 14, which depicts the subsoil profile along one of the investigation lines. The soft clay layer is about 10-12 m thick and the typical engineering properties are summarized in Figure 15. The piezometric condition in the subsoils at the airport site is similar to those found in other areas of Bangkok for which the piezometric drawdown exists in the underlying stiff clay and dense sand layer underneath the soft and medium clay layers. The drawdown was mainly attributed to deep well pumping (Phienwej et al., 2006). Figure 16 summarizes data on measured piezometric pressures in the subsoils at the airport site. It can be seen that the drawdown has encroached into the lower part of the soft clay layer (about 10-12 m depth from the ground.
surface). This drawdown condition affects the design of PVD in accelerating consolidation of the soft clay in the preloading method. The designed length of the PVD should not extend to the full thickness of the soft clay and medium clay as normally done.

![Piezometric Pressure Diagram](image)

**Fig. 16 Piezometric Drawdown in Subsoils at the Airport Site**

![Depth and Material Diagram](image)

**Fig. 17 Piezometric Drawdown in Soft Clay Layer and Effective Depth of PVD**

The PVD length should be limited to the thickness part of the soft clay that has not been affected by the piezometric drawdown from deep well pumping (Figure 17). A deeper PVD extending into the piezometric drawdown zone of the soft clay and medium clay layers would result in recharge of water into the clay layers that would result in swelling of that portion of the clay instead of consolidation. It would further obstruct the discharging of pore
water from the consolidating soft clay to the ground surface, which could reduce the efficiency of the PVD preloading.

![Pile Capacity Diagram](image)

**Fig. 18 Decrease in Pile Capacity Due to Future Piezometric Rebound in Bangkok Aquifer**

**Pile Capacity**

The present piezometric drawdown in the subsoils at airport site gives rise in bearing capacity of the foundation piles of buildings and other structures. In the future, the present pile capacity may reduce if the piezometric recovery of groundwater pressure occurs following the stoppage of groundwater pumping in the area. Calculation shows that the effect of groundwater rise on the reduction of pile capacity is more critical for short piles (tips in the first sand) than for long piles (tips in the second sand) (Figure 18). The 3-D groundwater flow simulation model MODFLOW (DGW, 2004) suggests that once the groundwater pumping in the Bangkok Plain has reduced to the permissible yield amount, the piezometric level in the Bangkok Aquifer would rise to the level of about 12 m below the ground surface from the present level at about 24 m below the ground surface.

**CONCLUSIONS**

Land subsidence induced by deep well pumping has been an environmental calamity affecting Bangkok for the past 35 years. Its past situations and current trend are as follows.

1. Despite the measures implemented to mitigate the crisis, the amount of groundwater pumping from the massive aquifer system of the Bangkok plain continued to increase from 1.2 million m$^3$/day in the early 1980s to around 2.0 million m$^3$/day during 1996-2003. As a result, land subsidence still continues in the Bangkok Metropolitan area, even though its rate has decreased, and location of the most affected zones has changed.

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2. At present, the most affected areas that undergo subsidence at the rate of 30 mm/year are the southeast suburb where Suvarnabhumi Airport is built. The most critical zone at the peak of the crisis 25 years ago was the eastern area of the city, which experienced the largest subsidence rate of 120 mm/year. This area still continues to subside at 20 mm/year.

3. Major impacts of land subsidence include increased flood risk and settlement problems of ground and buildings. The problems are particularly critical because of the flat low-lying topography of the city area and the presence of a thick soft clay layer underlying the ground surface.

4. The on-going Bangkok land subsidence affected the design development of Suvarnabhumi Airport in various ways including the design of the flood protection dike, the design of PVD length in improvement of airfield pavement areas by preloading method and the design of long-term pile foundation capacity.

5. The flood protection dike enclosing the low-lying airport area must allow for anticipated amount of future land subsidence from deep well pumping.

6. The piezometric drawdown in the bottom part of the soft clay layer limits the effective depth of the PVD to be around 10-12 m.

7. The design of pile capacity must consider the rebounded condition of the piezometric pressure in the sand layers in the future after the excessive deep well pumping has ceased.

REFERENCES


ELECTRIC IMAGING OF BANGKOK CLAY AT THE SITE OF THE NEW BANGKOK INTERNATIONAL AIRPORT (NBIA)

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ABSTRACT

The clay deposit underlying Bangkok city has been extensively investigated by geotechnical testing techniques for infrastructure development during the last three decades, but it has not much investigated by geophysical techniques despite the fact that the later have been increasingly used as an advanced tool of site investigation in many parts of the world. This paper deals with application of electric imaging technique (El) in mapping the shallow subsoil at the NBIA site. Technical aspects in design of a field survey using El such as selection of electrode configuration and spacing, depth of investigation, data analysis are discussed. Some initial results of a time-dependent monitoring of the subsoil resistivity change during the ground improvement process at the site are shown. The resistivity measurement of Bangkok clay in the laboratory and its correlation with some other geotechnical parameters are also presented.

INTRODUCTION

Bangkok metropolis with a population of more than 8 millions has expanded rapidly during the last 40 years over an area of about 4,000 km² in the Chao Phraya River Deltaic plain, consisting of fluvial and marine deposits. The Chao Phraya delta is located in a depression filled with Tertiary and Quaternary sediments with a thickness of more than 2000 m (Sinsakul, 2000). The depth to the basement of the Chao Phraya River Plain is still not well determined, except at a few locations in Bangkok area where it varies from about 450 m to more than 2500 m. Only the upper 450-m depth of the sediments is quite clearly defined and is referred to as the Bangkok aquifer system. Due to a global sea level rising in the past 20 kyr, the Chao Phraya River plain was covered by a shallow marine (Hanebuth et al., 2000). After the mid-Holocene (8-7 kyr), the delta prograded southward in the gulf, forming the present delta plain (Rau and Notalaya, 1983; Sinsakul, 2000). The deltaic and shallow marine Holocene sediments forming the delta plain are called the Bangkok Soft Clay and those of the Late Pleistocene are called the Bangkok Stiff Clay (Rau and Notalaya, 1983). The plain was a vast tidal flat that was gradually filled, resulting in formation of the soft Bangkok clay. The sea had then withdrawn and the soft clay was exposed at the surface. This soft clay has been little changed since its deposition and only the uppermost 2 m has been weathered. The Holocene marine sediments are about 8-12 m thick at Bangkok and increases up to 30 m in the coastal area south of the city (Phien-wej et al., 2006).

In the last three decades, strong economic and infrastructure development of Bangkok have resulted in very large construction projects, including a dramatic expansion of the city and its vicinity, highways, expressways, toll ways, sky trains, subways, tunnels, large bridges, flood protection system, port and airport facilities. As noted by Balasubramaniam (1991), the geological and soil conditions of the soft and compressible clay in which the construction is founded is a very important factor when viewed in terms of the cost and sustainability of the project. Therefore, study on Bangkok soft clay has constantly been one of the priority research topics pursued at the Geotechnical Engineering Division, the Asian Institute of
Technology (AIT). With a newly setup interdisciplinary program of Geosystem Exploration and Petroleum Geoengineering (GEPG), more researches on using engineering geophysical techniques are encouraged to help solving the geotechnical problems of Bangkok clay.

![Bangkok Subsurface Diagram](image)

**Fig. 1 Bangkok Subsurface (Phienwej et al., 2006)**

This study has the purpose to apply Electric Imaging (EI) in investigation of the Bangkok subsoil. Electrode configuration, distance between measuring point, depth of investigation of an EI survey will be investigated. Some results of field EI surveys that have been performed in the last three years will be presented.

**SURVEY PROCEDURE USING ELECTRIC IMAGING**

Soil resistivity measurements have various purposes. It can be used to make a subsurface geophysical survey as an aid in identifying ore locations, depth to bedrock and other geological phenomena. In addition, soil resistivity also directly affects the design of grounding system in order to archive the optimum economical grounding installation.

Clays are not a favorite target for geophysical methods as they absorb the geophysical signals. Traditional procedures of DC electric survey like Vertical Electric Sounding (VES) or Electric Profiling (EP) have not always been useful to map a thick clay layer in a certain geological section. The electric resistivity method was first proposed by Conrad Schlumberger and his colleagues in 1912. An important development benchmark was the work by Stefanescu and Schlumberger (1930). The method has become more widely used since 1970s primarily due to the availability of the computers. Two main traditional techniques are Vertical Electric Sounding (VES) and Electric Profiling (EP). Some of pioneering works by Barker (1981), Overmeer van and Ritsema (1988), Noel and Walker (1990), Griffiths and Baker (1993), Loke (1999), Abdul et al. (2000) etc. have led to
development of a new technique of electric imaging (EI) in the 1990s years. With development of modern Resistivimeters (e.g., SYSCAL, ABEM, STING/SWIFT, OYO etc.) having an automatic compensation of spontaneous potential (SP), the weak DC electric signals (low value in potential difference) in soft clay deposits can be better acquired. Some initial investigations to map the deposit of clayey soils at locations of Pusan and Mokpo in Korea were performed and reported by Giao (2001) and Giao et al. (2001, 2003). This paper presents in details how an EI field work has been carried out at the NBIA site and the results obtained.

In practical surveys, most of electric resistivity surveys are carried out using a four-electrode configuration installed on the ground surface as shown in Figure 2, in which A and B are called the current electrodes, while M and N are called the potential electrodes.

![Figure 2: A Common Four-Electrode Configuration](image)

The measured resistivity or apparent resistivity is determined by the following formula, where \( K \) is a coefficient depending on arrangement of electrodes and known as the geometrical factor or the arary coefficient:

\[
\rho_a = \frac{2\pi}{\left(\frac{1}{AM} - \frac{1}{AN} - \frac{1}{BM} + \frac{1}{BN}\right)} \times \frac{\Delta V_{MN}}{I} = K \times \frac{\Delta V_{MN}}{I}
\]

(1)

where \( \Delta V_{MN} \) is the measured value of potential difference between M and N, and I is the measured value of the current intensity injected into the ground through A and B.

**Field Procedure of Electric Imaging**

Field procedure of an electric imaging over a 240-m survey line is illustrated in Figure 3. Initially, the unit dipole spacing of 10 m was used to scan the whole line. After a scan of the survey line is completed, the electrode spacing increased by 10 m for the next scanning. Both dipole-dipole and Wenner arrays were employed at the NBIA site. For example, implementation of electric imaging with the Wenner array over a survey line of 240 m long is shown in Figure 4. It should be noted that besides the fully automatic procedure, we have also performed the manual and semi-automatic electric imaging for Bangkok clay to fit in conditions of a developing country, where fully automatic equipment is not always available. The advantages of a manual procedure were discussed by Giao et al. (2003), while the semi-
automatic procedure was studied and reported in Giao & Adisornsupawat (2004), respectively. For geotechnical engineers who may not have a fully automatic equipment at hand, the manual and semi-automatic procedures can allow them to carry out an electric imaging by means of the stand-alone units of resistivimeter that are much cheaper and easily available.

![Diagram of Electric Imaging Procedure in the Field](image)

**Fig. 3 Electric Imaging Procedure in the Field**

**Electric Imaging Results**

![Map of NBIA location](image)

- a) NBIA location
- b) EI testing site on airport runway in NBIA

**Fig. 4 Testing Location NBIA**

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The testing location at NBIA (Figure 4a) is along the taxiway and apron of east runway, where the soft clay has been improved by installation of prefabricated vertical drain (PVD) and preloading (Figure 4b). Under the impact of preloading from surface (i.e., weights of the sand and crushed stone layers, which will be removed later on when the underlying soft clay reached most of its primary consolidation), soft clay layer is compressed, releasing pore water to the vertical drains, thus accelerating the consolidation of the clay, and reducing the construction time. Consequently, the water content will be decreased and clay will become stiffer. Such stiffening can be related and detected by the change in resistivity of clay as studied by Giao et al. (2001 and 2003) in the case of the Kimhae international airport in Busa, Korea. The soil profile and some geotechnical properties at the testing site is shown in Figure 5. The imaging results of the change in electric resistivity of the improved clay at NBIA at the beginning and after six months of preloading are shown in Figures 6a-c. At this site, the clay layer is extended to 16 m deep. It can be observed an increase in resistivity of clay layer after six months of preloading is displayed.

LABORATORY MEASUREMENTS OF CLAY RESISTIVITY

As discussed by Giao et al. (2003), for the electric resistivity to be a useful parameter for soil characterization like many other geotechnical parameters such as water content, liquid and plastic limits, unit weight and so on, more tests results would be needed as the electric resistivity depends not only on soil material but also content of electrolyte, water content and salt content and it can widely vary for a soil type.

For Bangkok clay, a simple scheme of measuring electric resistivity on soft clay samples similar to that used for Pusan clays (Giao et al., 2003) was employed as shown in Figure 7. The 4-electrode configuration was used. The current electrodes A and B, made of copper disks of 0.5 cm thick and 75 mm in diameter, were put on top and at the bottom of the core sample. The potential electrodes M and N, made of copper needles of 1mm in diameter and 3 cm in length, were inserted at 4 or 8cm apart. The resistivimeter was SYSCAL R+1 with internal voltage supplied.
A few geotechnical properties of NBIA soil samples collected at the testing site were tested, including water content, unit weight, Atterberg limits. Some empirical relationship between resistivity and chemical parameters are attempted as shown in Figures 8a-d. The amount of salt content and sodium chloride is inverse proportion to resistivity value as shown in Figures 8a-b, respectively. The reduction of water content leads to increase in resistivity. PH value, however, doe not show any relationship with resistivity as seen in Figure 8d.
CONCLUSIONS

1) Electric imaging proved to be a value-added tool for site investigation of the Bangkok clay. Among two electrode configurations that have been tested, i.e., Wenner and dipole-dipole, the former proved to be more suitable to map the horizontal soil layers of the NBIA subsoil.

2) When a soft clay ground is subjected to an improvement its resistivity may change with the reducing water content and increasing stiffness during the consolidation process. While such a relationship, if exists, can be meaningful to monitor quality of a time-dependent improvement process it is the author’s opinion that it needs more studies on the issue for its clarification.

3) Laboratory resistivity measurements were made on the clay samples taken from the NBIA site. The resistivity values for soft clay are in the range of 1.0 to 2.5 \( \Omega \text{m} \). It is interesting to observe that this value is lower than that of soil samples collected at the AIT site, more
to the north of Bangkok, which is around 4 to 5 Ωm. This finding may be helpful to elucidate the depositional environment of Bangkok clays in different parts of the Chao Phraya plain, i.e., a tidal flat of fresh, brackish or marine clays. The resistivity of Bangkok clay is then compared with those of the other clays in the world as shown in Figure 9, confirming the conclusion drawn by Giao et al. (2003) that, except for some of diatomaceous Japanese clays, most of the marine and deltaic clays have resistivity found in a small range from 1 to 10 Ωm. This finding is useful in mapping of soft clay deposits.

![Figure 9 Comparison of Bangkok Clay Resistivity with Those of Other Soft Clays Measured on the Clay Samples in the Laboratory (Modified from Giao, 2005)](image)

**REFERENCES**


INVITED LECTURES
PVD GROUND IMPROVEMENT WITH VACUUM PRELOADING AT SUVARNABHUMI AIRPORT

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ABSTRACT

Cofra has introduced in the past a number of under pressure systems combined with vertical drains, that are very effective for applications where stabilization of the embankment is at risk or where not sufficient surcharge material is available. This paper will focus on the latest developments in vacuum drainage called Beaudrain and Beaudrain-S and the application of Beaudrain-S at the ground improvement of Suvarnabhumi Airport. The Beaudrain systems are characterized by the absence of a liner construction. At both systems the topsoil acts as an impermeable layer that separates the atmospheric pressure from the subsoil so that an effective vacuum system can be realized. At Suvarnabhumi Airport the Beaudrain-S system is successfully applied at a number of taxiways and a large portion of the apron. In total an area of 40 acres are treated with the new vacuum-PVD ground improvement method.

