2001 MILTON E. BENDER LECTURE

GEOTECHNICAL ENGINEERING IN INFRASTRUCTURE DEVELOPMENT

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ABSTRACT: Four major infrastructure projects in Taipei and Bangkok were discussed to illustrate the importance of geotechnical engineering in any infrastructure development in terms of safety and economy. The four projects are the Taipei Rapid Transit Systems (TRTS), the Taipei Airport Underpass, the Bangkok-Chonburi New Highway (BCNH) and Ground improvement of the Second Bangkok International Airport (SBIA). They were selected not only because of the mega size in terms of construction cost but also because of the complexity or uniqueness of the project. Emphasis has been placed on the importance of adequate and reliable subsurface information, appropriate selection of analysis principles and construction methodology/details, ability to copy with variation in ground conditions and timely interpretation of field performance data.

INTRODUCTION

This lecture is the eighth in the series of lectures organized by the Asian Institute of Technology started in 1993 to honor the Founding President of the Institute—Professor Dr. Milton E. Bender. This is also the first memory lecture since the demise of Dr. Bender last year. I felt greatly honored to be invited today to deliver this lecture, not only this is the first memory lecture but also being the first former faculty member of AIT who has worked under Dr. Bender to be the Bender Lecturer.

I had a profound memory of Dr. Bender as a boss, a colleague and a friend. It was Dean Bender who appointed me to the faculty of the SEATO Graduate School of Engineering in 1965 and pursued me to remain at AIT for 11 years. In the following years, I had the opportunity to work closely with him in developing a new field of study—soil engineering. I also had many opportunities to listen to his many up and downs relating to his historical accomplishment in transforming the SEATO Graduate School to an independent and truly international Asian Institute of Technology, and later worked with him on the development of the new campus at Pathumthani.

At AIT, I started the first graduate program in Geotechnical Engineering, probably one of the first in the region, the first regional information center, organized the first regional conference and then founded the first regional professional society, (i.e. the Southeast Asian Society of Soil Engineering). Without the vision and unfailing support of Dr. Bender, these activities were not possible. Dr. Bender was an engineer, academic, diplomat and promoter. He was a man with principle and determination. He had no hesitation to make decisions but he always was willing to listen and to adopt good ideas.

Recognizing infrastructure development is the key to economic development of developing countries, D. Bender has placed major emphasis on the development of academic programs that uniquely fit the needs of the region. In the early days, most of the major fields of study are related to civil engineering, i.e. the base of physical infrastructure development. Even today, those basic fields of study are still playing major roles in AIT's academic program. For this reason, I have chosen the topic "Geotechnical Engineering in Infrastructure Development" for today's lecture.

For any physical infrastructure development, whether they are founded on ground, built in the ground or utilize earth material as construction material, the role of geotechnical engineering in design and construction cannot be over-emphasized. However, geotechnical engineering, not like other branches of civil engineering, up to the present, still has not been developed into an exact science due to the complex nature of the earth. Sound geotechnical practice depends on integration of theory, reliable data, and experience. Observational method by utilizing field monitoring data during construction for design revisions is the best approach to achieve economy and safety in major infrastructure development. This lecture presents case studies of four major infrastructure projects to illustrate the complexity of geotechnical engineering.

SOFT GROUND CONSTRUCTION IN TAIPEI BASIN

Projects

(A) The Initial Network of the Taipei Rapid Transit Systems

The Initial Network of the Taipei Rapid Transit Systems (TRTS) comprises of six lines with a total of 88km of track and 77 stations. About half of the stations and tracks are underground. Except a short section of one of the lines, the majority of the Initial Network is located in soft ground. A system map is depicted in Fig. 1. Of the six lines only the Mucha Line is of medium capacity. The other five lines are all of heavy capacity. Planning and design of the Initial Network started in 1987. The first line, i.e., the Mucha Line was completed and open to operation in March, 1996. At present, except for a short section of the Panchiao Line, all six lines of the Initial Network are open to revenue services with average daily traffic of 900,000 passenger-trips per day and holiday traffic exceeding 1.2 million per day. The total construction cost of the six lines, including the extensions to Neihu and Tucheng, is about NT$440 billion (about 13 billion US dollars).

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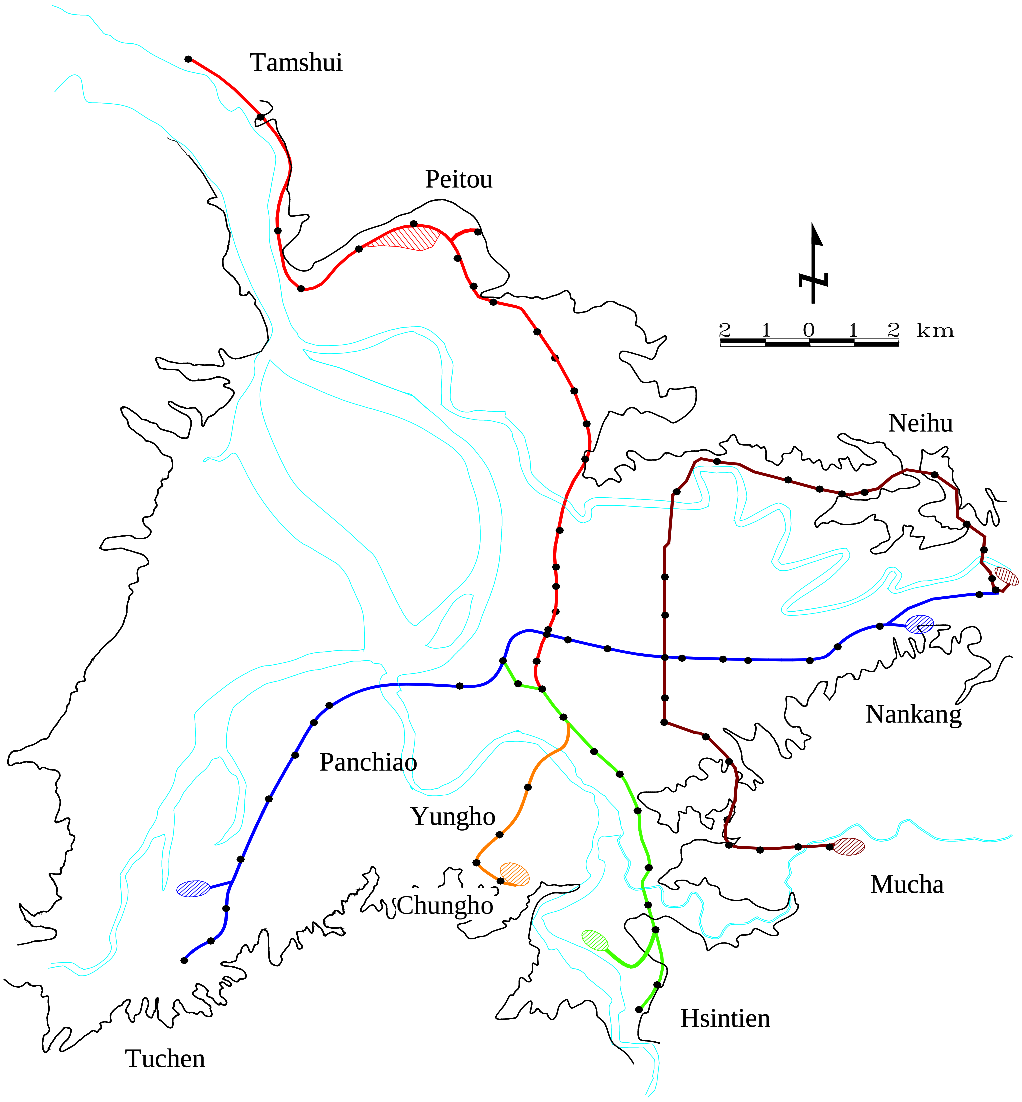


Fig. 1 Initial Network of Taipei Rapid Transit Systems

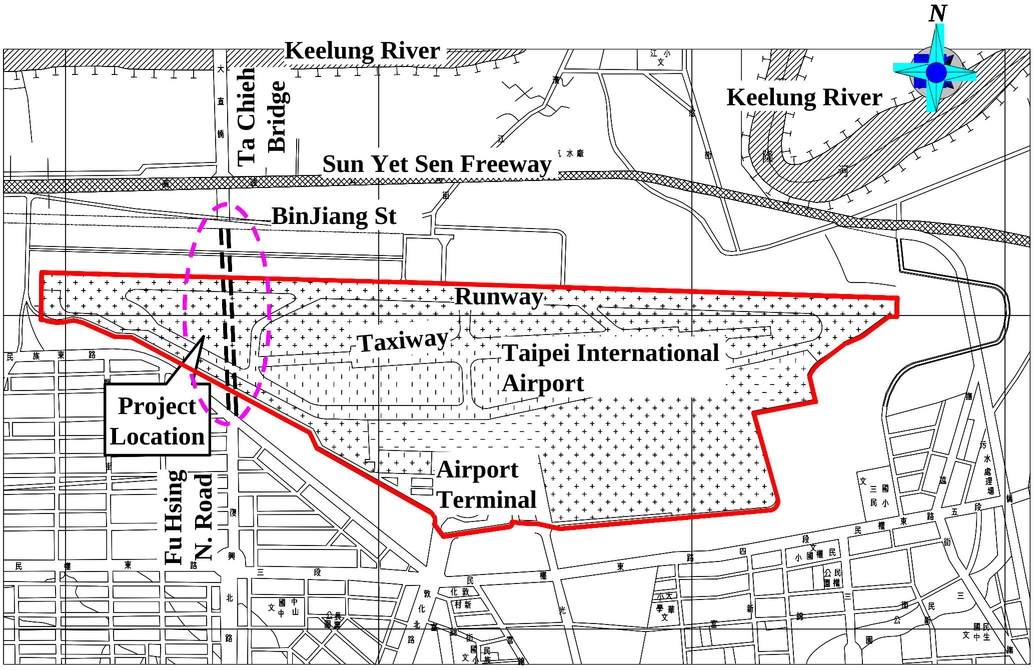
For the underground construction, about 45 km of diaphragm walls, with thickness varying from 800 mm to 120 cm, were constructed in cut-and-cover excavations whilst bored tunnels have a total route length of 22km.