INTRODUCTION

The innovative Beaudrain systems recently developed to accelerate the settlement rate and to increase the shear strengths of these fine grained, soft soils by means of the proven concept of vacuum consolidation. Applying the atmospheric pressure as a surcharge offers excellent possibilities to contribute to the reduction of risks, environmental nuisance and future maintenance.

BEAUDRAIN SYSTEM

The Beaudrain system is a recently developed vacuum consolidation technique with an innovative installation procedure. Through a specially designed plough that is pulled by a hydraulic crane, prefabricated vertical (wick) drains are installed and cut at predefined depths below ground level. While the plough is moving a horizontal collection drain is placed at a depth of approximately 3 m below ground surface and is connected to the vertical drain. Before it leaves the plough, the horizontal drain is also covered by an impervious liner in order to ensure a proper sealing between the horizontal drain and the atmospheric conditions.
The total system, which is usually referred to as a drainage curtain, consists of a row of vertical drains, a horizontal drain and seal. It is placed in a single pass of the plough. Figure 1 illustrates schematic the installation of the system. After passage of the plough the compressible soil closes in on itself above the horizontal drain creating a natural seal additional to the liner. The total system consists of a number of drainage curtains connected to vacuum pumps. The crane with plough is depicted in Figure 2.

The vacuum measured at the pumps generally varies between 80 kPa and 90 kPa (0.8-0.9 bar). Depending on the height difference between the pump and the horizontal drain, this usually results in a reduced pressure of approx. 50 to 60 kPa in the horizontal drain.
This vacuum pressure of 50 to 60 kPa, corresponding to a surcharge equivalent of approximately 3 m (dry) sand, acts on the compressible strata as a load. Figure 3 clearly shows the increase of the effective stress as a result of the reduced atmospheric pressure in the soil mass. The net effect of this is an additional surcharge, which will ensure an early attainment of the required settlement, and an increased shear strength which will favor the stability (accelerated loading schemes, steeper slopes in areas with limited space).

The Beaudrain system has some major advantages over the traditional vacuum systems where liners are applied:

- Very quick and clean installation
- No liner needed for sealing of the area
- Instant accessibility of the working platform
- No hydraulic resistance in the dewatering system and thus a higher vacuum.
- No blocking of the horizontal drains by fines of the sand in the working platform.
- No drainage layer needed
- No border trench needed

BEAUDRAIN-S SYSTEM

The Beaudrain-S system is developed as an alternative for the Beaudrain system. The principle is the same. Also at Beaudrain-S the topsoil itself acts as an impermeable layer. The big advantage of Beaudrain-S is that standard installation machines can be used for the installation of the system.

Beaudrains are installed till 1m under the top of the compressible layer. The top of the drain is connected to a polyethylene tube using a special fitting that also fits in the mandrel from the installation rig and allows free flow from the drain in the PE-tube. The drains with the
tings and anchor plates are prefabricated and pulled in the mandrel from the bottom. After installation to the designated depth, the PE-tube is sticking out of the ground for at least 1 m.

Fig. 4 Prefabricated Beaudrain-S

At Suvarnabhumi Airport the drains were installed from the original surface. There was no working platform or drainage layer placed because both were superfluous. To avoid air leaks the holes around the tubes were filled with clay.

Fig. 5 Installation Rigs at Suvarnabhumi Airport

The PE-tubes are cut at ground level and provided with a T-coupling that was especially developed for vacuum applications. In total approximately 30 drains were connected to one collecting tube of the same diameter. To avoid pressure drop in the collection system there has to be a laminar flow in the tubes. As soon as turbulent flow occurs the pressure drop will be not acceptable. About ten collection tubes are connected to a manifold and from the manifold a much larger flexible tube with transport the expelled pore water to the vacuum pump. In total
about 1000 to 3000 m² area is connected to 1 pump. The water is feed back into an infiltration system that keeps the surface wet so that the top soil will be saturated and able to act as a seal.

![Fig. 6 Collection System](image)

![Fig. 7 Beaudrain Pumps](image)

The Beaudrain pumps are also especially developed for this application. They can create a very high vacuum of up to 95 kPa because they are provided with a separate water and air pump. First the water is separated from the air. The air is injected with oil to create a higher vacuum in the air pump and the oil is separated from the air after the pump. Both air and water pump are only running when it is needed, thus saving energy during operation. The complete vacuum system is adapted for tropical conditions.

Before the collection system is covered with surcharge material the system is tested. Every collection tube is separately connected to the vacuum pump to check if sufficient vacuum can be achieved and if all the connections are watertight. After inspection the surcharge can be placed. Beaudrain-S has some big advantages over the standard vacuum drainage systems.

- No liner needed that stays behind in the soil or has to be taken away.
- Direct connection of every drain to vacuum pump with little flow resistance
- No border trench needed
- Standard drain machines can be used.
- Better control on functioning due to separate testing of the drain sections
- No drainage layer needed
- Fast mobilization
- Also applicable at layered sub-soils

**GEOTECHNICAL ANALYSIS**

In this paragraph a geotechnical analysis is made about the Beaudrain–S design for the taxiways toward the third landing strip and the midfield apron on Suvarnabhumi Airport. The total area that has to be drained is about 400,000 square meters, divided into 11 separate enabling work areas in total 180,000 m² and taxiways around the midfield apron in total 220,000 m². The client required 60% consolidation for the enabling work areas and 70% for the taxiways at the midfield apron based on 60 kPa vacuum and a surcharge of 2.8 m.
The following boundary conditions were used in the design.

- Installation time drains 6 months
- Pumping time 4 months
- Vacuum 60 kPa
- Depth drains till 10 meters below present surface
- Drain distance maximal 1 m square, from tender document.
- Embankment 2.8 meters (18 kN/m³)
- Foundation 1.0 meter
- Embankment is constructed in two phases
  - Phase 1 1.5 meter day 0
  - Phase 2 1.3 meter day 14
- These assumptions were not based on calculations

GROUND CONDITIONS

The following information on the ground conditions were provided by the client. A short review are given in Tables 1 and 2.

<table>
<thead>
<tr>
<th>Table 1 The Stratigraphy</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Present surface</td>
<td>0.00 m</td>
</tr>
<tr>
<td>Water level</td>
<td>-0.50 m</td>
</tr>
<tr>
<td>Type</td>
<td>Top layer(m)</td>
</tr>
<tr>
<td>Top layer, weathered clay</td>
<td>0.00</td>
</tr>
<tr>
<td>Very soft clay1</td>
<td>-2.00</td>
</tr>
<tr>
<td>Very soft clay2</td>
<td>-5.00</td>
</tr>
<tr>
<td>Soft clay</td>
<td>-10.00</td>
</tr>
<tr>
<td>Soft to medium clay</td>
<td>-13.00</td>
</tr>
<tr>
<td>Stiff clay1</td>
<td>-15.00</td>
</tr>
<tr>
<td>Stiff clay2</td>
<td>-17.00</td>
</tr>
<tr>
<td>Stiff clay3</td>
<td>-20.00</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 2 The Used Consolidation Parameters</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight [kN/m³]</td>
<td>Compressibility</td>
</tr>
<tr>
<td>Top layer, weathered clay</td>
<td>18.50</td>
</tr>
<tr>
<td>Very soft clay1</td>
<td>13.80</td>
</tr>
<tr>
<td>Very soft clay2</td>
<td>14.00</td>
</tr>
<tr>
<td>Soft clay</td>
<td>15.00</td>
</tr>
<tr>
<td>Soft to medium clay</td>
<td>15.70</td>
</tr>
<tr>
<td>Stiff clay1</td>
<td>18.50</td>
</tr>
<tr>
<td>Stiff clay2</td>
<td>19.00</td>
</tr>
<tr>
<td>Stiff clay3</td>
<td>20.40</td>
</tr>
</tbody>
</table>

where:
- POP = pre-overburden pressure
- CR = compression ratio
- RR = recompression ratio
- C_v = creep coefficient
- C_v = vertical consolidation coefficient
In Table 2 the pre-overburden pressure (POP) is derived from the given OCR value in. The numbers are rounded and taken as an average value of each layer. The consolidation coefficient is estimated from an article by Athanasiu et al [2] (see following section). In this article the $C_v$ as function of the effective stress has been back calculated from settlement data from a case history on the terrain of the airport. The $CR/C_a$ correlation is estimated to be 25.

**HISTORIC DATA**

A few case histories with vertical drains on the airport are described in literature. The articles reviewed are included in the literature. Articles indicate several different soil profiles. These profiles are generally the same, but there are minor differences regarding the thickness of the layers and the volumetric weights of the clay layers. The profiles show some variation in the thickness of the layers. It is not sure to what extent the ground profile provided by the client represents the actual site conditions. The design parameters also vary between authors. The following Table 3 gives the compression ratios, derived from three articles. Ref. [1] gives the compression ratio ($CR$) and recompression ratio ($RR$) in the table. Reference [4] gives kappa and lambda values. These values were transformed into $RR$ and $CR$ values using the $e_0$, also given in the article. The $CR$ and $RR$ value given under [3] were calculated using the $e_0$ from [3], as this value was missing in the article.

**Table 3 The Compression Parameters**

<table>
<thead>
<tr>
<th></th>
<th>[4]</th>
<th>[1]</th>
<th>[3]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$CR$</td>
<td>$RR$</td>
<td>$CR$</td>
</tr>
<tr>
<td>Layer 1</td>
<td>0.11</td>
<td>0.01</td>
<td>0.35</td>
</tr>
<tr>
<td>In-between layer</td>
<td></td>
<td></td>
<td>0.5</td>
</tr>
<tr>
<td>Layer 2</td>
<td>0.25</td>
<td>0.03</td>
<td>0.42</td>
</tr>
<tr>
<td>Layer 3</td>
<td>0.18</td>
<td>0.02</td>
<td>0.4</td>
</tr>
<tr>
<td>Layer 4</td>
<td>0.04</td>
<td>0.00</td>
<td>0.3</td>
</tr>
<tr>
<td>Layer 5</td>
<td>0.04</td>
<td>0.00</td>
<td>0.08</td>
</tr>
</tbody>
</table>

The differences between these compression parameters are significant. The OCR value is in all articles reasonably high. The values are given in Table 4. Reference [1] gives the highest OCR values. Also in Table 4 differences between the various authors are present.

**Table 4 The Preconsolidation Pressure**

<table>
<thead>
<tr>
<th></th>
<th>[5]</th>
<th>[1]</th>
<th>[3]</th>
<th>[4]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\sigma_p$ [kPa]</td>
<td>OCR</td>
<td>OCR</td>
<td>POP [kPa]</td>
</tr>
<tr>
<td>Layer 1</td>
<td>42</td>
<td>3.62</td>
<td>5</td>
<td>60</td>
</tr>
<tr>
<td>In-between layer</td>
<td></td>
<td>1.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Layer 2</td>
<td>38</td>
<td>1.17</td>
<td>1.6</td>
<td>30</td>
</tr>
<tr>
<td>Layer 3</td>
<td>67</td>
<td>1.3</td>
<td>1.8</td>
<td>45</td>
</tr>
<tr>
<td>Layer 4</td>
<td>90</td>
<td>1.24</td>
<td>2</td>
<td>20</td>
</tr>
<tr>
<td>Layer 5</td>
<td></td>
<td>4</td>
<td></td>
<td>50</td>
</tr>
</tbody>
</table>

Table 5 gives the parameters that control the rate of consolidation, the consolidation coefficient $c_v$ or, if the necessary stiffness parameters are missing, the $k_v$ values only. The numbers given under [2] were interpreted from Figure 4 in [2].
Table 5 The Consolidation Coefficient

<table>
<thead>
<tr>
<th></th>
<th>[2] $c_v$ [m²/s]</th>
<th>[5] $c_y$ [m²/s]</th>
<th>[3] k [m/s]</th>
<th>[4] $k_v$ [m/s]</th>
<th>[1] $k_v$ [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer 1</td>
<td>3.17E-08</td>
<td>3.00E-09</td>
<td>2.60E-07</td>
<td>1.00E-08</td>
<td></td>
</tr>
<tr>
<td>In-between layer</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>8.00E-09</td>
</tr>
<tr>
<td>Layer 2</td>
<td>2.80E-08</td>
<td>6.37E-08</td>
<td>6.80E-09</td>
<td>5.50E-08</td>
<td>7.30E-09</td>
</tr>
<tr>
<td>Layer 3</td>
<td>3.17E-08</td>
<td>7.04E-08</td>
<td>3.00E-09</td>
<td>2.60E-08</td>
<td>5.30E-09</td>
</tr>
<tr>
<td>Layer 4</td>
<td>3.81E-08</td>
<td>8.28E-08</td>
<td>2.60E-09</td>
<td>2.30E-09</td>
<td></td>
</tr>
<tr>
<td>Layer 5</td>
<td></td>
<td></td>
<td>2.60E-09</td>
<td>1.70E-09</td>
<td></td>
</tr>
</tbody>
</table>

During the analyses of the articles, it was noticed that the final settlements of all the discussed embankments was less than 1.5 m: 1.42 m Horpibulsuk (510 days, 68kPa), 0.96 m Bergado (120 days, 105 kPa), 1.1 m Athanasiu, (650 days, not specified load) and 1.30 m Indraratna, (425 days 75 kPa).

Table 6 The Settlement Data from Areas Next to the Proposed Beaudrain Areas

<table>
<thead>
<tr>
<th>Project</th>
<th>Surcharge Load, kPa</th>
<th>Waiting Period</th>
<th>GI Method</th>
<th>Settlement (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GI Airside Pavements</td>
<td>75</td>
<td>8 months</td>
<td>10 m long PVD in 1 m spacing</td>
<td>1.3 (1.10-1.51)$^2$</td>
</tr>
<tr>
<td></td>
<td>85</td>
<td></td>
<td></td>
<td>1.6 (1.20-1.85)$^2$</td>
</tr>
</tbody>
</table>

Notes: 1. After reaching final fill height.
2. Indicate average, minimum and maximum settlement.

Reviewing Table 3, it can be concluded that Beaudrain-S with an average vacuum pressure of 60 kPa and a surcharge of 56 kPa (total load = 116 kPa) has to achieve in 4 months what 85 kPa achieves in 8 months, assuming the same geotechnical conditions. It is not possible to back calculate this due to a lack of data. The article by Bergado [4] gives after the installation of a vacuum system with the same surcharge a final settlement of around 1.0 m. After reviewing the past section, it can be concluded that the soil parameters are not univocal. It was therefore assumed that with their on-site experience the data given by client are the most reliable data available. Therefore the calculations were made using the soil parameters as presented in [1].

**CALCULATION RESULTS**

Settlement calculations were made using the data provided in [1]. To have a better understanding of the influence of the different parameters on the design the relation was plotted between the consolidation coefficient and the expected settlement for various drain spacing's and a fixed consolidation period of 4 months. See Figure 7. The same was done for the rate of consolidation. See Figure 8.
Fig. 7 The Relation between the Consolidation Coefficient and the Expected Settlement for Various Drain Spacing’s and a Fixed Consolidation Period of 4 Months

Fig. 8 The Relation between the Consolidation Coefficient and the Rate of Consolidation for Various Drain Spacing’s and a Fixed Consolidation Period of 4 Months

Using the graph with the correlation between the $c_v$, the settlement and the drain spacing, the boundary condition of a minimal settlement of 1.2 to 1.4 m should be reached with the following configurations. With an average $c_v$ value of 2.5 E-08 m²/s, which is derived from [2], a drain spacing of 0.9 x 0.9 is needed to achieve the 1.3 meters of settlement. If the $c_v$ is higher, than it is possible to increase the drain spacing. A consolidation of 60% would be reached with also a 0.9 x 0.9 m drain spacing within the time constraint of the enabling work.
Reviewing the articles, it has to be kept in mind that the settlement data provided by the client shows large variations. It is possible that local geological variations give less or more settlement than the calculated. Therefore a requirement in terms of an absolute amount of settlement is not advisable. A rate of consolidation was much more preferable. Therefore a consolidation of 60% would be a reasonable demand. To make sure this demand is reached, the drains should be installed at 0.85 x 0.85 m drain spacing.