For the initial network of the TRTS, besides being detailed design consultant for one of the design package, Moh and Associates, Inc. was the overall Geotechnical Engineering Specialty Consultant for the entire project from planning to construction.

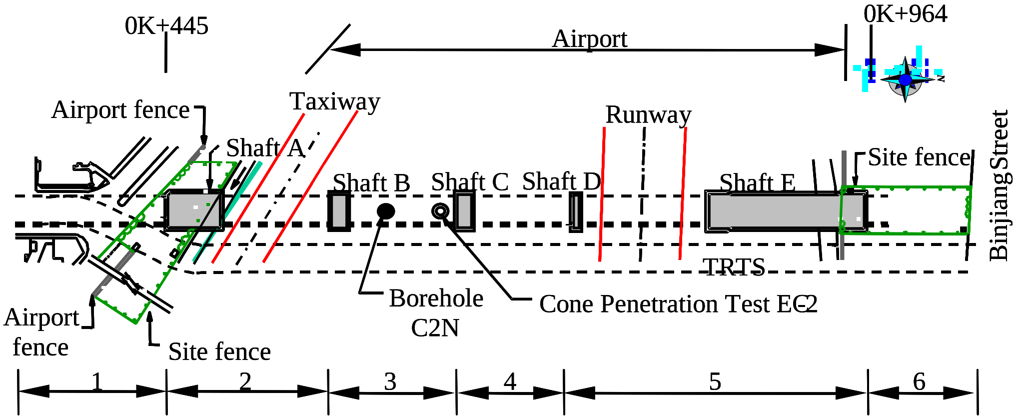
(B) Taipei Airport Underpass

To ease the local traffic congestion in the northern part of Taipei City, an underpass is currently being constructed to extend Fuhsing North Road northward to connect to Tachieh Bridge which crosses the Keelung River. A major portion of this extension is underneath the field of the Taipei International Airport which is a busy airport serving both civilian and military air traffic with more than 300 commercial flights per day. As shown in Fig. 2(c), the underpass has to dive to a depth of 21.37m (road level) at its south end because of the provision of a tunnel box for the TRTS on the top and also because of the presence of a drainage box culvert. At its northern end the underpass is to meet the existing Bingjiang Street and therefore has to pass underneath the runway with a very thin cover of less than 5m in thickness above its roof. As depicted in Fig. 3, the underpass has 2 lanes in each direction and the twin-cell box is 22.20m in width and 7.80m in height.

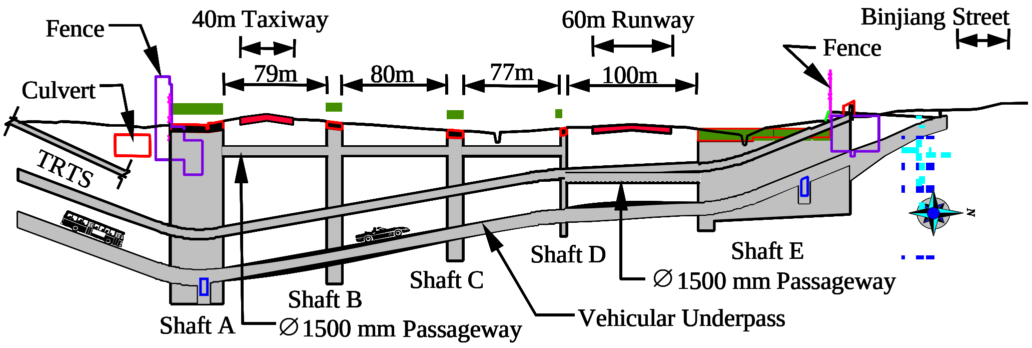
Since the Taipei International Airport is the only airport in the vicinity of Taipei metropolis, the air traffic must be maintained all the time and there are several constrains which must be considered in the design and construction of the underpass. Major constrains include: (1) construction activities above the ground surface are limited to the period between 11pm and 5am within the entire boundary of the airport; and (2) settlement of the runway during construction must be maintained within 25mm. To assure safety of the airport operation, a comprehensive instrumentation program has been implemented. There are more than one thousand pieces of instruments, including settlement points, horizontal inclinometers, installed at the site. Majority of these instruments can be read automatically at any desirable frequencies. Data are transmitted from data loggers to the central control room at the site office through cables for immediate processing and



(a) General layout

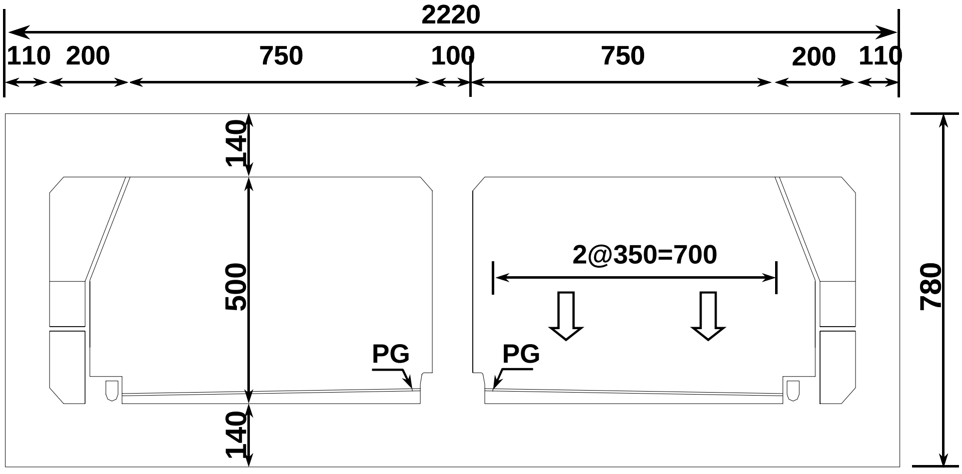


(b) Site Plan



(c) Longitudinal section

Fig. 2 Underpass beneath Taipei International Airport



(unit:cm)

Fig. 3 Typical tunnel section of the airport underpass

analyses. An automatic alert system is adopted for prompt actions to ensure operational safety of the airport.

For this project, Moh and Associates, Inc. is the consultant responsible for the design and construction supervision.

Difficult Geological Features

Taipei Basin was formed about 10,000 years ago as a result of tectonic movement. It was a lake until a few thousands of years ago and particles brought from surrounding hills by rivers accumulated at the bottom to form thick alluvial strata. As the sea retreated, the lake was drained and the bottom of the lake was exposed. Figure 4 shows typical profiles of subsoils in the central city area of Taipei. As can be noted that there exists a thick layer of young sediments, i.e., the so-called Sungshan Formation, underlain by the Chingmei Gravels. The Sungshan Formation contains interbedded soft silty clay and loose silty sand layers, both of which are fairly compressible. Therefore, the city has experienced large ground subsidence as the basin was inhabited and excessive withdrawal of groundwater for domestic and industrial uses led to considerable drop in piezometric levels in the Chingmei Gravels and the subsoils in the Sungshan Formation.

The results of a cone penetration test carried out in the central city area are shown in Fig. 5. As can be noted that the six sublayers in the Sungshan Formation are clearly identifiable. The weak strengths together with the high compressibility of the surface layers necessitate thick retaining walls and deep penetrations of walls in deep excavations to limit wall deflections and reduce seepage flows. In addition, there are numerous drift woods buried in the Sungshan Formation and obstacles were frequently encountered during tunneling and/or pipe jacking. Accidents indeed occurred during the construction of the rapid transit systems and some of these accidents caused considerable financial loss and delays of the project.

The presence of methane in the Sungshan Formation is another unique feature which calls for special attention in underground works. Methane is potentially dangerous as quite many fatal accidents have occurred in the past as workers entered manholes, storage tanks, etc for maintenance. These accidents are not what the author intends to discuss herein because the methane encountered in these cases was not released from soils but from organic wastes. What of interest is the methane entrapped in soils because it might seep into confined underground spaces and accumulate to a concentration high enough to become harmful to people. It might even explode if its concentration reaches the critical concentration for ignition.

The Chingmei Gravels underlying the Sungshan Formation, refer to Fig. 4, is equally problematic. It is extremely permeable and is very rich in water reserve. Although its presence has been carefully considered in the design, accidents still occurred during the construction of the rapid transit system. In fact, the Chingmei Gravels was responsible for all the disastrous accidents which occurred during the construction of the said system. Problems usually started with small leakages of water either on retaining walls or at the bottom of excavations and the situations soon became uncontrollable as the water paths connected to the Chingmei Gravels. The lessons learned are particularly valuable in the sense that it is anticipated that the tunnels in the future lines of the rapid transit systems are inevitably deeper in depth than before. The situation is aggravated by the fact that, as to be discussed in a later section, the piezometric levels in the Chingmei Gravels are rising with time.