CONCLUSION

To reach 60% consolidation within the available time a triangular spacing of 0.85 m is required for the EW areas. The embankment should also be constructed as quickly as possible to use the maximum available consolidation period with the largest surcharge. The rate of loading was restricted by the stability of the embankment. To avoid instability calculations had to be made to set up a loading schedule that meet stability as well as the required consolidation. If the measured settlements did not fit with the prognosis or the calculated residual settlements are larger than expected the following measurements were taken:

- Placing extra vacuum pumps to increase amount of vacuum
- Placing of extra fill
- Extending of the loading period

The measured settlements corresponded to the predicted consolidation rate. There for the conclusion can be made that Beaudrain-S was a very effective system to eliminate future differential settlements at Suvarnabhumi Airport.

REFERENCES

Table 1: Input Soil Parameters Used for Finite Element and Stability Analyses.


Bergado D.T. et al, Vacuum Consolidation with PVD at SBIA Site on soft Bangkok Clay

Horpibulsuk S., Evaluation of Ground Improvement and Recharge using Prefabricated Vertical Drain for the Second Bangkok International Airport
CONSTRUCTION CONTROL AND MANAGEMENT OF GROUND IMPROVEMENT OF AIRSIDE PAVEMENTS

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S. Koslanant
Ph.D. Candidate, Saga University

Z.C. Moh
MAA Engineering Consultants International Ltd.

ABSTRACT

Due to the underlying soft marine clay, ground improvement by using Prefabricated Vertical Drains (PVDs) with preloading embankment was selected at Suvarnabhumi Airport to reduce the post-construction settlement prior to construction of permanent airport facilities. Ground improvement for airside pavement at airport site was commenced in November 1997 and completed in September 2004. Over 42,000,000 m PVDs have been installed at the project site. A comprehensive instrumentation program was undertaken to observe the ground improvement performance. Total construction cost is over 9.7 billion Baht under two different construction contracts. Details of ground improvement work related to construction management for this large-scale ground improvement project are described in the paper.

INTRODUCTION

The Suvarnabhumi Airport (SA) Project was initiated by the Royal Thai Government for more than forty years ago. The Project was designed to replace the overloaded Bangkok International Airport at Dong Muang in order to meet the need of a growing demand for aviation transport worldwide. It will also help promoting the Bangkok Airport System to be developed into a modern international aviation hub in Asia. With this “hub development concept”, the SA is catered to serve all international long haul flights, flights to cities within Bangkok’s key services area in Asia, as well as providing a full range of domestic services.

The Project site is located at Nong Ngu Hao, Bangphli District, Samut Prakan Province, about 30 km east of the Bangkok Metropolis. It occupies an area of about 3,200 ha. with an approximate boundary length of 8 km in the north and south and 4 km in the east and west. The north boundary is the Samut Prakan Province boundary, having Khlong Nong Ngu Hao as the east boundary, Khlong Thewa Trong in the south and Khlong Lad Krabang in the west.
The Project site initially consisted of many ponds, plants and other surface structures such as houses. Figure 1 shows the project location and the general airport layout plan.

During the 20 years of planning prior to commencing of construction, various studies such as accessibility and ground surface hydrology have been conducted to evaluate the suitability for the Nong Ngu Hao site to be developed into a new international airport. Due considerations were also made in the evaluation of subsoil conditions that form an important part of the foundation of all land based infrastructures, namely, roads and runways. The overall geological conditions of the 35 m deep subsoil consist of five soil strata including weathered clay, soft to medium clay, stiff clay and dense to very dense sand. The underground soil consisting of very soft marine clay with thickness ranged from 7 to 10 m. is a major problem on ground infrastructures due to unpredictable and excessive settlement leading to potential failure of the structures. The overall site development concept for the airport is to construct facilities at existing ground level with polder system around the site for flood prevention instead of elevating the airport facilities above potential flood levels. It is necessary to improve the in-situ soil properties sufficiently to support the applied loads. The preferred engineering approach to the site soil improvement program is pre-consolidating the subject area by applying temporary preloading surcharge load which should be greater than the permanent load to reduce post-construction settlement and strengthen the soil. Installations of sub-surface drainage system such as Sand Drains or Prefabricated Vertical Drains (PVD) are necessary to accelerate the primary consolidation settlement.

PROJECT OUTLINE

Construction of the entire new airport is managed by the New Bangkok International Airport Co., Ltd. (NBIA), a state enterprise under the Ministry of Transport and Communications, Thailand. Design of the ground improvement for the Airside Pavements was carried out by the Airside Design Group (ADG) in 1995. The entire ground improvement work of Airside Pavement was divided into two phases under separate construction contracts with different supervision teams. The two phases had different implementation schedule but with the same design scheme.

![Fig. 1 Project Location and Layout Plan](image-url)
The Ground Improvement Phase I (G.I. Phase I) for Airside Pavement, one of the initial projects for the Suvarnabhumi Airport, was commenced in November 1997 in order to prepare the site for construction of permanent structures in future. Besides site clearing and levelling for about 53 percent (1,699 ha) of all airport sites, ground improvement by using PVD and surcharge load was applied to the Airside Pavements area including the West Runway, Taxiways, Apron, part of East Taxiway and two emergency access roads to accelerate consolidation settlement before construction of the pavement. The total improved area is about 10 percent (308 ha) of the total airport site. Total contracted amount for the Ground Improvement Phase I work was about 8.24 billion Baht (or 206 million US dollar) and was completed in June 2002. Pavement construction on West Runway and Aprons has been started since September 2003.

The Ground Improvement Phase II (G.I. Phase II) work covering the 4,000m long East Runway with two Taxiways and the Landside Road System of 21 access roads inside the airport was commenced in December 2002 and completed in August 2004 with total construction cost of 3 billion Baht (or 75 million US dollar). Pavement construction on the East Runway was started immediately after the surcharge removal. Figure 2 illustrates the working area of Phases I and II and Table 1 summarizes the contract details.

<table>
<thead>
<tr>
<th>Table 1 Contract Data of G.I. Phases I and II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Construction Cost (Thai Baht)</td>
</tr>
<tr>
<td>G.I. Phase I</td>
</tr>
<tr>
<td>8,419,205,480</td>
</tr>
<tr>
<td>G.I. Phase II*</td>
</tr>
<tr>
<td>1,308,647,400</td>
</tr>
<tr>
<td>Financial Source</td>
</tr>
<tr>
<td>Government Budget</td>
</tr>
<tr>
<td>OECF Loan (Japan)</td>
</tr>
<tr>
<td>Construction Period</td>
</tr>
<tr>
<td>01/11/97 - 30/04/02 (54 mos.)</td>
</tr>
<tr>
<td>09/12/02 - 07/08/04 (21 mos.)</td>
</tr>
<tr>
<td>Contractor</td>
</tr>
<tr>
<td>Italian - Thai Development Public Co., Ltd.</td>
</tr>
<tr>
<td>(ITD)</td>
</tr>
<tr>
<td>Vichithan Construction Co., Ltd., Krung Thon</td>
</tr>
<tr>
<td>Engineers Co., Ltd., and Prayoonvisava</td>
</tr>
<tr>
<td>Engineering Co., Ltd. (VKPJ)</td>
</tr>
<tr>
<td>Construction Supervision Consultants</td>
</tr>
<tr>
<td>TMSUM (TEC/MAA/SIGEC/UIC/MTL)</td>
</tr>
<tr>
<td>TNM (TEC/MAA/NK)</td>
</tr>
</tbody>
</table>

* Not including Landside Road System

CONSTRUCTION PROCEDURES

Ground improvement target is to reach 80 percent of primary consolidation at 6 months (or 11 months under 8.67 t/sq. m surcharge load at Aprons near the Terminal Building and the Concourses, and at both ends of runway) after final stage loading. The minimum design
surcharge of 7.65 t/sq. m is to reach stress well above the apparent pre-consolidation stresses and to achieve a minimum thickness of sand fill in the lowest elevated pavement areas. The major design features of the ground improvement for Airside Pavements are summarized as below:

- Embankment fill thickness: 3.8m-4.2m (7.65 t/sq. m to 8.67 t/sq. m)
- Counterweight berm thickness: 1.7m
- Thickness of soft clay layer: 9.0~11.0m
- Total unit weight of soft clay: 1.43 t/cu. m
- Undrained vane shear strength: 1.5 t/sq. m
- PVD length & spacing: 10m with 1m spacing in square pattern
- Sand drainage thickness: 1.5m
- Coefficient of vertical/horizontal consolidation, cv/ch: 2.0 sq. m/year & 4.0 sq. m/year
- Coefficient of compressibility, Cc: 1.45

Fig. 2 Ground Improvement Work Area

The ground improvement work, as illustrated in Figure 3, consists of site preparation (including site clearing/leveling/backfilling), subsoil investigation, instrument installation and monitoring, first layer filter fabric placement (on original ground), sand blanket construction, drainage facilities installation, PVD installation, sand drainage layer construction, second layer filter fabric placement (on sand drainage layer), surcharge fill construction (in two stages) and removal. It should be noted that several large canals with depth up to 6 m were backfilled with clay and/or sand during site cleaning and leveling.

Drainage facilities include collector pipes, sub-drainage pipes and manholes were installed. Therefore, the generated pore water under preloading was not only be drained out from the sand blanket/drainage layer, but also be collected (and the manholes be pumped out
from). Water pump with adequate capacity (more than 10 liter/sec) was provided at each manhole for pumping water from manhole to flood protection system. Pumping operation was carried out continuously throughout preloading period. Temporary dikes and ditches with pumping facilities were also built and installed to protect ground improvement area from flooding during construction period. Sand blanket was designed to drain out the excess pore water as well as being the working platform for PVD installation.

![Flowchart of Ground Improvement Procedure](image)

Fig. 3 Ground Improvement Procedure

The surcharge fills by using crushed rock was placed in two stages to reach the final preloading height. It was placed in layer of maximum 0.20m thickness and compacted to the required density of 2.14 t/m³ as minimum. The counterweight berms with 1.7 m height and 15.4 m width were constructed to stabilize the preloading embankment. The embankment
construction in the first stage was to reach a total of 2.8m fill thickness followed by 3 months waiting period for increase of subsoil strength prior to the next stage of construction. Due to limited construction schedule, the 3 months waiting period at most of East Runway and Taxiways were reduced to 1.5 to 2 months. The final preloading height under 7.65t/m² and 8.67t/m² was 3.8 m and 4.3 m in the final design, respectively. The surcharge load of 8.67t/m² was required at Aircraft Stands, Apron close to Terminal Building and end of both Runways. According to the results obtained from the Reference Section, the average surcharge load in a total fill height of 3.8 m equivalent to 8.27t/m², which was about 8% more than the design load due to the high field compaction requirement. Therefore, the final fill thickness was then adjusted according to the accumulated field density to satisfy the design surcharge load of 7.65t/m² and 8.67t/m² in order to save the construction cost. In general, final preloading height was about 10 to 20cm less than the design fill height.

An initial section as called “Reference Section”, was first constructed at West Taxiway with one canal crossing during G.I. Phase I in order to:

- Confirm the design assumptions and the criteria for accepting the improved ground;
- Check the Contractor’s working methods; and
- Check the installation procedures and suitability of the instruments used.

The required criteria for surcharge removal after reaching the final stage loading were specified as below:

- Minimum 6 (under 7.65t/m²) or 11 (under 8.67t/m²) months waiting period;
- Pore water pressure dissipation $\geq 75$ percent or degree of consolidation (settlement) $\geq 80$ percent; and
- Settlement ratio (the last monthly settlement to the cumulative settlement) $\leq 4$ percent (under 7.65t/m²) or 2 percent (under 8.67t/m²).

In general, the final waiting period was actually one or two month(s) longer than the estimated time at most of Airfield Pavement area (both Phase I & II) mainly to satisfy the settlement ratio criterion. Preloading embankments were removed to MSL elevation during G.I. Phase I and to sub-grade elevation during G.I. Phase II for the following pavement construction. The lower part of the manholes, 0.50 m. below the sub-grade level, were filled with sand after surcharge removal. Table 2 summarizes the total quantity of major items used in both projects.

In order to save ground improvement cost, surcharge fill was removed either as reuse or as stockpile for future use. The use for surcharge fill during G.I. Phase I was planned as follows:
(1) Imported crushed rock surcharge fill was placed at Apron area and Reference Section (in west runway area).

(2) Excavation, moving and replacing crushed rock were performed in 2 cycles:
   ➢ The first cycle - from apron to replace at cross taxiway and part of west runway.
   ➢ The second cycle - from cross taxiway and part of west runway to replace at the rest areas of west runway, part of east runway and emergency roads no. 4 and 9.

(3) Excavation and removing surcharge fill to stockpile area (as shown in Table 3) were separated in three kinds of material namely drainage sand (surplus from drainage layer), contaminated crushed rock (consisted of crushed rock, sand and pieces of filter fabric) and crushed rock. As comparing the import quantity with the final stockpile quantity, about 90% of the crushed rock material was recovered.

Table 2 Material Quantities in Airside Pavements Ground Improvement Work

<table>
<thead>
<tr>
<th>Item</th>
<th>Unit</th>
<th>Phase I</th>
<th>Phase II</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drainage sand</td>
<td>m³</td>
<td>4,447,453</td>
<td>1,525,000</td>
<td>5,972,453</td>
</tr>
<tr>
<td>Filter fabric</td>
<td>m²</td>
<td>6,793,294</td>
<td>2,188,256</td>
<td>8,981,550</td>
</tr>
<tr>
<td>PVD</td>
<td>m</td>
<td>31,288,708</td>
<td>9,453,785</td>
<td>41,042,493</td>
</tr>
<tr>
<td>Crushed rock</td>
<td>m³</td>
<td>2,947,025</td>
<td>2,340,900*</td>
<td>5,287,925</td>
</tr>
<tr>
<td>Subdrainage pipe</td>
<td>m</td>
<td>255,755</td>
<td>79,350</td>
<td>335,105</td>
</tr>
<tr>
<td>Collector pipe</td>
<td>m</td>
<td>12,799</td>
<td>3,517</td>
<td>16,316</td>
</tr>
<tr>
<td>Manhole</td>
<td>No.</td>
<td>142</td>
<td>41</td>
<td>183</td>
</tr>
</tbody>
</table>

*Obtained from stockpile

Table 3 Total Quantities at Stockpile Area in G.I. Phase I

<table>
<thead>
<tr>
<th>Item</th>
<th>Unit</th>
<th>Q'ty</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crushed Rock</td>
<td>m³</td>
<td>2,362,869</td>
<td>Some material were used under instruction by the NBIA</td>
</tr>
<tr>
<td>Contaminated Crushed Rock</td>
<td>m³</td>
<td>406,998</td>
<td></td>
</tr>
<tr>
<td>Drainage Sand</td>
<td>m³</td>
<td>128,526</td>
<td></td>
</tr>
</tbody>
</table>

INSTRUMENTATION

A well-planned instrumentation and monitoring program is necessary to provide sufficient and important information to ensure a safe construction and to evaluate the performance of ground improvement. Precaution measures could then be taken if there was any abnormal situation occurring during the construction. In general, monitoring results of instrumentations are used:

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- To evaluate the ground improvement performance
- To monitor the efficiency of drainage system
- To control the embankment safety during construction
- To provide data for the decision on surcharge removal
- To monitor the long-term ground subsidence and groundwater condition

Instruments installed at site included surface settlement plates, deep settlement gauges, surface settlement monuments, piezometers, inclinometers and observation wells. Purpose and installed depth of each instrument are described below:

**Permanent Benchmark (PB)**
- Installed to depth of 150m at east and west of project site for survey reference

**Surface Settlement Plate (SP)**
- Installed above sand blanket layer (except for the Reference Section where SP were installed on the original ground) to measure vertical settlement under surcharge fill

**Deep Settlement Gauge (DG)**
- Installed at various depths to measure the vertical settlement of soils underlying the specified depth

**Surface Settlement Monument (SM)**
- Installed at top of preloading embankment to measure the fill settlement.

**Inclinometer (IM)**
- Installed at intersection between berm and main embankment or on the main embankment to a depth of 20m below original ground surface to measure lateral displacement.