Problems with Soft Ground

The soft clays in the Sungshan Formation are highly compressible as evidenced by the large ground subsidence experienced in the past. The ground subsidence in the Taipei Basin is comparable in magnitude with, if not more than, the subsidence experienced in many other cities located on recent alluvium. Figure 6 shows the ground subsidence recorded in the past in the central city area, and as can be noted, a settlement of 2.2 m ccurred in a 20-year period between 1960 and 1980. This was a result of lowering of piezometric levels in the Chingmei Gravels due to excessive pumping of groundwater in the formation. Fortunately, in the early days, buildings were mostly 6 stories or lower till the late 80’s and very few buildings were founded on piles. Therefore, to the author’s

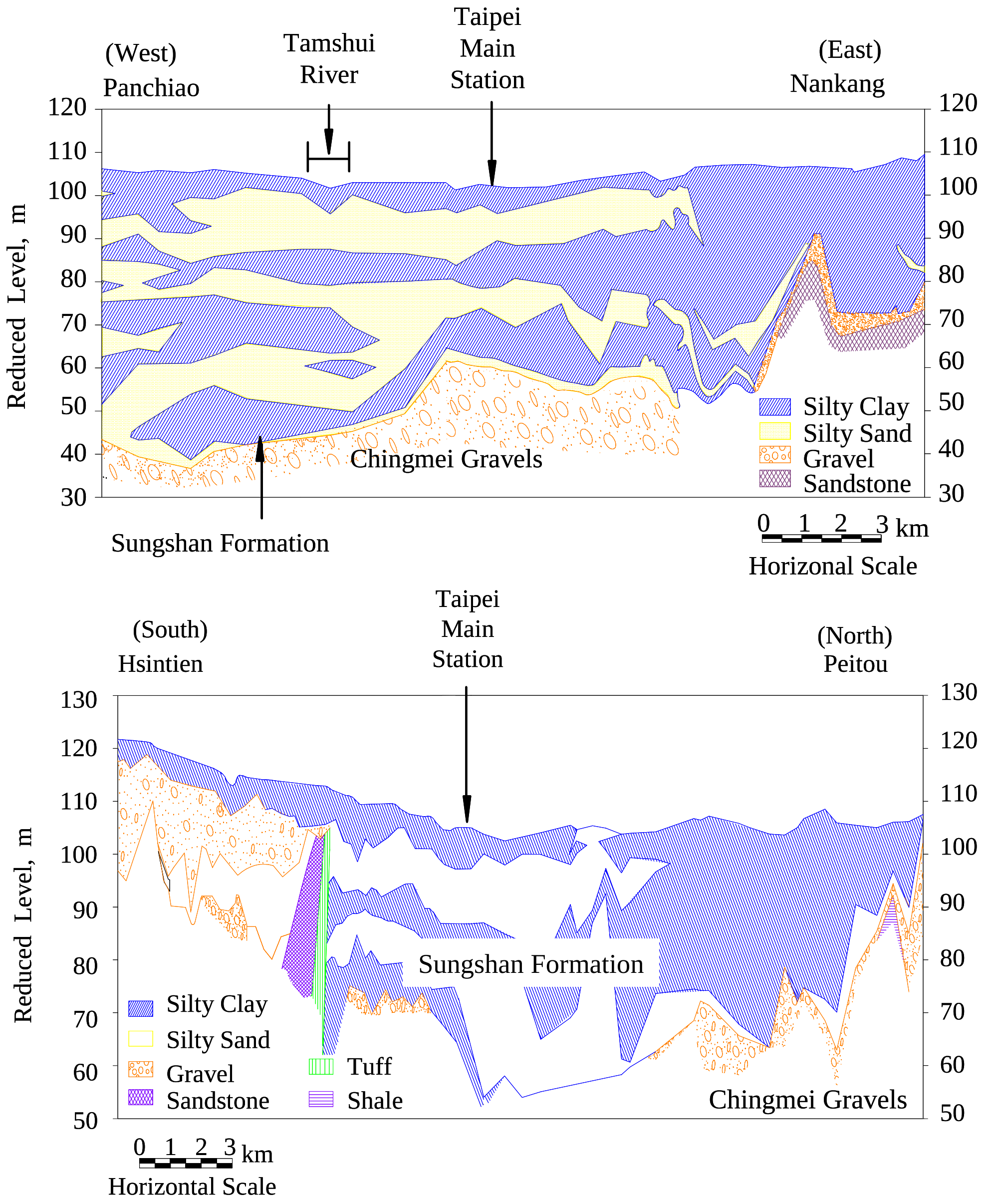


Fig. 4 Geological profiles of the taipei Basin

knowledge, there are no reported cases of failed piled foundations associated with ground subsidence. It was reported that groundwater in the city was in an artesian condition at the beginning of the 20th century. This means, refer to Fig. 6, with a current ground level at RL 102m (mean sea level = RL 100m) or so, the piezometric level in the Chingmei Formation once dropped by more than 50m. After the completion of the Fei Tsui Reservoir in the late 80's which has since supplied sufficient water to the entire Taipei City and part of Taipei County for domestic and industrial uses, deep well pumping in the city area was strictly regulated. The piezometric levels in the Chingmei Gravels, and those in the Sungshan Formation as well, have gradually recovered as depicted in Fig. 7 and ground subsidence has been small in recent years as depicted in Fig. 6.

The high compressibility of the soft clays in the Sungshan Formation, however, necessitates deeper penetrations of retaining walls and better water tightness of the walls for avoiding large drawdown of groundwater table due to seepage which might lead to intolerable ground settlements in the vicinity of excavations. Furthermore, the weak strengths of the subsoils call for thick walls and heavy strutting for limiting wall movements which will inevitably cause ground to settle. In comparison with the practice of the old days, design concept has advanced with due considerations given to ground movements and workmanship has been much improved in the construction of the rapid transit systems. As a result, drawdown of groundwater was generally small and lateral movements of diaphragm walls were less than one third of what was experienced before. Ground settlements in the vicinity of excavations were generally within tolerance and few buildings, if any, suffered from structural damages except in a few unexpected accidents.

The same can be said on ground settlements over tunnels. The construction of the trunk lines of the sewerage systems along Mingtsu Road in 1977 was the first application of shield machines in Taiwan and the tunneling operation had to be suspended shortly after launching as a result of excessive inflow of water into the tunnels and loss of compressed air. The tunneling technique has evolved

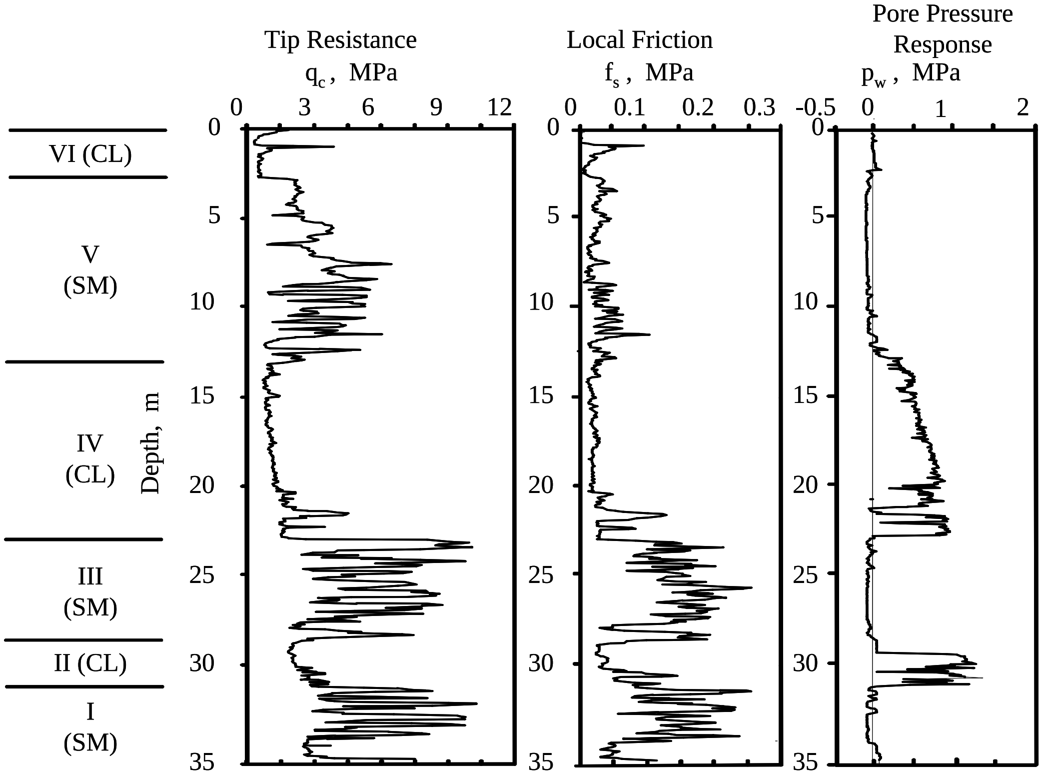


Fig. 5 **Typical CPT profile in Central Taipei**

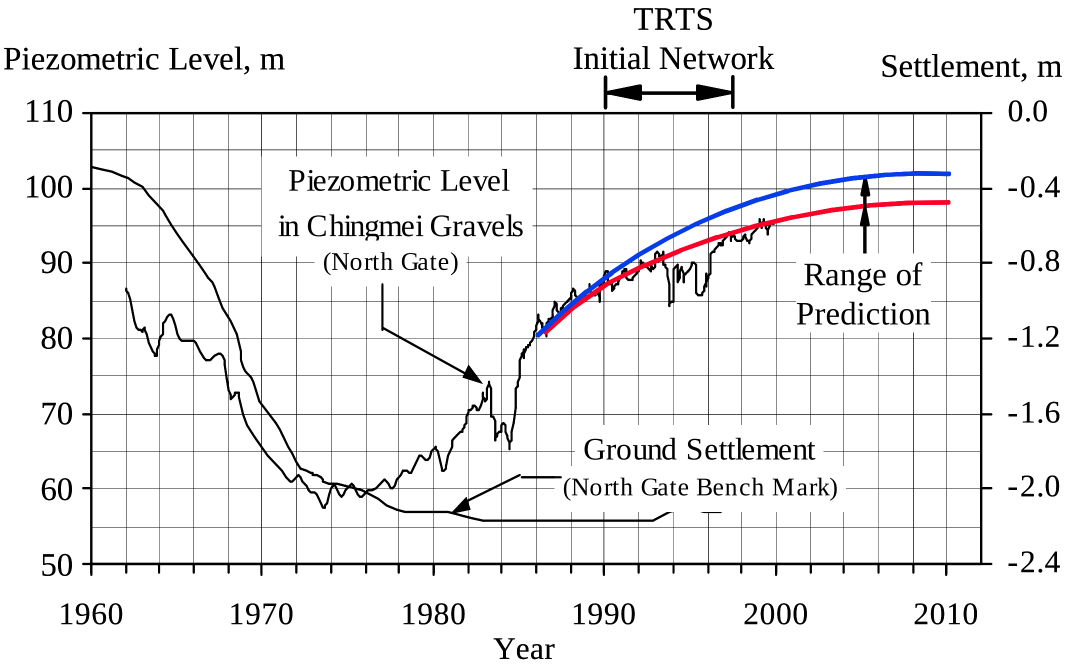


Fig. 6 **Piezometric level in the Chingmei Gravels and historical ground settlement in Central Taipei**

ever since and the use of earthpressure balancing type and slurry type shield machines in the construction of the rapid transit systems has drastically reduced ground settlements to less than a half of what was before.

A rather unique situation was faced in constructing the underpass beneath the Taipei International Airport (Moh et al, 1999). This underpass is a 4-lane highway tunnel and, as depicted in Fig. 2, is to go underneath a taxiway of 40m and also a runway of 60m in width. The top of the underpass is only 5.6m below the surface of the runway. The service of both the taxiway and the runway shall in no case be interrupted except in the period between 11pm and 6am. This makes the project technically challenging and difficult to manage. As depicted in Fig. 8, because of the poor nature of the ground, interlocked steel pipes of 812.8 mm in diameter were installed by jacking to provide an enclosed protective shelter for excavation to be carried out

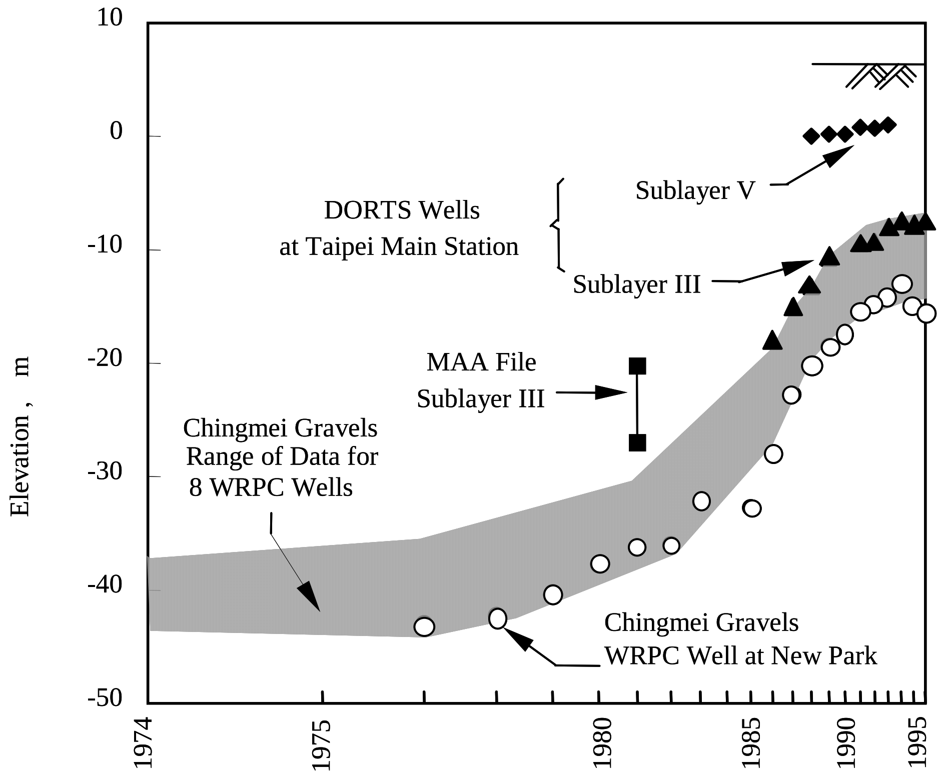
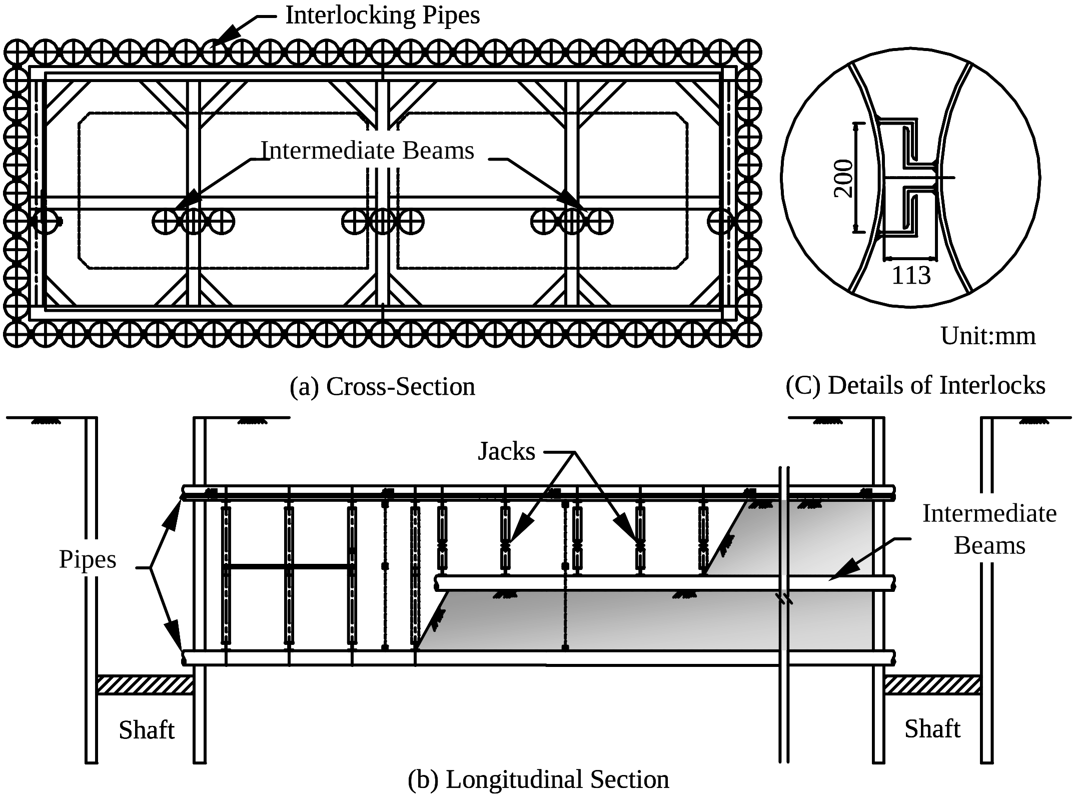


Fig. 7 Piezometric levels in the Sungshan Formation



(a) Cross-Section

(C) Details of Interlocks

(b) Longitudinal Section

Fig. 8 Tunneling in sections 3 and 4 of the Underpass

inside safely. This, however, still does not guarantee that the stringent settlement criteria can be made. To further reduce ground settlements, as depicted in Fig. 9, the precast tunnel boxes are to be jacked into the shelters under the taxiway and the runway by using the so-called endless-self-advancing method (ESA) in full-section segments of 10.5 m each, except the head segments which are only 6.5m, in length.

Five working shafts, refer to Fig. 2, have been sunk and are inter-connected by a 1,500mm passageway for transporting materials and crew so work can be carried out underground without surface activities. The section of the passageway under the runway is in the way of the tunnel box and has to be removed, piece by piece, while the box is being jacked into the shelter. There will be five guide tunnels, as depicted in Fig. 10, in which rails will be laid for tunnel segments to rest. In addition, there are 12 cable ducts. The basic concept of the ESA method is that each time a segment is being driven, either by jacking or pulling, the majority of the reaction comes from the frictional resistance acting on the rest of segments. For example, refer to Fig. 9b, when Segment 3 is being driven by jacking against Segment 4, part of the jacking force is taken by the frictional resistance acting on segments 4, 5 and 6, part of the jacking force goes to the end of Segment 6 and is taken by cables. The force taken by the cables which run through guide tunnels, refer to Fig. 10, is transmitted to the reaction pad in the arrival shaft. The force taken by cables which run through cable ducts is transmitted to Segment 2 and is resisted by the frictional force acting on Segment 2. This way, the size and the rigidity of the reaction pad can be much reduced. During jacking, lubricant will be injected from grouting holes on the segments to reduce frictional resistance between the tunnel box and sheltering pipes. In theory, the forces taken by the cables and the reaction pad are not affected by the length of the tunnel box and the tunnel box can be as long as desired.

At the time this paper was prepared (April, 2001), all the sheltering pipes have been installed and precast tunnel segments, refer to Fig. 9, have been erected in Shaft E to prepare for launching. Because of the poor nature of the soils, as depicted in Fig. 11, extensive ground improvement has been carried out for increasing face stability and reducing water seepage. In principle, the soil core was solidified by cement-bentonite-water mixture. The soil surrounding the guide tunnels at the two corners was treated by using the double-packer grouting method to yield a minimum unconfined compressive strength of 160 kPa while the unipack method was used elsewhere to yield a minimum unconfined compressive strength of 80 kPa.

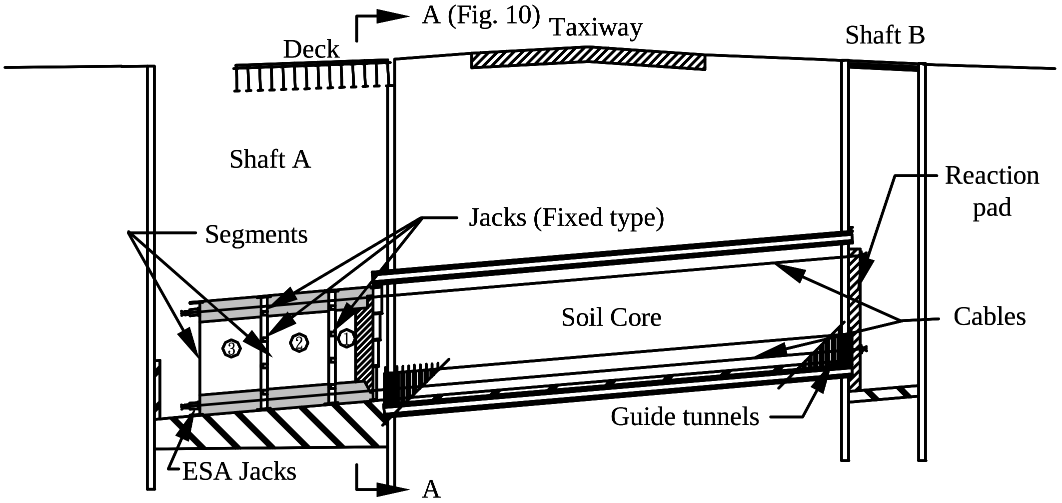
Problems with Drift Woods

Jacking of sheltering pipes at the airport underpass was completed not without problems. Pieces of drift woods were encountered at a depth of roughly 6m during jacking of one of the pipes between Shafts C and D and a temporary shaft had to be sunk from surface for them to be removed. Two of the pieces removed measured about a half meter in length and 200mm in diameter.