**Electric Piezometer (EP)**
- Installed at various depths to monitor the dissipation of excess pore water pressure.

**AIT-Type (open stand pipe) Piezometer (AP)**
- Installed at various depths outside preloading embankment to measure the pore water pressure in the sand layer

**Observation Well (OW)**
- Installed at ground level to measure the ground water level under surcharge fill

Dummy instruments including piezometers, deep settlement gauges, and AIT-Type piezometers were installed at a distance of 30m from the toe of the preloading embankment to provide base readings. A typical instrumentation profile is shown in Figure 4. Monitoring work was carried out mainly on weekly basis and before and after each change in loading until the approval of surcharge removal. Table 4 summaries the total quantities of instruments and monitoring work at the project site.
Three important features are often analysed through monitoring data during preloading construction, which include (i) Deformation Characters, (ii) Stability Problems, and (iii) Pore Pressure Developments. The monitoring data obtained from instruments were further to be managed into the following formats to evaluate embankment performance:
- The relationship of settlement at varied depth under surcharge fill with time
- The relationship of settlement with time at dummy area
- The relationship of lateral movement at varied depth under surcharge fill with time
- The relationship of pore water pressure with time at dummy area
- The dissipation of excess pore pressure generated under surcharge fill with time

Ratio of lateral movement to vertical settlement was used as the criterion for safety control in both projects. Special attention was given to the control of construction site if the ratio exceeds 0.25. Settlements at small loop of West Runway and East Taxiway (under same surcharge load) where PVD was not required was about 30 to 35% of settlement measured at the nearby PVD area. The settlement results have shown the effectiveness of PVD installation.
Table 4. Total Quantities of Instruments and Monitoring Work

<table>
<thead>
<tr>
<th>Item</th>
<th>Phase I</th>
<th>Phase II</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP</td>
<td>1,724</td>
<td>-</td>
</tr>
<tr>
<td>SM</td>
<td>553</td>
<td>-</td>
</tr>
<tr>
<td>PB</td>
<td>-</td>
<td>2</td>
</tr>
<tr>
<td>IM</td>
<td>56</td>
<td>-</td>
</tr>
<tr>
<td>DG</td>
<td>555</td>
<td>55</td>
</tr>
<tr>
<td>EP</td>
<td>444</td>
<td>46</td>
</tr>
<tr>
<td>AP</td>
<td>-</td>
<td>40</td>
</tr>
<tr>
<td>OW</td>
<td>1,722</td>
<td>-</td>
</tr>
<tr>
<td>Total</td>
<td>5054</td>
<td>143</td>
</tr>
</tbody>
</table>

SUBSOIL INVESTIGATION

Subsoil investigation including undisturbed sampling, field vane shear test and PCPT was implemented in two stages:

➢ Before ground improvement to verify the soft clay thickness and subsoil condition
➢ After surcharge removal to evaluate the change of soil properties.

Purpose of each field investigation method is described below:

Undisturbed Sampling — To obtain continuous undisturbed soil samples for laboratory testing through Shelby tube up to a maximum depth of 15m.

Field Vane Shear Test — To obtain profile of undrained field vane shear strength to the maximum depth of 15m.

Piezocone Penetration Test — To obtain cone resistance, local friction and friction ratio data through cone penetrometer with tip and sleeve to a maximum depth of 20 m. Pore pressure dissipation tests were also performed by pore pressure filter at depth 3 m, 5 m, 8 m and 12 m.
Standard Penetration Test — To obtain the SPT blow counta at end of undisturbed sampling borehole.

Quantities of subsoil investigation implemented before and after ground improvement work including both Phases I and II are summarized in Table 5.

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Locatio</th>
<th>Quantity</th>
<th>Before GI</th>
<th>After GI</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Nos.</td>
<td>m</td>
<td>Nos.</td>
<td>m</td>
<td>Nos.</td>
</tr>
<tr>
<td>1</td>
<td>PCPT</td>
<td>RF</td>
<td>41</td>
<td>752.8</td>
<td>4</td>
<td>71.60</td>
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<tr>
<td></td>
<td></td>
<td>A</td>
<td>111</td>
<td>2,049.</td>
<td>24</td>
<td>360.0</td>
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<tr>
<td></td>
<td></td>
<td>W</td>
<td>159</td>
<td>3,157.</td>
<td>17</td>
<td>255.0</td>
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<tr>
<td></td>
<td></td>
<td>E</td>
<td>75</td>
<td>1,479.</td>
<td>64</td>
<td>1,241.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C</td>
<td>28</td>
<td>539.5</td>
<td>5</td>
<td>75.00</td>
</tr>
<tr>
<td></td>
<td><strong>Total</strong></td>
<td></td>
<td><strong>414</strong></td>
<td><strong>7,978.</strong></td>
<td><strong>114</strong></td>
<td><strong>2,870.</strong></td>
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<tr>
<td>2</td>
<td>Vane Shear Test</td>
<td>RF</td>
<td>19</td>
<td>278.0</td>
<td>4</td>
<td>59.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>A</td>
<td>51</td>
<td>756.5</td>
<td>22</td>
<td>342.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>W</td>
<td>73</td>
<td>1,092.</td>
<td>17</td>
<td>253.5</td>
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<tr>
<td></td>
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<td>E</td>
<td>56</td>
<td>845.0</td>
<td>45</td>
<td>678.0</td>
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<tr>
<td></td>
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<td>C</td>
<td>14</td>
<td>208.5</td>
<td>8</td>
<td>120.0</td>
</tr>
<tr>
<td></td>
<td><strong>Total</strong></td>
<td></td>
<td><strong>213</strong></td>
<td><strong>3,180.</strong></td>
<td><strong>96</strong></td>
<td><strong>2,027.</strong></td>
</tr>
<tr>
<td>3</td>
<td>Undisturbed Sampling</td>
<td>RF</td>
<td>11</td>
<td>153.6</td>
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<td></td>
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<td>A</td>
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<td>303.3</td>
<td>18</td>
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<tr>
<td></td>
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<td>W</td>
<td>26</td>
<td>376.0</td>
<td>18</td>
<td>251.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>E</td>
<td>28</td>
<td>389.4</td>
<td>24</td>
<td>351.4</td>
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<tr>
<td></td>
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<td>C</td>
<td>7</td>
<td>102.5</td>
<td>5</td>
<td>74.55</td>
</tr>
<tr>
<td></td>
<td><strong>Total</strong></td>
<td></td>
<td><strong>91</strong></td>
<td><strong>1,324.</strong></td>
<td><strong>52</strong></td>
<td><strong>760.4</strong></td>
</tr>
<tr>
<td>4</td>
<td>Standard Penetration Test</td>
<td>RF</td>
<td>11</td>
<td>55.85</td>
<td>4</td>
<td>1.90</td>
</tr>
<tr>
<td></td>
<td></td>
<td>A</td>
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<td>29.20</td>
<td>18</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>W</td>
<td>26</td>
<td>12.60</td>
<td>18</td>
<td>35.60</td>
</tr>
<tr>
<td></td>
<td></td>
<td>E</td>
<td>18</td>
<td>13.05</td>
<td>16</td>
<td>22.65</td>
</tr>
<tr>
<td></td>
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<td>C</td>
<td>7</td>
<td>3.15</td>
<td>5</td>
<td>0.45</td>
</tr>
<tr>
<td></td>
<td><strong>Total</strong></td>
<td></td>
<td><strong>83</strong></td>
<td><strong>113.8</strong></td>
<td><strong>61</strong></td>
<td><strong>60.60</strong></td>
</tr>
</tbody>
</table>

CONSTRUCTION MANAGEMENT AND QUALITY CONTROL

Project Organization

In general, the project organization was divided into three parts: NBIA (the client), CSC (the engineer for supervision) and the contractor as described below:
The NBIA – The Construction Supervision Services Committee (CSSC) and the Inspection Committee (IC) was set up by the NBIA to handle the payment and other general matters for CSC and Contractor, respectively. All technical issues and overall site planning were handled by GEC or PMC (after October 1999), the project management consortium for NBIA. A project manager was also assigned by the NBIA to act as the representative and coordinator between the NBIA and all concerned parties during the project implementation.

The CSC – The CSC of Phases I and II (TMSUM and TNM, respectively) was generally divided into three major sections based on their specialties: Administration, Earthwork and Geotechnic, handled by it’s Deputy Project Managers under the Project Manager. The Administration section was to manage site administration, documentation, and cost and schedule control etc. The Earthwork section was responsible in survey, site clearing/leveling/discing, canal backfilling, sand and crushed rock material, drainage pipes and manholes etc. The Geotechnical section was to deal with soil investigation, instrumentation, PVD, filter fabric, monitoring and data interpretation etc. The engineers in charge on QA/QC, Data System and Construction Safety were under the Project Manager directly. There were also special advisors in ground improvement and airport system to help the Project Manager in handling related technical issues.

The Contractor – The ITD, contractor of G.I. Phase I, was led by a Project Director with several project managers underneath in handling Production I (Aprons and Cross Taxiways), Production II (West and Runways and East Taxiway), and other administration issues. Construction work on the East Runway was divided into four sections (Section I to IV) handled by Project Manager of the VKPJ, contractor of G.I. Phase II.

Communication channel in the project was divided into two types: NBIA vs. CSC and CSC vs. the Contractor. In principle, there should not have direct contact between NBIA and the Contractor or GEC (PMC) and CSC according to the responsibility defined in the contract. Weekly progress meetings were held between CSC and the Contractor with the attendance of NBIA and GEC (PMC).

Document Control

Major documents required in the project included Daily Request, Progress Report, and Submission as described below:
Daily Request

Contractor was required to make its fieldwork request on daily basis. The contents in the request should include type of work (survey, site preparation, soil investigation, filter fabric, earthwork, drainage, PVD, instrumentation or others), type of request (approval or information), work description, location and time with other necessary information. The daily request was submitted to CSC not later than 5:00 p.m. on the previous day.

Progress Report

Both CSC and Contractor had to prepare and submit their progress report. Contractor’s progress report had to be prepared on daily, weekly and monthly basis. The daily progress report summarized the field activities during the day including working item with quantity and location, manpower, equipment, delivered material and weather record etc. and had to submit to the CSC office not later than 9:00 am on the following day and then forward to NBIA after approval. Weekly/monthly reports included the details of weekly/monthly activities and were used for discussion during weekly/monthly progress meeting. The CSC had also to prepare weekly/monthly progress report and bi-week cost estimation report. The CSC’s report included more detailed information on monitoring data and ground improvement evaluation, quality control matters and financial information.

Submittal

The submittals included all non-regular documents such as letter, memo, drawings, report or invoice etc. A numbering system with four components including addressee code, correspondence type, serial number and originating section was specified in order to categorize all documents systematically. The correspondence numbering system used in GI. Phase I is shown as below:

```
Corresp| Addressed | Originating Section
       |           | Serial Number
```

Addressee: The addressee code refer to organizations in which the correspondence is addressed to and four types of addressee codes has been define as below:

NBIA – The Client
ENG – The Engineer (TMSUM)
ITD – The Contractor
GEN – The Others
Correspondences: The correspondences are divided into 6 types as below:
L – Letter; F – Facsimile; S – Submittal; M – Meeting Minutes; R – Report; I – Invoice

Serial number: A continuous four-digit serial number is used.

Originating section: The originating section refers to the organization responsible for the correspondences and defined as below:
N – NBIA; E – TMSUM; C – ITD

A total of four action codes were used by CSC after reviewing the submittal from Contractor, which are:

- Action Code 1 – No exceptions taken;
- Action Code 2 – Make corrections noted
- Action Code 3 – Amend and resubmit
- Action Code 4 - Reject

Since there were tremendous monitoring data to be reviewed and interpreted by CSC, e-data were normally required by the geotechnical engineer of CSC prior to formal submission in order to handle the data more efficiently, especially for the abnormal condition occurred at site.

QUALITY CONTROL

The relationships and responsibilities of the key personnel under the Quality Plan were organized to provide a systematic and complete coverage of all tasks and procedures necessary to ensure satisfactory implementation and control of the quality program. The functional organization in quality plan was divided into four distinct levels: Executive, Administration, Implementation and Field. At Executive level, the Project Manager was responsible for all QA/QC activities with NBIA. At Administrative level, the senior QA/QC Engineer was responsible for approval of QA/QC procedure, reviewing all material testing and inspection reports, cooperating with material engineer on the issues that need to be resolved, initiating random testing for quality audit as part of compliance checking. At implementation level, key personnel including the Deputy Project Manager (Geotechnics), the Deputy Project Manager (Earthworks and Drainage) and Material Engineer were responsible for all QA/QC activities within their respective sections and ensuring all procedures and checklists were being implemented according to the quality plan. The field level represented personnel who directly inspected and witnessed the contractor’s QA/QC activities. In general, QA/QC activities in this project focus on Survey, Earthwork/Drainage, and Geotechnics are described below:
Survey
- Checking and verifying that the project control points are within acceptable tolerances
- Monitoring instrumentation elevation (i.e., settlement plate, observation well)
- Measurement of placed material for the approval of payment

Earthworks/Drainage
- Inspection and approval of material source (sand and crushed rock (on G.I. Phase I only)
- Approval of material stockpile (sand and crushed rock)
- Control of material haulage (sand and crushed rock)
- Approval of drainage system material (subdrainage pipe, collector pipe, manhole)
- Verifying and monitoring the placement of materials on site
- Monitoring the removal of crushed rock material
- Inspecting that Contractor's laboratory testing equipment with the required standard
- Witness and approval of material testing (sand, crushed rock, drainage pipe)

Geotechnics:
- Review/Approval of the proposed materials (PVD, Filter Fabric, Instruments)
- Witness and approval of the delivery, stockpile and laboratory test of PVD and Filter Fabric
- Monitoring the placement of filter fabric and the installation of PVD on site
- Monitoring the subsoil investigation
- Monitoring the instrument installation and data processing work

To ensure adequate quality control on procedures and measures in all materials used in the project, “Quality Plan” was prepared and used by the CSC to monitor and control the construction works in compiling with the specifications, drawings and other contract documents. There were specified procedures and varied forms in work request, test result, inspection and checklist in the "Quality Plan". For example, the approval procedures in source and stockpile for sand and crushed rock are shown in Figure 5. G.I. Phase I approved seventy-two and twenty-two quarry sources for sand and crushed rock, respectively.

Beside the initial tests, routine and general tests were also performed on every 50,000 m² of filter fabric placement on the items of weight, strip tensile strength, elongation at failure and CBR puncture strength as shown in Table 6. Quality control procedure on PVD material is illustrated in Figure 6. According to the specification, the approval of PVD material should also be agreed by the NBIA. PVD layout plans were required for approval by the Engineer prior to the field installation. Maximum of twenty PVD installation rigs had been working
together at same time during GI Phase I. Each rig had to demonstrate it's working capability at site for approval by the Engineer's representative prior to the use. In general, the PVD daily installation rate was about 8,000 m to 10,000 m per rig. During PVD installation, the accuracy of penetration depth was verified randomly by checking the marked depth on PVD roll. Three types of PVD quality control tests were specified in the project including initial test, general test and routine test. Required test items of each test are summarized in Table 7.