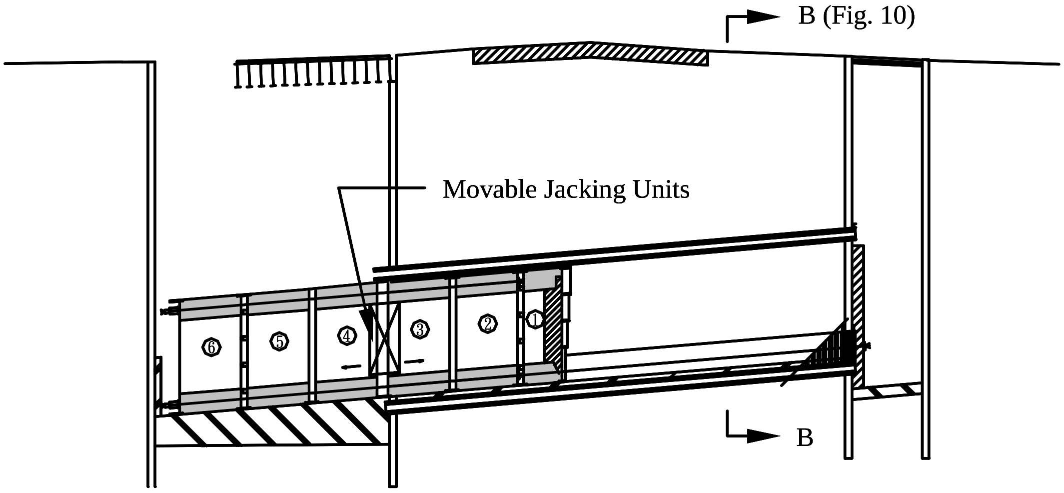
Chunks of drift woods of 1m or so in diameters were recovered in numerous excavations in the past and they were as long as 5m in length and as thick as 1 m in diameter. Most of the woods recovered appear to be reasonably fresh. For academic interest, a piece of wood found at a depth of 9m in Observation Well 2 in Panchiao was sent to laboratory for dating and the results indicated an age of 6,760 years and another piece found at a depth of 23m dated back to 7,950 years (Liew, 1994). Although drift woods were frequently encountered, to the author's knowledge, few problems occurred prior to the construction of the rapid transit system.

The problem with drift woods was, however, serious during shield tunneling in the constructions of the rapid transit systems. Although the presence of drift woods was recognized in the design stage and was well reported, however, it is still very difficult to predict the locations and depths of drift woods with any degree of accuracy. To prepare for the problem, all the shield machines adopted in the construction of the rapid transit system have sufficient strength and power to cut through the woods as long as they are not too large in size. Pieces of drift woods were indeed encountered at many locations along the Panchiao Line and the Chungho Line and in most of the cases the shield machines were able to advance with efforts . However, there was one occasion in which the presence of drift wood caused the ground to collapse as the shield machine was slowed down while mud kept on running into the earth chamber. Finally, a sinkhole of 5m in diameter occurred right above the head of the shield machine. As depicted in Fig. 12 (Lin et al., 1997a), ground treatment was carried out in front of the machine to enable workers to go out of the machine to remove the obstacles and to repair damaged cutting blades. Two pieces of drift wood, 500mm and 400 mm each in length, were recovered. To avoid road surface from collapsing should drift woods be encountered again, the remaining section of tunnel was treated as illustrated in the figure.

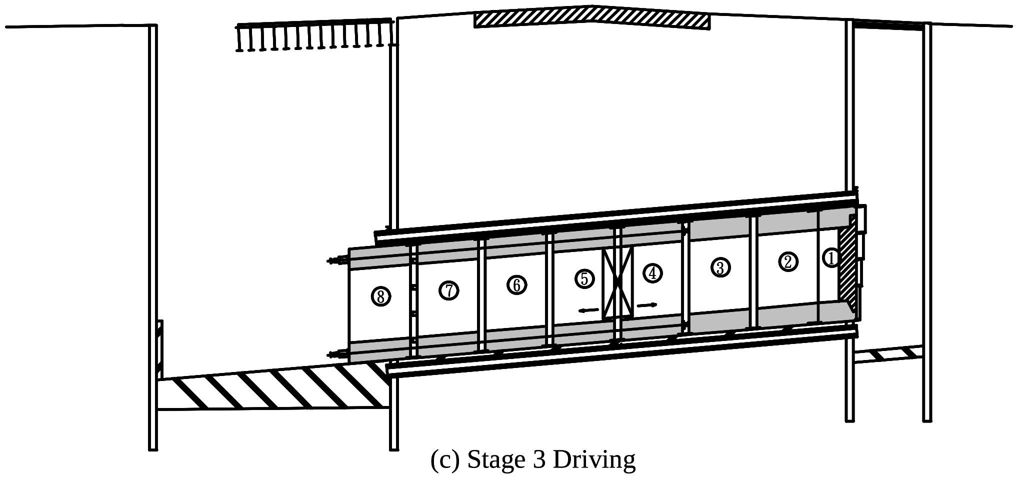
The problem with drift woods was the most serious in the Panchiao Line and several disastrous accidents occurred during tunneling. Although shield machines were able to chop drift woods into pieces, the movements of the woods as they were stirred by the cutting blades destroyed the integrity of ground treatment in front of working shafts and created fissures leading to leakage of water into these shafts. Figure 13 shows one of such cases in which groundwater spurted into the arrival shaft when an opening was made on the diaphragm wall for receiving the shield machine (Lin et al., 1997b). The flow soon became uncontrollable as the water path connected to the Chingmei Gravels and a sinkhole of roughly 4,000 m^3 in volume was created. The shaft had to be recharged to stop water from running into the shaft. Both tunnels were flooded and one of them had to be abandoned as a result. Probing from surface indicated that 34 rings in the down-track tunnel and 39 rings in the up-track tunnel were damaged and had to be replaced. The opening was later sealed by a gravity retaining wall formed in water and by ground freezing to solidify the surrounding soils (Ju, et al. 1998). When the opening was re-opened after damaged rings were replaced, a piece of wood was found at the location where water spurted. Right next to this piece of wood was a PVC pipe, which is believed to be an abandoned pumping well, extending all the way down to the Chingmei Formation. It is hypothesized that the ground treatment, which was carried out by using the CJG method of grouting, was disturbed as the wood was stirred by the shield machine and groundwater was able to find its way into the shaft. The PVC pipe was chopped off and the lower portion of



(a) Stage 1 Driving



(b) Stage 2 Driving



(c) Stage 3 Driving

Fig. 9 ESA Tunneling in Section 2 of Underpass

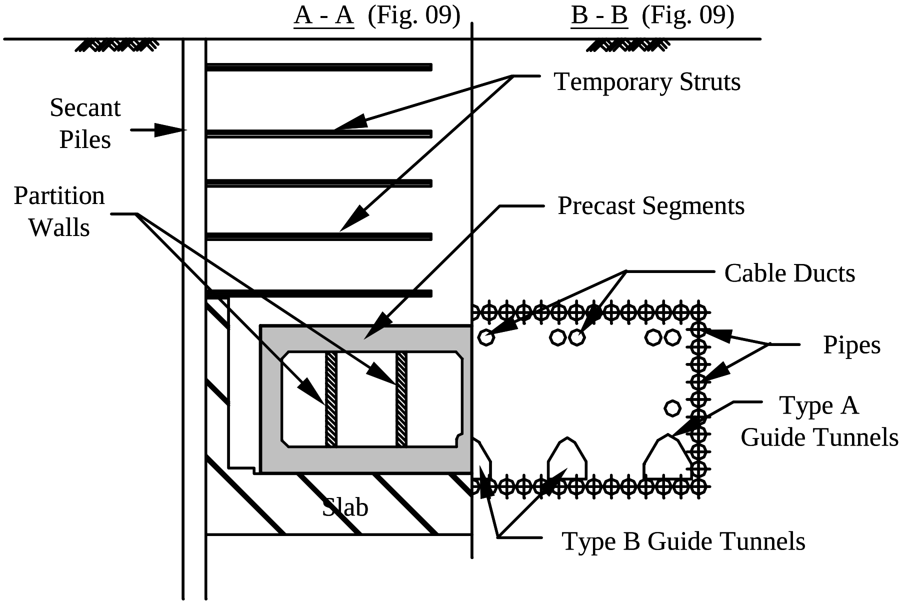


Fig. 10 Cross-section of ESA layout of the Underpass

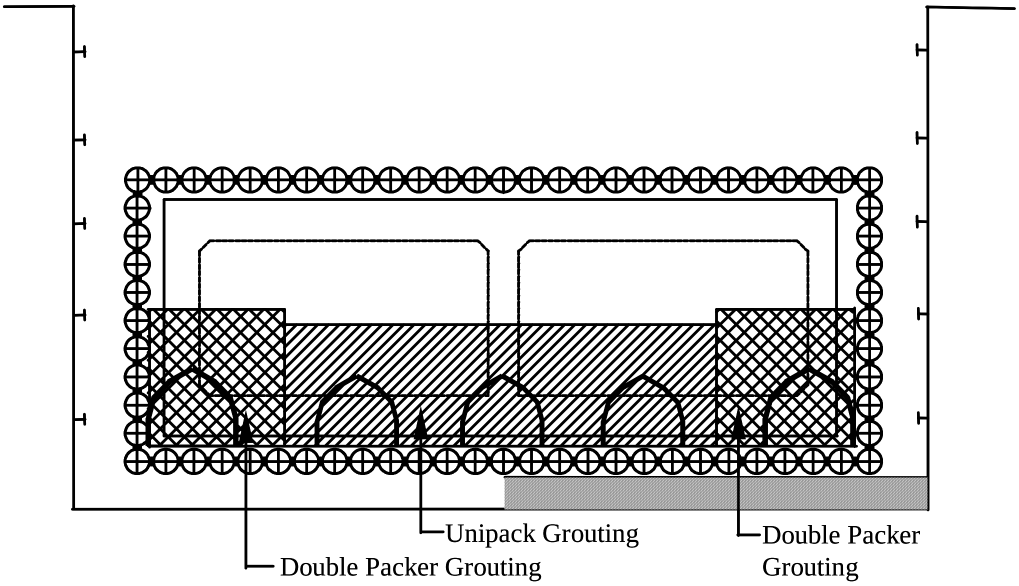


Fig. 11 Ground improvement for ESA work

the pipe became a water path connecting to the Chingmei Gravels which practically became a reservoir with a huge water supply.

Problems with the Chingmei Gravels

The problems with the underlying Chingmei Gravels were many and the above-mentioned case is definitely not an isolated one. This gravelly layer is extremely permeable and very rich in water. The fact that it was the sole water supply for the entire Taipei City prior to the 70's tells how ample the water reserve is in this layer. Experience indicates that once water path connects to the Chingmei Gravels, it will be extremely difficult to stop water from running into excavations. Recharging appears to be the only option although such a decision is painful to make because the consequences are usually grievous. Another accident as disastrous as the one mentioned above happened in the Hsintein Line when a tunnel portal on the diaphragm wall of a working shaft was enlarged to enable the expansion joint to be installed between the tunnel and the shaft (Hwang, et al. 1998). As shown in Fig. 14, although the surrounding ground had been treated before, water was able to find its way into the shaft. Because the

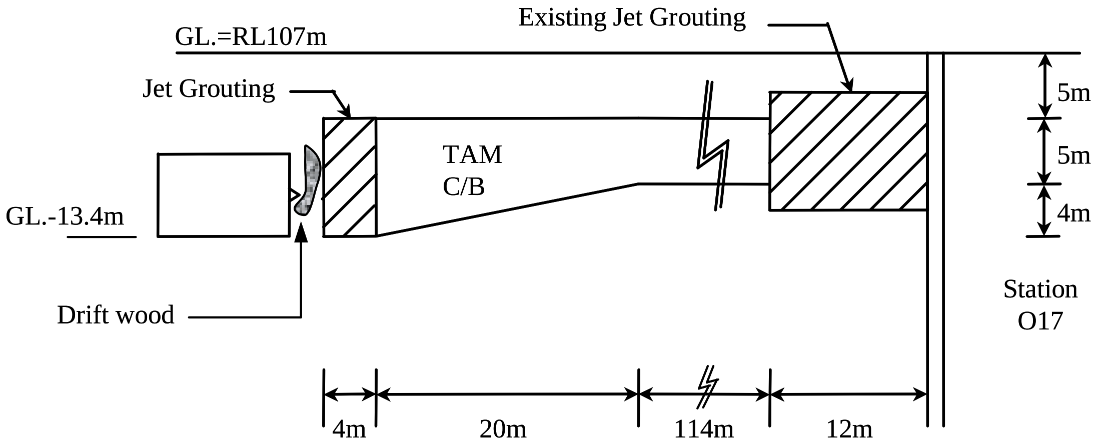


Fig. 12 **Ground treatment at the up-track tunnel of Contract CC277 of TRTS**

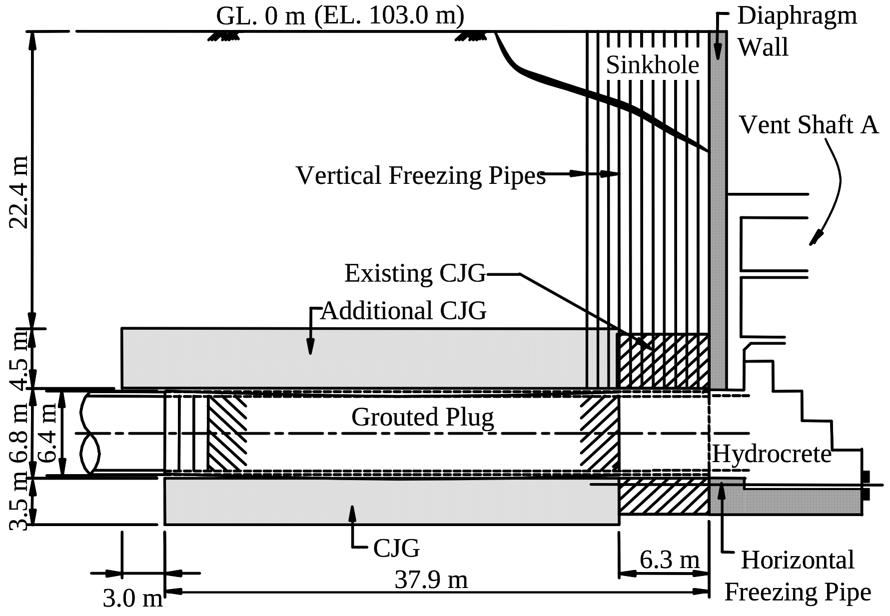


Fig. 13 **Settlement profile at the arrival shaft in Contract CP262 of the TRTS**

portal is so close to the Chingmei Gravels, flow soon became uncontrollable. The shaft also had to be recharged to balance the groundwater pressure. A section of the up-track tunnel was damaged and 23 segments had to be replaced. The down-track tunnel, fortunately, was not affected. To enable the remedial measures to be taken, the portal was sealed by ground freezing to form a patching pad. A steel diaphragm was installed at the other end of the damaged section of the tunnel and the damaged segments were replaced in compressed air.

The problems with the Chingmei Gravels were not limited to tunneling. Blow-in and piping are serious concerns in deep excavations wherever the Chingmei Gravels exists and provisions had to be made for excavations to be carried out safely. For examples, excavations for constructing the three ventilation shafts of TRTS were carried out to depths exceeding 30m and could not be carried out safely unless measures were taken to resist the groundwater pressures. For Ventilation Shaft A in the Panchiao Line shown in Fig. 13, continuous pumping had to be carried out to reduce the piezometric level in the Chingmei Gravels, as shown in Fig. 15, from RL. 88.3m to 77.6m to give a factor of safety of 1.25 against blow-in. The rate of pumping was as much as 4170 m^3/hr. For constructing Ventilation Shaft B in the same line, refer to Fig. 16, pumping was carried out at a rate of 3600 cmh to lower the piezometric level in the Chingmei Gravels from RL. 89m to RL. 79.5m. As shown in Fig. 17, groundwater drawdown was significant at distances exceeding 5 km or so in these two cases.

In each of the two ventilation shafts in the Panchiao Line, refer to Figs. 15 and 16, there were a soil plug of significant thickness below the bottom of excavation and above the Chingmei Gravels. For constructing the ventilation shaft in the Hsientein Line shown in Fig. 14, the situation was more critical because the excavation was carried out all the way down to the top of the Chingmei Gravels. Instead of dewatering, the contractor opted to extend the diaphragm wall to a depth of 30m into the Chingmei Gravels and seal off the bottom by grouting to form an artificial soil plug. Installation of diaphragm wall in this gravelly layer to such a depth was, however, a difficult task. One of the panels took 33 days to dig and it took half of a year to complete the entire installation. The grouted pad at the bottom, however, did serve the purpose of sealing off the seepage water and the pit was dry all the time during the excavation.

In all the three cases depicted in Figs. 14, 15 and 16, excavations, per se, were completed without accidents. An unexpected event, however, refer to Fig. 18, did occur at one station during excavation where drilling for replacing a malfunctioned piezometer penetrated through the clay blanket covering the Chingmei Gravels and caused water in the Chingmei Gravels to rush into the excavation (Moh et al., 1997). At the time when this happened, excavation had reached its final level and the bottom of the excavation had been paved by using lean concrete. Similar to the two cases mentioned above, the pit had to be flooded to balance the water pressure. The water path was sealed by grouting for the rehabilitation work to be carried out.

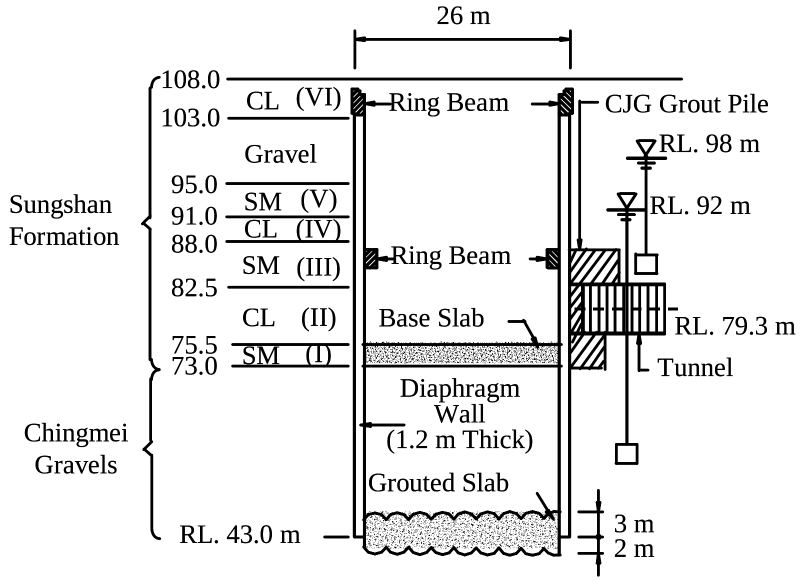


Fig. 14 Ground conditions and configuration of the Contract CH221 ventilation shaft, TRTS