**Fig. 5 Flow Chart of Quality Control in Material Source and Stockpile**

**Fig. 6 Flow Chart of Quality Control in PVD Installation**
Table 6 Required Testing Criteria on Filter Fabric

<table>
<thead>
<tr>
<th>Item</th>
<th>Required Criteria</th>
<th>Test Standard</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Weight, g/m²</td>
<td>130</td>
<td>ASTM D 3776</td>
</tr>
<tr>
<td>Minimum strip tensile Strength, kN/m</td>
<td>8</td>
<td>ASTM D 4595</td>
</tr>
<tr>
<td>Maximum Elongation at Failure, %</td>
<td>50</td>
<td>ASTM D 4595</td>
</tr>
<tr>
<td>Minimum CBR puncture Strength, N</td>
<td>1,000</td>
<td>BS 6906/4</td>
</tr>
</tbody>
</table>

Table 7 Testing Requirements on PVD Material

<table>
<thead>
<tr>
<th>Item</th>
<th>Required Criteria</th>
<th>Test Standard</th>
<th>Type of Test*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Apparent Opening Size, μm</td>
<td>Less than 90</td>
<td>ASTM D 4751-87</td>
<td>I,G,GR</td>
</tr>
<tr>
<td>Grab Tensile Strength, kN</td>
<td>Greater than 0.35</td>
<td>ASTM D 4632-91</td>
<td>I,G</td>
</tr>
<tr>
<td>Trapezoidal Tear Strength, kN</td>
<td>Greater than 0.10</td>
<td>ASTM D 4533-91</td>
<td>I,G</td>
</tr>
<tr>
<td>Puncture Resistance, kN</td>
<td>Greater than 0.10</td>
<td>ASTM D 4833-88</td>
<td>I,G</td>
</tr>
<tr>
<td>Burst Strength, kPa</td>
<td>Greater than 900</td>
<td>ASTM D 3768-80A</td>
<td>I,G</td>
</tr>
<tr>
<td>Discharge Capacity at 7 days, 200 kPa at</td>
<td>Greater than 500</td>
<td>ASTM D 4716-87</td>
<td>I,G</td>
</tr>
<tr>
<td>Hydraulic gradient of 1. (In-Plane Flow), m³/yr</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Discharge Capacity @200 kPa and</td>
<td>Greater than 500</td>
<td>Modified Triaxial</td>
<td>I,G</td>
</tr>
<tr>
<td>hydraulic gradient of 1 (Modified Triaxial), m³/yr</td>
<td></td>
<td>(Straight)</td>
<td></td>
</tr>
<tr>
<td>Equivalent Diameter (Length + Width) /2, mm</td>
<td>Greater than 50</td>
<td>None</td>
<td>I,G,GR</td>
</tr>
</tbody>
</table>

*I: Initial Test; G: General Test; R: Routine Test

A sample on each PVD brand was collected from the underground to evaluate the change of PVD properties after completion of ground improvement work. The discharge capacity of all brands was about 70% less than its original value after one to two year(s) installation. However, most of strength characters still satisfied the requirement after the completion of ground improvement works.
Cost and Schedule Control

The objective of cost and schedule control was to review, monitor and provide recommendation on Contractor’s monthly progress payment, variation order or force account work in accordance with the approved construction schedule; and to provide all necessary actual and forecasted data in payment and progress with detailed productivity statistics to the NBIA. The following items were prepared and included in the CSC’s monthly progress report:

➢ Curves illustrating schedule and actual performance regarding time and budget
➢ Curves illustrating schedule and actual production rates for installation of major work activities
➢ Records of payment certificates
➢ Charts of forecast and actual cash flow

The CSC also looked for any possible cost saving and makes recommendation to NBIA provided such exercise will not reduce the quality of work as following examples:

➢ Use unit weight to control surcharge fill thickness (according to the design assumption) instead of a fixed height
➢ Reuse of sand blanket/drainage layer in concourse area to west runway in order to reduce imported material
➢ Reuse surface settlement monument after surcharge fill removal

MAJOR ISSUES DURING GROUND IMPROVEMENT CONSTRUCTION

Major concerned issues during ground improvement construction are described below:

Additional Surcharge Fill

In order to compensate for the effects of additional long term loading at aircraft stand areas within the Apron Area that would be subjected to aircraft parking, the surcharge loading was increased from 7.65t/m² to 8.67t/m² with an additional 4-month waiting period in these areas. This area of increase surcharge fill thickness was not included in the original design and construction drawings. The additional work resulted in both an increased volume of surcharge fill material to be removed and replaced and longer waiting periods, which had an delaying impact on the contract. Although no additional material was required, the Contractor had to determine a revised construction schedule in order to include the additional material moving time and waiting periods. Analysis indicated the optimal method resulted in an additional 6 months to the contract period. This revised schedule was subsequently accepted and approved
by the NBIA and both the Engineer and the Contractor were given 6-month extension to their respective contracts.

**Wat Nong Prue**

Wat Nong Prue was located within the project site and directly affected a small part of the area of ground improvement work for Part of the East Runway. However, due to delays associated with the construction of the new wat at a location outside the project site, the wat buildings were not completely removed until 6 June, 2001. As a result, part of one connecting taxiway for the East Runway System could not be fully constructed within the period of the contract, as the area for the construction of the counterweight berm was not available. Therefore the surcharge fill loading and removal was not undertaken and these works were removed from the contract (and transferred to the contract for Phase 2 Ground Improvement Works for East Runway). The work area for surcharge fill loading was set back by about 10m.

**Cracks**

Longitudinal cracks were observed at western bound of East Runway beginning from X=11,000 to X=11,200 as the ratios of maximum lateral movement to the vertical settlement were up to 0.4. The cracks located at western bound of Runway beginning from X=11,000 to X=11,200. The “Crack Indicators” were installed to record the cracking width regularly. The monitoring frequency was also increased on the cracking area to be twice per week in order to monitor the embankment behavior closely. It was further found that the cracks appeared on the backfilled Khlong Nong Pru area. In general, cracks may be resulted from high excess pore water pressures which decreased the embankment stability against slope failure and/or differential settlement due to recorded less settlement at center as comparing with both side of the main embankment. The Contractor was instructed to monitor the embankment behavior closely and to increase the water pumping frequency on the surrounding area to accelerate the dissipation of excess pore pressure during consolidation process. Additional pumping wells had been installed to accelerate the consolidation process as requested by the CSC. Longitudinal cracks also occurred on the counterweight berm at several areas of G.I. Phases I and II construction. The cracks mainly resulted from differential settlement between the PVD and Non-PVD section.

**CONCLUSIONS AND RECOMMENDATIONS**

Ground improvement work is the fundamental project for airport construction but the results and performance will not be visible easily in a short period of time. The conclusions and recommendations are made based on the performance and experience during project implementation:
1. Based on the ground improvement performance, it can be concluded that preloading with PVD installation is a suitable technique for accelerating consolidation of the Bangkok Clay under a careful design, effective drainage system and proper monitoring work.

2. Since settlement mainly resulted from the dissipation of excess pore water during consolidation process, sufficient sand blanket with an extensive network of drainage system with proper and prompt monitoring work will be the key factors for a successful ground improvement work by using PVD and preloading embankment. It is recommended that total thickness of sand drainage layer should be over the estimated settlement unless sufficient pumping facilities are provided.

3. The successful implementation of ground improvement project will depend on a clear understanding among the client, the engineer and the contractor of the project objective. The choice of the right contractor with good track record, experience and necessary technical back up is vital for the success of the scheme.

4. Surcharge removal was about one to two month(s) delay mainly to satisfy the required settlement ratio criterion.

5. Settlement at non-PVD area of West Runway was about one-third of settlement at PVD area under same preloading and time.

6. The three months waiting period after 1st stage loading should be possible reduced to one and half or two months under current preloading embankment configuration to reduce the overall construction period and to compensate the longer final waiting period as occurred at most sections.

7. The ground improvement work should start two years earlier than the pavement construction to ensure a sufficient time for satisfying the design assumption, especially when the cycling use of preloading material is also planned. It is possible to have longer preloading period with less surcharge load (but still have to be over the permanent load) to reduce the ground improvement cost.

8. Since many subsoil investigation had been done at site in the past years, it is the Engineer’s recommendation that all data obtained from previous soil investigation, field tests and monitoring records should be summarized as “Geotechnical Data Bank” for future use.

ACKNOWLEDGEMENT

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CURRENT PRACTICE AND FUTURE TRENDS OF DEEP-SEATED BORED PILES AND BARRETTES FOR MEGA PROJECTS IN THAILAND

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SEAFCO Public Company Limited, Bangkok, Thailand

ABSTRACT

Based on the authors’ experiences, this paper is intended to summarize the current practice of deep-seated bored piles and barrettes construction in Thailand, in comparison with that of the past. Literature related to wet-processed bored piles and barrettes in Thailand published throughout the past 30 years is summarized together with recent research works. An attempt is also made to briefly discuss the future trend of bored pile and barrette foundations. Result of the static pile load test performed on pilot bored pile at the Second Bangkok International (Suvarnabhumi) Airport Project during design phase is discussed and compared with that of bored piles constructed in other parts of Bangkok.

INTRODUCTION

During the past three decades, owing to the acceleration of development in mega infrastructure projects, techniques and practice of cast-in-place deep foundations, particularly wet-processed deep-seated large diameter bored piles and barrettes have experienced enormous progress in Thailand. The versatility of the construction method and the high-load capacity which in turn offered the constructibility and cost-saving are the main factors contributed to the increasing use of those cast-in-place deep foundations. In the initial stage, there were a number of questions on design and construction aspects of these foundation systems particularly in Bangkok subsoil. With the passage of time, the construction equipment, installation techniques and testing of deep foundation elements have been developed. Large numbers of instrumented full-scale static pile load tests were conducted throughout the 1990s mainly in mega projects, which provided better understanding on behavior of these deep-seated foundations.

Research focused on the design parameters and methods were produced based on these test results. The design parameters became so well established that wet-process cast-in-place foundations became regarded as reliable foundations for practitioners involved in construction industry in Thailand. This paper presents the past and current practice of wet-processed bored piles and barrettes in Thailand together with a brief discussion on future trend of these cast-in-place deep foundations. The information contained in the paper is mainly from the mega projects constructed in Bangkok and adjacent areas.
SUBSOIL AND EXISTING PIEZOMETRIC PROFILE

Subsoil profile and the present piezometric drawdown condition of Bangkok are presented in Figure 1 below. A typical subsoil profile is relatively consistent in different localities in Bangkok. It is characterized by alternating layers of clay and sand deposits as shown in Figure 1.

![Subsoil Profile Diagram]

Fig. 1. Typical Soil Profile of Bangkok with Piezometric Drawdown Condition (Thasnanipan, et. al., 2002)

CONSTRUCTION METHOD IN GENERAL

Due to the prevailing subsoil and groundwater conditions, deep-seated bored piles of toe depth over 24m are constructed by wet-process or slurry displacement method. Wet-process bored piles can be constructed by 2 methods, reverse-circulation and rotary drilling. As construction procedure of historic reverse-circulation method has been well documented in various studies and since it is less frequently utilized nowadays in Thailand, only a rotary-drilling method is briefly presented in this article as follows.

In the rotary drilling method, a temporary casing of appropriate length (12 to 18m in Bangkok depending on the thickness of soft clay) with required diameter (internal diameter not less than that of design bored pile diameter) is first installed to ensure the stability of the borehole in the top soft or loose soil layers. In some projects where the vibration is strictly limited, a standard length of casing with oscillator or short temporary casing of 5 to 8 m is pushed down in combination with the pre-boring process. Drilling is commenced by auger to drill out the soil inside the temporary casing. Auger-drilling is commonly continued up to the top of the first water-bearing sand layer or bottom of the casing when using the short casing method. Drilling slurry or supporting fluid is then supplied to the borehole and drilling is proceeded with a bucket down to the design final depth of the pile. Before lowering the reinforcement cage, a special cleaning bucket is used to clean the base of borehole. If bentonite slurry is used, recycling method by air-lift or pump is applied as the base cleaning process. Reinforcement cages are then lowered into the borehole and concreting is carried out by tremie method.
A mechanical or hydraulic cable-suspended grab is commonly used for barrette construction. Excavation of the trench is carried out by the cyclic-process of lifting and lowering of the grab under gravity and tangential force of the clamshell operated by cables (mechanical grab) or hydraulic action (hydraulic grab). Different from bored pile construction, a guide wall of depth 1 to 1.5 m with inside clear dimensions slightly larger than the nominal size of the barrette is used to guide the grab during initial bites. Bentonite slurry is introduced to the trench as soon as the initial excavation commenced. The excavation is continued under the bentonite slurry to the final depth. After recycling of slurry and lowering the rebar cage, concreting is done by tremie method.

OVERVIEW OF APPLICATION

Bored Pile

The first wet-process large diameter bored pile was constructed for Pinklao Bridge in Bangkok 30 years ago. Using reverse circulation method, a 1.50m diameter bored pile was installed up to 45m in the second sand layer. Three major bridges were constructed by bored pile using the reverse circulation method in Bangkok from early 1970 to 1980. The summarized information of these bridges is tabulated below.

Table 1 First 3 Major Bridges Constructed by Large Diameter Wet-Processed Bored Piles in Bangkok

<table>
<thead>
<tr>
<th>Project Name</th>
<th>Year of Construction</th>
<th>Construction Method</th>
<th>Diameter (m)</th>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pinklao Bridge</td>
<td>1971</td>
<td>Reverse circulation</td>
<td>1.50</td>
<td>45</td>
</tr>
<tr>
<td>Sathorn Bridge</td>
<td>1979</td>
<td>Reverse circulation</td>
<td>1.50</td>
<td>46</td>
</tr>
<tr>
<td>New Memorial Bridge</td>
<td>1982</td>
<td>Reverse circulation</td>
<td>1.50</td>
<td>49</td>
</tr>
</tbody>
</table>

The first wet-processed bored pile utilizing the rotary-drilling method down to first sand layer was constructed in Bangkok in the late 1970s for high-rise building project, the Royal Orchid Hotel located at the bank of Chao Phraya river. Since then bored piles constructed by rotary-drilling method have been extensively used for foundations of various heavy structures such as high-rise buildings, elevated expressways, overpass-bridges, underground car park buildings, waste-water treatment plants and most recently underground train stations of Bangkok’s first subway project.