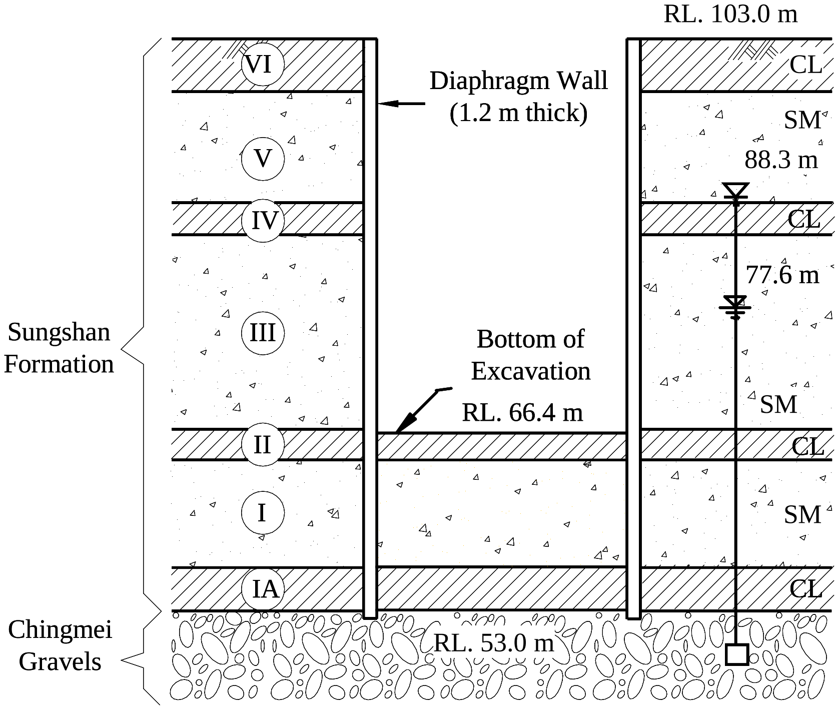


Fig. 15 Pumping at Ventilation Shaft A in Contract CP262, TRTS

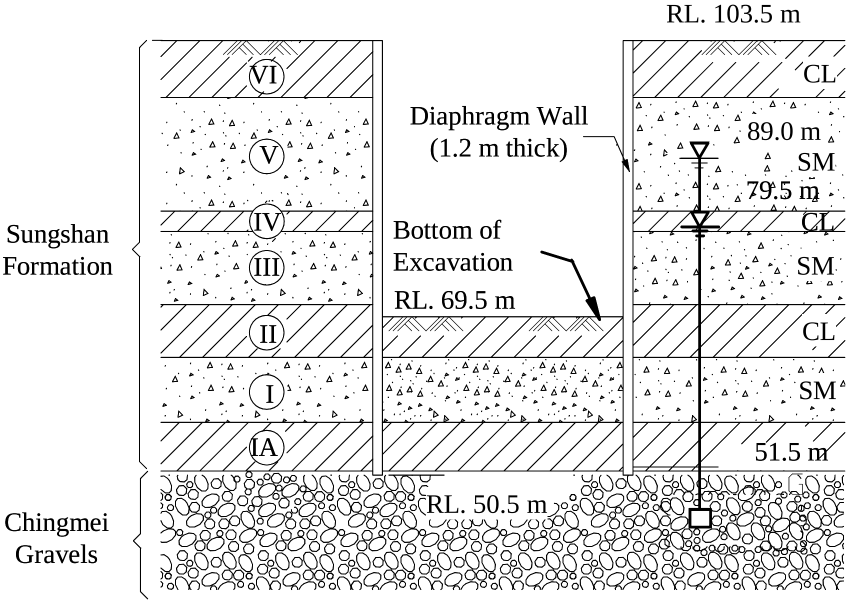


Fig. 16 Pumping at Ventilation Shaft B in Contract CP261, TRTS

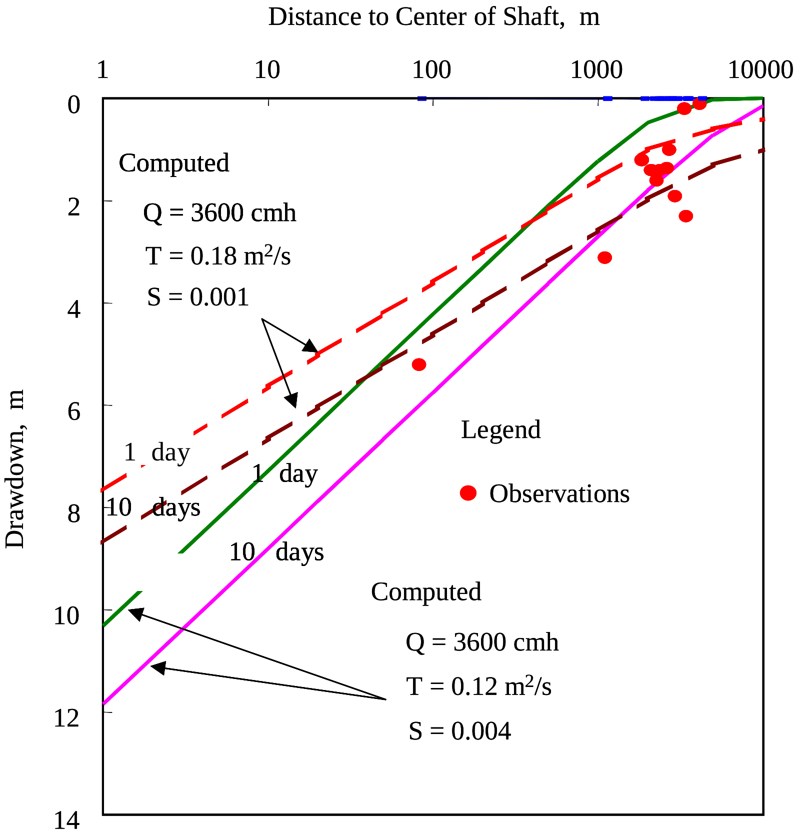


Fig. 17 Influence of dewatering in the Chingmei Gravel

Problems with Methane

Methane was encountered in several boreholes during site investigation. Its presence is related to the capture of gas in domes encapped by impermeable clays (Lin et al., 1997a). The depths at which methane was encountered vary from 20 m to 33 m and correspond to Sublayer 1 of the Sungshan Formation. In one case, eruption sent a jet of methane-water mix to a maximum height of about 10 m into the air and continued for more than 3 days. A piezometer which was installed previously at a depth of 34.75m in the hole was blown out. It took 2 months for the pressure to totally release. Measurements were taken to determine the concentration of methane and the results indicated concentrations up to 5%. Laboratory tests indicated a chemical composition of CH:24 ppm, O:0.3 ppm, CO:18 ppm and H_2S:0 ppm.

Methane is usually dissolved in groundwater and might be carried by seepage water into tunnels wherever leakage exists. Therefore, as long as the tunnel is watertight and provided there is some circulation of air, the danger for methane to accumulate to a harmful concentration is practically small. Methane is potentially explosive when its concentration is in the range of 5% to 15% and the results could be fatal. In the construction of the Taipei rapid transit system, contractors had been warned of the possibility of encountering methane in tunnel drives and possible consequences before they tendered. As a precaution, specifications required the concentration of methane be continuously monitored during tunneling and shield machines be equipped with detective devices and alarm systems. The capacity of ventilation in tunnels was increased from 780 m^3/min to twice as much to reduce the concentration of methane, if any. The power supply would be automatically cut off whenever the concentration of methane reached 1.25% and resume only after the concentration dropped to 1% or below.

Although methane was encountered at locations scattering all over the entire Taipei Basin during investigation, the problems, to the author's knowledge, were limited to the Chungho Line during the construction of the rapid transit system. To play safe, whenever methane was found in a drilled hole, relief wells were sunk in the neighborhood of the hole to release methane. Tens of such relief wells were sunk for the purpose. During the drive of down-track tunnel of Contact CC 275 of the Chungho Line, methane was indeed encountered at Rings No. 127, 150, 176, 187 with concentrations varying from 17% to 57%. However, because of all the precautionary measures, its presence did not cause any consequences.

SOFT GROUND CONSTRUCTION IN BANGKOK REGION

Projects

(A) Bangkok-Chonburi New Highway

Traffic congestion is always a major problem in Bangkok and its outskirts. During the recent years, the industrial activities and associated urbanization in the east and southeast of Bangkok have been growing up rapidly. Two ports at Laem Chag and and Map Ta Pud and the future Second International Airport at Nong Ngu Hao are also expected to increase the traffic volume. In order to increase the capacity and carriageways in the regional network, the Department of Highway (DOH) decided to construct its first motorway project – the Bangkok-Chonburi New Highway (BCNH) with fully-controlled access facilities including grade separated interchanges and flyovers. Table 1 summarizes the design criteria of the BCNH.

Table 1 Design Criteria of BCNH

Project Length	81.7 km
Design Standard	Divided 4 lanes
Design Speed Main Road Frontage Road Ramp	 110 km/hr 80 km/hr 40 km/hr
Design Service Life	15 years
Design Elevation	30 cm min above high water level during service life
Traffic Expectation Year 2000 Year 2008	 16,500~39,700 vehicles/day 25,800~61,300 vehicles/day

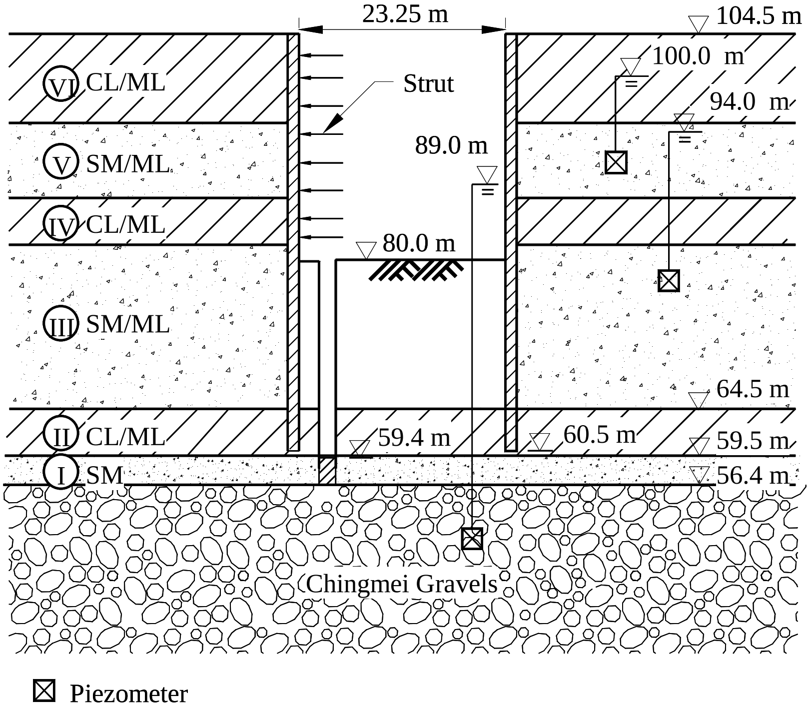


Fig. 18 Soil profile and configuration of excavation

As shown in Fig. 19, the BCNH begins at Sri Nakarin Road connecting the Second Stage Expressway and extends eastward to join Route No. 36, the New Chonburi-Pataya Highway. The total length of the BCNH is 82 km including eight interchanges. The total construction cost of the BCNH was 12,750 million Bahts (about 500 million US dollars), of which 50% was OECF loans from Japan. The construction of the BCNH was commenced in June 1994 and the highway was open to traffic in December 1997.