By mid 1980s, bored piles became the foundation of choice for heavy structures particularly in the urban area of Bangkok. The versatility of the construction method and the high-load capacity which in turn offered the constructability and cost-saving, are the main factors contributing to the increasing use of deep-seated large diameter bored piles and barrettes. Most of the early wet-process bored piles (1980s) in Bangkok were constructed up to 50 m depth. Sizes of bored piles constructed in early days ranged from diameter 0.6 to 1.5 m. Summarized information of early major high-rise building projects in Bangkok is presented in Table 2.
Table 2 First High-Rise Building Projects Constructed with Large Diameter Wet-Processed Bored Piles in Thailand (from 1979 to 1983)

<table>
<thead>
<tr>
<th>Project Name</th>
<th>Construction Method</th>
<th>Diameter (m)</th>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Royal Orchid Hotel</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 - 1.00</td>
<td>33.0</td>
</tr>
<tr>
<td>(1979)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Taiping Tower</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 - 1.50</td>
<td>32.0</td>
</tr>
<tr>
<td>(1980)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>River City Hotel</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 – 1.00</td>
<td>27.5</td>
</tr>
<tr>
<td>(1982)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Asoke Tower</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 - 1.50</td>
<td>50.0</td>
</tr>
<tr>
<td>(1983)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Time Square Building</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 - 1.50</td>
<td>50.0</td>
</tr>
<tr>
<td>(1983)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3 Summary of Barrette Selection in Major Projects

<table>
<thead>
<tr>
<th>Selection Criterion</th>
<th>No. of Project</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>High Load Capacity</td>
<td>6</td>
<td>Incorporated with bored piles</td>
</tr>
<tr>
<td>Minimize Construction Equipment</td>
<td>7</td>
<td>Alternative for bored piles</td>
</tr>
<tr>
<td>Limited Head Room for Excavation</td>
<td>3</td>
<td>Under existing structures such as bridges, elevated expressway and power lines</td>
</tr>
<tr>
<td>Combination with Diaphragm Wall</td>
<td>16</td>
<td>As diaphragm wall legs</td>
</tr>
<tr>
<td>Foundation as well as portion of column</td>
<td>3</td>
<td>Provision for the future requirement</td>
</tr>
</tbody>
</table>

Barrette

In some projects where bored piles were not feasible due to the site constraints, applicable construction methods, and/or extensive bearing capacity requirements, the use of barrette foundations would make a suitable alternative. Barrettes with dimensions ranging from 0.80 m x 2.7 m to 1.5 m x 3.0 m for safe working load capacity from 11,000 to 23,000 kN have been used in some major projects.

The first barrette in Bangkok was believed to be constructed in the late 1970s for the foundation of the Bangkok Bank Head Office Building at Silom road with size of 0.6-0.8 x 2.5m and toe depth of 33m. Barrettes can be constructed with flexible layout plans for both vertical and lateral loads. The layout pattern of barrettes can be arranged in a continuous row or column, radial, alternating long and short axis of barrette and a combination of two or more of such patterns. To minimize the need of additional equipment, a mechanical grab mounted on crawler crane can be used for constructing both barrettes (as piled-foundation) and diaphragm walls (as earth retaining structure) in some suitable projects without employing a bored piling rig, which could save the extra mobilization cost. In addition to the large bearing capacity requirements, on site difficulties such as limited head room where piling rigs cannot be utilized, under such situations like presence of overhead high voltage power cables, existing overpasses or structures for elevated expressways and planned subway stations, also demands the barrettes. Static load test up to 52,900 KN conducted on barrette set the record as the highest load ever tested for a single barrette foundation in Thailand (Thasnanipan et al. 1994).
A summary of completed 35 projects constructed with barrettes with respect to selection criteria is presented in Table 3.

PROBLEMS AND DIFFICULTIES IN CONSTRUCTION OF WET-PROCESSED BORED PILES AND BARRETTEs IN EARLY DAYS

Basic but extensive problems were experienced in early stages of bored pile construction in Thailand as summarized below.

- Limited availability and capacity of equipment
- Lack of skills in operation of equipment (particularly the drilling rig)
- Adverse effect due to a slow rate of drilling (e.g. excessive formation of filter-cake by using bentonite slurry)
- Limited knowledge and less advanced techniques in control of bentonite slurry
- Limited experience in construction method and related negative impact
- Quality of concrete for tremie concreting method
- Lack of experienced engineers and foremen
- Improper construction and quality control specifications and guidelines for deep-seated piles in local soil
- Improper design for constructability

IMPROVEMENT IN CONSTRUCTION

Over the past three decades, along with the development of wet-processed piling technology in other parts of the world, equipment, construction technique and design methods as well as better understanding of construction impact on the performance of this type of foundation have significantly improved in Thailand. Table 4 summarizes the areas of improvement in bored pile and barrette construction and main factors contributed to these developments. Some of these improvements were published both locally and internationally. The design, construction and behavior of bored cast in-situ concrete piles in Bangkok Subsoil was presented by Thasnanipan et. al., (1998a). The construction and performance of barrettes in Bangkok Subsoil was also reported by Thasnanipan et. al., (1998b). Effect of construction time and bentonite viscosity on shaft friction of bored piles was also pointed out by Thasnanipan et al., (1998c). Thasnanipan et. al., (2004a) also reported a comprehensive study on the effectiveness of two different toe-grouting methods, known as tube-à-manchette and drill-and-grout, applied in Bangkok.

Fig. 2 (a) Guide wall for cruciform barrette
(b) View of cruciform barrette after installation of reinforcement
Deep-seated T-shape and L-shape barrettes were constructed as leg-piles for diaphragm wall in recently completed projects in Bangkok. Construction of cruciform barrette (shown in Figure 2) for monopole type high-voltage power transmission line was described in the work of Thasnanipan et al., (2000a).

Above-mentioned research works reflect the development history of bored pile and barrettes in Thailand and offered useful information to the local construction industry and perhaps to the international deep foundation engineering society.

Table 4 Summary of Development in Bored Pile and Barrette Construction

<table>
<thead>
<tr>
<th>Area of development / improvement</th>
<th>In the past (Early 1980’s)</th>
<th>At present</th>
<th>Main factor contributed to development</th>
</tr>
</thead>
<tbody>
<tr>
<td>Speed of construction</td>
<td>Minimum 3 days required to complete diameter 1.5m tip 50m bored pile</td>
<td>Less than 1 day to complete diameter 1.5m tip 50m bored pile</td>
<td>Better equipment, operating skills as well as improved knowledge in construction method, management and local soil condition</td>
</tr>
<tr>
<td>Pile size and depth</td>
<td>Bored pile Maximum diameter 2.0m and common depth 25-50m for bored pile. Barrette Limited in size and depth</td>
<td>Bored Pile Maximum Diameter 2.0m and common depth 25-60m Barrette Various sizes &amp; depth over 60m</td>
<td>Better equipment, operating skills as well as improved knowledge in construction method, management and local soil condition</td>
</tr>
<tr>
<td>Base grouting</td>
<td>Not available</td>
<td>Available</td>
<td>Equipment availability and advance technology</td>
</tr>
<tr>
<td>Application of polymer-based slurry (for bored pile only)</td>
<td>Not available</td>
<td>Extensive use</td>
<td>Material availability and research</td>
</tr>
<tr>
<td>Construction impact on quality and performance</td>
<td>Not well understood</td>
<td>Improving</td>
<td>Experience from past projects and research</td>
</tr>
<tr>
<td>Quality control in construction process</td>
<td>Not well established and systematic</td>
<td>Well established and systematic</td>
<td>Experience from past projects and research</td>
</tr>
<tr>
<td>Quality control test method and interpretation</td>
<td>Fewer methods available and limited knowledge in interpretation</td>
<td>Better equipment available and better knowledge in interpretation</td>
<td>Advance equipment, experience from past projects and research</td>
</tr>
</tbody>
</table>

Experience from past projects and extensive research works provided better understanding of construction impact of quality and performance of these wet-processed deep foundations. Marked difference between outcome quality of bored piles constructed in 1980s
and 1990s can be observed by significant fewer defects found in the latter. It can also be observed from load test results that bored piles constructed in late 1990 have higher capacities than those of 1980s as shown in Table 5.

Table 5  Load Test Results of Bored Piles Constructed in 1970 to 1980 and 1990s

<table>
<thead>
<tr>
<th>Year of construction</th>
<th>Project Name</th>
<th>Pile Dimension (Dia. &amp; Depth)</th>
<th>Design Load (KN)</th>
<th>Test Load (KN)</th>
<th>Total settlement at max. test load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1971</td>
<td>Pinklao bridge</td>
<td>1.5m x 45m</td>
<td>4100</td>
<td>8200</td>
<td>5</td>
</tr>
<tr>
<td>1995</td>
<td>Central Plaza Pinklao</td>
<td>1.2m x 45m</td>
<td>7000</td>
<td>14000</td>
<td>27</td>
</tr>
<tr>
<td>1980</td>
<td>Taiping Tower</td>
<td>1.0m x 32m</td>
<td>3000</td>
<td>11250</td>
<td>118</td>
</tr>
<tr>
<td>1990</td>
<td>High-rise building at Ekamai Rd.</td>
<td>1.0m x 32m</td>
<td>3900</td>
<td>9750</td>
<td>23</td>
</tr>
</tbody>
</table>

Bored piles have been extensively used for foundations of the majority of the elevated expressways since 1991. Thousands of base-grouted bored piles were constructed for these infrastructure projects including Second Stage Expressway, constructed in 1991; Don Muang Tollway Extension, constructed in 1997; Bangna-Bang Pli-Bangprakong Expressway constructed in 1998; and Wat Nakorn-In project, constructed in 2001. Over 700 bored piles of diameter ranging from 0.8m to 1.5m with depth from 35m to 54m were constructed between 1999 and 2000 using polymer-based slurry for the foundation of Rama VIII Bridge, one of the initiatives of his majesty the King Bhumibol Adulyadej. Progress of piling and superstructure works of the Rama VIII Bridge at the bank of Chao Phraya river is depicted in Figure 3.

Fig. 3  Rama VIII Cable-Stayed Bridge and Connecting Elevated Flyovers – More Than 700 Deep-Seated Bored Piles were Constructed Using Polymer-Based Slurry

Fig. 4  Rock-Socket Bored Pile Construction for Mool River Project Ubon Ratchathani Province
The availability of more reliable and powerful equipment and tools for drilling makes it possible to construct the rock-socket bored piles by rotary drilling method. Figure 4 shows the drilling rig equipped with a rock auger used for construction of highway bridge across Mool River in 2001, in the north-eastern part of Thailand. Drilling was carried out through weathered-sandstone by powerful rotary drilling rig with core barrel and rock auger.

Bored pile of diameter 2m founded at 62m depth for the foundation of Rama 8 bridge, a cable-stayed bridge across Chaopahaya river constructed in early 2005 was the largest deep-seated bored pile in Bangkok installed by rotary drilling method. In the same project, both conventional static pile load tests and bi-directional load tests were carried out on diameter 2m bored piles. Simplified version of load vs. settlement of these tests are presented in Figure 5. Apparently, from the test results, the test piles could carry a significantly higher load than the original pre-defined safe design load of 1,600 ton. Thasnanipan et. al. (2006) reported the experiences gained in practical construction of these largest diameter bored piles at the river bank. Pictures of some activities during construction of bored piles in the said project are presented in Figure 6, 7 and 8.

Fig. 5 Load-Settlement Relationship of Diameter 2.0m Bored Piles (Conventional Static Pile Load Test and Bi-Directional Load Test)

Fig. 6 View of Progress in Temporary Casing Installation and Drilling for Bored Pile Diameter 2 m at the Bank of Chaophaya River
Fig. 7 View of Progress in Rebar Cage Preparation for Bi-Directional Load Test and Installation of Rebar Cage

Fig. 8 View of Conventional Static Pile Load Test and Bi-Directional Load Test

The availability of more powerful and efficient hydraulic grabs offers faster construction of deep-seated barrettes in comparison with mechanical cable-hung grabs. Figure 9 shows the construction of barrettes using hydraulic grab.

Fig. 9 View of Barrette Construction by Hydraulic Grab
IMPROVEMENT IN DESIGN CONCEPT AND PARAMETERS SELECTION

In the initial stage of introducing bored piles in Thailand, the design concepts and parameters were mainly based on the available literature from research carried out in other parts of the world such as Tomlinson (1957), Skempton (1959), Broms (1966), Bowles (1968), Meyerhof (1976) etc. The work of Chiruppapa (1968) was believed to be the first research data available for design parameters of bored piles in Bangkok soft clay. The author conducted the study based on 6 dry-processed small diameter bored piles with load cells installed at the pile toes on AIT campus in Phathumthani. Though the piles were simply constructed using the dry-process with casing method, this early-stage study provided some important information such as load-settlement behavior, adhesion factor (\(\alpha\)) and bearing capacity factor (N) of bored piles in Bangkok soft clay.

With the passage of time, design method and selection of parameters for local subsoil were improved as a result of research works carried out in 1980s. Differences between the behavior of driven piles and bored piles were well realized from these studies. Ng (1983) presented the load distribution characteristics of wet-processed bored piles founded in both the first and second sand layers of Bangkok based on instrumented (strain gauges) pile load test results. Chiewcharnsilp (1988) reported the shaft friction factor \(\beta\) values of sand layers in Bangkok based on the instrumented load test results. In addition to the literature previously available (Chiruppapa, 1968; Suwanakul, 1969; Promboon, 1981; Ng, 1983; Chiewcharnsilp, 1988), Pimpasugdi (1989) determined the shaft friction factor (adhesion factor, \(\alpha\)) of Bangkok clay layers based on 11 bored piles of diameters ranging from 0.50 to 0.80m with the embedded length varying between 21.50m and 46m. It should be noted that the design parameters obtained from the research works of 1980s were mainly based on the estimation of shaft friction loads from plain static pile load test results with numbers of assumptions since instrumented pile load test results were limited.

With the peak of construction-boom, large numbers of instrumented full-scale static load tests on bored piles were conducted throughout 1990's which provided better understanding on behavior of these deep-seated foundations. Researches focused on the design parameters and methods were published based on these test data. With improved design methods the wet-process cast-in-place foundations came to be regarded as reliable foundations in the construction industry of Thailand. With more confidence on soil parameters selection and better understanding on behavior of these deep foundations, the designer designed higher load capacity bored piles and barrettes in late 1990 than those in 1980s. Thasnanipan et. al., (1999) reported the failure mechanism of long bored piles in layered soil of Bangkok. The authors cited that for the bored piles embedded in the multi layered soils of Bangkok, estimation of ultimate shaft friction capacity needs to consider the brittle type of failure mechanism of stiff to hard clay layers and the \(\alpha\) values selected need to be adjusted accordingly. Peak and residual \(\alpha\) values mobilized in the stiff clay layers analyzed by the authors were plotted as shown in Figure 10 along with the suggested curves by different researchers. It can be seen from the figure that the residual \(\alpha\) value of 2nd stiff clay layer (undrained shear strength values of 25 ton/m2), at the maximum test load, drops below the curve suggested by Pimpasugdi (1989). So the \(\alpha\) values proposed by the author in Figure 10 for the stiff to hard clay layers overestimate the ultimate shaft friction under these conditions.
Fig. 10 Comparison of Adhesion Factor $\alpha$, Suggested by Different Researchers with the Actual Mobilized in the Stiff Clay Layers (Thansnanipan et al. 1999)

Fig. 11 Back-Calculated $\beta$ Values of Polymer Bored Piles at Maximum Test Load Plotted on Design Line of Bentonite Bored Piles Constructed in Bangkok Subsoil (Thansnanipan et al. 2002b)

Introduction of polymer-based slurry for wet-process bored piles marked a major breakthrough for both construction and design engineers. Thansnanipan et al. (2002b), reported that bored piles constructed with polymer-based slurry have higher capacity than those
constructed with bentonite slurry. Figure 11 shows the shaft friction factors β of sand layers for polymer-based bored piles in comparison with the design line of bentonite bored piles. The higher load capacity of bored piles constructed with polymer-based slurry allows the use of a single, large diameter deep-seated bored pile in place of a group of smaller size shallow-seated bored piles or driven piles. Figure 12 depicted the static pile load test result of 1.80m diameter bored pile of 60m depth constructed with polymer-based slurry in urban area of Bangkok. The test was conducted in late 2003. The maximum applied load 48,000 KN in this test is believed to be the highest static pile load test performed on a single bored pile in Thailand. Apparently, from the test result, the pile (diameter 1.80m x 60m deep) could carry a significantly higher load than the original pre-defined safe design load of 16,000 KN.

![Graph showing Applied Load (KN) vs. Pile head displacement (mm)]

Fig. 12 Static Pile Load Test Result of 1.80m Diameter Bored Pile of Depth 60m in Bangkok

DEVELOPMENT IN PERFORMANCE MONITORING AND QUALITY CONTROL TESTING

Improvement of testing equipment and powerful computer facilities are key factors that contributed to the development in performance monitoring and quality control testing. Interpretation skills relevant to local soil conditions and construction methods of these tests were significantly improved in local industry. For instance, in early 1980, the sonic integrity (seismic test) test results were needed to send to the specialists abroad for interpretation which in turn made the testing cost more expensive. Significant cost-saving were achieved in some major projects of late 1990, as more practical and precise interpretation were locally made to verify and establish acceptance criteria in proving the quality of suspected piles with anomalies.

Koden Drilling Monitoring System

In Thailand, prior to the availability of Koden testing equipment, borehole verticality was checked by mechanical type equipment such as plumb float and adjustable globe. However, the reliability of these methods was doubtful. The Koden drilling monitor system was believed to be first used in Thailand in 1980. The verticality of slurry-filled borehole can be monitored rapidly from the electronic plot of the continuous profile by Koden equipment.
This system is very useful for the verticality-control of long barrettes - unlike bored piles, guiding by temporary casing is not available for barrette drilling.

**Sonic Integrity Test**

Sonic integrity test, also known as seismic test is the most common method of integrity testing for both driven and bored piles in Thailand. Sonic integrity test is usually selected for both quality check (control test) and retrospective investigation. It is the cheapest in terms of cost and the simplest in terms of testing process. The main advantage of this test is that since no particular preparation is necessary during the pile construction, phase, it is more flexible to select which pile is to be tested. However, interpretation of sonic integrity testing needs considerable experience and knowledge in testing, subsoil condition and construction method. In many projects it is a part of the contractual requirement to conduct sonic integrity test. Minimum 10% to maximum 100% of production piles are commonly tested. It is also a reasonably acceptable method as a retrospective investigation in determining integrity of the pile. The signal characteristics and their interpretations of sonic integrity test on piles founded in Bangkok subsoil were reported in details by Thasnapan et al. (1998d).

**Cross-Hole Sonic Logging Test**

The first sonic logging test was believed to be conducted in 1982 for the wet-processed bored piles of the Memorial Bridge Project where bad concrete zones were detected at depth about 20m and 1m Ng, 1983. Use of the Sonic logging test for checking pile integrity has increased in Thailand particularly in mega projects. Cross-hole sonic logging test is relatively expensive. It is mainly employed as a pre-planned site quality control testing. The major advantage of this method is that test can be carried out shortly after pile construction. Hence, rectification measures can be implemented if the pile is defective while the foundation contractor is on site. However, this method is generally not applicable if pile integrity is in question due to post-construction activities, as access tubes are usually grouted after completion of the test. The results from sonic logging test conducted on model piles in Bangkok helped to extend the knowledge of the signal characteristics and interpretation (Thasnapan et al., 2000b). Thasnapan et al., (2004b) also demonstrated the application of cross-hole sonic logging test in identifying over-cast length of bored piles prior to exposing the pile top.

**High Strain Dynamic Load Test**

High strain dynamic integrity test has become a well-accepted method especially for evaluating the pile capacity in today's foundation industry of Thailand and it is applied for both driven and bored piles. A large number of related technical papers and case histories of the test have been published and it is a part of standards and specifications such as ASTM D4945-89 (Standard Method for High-strain Dynamic Testing of Piles). Thasnapan et al. (2000b), reported the application of dynamic load testing on piles in Thailand.

**BORED PILES AND BARRETTES IN THE SECOND BANGKOK INTERNATIONAL AIRPORT**

Large quantities of bored piles were constructed for foundation of Passenger Terminal Complex and associated structures of the SBIA Project. Barrettes were mainly used for Underground Train Station. Conventional static pile load tests and bi-directional load tests
carried out on bored piles were studied by Sakaret (2004). It is expected that more research works are published in Symposium on Geotechnical Aspects of Second Bangkok International Airport and other publications so that source of references will be available for the future projects of similar nature.

STATIC PILE LOAD TEST ON PILOT BORED PILE AT THE SBIA PROJECT

Figure 13 shows the load vs. pile head movement of pilot bored pile tested at the Second Bangkok International (Suvarnabhumi) Airport Project during the design phase prior to the award of the piling contract. Summary of load tests are summarized in Table 6. The pilot pile at SBIA site and Test Pile No. 2 were failed before reaching to 2 times of proposed design loads, at 14,200 kN and 12,000 kN respectively.

![Graph showing load vs. pile head movement](image)

Fig.13 Load vs. Pile Head Movement of the SBIA Pilot Test Pile and Test Piles Conducted in Other Parts of Bangkok

Table 6 Summary of Load Test Data of SBIA Pilot Pile Test and Same Pile Tip Level of Different Pile Size in Other Area of Bangkok

<table>
<thead>
<tr>
<th>Description</th>
<th>SBIA Pilot Pile</th>
<th>Test Pile No. 1</th>
<th>Test Pile No. 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter (m)</td>
<td>1.00</td>
<td>0.80</td>
<td>1.20</td>
</tr>
<tr>
<td>Tip Level (m)</td>
<td>41</td>
<td>41</td>
<td>41</td>
</tr>
<tr>
<td>Slurry Used</td>
<td>Polymer-based</td>
<td>Polymer-based</td>
<td>Bentonite</td>
</tr>
<tr>
<td>Proposed Design Load $Q_{pd}$ (KN)</td>
<td>7,100</td>
<td>3,300</td>
<td>6,000</td>
</tr>
<tr>
<td>Calculated Design Load (kN), $Q_{pd}$ with FoS=2</td>
<td>4,100</td>
<td>3,800</td>
<td>6,000</td>
</tr>
<tr>
<td>Load at pile head movement 10% of pile diameter (kN)</td>
<td>12,420</td>
<td>N.A</td>
<td>15,000</td>
</tr>
<tr>
<td>Load (kN) at 15mm total pile head movement</td>
<td>8,600</td>
<td>9,900</td>
<td>10,000</td>
</tr>
<tr>
<td>Total pile head movement (mm) at $Q_{pd}$</td>
<td>8</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>at 1.5 $Q_{pd}$</td>
<td>41</td>
<td>5</td>
<td>6</td>
</tr>
<tr>
<td>at 2 $Q_{pd}$</td>
<td>NA</td>
<td>8</td>
<td>50</td>
</tr>
</tbody>
</table>

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FUTURE TRENDS OF CAST-IN-PLACE DEEP FOUNDATIONS IN THAILAND

As the practitioners in construction industry have gained more experience and confidence in using higher capacity bored piles and barrettes, these types of deep foundations are expected to be of more demanding in the future. The higher load capacity achievable by deep-seated large bored piles and barrettes will allow the use of a single foundation element in place of a group of smaller size shallow-seated bored piles and driven piles. It is expected that improvement in installation techniques will be of key advance in the future of deep foundation industry in Thailand.

It is anticipated that barrettes will be more popular in the future for the following reasons.

- The availability of more powerful and efficient hydraulic grabs which will offer faster construction of deep-seated barrettes
- Less noise which will offer main advantages in congestive and sensitive neighbourhood
- Less equipment requirement which will offer advantages construction in limited space
- More versatility (e.g. able to construct in the area with limited headroom)
- Better in safety aspects as less equipment are required (especially for the construction along the public roads, subway stations, elevated expressway etc.)

CONCLUDING REMARKS

In Thailand, according to the authors’ experience as a deep-foundation contractor, development in both construction and design aspects of wet-process deep bored piles and barrette foundations in past decades were significant. With recognition of technical and economic advantages of using these high capacity cast-in-place foundations by local practitioners, it is expected that they will be more popular in the future construction industry of Thailand. However, in the authors’ opinion, there is much work to be done with particular focus on constructability issues, concrete technology for wet-processed bored piles and barrette, reliable but cost-effective quality control testing and value-engineering.

Starting from the planning stage, site investigation, design, construction and inspection should be integrated so that designers, contractors and construction inspectors can participate as a team with a common goal. Appropriate and practical specifications should be established jointly by these parties for local soil conditions and construction methods. Practical acceptance criteria should be developed to verify bored piles and barrettes with suspected anomalies. Continuing education should be promoted for designers, inspection engineers, and contractors. The Geotechnical Chapter of the Engineering Institute of Thailand under the royal patronage of his majesty King Bhumibol Adulyadej, has started to establish the standard code of practice and guidelines for wet-processed bored piles which will serve as a yardstick for the deep foundation industry upon its completion in the near future.
REFERENCES


BORED PILES AND BI-DIRECTIONAL LOAD TESTS

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Director, Samart Project Consultants Co Ltd
[formerly Resident Project Manager with TAMS Consultants]

ABSTRACT

The Passenger Terminal Complex of the Second Bangkok International Airport is founded on over 25,000 piles, of which approximately 10,000 are bored piles. The design, using effective stress methods and the concept of a Neutral Plane, is described, including special features to deal with rising groundwater. In addition the analyses carried out to predict and control differential settlement are discussed. A total of 22 pile tests have been carried out, including 15 on bored piles. Of these two were tested using a bi-directional load test for the first time in Thailand. The pile tests are described, and the results compared with the design.

INTRODUCTION

The Passenger Terminal Complex (PTC) of the Second Bangkok International Airport is a unique structure, illustrated in Figure 1. It was designed between 1995 and 2000 by Architects Murphy-Jahn, with TAMS as the engineering managers. Martin/Martin of Colorado provided

![Fig. 1 Plan of Passenger Terminal Concourse showing Main Terminal Building, Concourses, Elevated Frontage Roads, Chiller Buildings and Short-term Car Parks](image-url)
specialist design services for the steel roof structure over the Main Terminal Building (MTB) while Werner Sobek Ingenieure (WSI) of Stuttgart provided specialist design services for the Concourses and ACT Consultants of Bangkok designed the reinforced concrete structures. The total area of the PTC is 563,000 m², making it the largest passenger terminal building in the world at that time. The design contract also included 8,600 m of elevated roads to give access to the MTB, two Chiller Plant Buildings and two short term car parks each housing 2,500 cars.

The main roof over the MTB, seen in the center of Figure 1, covers an area of 560 x 210 m, and is about 30 m above ground level (the two outer spans shown in this early model were deleted before construction). It is made of steel and glass, basically a flat roof, supported on secondary trusses spanning 81 m between primary trusses. These primary trusses have become known as "Supertrusses", simply because they are each 210 m long, with a clear span of 126 m and weighing approximately 1,500 tonnes. To support them at a height of 30 m they are fixed to "Superpylons", each made of four legs fabricated from thick steel plate in square section.

The foundations for the PTC were designed by TAMS in their New York office, and comprised over 25,000 piles. Many of the foundations were conventional, with groups of 4 to 8 piles per pile cap supporting the reinforced concrete structure up to four storeys high with a single basement. However the Elevated Frontage Roads had larger loads, with their two levels of elevated structure, and required up to 42 piles for each pile cap. The Superpylons were also exceptional, with downloads of up to 15,000 tonnes. These required large diameter bored piles in groups of up to 25, some of which had an additional 10 piles to help support Elevated Frontage Road as well.

SOIL CONDITIONS

The soil conditions in and around Bangkok are quite well known and have been referred to in numerous technical publications over the last three decades. Most commonly they are characterized by a crust of weathered clay; a layer of soft Marine Clay up to about 12 m thick, which is followed by a firmer clay, probably resulting from drawdown of the water table; a First Stiff Clay layer; a First Sand layer; a Second Stiff Clay; and finally a Second Sand layer. This profile extends down to about 60 to 80 m, which is the practical limit of most soil boreholes around Bangkok. Only in a few exceptional cases has rock been proven, at depths of 500 to 100 m.

There are exceptions to this general pattern, such as in Wireless Road where the First Sand Layer appears to be absent. Another notable exception is at the PTC site, where the Second Stiff Clay appears to be absent and the First Sand layer seems to join straight into the Second Sand layer. This has a significant effect on pile design, especially with regard to ground water level.

As has been described in other papers to this Symposium, the soil conditions around the 32 km² site of the Second Bangkok International Airport have been investigated over a number of years in various investigations for different design purposes. As a result 19 boreholes were available to the PTC foundation designers at the design stage. Design parameters were therefore based on laboratory test results combined with local experience, particularly of the concrete structural designers ACT Consultants. A further 20 boreholes and
7 PCPT tests were carried out during the foundation construction stage to better define the soil profile and investigate anomalies in the 1 km² of the PTC foundations.

PILE DESIGN - CAPACITY

Most piles in Bangkok are designed using undrained cohesion and adhesion factors, often referred to as the alpha-method, because adhesion is often defined as $\alpha$. Pile lengths are typically 50 to 60 m for large diameter bored piles. The PTC foundation designers decided that the most appropriate method would be the beta-method, based on the coefficient used to define the skin friction in effective stress terms, $\beta$. They were also aware that the deep pumping of aquifers under Bangkok, to depths of 100 to 200 m and producing drawdowns by the mid-1990s of up to 60 m, was causing ongoing subsidence with consequent effects on Negative Skin Friction (NSF). They therefore adopted the concept of a Neutral Plane, proposed by Fellenius (1996). In this concept a pile under load first moves down relative to the soil, generating supporting upward forces in positive skin friction (see Figure 2). Then the upper soil moves down relative to the pile, generating NSF which increases the load on the pile. The Neutral Plane is defined as the horizontal plane at the depth at which the Dead Loads and NSF forces acting down on the pile are just balanced by the positive skin friction forces acting upwards. At this plane there is no relative pile/soil movement.

The structural requirements of the pile are therefore defined at two locations: (i) by the forces acting at the head, which include Dead Loads and Live Loads including Permanent Live Loads, and will include any bending forces, and (ii) by the forces at the Neutral Plane, where Dead Load and NSF apply, without bending, and probably involve the maximum axial loads. The geotechnical capacity of the pile based on full soil strength was related by the Factor of Safety to the Dead Load, Live Load and Permanent Live Load, without considering NSF, since NSF is extremely unlikely to cause a pile to fail, and is therefore unlikely to be applied in a failure mode where large vertical movements are expected.

![Fig. 2 Illustration of Neutral Plane](image-url)
Table 1 Pile cost comparison

<table>
<thead>
<tr>
<th>Type of pile</th>
<th>Estimated construction cost (Bt/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bored pile, compression, 1000 mm</td>
<td>3,950</td>
</tr>
<tr>
<td>Bored pile, compression, 600 mm</td>
<td>1,280</td>
</tr>
<tr>
<td>Bored pile, tension, 600 mm</td>
<td>1,400</td>
</tr>
<tr>
<td>Driven spun pile, compression, 600 mm</td>
<td>800</td>
</tr>
<tr>
<td>Driven spun pile, tension, 600 mm</td>
<td>1,400</td>
</tr>
</tbody>
</table>

In order to assist in selecting the most appropriate pile type for general use, a cost analysis was carried out based on local information. In this analysis the cost per metre of installed pile, was determined for bored piles of 1000 mm and 600 mm diameter (the latter in both compression and tension), and for driven precast spun concrete piles, also in both compression and tension. In analyzing costs, especially for tension, the extra difficulty of splicing together precast concrete piles, the extra excavation for pile caps and the material lost in pile cut-offs to make tension connections were also taken into account. The results are shown in Table 1 and, based on this, the 600 mm diameter spun concrete pile was seen to be the most economical.

The pile of choice has therefore been the 600 mm spun concrete pile, except for:

- 1000 mm bored piles for Superpylons; here the pile cap size for 600 mm diameter piles would have been unmanageable, and the large groups lost efficiency
- 600 mm bored piles for two rows of MTB pile caps around Superpylons; these were chosen to minimize soil disturbance effects on the major foundations and to minimize differential settlement
- 600 mm bored under service tunnels; in this location tension loads to resist flotation were required

The design of the piles has considered two distinct design cases, related to groundwater levels. The first is the current groundwater condition, with a hydrostatic profile from near the surface, which then drops to zero at about 20 m depth. Below this the profile is again approximately hydrostatic. This is illustrated in Figure 3 which also shows the hydrostatic pressure distribution which would apply if pumping stops and ground water rises to ground level, together with the approximate effective stress distributions both under current condition and future conditions.

In fact there have already been significant changes in the groundwater regime around the airport. In 1995 and 1996, while carrying out the design of the ground improvement for the runways, taxiways and aprons, enquiries were made through the Department of Mineral Resources (DMR) and the Royal Thai Army Survey Department (RTASD). The DMR have an ongoing project funded by JICA in which the groundwater level in a number of deep aquifers is recorded monthly at a large number of locations around Bangkok. The RTASD has also been monitoring surface settlement by leveling a series of survey markers over the same area. In the period up to 1997, the nearest groundwater monitoring station to the SBIA site was at Wat King Kaew on King Kaew Road. Here the groundwater levels in the Phrapradaeng, Nakornluang and Nonthaburi aquifers at depths of about 100, 150 and 200 m below ground level were being measured, see Figure 4, and the trend at that time was clearly progressing steadily downward, despite Government attempts to reduce the pumping.
Fig. 3  Pore Pressure and Effective Stress Distributions

Fig. 4  Groundwater Levels to 1997 in Phrapradaeng, Nakornluang and Nonthaburi Aquifers as Measured at Wat King Kaew
However, during 2002 as part of the Piling Works contract, the data was updated and has been updated each year since. The original site at Wat King Kaew has been lost, since the road was widened to a dual carriageway in about 1997. However data has been obtained for Wat Khuwararam on the east side of the SBIA, and at Hua Mark Golf Club which is not too far away to the north. The results are quite surprising, as seen in Figures 5 and 6.

Fig. 5  Groundwater Levels to 2006 in Phrapradaeng, Nakornluang and Nonthaburi Aquifers as Measured at Wat Khuwararam

Fig. 6  Groundwater Levels to 2006 in Phrapradaeng, Nakornluang and Nonthaburi Aquifers as Measured at Hua Mark Golf Club
Although there are obvious anomalies in the data, which suggest coupling between certain aquifers, probably at the monitoring location, the overall trend is clearly defined and identified by many data points. Since about 1996 there has been a steady recovery of groundwater in the three aquifers, and in the lower two the increase in groundwater level has been over 25 m. Based on the rates of recovery, estimates were made of the time for the groundwater levels to reach ground level for each aquifer, based on the 2005 and 2006 data. The results are tabulated below.

Table 2 Recovery of Groundwater Levels

<table>
<thead>
<tr>
<th>Station</th>
<th>Aquifer</th>
<th>Year to reach ground level</th>
</tr>
</thead>
<tbody>
<tr>
<td>2005</td>
<td>Hua Mark Golf Club</td>
<td>Nonthaburi and Nakornluang</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Phrapadaeng</td>
</tr>
<tr>
<td></td>
<td>Wat Khuararam</td>
<td>Phrapadaeng</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Nonthaburi</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Nakornluang</td>
</tr>
<tr>
<td>2006</td>
<td>Hua Mark Golf Club</td>
<td>Nonthaburi and Nakornluang</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Phrapadaeng</td>
</tr>
<tr>
<td></td>
<td>Wat Khuararam</td>
<td>Phrapadaeng</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Nonthaburi</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Nakornluang</td>
</tr>
</tbody>
</table>

This data shows two things. Firstly that there is a consistent rise in groundwater levels, with the potential to reach the ground surface in 15 to 20 years; and secondly that the rate is, if anything, increasing. This is probably the result of a greatly increased availability of piped water in the area, and justifies the use of the two effective stress conditions in the design, considering both the current situation and the possible future scenario.

Table 3 Pile capacities and working loads

<table>
<thead>
<tr>
<th>Pile type and diameter</th>
<th>Ultimate capacity (tonnes)</th>
<th>Allowable load (tonnes)</th>
<th>NSF (tonnes)</th>
<th>Maximum structural load (tonnes)</th>
<th>Depth to maximum structural load (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Effective stress condition (1); current condition with pumping</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bored 1000 mm; tip at -42m</td>
<td>1,646</td>
<td>713</td>
<td>466</td>
<td>1,179</td>
<td>32.5</td>
</tr>
<tr>
<td>Bored 600 mm; tip at -30m</td>
<td>287</td>
<td>140</td>
<td>74</td>
<td>214</td>
<td>19.7</td>
</tr>
<tr>
<td>Driven 600 mm; tip at -26m</td>
<td>425</td>
<td>176</td>
<td>125</td>
<td>301</td>
<td>22.6</td>
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<tr>
<td><strong>Effective stress condition (2); hydrostatic piezometric pressures</strong></td>
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<td></td>
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<tr>
<td>Bored 1000 mm; tip at -42m</td>
<td>1,068</td>
<td>466</td>
<td>301</td>
<td>767</td>
<td>28.8</td>
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<tr>
<td>Bored 600 mm; tip at -30m</td>
<td>216</td>
<td>104</td>
<td>56</td>
<td>160</td>
<td>18.3</td>
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<td>Driven 600 mm; tip at -26m</td>
<td>278</td>
<td>121</td>
<td>78</td>
<td>199</td>
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</table>

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When the two conditions were considered for each pile size, and adjustments made to the pile length using the Neutral Plane concept, the following relationships between capacity and allowable load were determined. It is seen that the limiting allowable loads relate to the long-term condition, while the limiting structural loads, at the Neutral Plane, are in the short-term.

PILE DESIGN - SETTLEMENT

With such large and highly loaded structures, design to limit settlement was even more critical than usual. In addition the structural form, in which the roof was free-standing on the Superpylons over the whole area; the Main Terminal Building was a reinforced concrete framed structure sitting under the roof; and the enclosure was a full height glass curtain wall unconnected to the concrete frame, was particularly sensitive to differential settlement. The limit was set at 1:300, which equated to 30 mm for each 9 m span.

Settlements were analysed by considering the soil below the Neutral Plane carrying the permanent loads. A closed form solution to Mindlin’s problem No. 1 (Poulos and Davies 1974), which determines the stress field caused by a point load inside an elastic half space, was programmed. The more commonly used Boussinesq solution determines the stress field for a point load on the surface of an elastic half space. As each pile was analysed the results were superimposed to examine the stresses within a pile group, and then group interaction was further determined by superposition again. Finally the $m_v$ value was applied to determine the settlements from the stresses.

Three key areas were identified for the detailed examination of settlement. These were:

1. Around the Superpylons
2. The MTB away from the Superpylons
3. The Concourses away from the Superpylons

Because of their high loads, up to 15,000 tonnes, the Superpylons have the largest settlement and differential settlement. At the same time each Superpylon pile cap has 25 to 35 bored piles, each 1000 mm in diameter and to a depth of –41 m. The adjacent MTB foundations typically had 4 to 8 piles per cap, and each pile was 600 mm diameter and to a depth of –30 m. The stressed zones around each were therefore quite different. The table below shows the maximum expected differential settlement of MTB foundations near some of the Superpylons.

<table>
<thead>
<tr>
<th>Location of Superpylon</th>
<th>B-20</th>
<th>B-29</th>
<th>B-38</th>
<th>R-20</th>
<th>R-29</th>
<th>R-38</th>
</tr>
</thead>
<tbody>
<tr>
<td>Settlement (mm)</td>
<td>15</td>
<td>30</td>
<td>30</td>
<td>20</td>
<td>15</td>
<td>20</td>
</tr>
</tbody>
</table>

By comparison the typical design differential settlements throughout the MTB were 10 mm, while for the Concourse the typical differential settlement was 15 mm. Another factor was the construction sequence. It was planned that the roof would be erected early in the schedule, and this allowed a significant part of the immediate settlement, in the sands, to take place during construction and be built out. This lead to the following expected total settlements.
Table 5 Settlement of Main Structures

<table>
<thead>
<tr>
<th>Location</th>
<th>Settlement (mm)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Construction</td>
<td>Post-construction</td>
<td>Total</td>
</tr>
<tr>
<td>Superpylons</td>
<td>20</td>
<td>90</td>
<td>110</td>
</tr>
<tr>
<td>MTB away from Superpylons</td>
<td>29</td>
<td>16</td>
<td>45</td>
</tr>
<tr>
<td>Concourse away from Superpylons</td>
<td>36</td>
<td>19</td>
<td>55</td>
</tr>
</tbody>
</table>

Table 6 Schedule of Load Tests

<table>
<thead>
<tr>
<th>Load Test Type</th>
<th>Load transfer measurements</th>
<th>Number of tests</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1000 mm diameter bored piles</td>
<td></td>
</tr>
<tr>
<td>Compression</td>
<td>Extensometers only</td>
<td>2</td>
</tr>
<tr>
<td>Compression</td>
<td>Strain gauges</td>
<td>1</td>
</tr>
<tr>
<td>Bi-directional</td>
<td>Strain gauges</td>
<td>2</td>
</tr>
<tr>
<td>Lateral</td>
<td>Inclinometer</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>600 mm diameter bored piles</td>
<td></td>
</tr>
<tr>
<td>Compression</td>
<td>Extensometers only</td>
<td>3</td>
</tr>
<tr>
<td>Compression</td>
<td>Strain gauges</td>
<td>4</td>
</tr>
<tr>
<td>Lateral</td>
<td>Inclinometer</td>
<td>1</td>
</tr>
<tr>
<td>Uplift</td>
<td>Extensometers only</td>
<td>1</td>
</tr>
</tbody>
</table>

PILE TESTING

In order to verify the design, an extensive programme of testing was planned, both before main production piling began and also during the construction process. A total of 19 test piles were installed, and 15 tests were carried out on bored piles, since some were subjected to both vertical compression and lateral load tests. Most of the vertical compression tests were carried out in the conventional manner, using hydraulic jacks reacting against beams held down by tension piles. Two special tests were carried out using a bi-directional loading system, similar in concept to the Osterberg Cell. The detail of the tests is shown in Table 6.

Vibrating Wire Strain Gauges (VWSGs) were incorporated in 5 of the conventional pile tests and both of the bi-directional tests. Their use has become quite common in bored pile testing since the early 1970s, and they allow estimates to be made of the load distribution down the pile, from which the load transfer to the soil can be determined. There are almost always some problems with the technique, since the measurements are of local strain, while it is actually the load which needs to be known. Conversion of strain to load through modulus and area is never quite as simple as would be wished, but overall the instruments performed well and good agreement with design values was obtained. Figures 7 and 8 below illustrate the overall load/settlement behaviour of two test piles:
TP17 which was a 600 mm bored pile to a depth of 35 m, and TP18 which was a 1000 mm bored pile to a depth of 41 m. It can be seen that the larger pile had some problems at the toe, which lead to significant settlement at a load of about 12,500 kN which was somewhat lower than expected, while still being sufficient for the design load. The smaller pile performed quite well. Also shown, as Figures 9 and 10, are the individual load distribution curves, which illustrate the amount of toe resistance mobilized by TP18 in the third cycle. Finally Figure 11 shows the load distribution for both piles, normalized by the pile diameter. This shows by the slope of the curves, especially TP17 II and TP18 III, that the skin friction mobilized over the majority of the depth was very similar, TP 18 carrying the higher load because of its greater base load and longer length.

As stated above, two bi-directional tests were carried out. In these tests there are no reaction piles, and no hydraulic jacks at the surface to apply a vertical compression load. Instead the jacks are cast into the pile, and various parts of the pile are jacked against
resistance provided by other parts of the pile. Various arrangements are available, but only
the method used, with two jacks, will be described.

With two jacks, the pile is divided vertically into three segments, as seen in Figure 12. One of the segments, below the lower jack, is generally much smaller than the other two. The jacks themselves, powered by hydraulics, have to be installed at the time of constructing the pile, along with all the piping, instrumentation and cabling. This in itself has a significant effect on the construction schedule. The jacks are available in a range of sizes and capacities, and have to be selected to give the required load at the required depth. There also has to be sufficient space around the jacks for the concrete to be tremied into place. For piles formed under drilling fluid, clearly the obstruction to concrete flow caused by the jacks will not assist in the scouring action of the concrete.

Once the pile has been formed, gained sufficient strength and been prepared for testing with all instrumentation connected, the lower jack is first used to push the base down against the combined skin friction of the middle and upper sections. This will normally be pushed to “failure”, at a displacement of about 10% of the diameter. When this has occurred, the lower jack is left open, while the upper jack is activated. Provided the positioning has been correct, the upper jack will cause the middle section of the pile to fail, leaving the upper section relatively unmoved. Once this has happened the lower jack is then closed off, to load the upper section of the pile to failure against the combined reactions of the middle section and the base. At all stages the instruments are used to monitor jack pressure, jack expansion, pile movement, and pile strain at various levels. The idea is then that the performance of the three elements can be added together to give the combined load settlement curve for a reaction force which would fail all three in vertical compression, assuming that the load to fail the upper section vertically downward is the same as that to fail it vertically upward. At small strains this is unlikely to be true, since the pile is loaded from the strongest part at the bottom first, whereas a normal pile is loaded from the weakest soil at the top first. However, if the pile is pushed to failure, the ultimate skin friction capacity is probably almost the same in both directions. It is also necessary to make adjustments to the settlement values for the elastic compression of the pile above that section.

In practice a number of problems occurred, some related to installation, some to monitoring and some to interpretation. The most serious problems relating to installation were in connection with the complications of installing the reinforcement cage fitted with two hydraulic jacks and all the appurtenant instrumentation. Based on previous experience, especially in Thailand (Littlechild and Plumbridge 1998), there is a close relationship between the time spent during construction with drilling fluid, especially bentonite, in the hole and the skin friction measured in a test. For this reason it was recommended and planned that a dummy hole be used to assemble the cage, so that it could be lifted out in one piece and placed into the completed bore as quickly after completion and preliminary cleaning as possible. However, the plan was changed at the last minute and the dummy hole was not used. The result was that there was a delay of 15 hours between completion of boring and start of concreting, which then took a further 3 hours to complete.

In the monitoring phase, as noted above the lower hydraulic jack is closed in the final stage to allow the middle section and lower section to react together against the upper section. It would appear to be obvious that monitoring of the lower hydraulic jack pressure is essential, in order to know how much load was applied to the lower section, especially the base. However no equipment was available to carry out this monitoring.
successfully on the My Thuan Bridge project in Vietnam, where 2.4 m diameter piles, 95 m long, were successfully tested in the Mekong River (Randolph, 2003).

CONCLUSIONS

1. The piled foundations of the Passenger Terminal Complex of the Second Bangkok International Airport project comprise over 25,000 piles, including over 10,000 bored piles. These have been designed on an effective stress basis and to take account of future rises in the groundwater level at the site.

2. The concept of a Neutral Plane has been used to take account of Negative Skin Friction, and to determine the depths at which loads will induce settlement.

3. The settlements have been analysed using a closed-form solution to Mindlin’s Problem No 1, which allowed the stress due to each pile to be considered as a point load within an elastic half-space. Additional piles in a group were then added by superposition, as were the effects of adjacent groups. Settlement was then predicted by applying an elastic stiffness parameter.

4. Pile tests have been carried out which verified the design method and parameters. A total of 19 tests were performed, 15 on bored piles, and 7 of these included Vibrating Wire Strain Gauges to measure load transfer.

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5. Two of the tests with Vibrating Wire Strain Gauges were carried out using the bi-directional test method, for the first time in Thailand.

6. Although the bi-directional tests were basically successful, the extra cost plus difficulties of construction and interpretation do not justify their use for conventional on land testing. They are extremely useful in conditions where reactions are difficult to arrange, such as in marine piling.

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