Almost all of this project route, except about 15 km at the end, is traversing on the flat central plain of Thailand where the underlying soils consist mainly of soft marine clay up to depths of 15 to 20m. Figure 20 shows the soil profile along the project route with related design profile grade. Based on the subsoil properties, the whole project route has been divided into five zones, i.e., Zone 1 to Zone 5. The basic properties of the very soft to soft clay in each zone are summarized in Table 2.

Detailed design of the highway was carried out by a consortium led by Thai Engineering Consultants Co. Moh and Associates was responsible for all the geotechnical related work during construction.

(B) Second Bangkok International Airport

Development of the Second Bangkok International Airport (SBIA) in Thailand has been under planning for more than 20 years in order to meet the foreseeing air traffic growth in the region. This growth is a result of Thailand's dynamic economical and tourism development and will accelerate the SBIA to be the region's key central aviation hub. Master plan and engineering studies for the new airport were carried out in 1984 and the government finally approved the SBIA project in May 1991. 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 SBIA project, which is scheduled to open for operation in 2004 to serve 30 million passengers and 1.46 million tons of cargo each year during the first phase. As a final goal, the SBIA will accommodate up to 100 million passengers and 4.6 million tons of cargo annually. The total construction cost is estimated to be 120 billion Bahts (about 3 billion US dollars) of which over 60% will be used for engineering.

The project site is located at Nong Ngu Hao in Samut Prakan Province, about 30 km east of central Bangkok (refer to Fig. 21). The site is located in a low-lying flood-prone area and covers an area of, approximately, 3,200 hectares with an average elevation of less than one meter above the sea level.

The subsoil conditions are relatively uniform over the entire airport site. The soil profile, as illustrated in Fig. 22, is typical of the deltaic deposit in the central plain of Thailand. The well known Bangkok Clay at the SBIA site consists of relatively distinct layers of weathered crust, up to 1.m thick, followed by 1.5 to 11 m of very soft to soft clay. The soft soil is underlain by a few meters of medium clay and then stiff clay. Below a depth of about 25m, the site is underlain by a dense sand stratum. The layer of very soft to soft clay possesses very high water content, usually over 100%, low shear strength and high compressibility. It is this layer of soil which dictates the design of the airport facilities.

Detailed design of the airside pavement for the SBIA was carried out by the Airport Design Group consisting of DMJM International, Scott-Wilson-Kirkpatrick, Norconsult International, SPAN and SEATEC. Construction supervision of the ground improvement work for the airside pavement is being carried out by a consortium of which MAA Consultants, Co. is a member and is responsible for all geotechnical related work.

Ground Improvement Scheme – Design and Construction

As described in the previous section, the subsoils at the two project sites are typical of the Bangkok Clay deltaic deposit in the central plain of Thailand. The underlying layer of very soft to soft clay with high natural water content, low strength, and high compressibility is the source of many failures or severe damages, including stability problem and large settlement. Undulating pavement surfaces and unstable embankment slopes are common scenes at many highways, particularly the existing Highways No. 3 and No. 34. Extensive research studies have been carried out in the last 20 years to improve the soft Bangkok Clay for large scale construction. Many different methods and schemes have been evaluated either to increase the strength or to accelerate consolidation or both, including preloading with surcharge or vacuum, vertical prefabricated drains (PVD), sand drains, large diameter non-displacement sand drains, cement/lime columns, etc. (e.g., AIT, 1974; Moh and Woo, 1987; Bergado, et al., 1998). Based on these study reports, the use of preloading with PVD appeared to be the most suitable method of ground improvement for the two projects since compression of the Bangkok Clay is dominated by primary consolidation and there were sufficient time available for the compression to take place prior to the target completion date of the projects.

According to available records, sand drains and PVD were used only in small scale projects or on a trial basis in the Bangkok area. The Bangkok-Chonburi New Highway was the first large scale highway project that used PVD to accelerate the consolidation of Bangkok Clay. A total of 23 million linear meters of PVD was installed in the BCNH. Ground improvement for the Airside Pavement, one of the initial projects for the SBIA, was commenced in November, 1997 in order to prepare the site for the construction of permanent structure in future. Besides site clearing and leveling for about 53% (1,699 hectares) of all the airport sites, ground improvement by using PVD and surcharge load is applied to the Airside Pavements including West Runway, Taxiways, Apron, part of East Runway and two access roads to accelerate consolidation settlement before the construction of pavement or the facilities and to minimize maintenance cost after the operation (Fig. 23). The total improvement area is about 10% (308 hectares) of the total airport site. The contract amount for the first phase ground improvement project is 8.24 billion Bahts with 32 million linear meters of PVD installed. The project will be completed in April 2002. Further ground improvement work on the remaining East Runway, Cargo

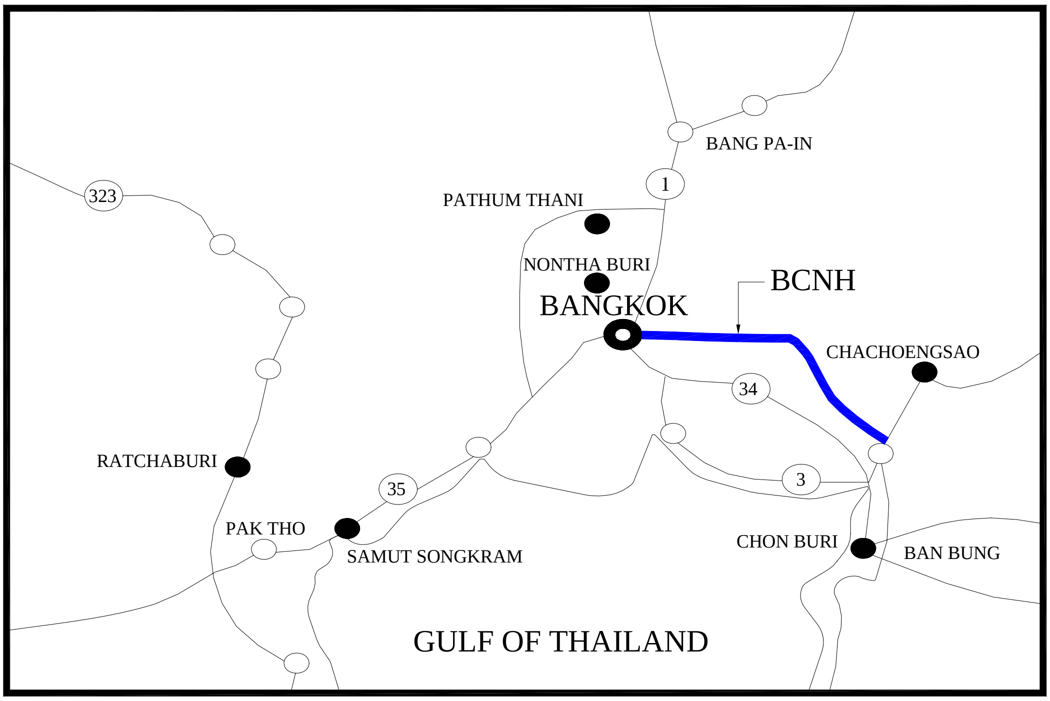


Fig. 19 Location Map of Bangkok-Chonburi New Highway

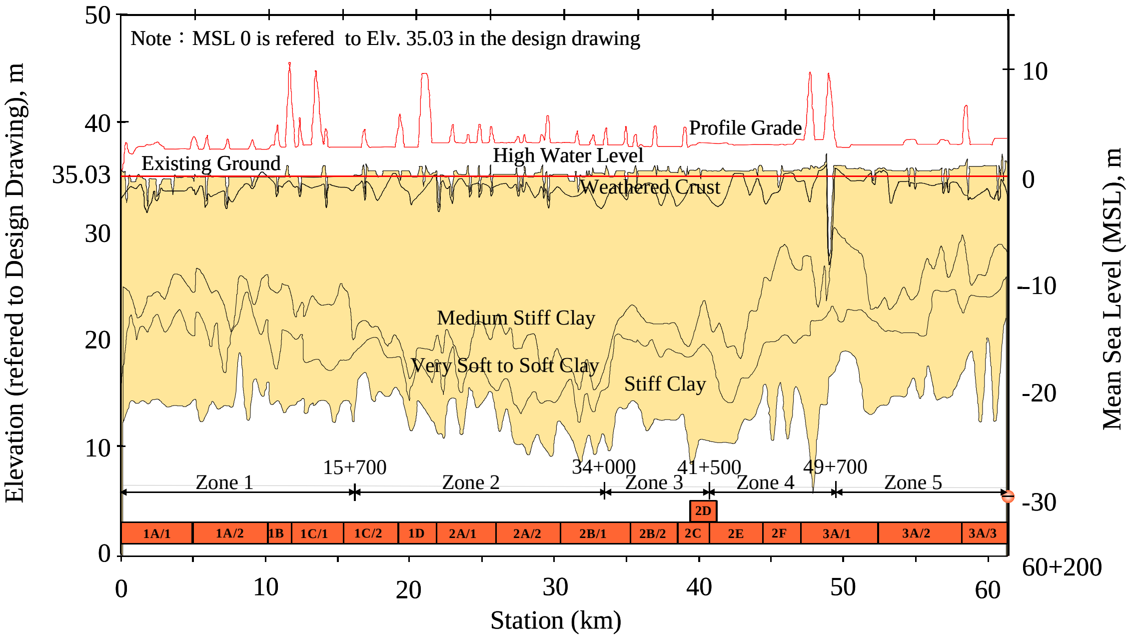


Fig. 20 The soil profile within PVDs' installation area along the BCNH

Table 2 Soil Properties of Soft Clay along BCNH

Section (km)	w (%)	γ_t (t/m^3)	w_L (%)	Ip (%)
Zone 1 (0+000~15+700)	60~120	1.4~1.5	70~140	30~40
Zone 2 (15+700~34+700)	70~160	1.4~1.5	90~140	30~60
Zone 3 (34+000~41+500)	70~120	1.4~1.5	70~120	40~60
Zone 4 (41+500~49+700)	70~90	1.4~1.5	70~90	30~40
Zone 5 (49+700~60+200)	70~120	1.4~1.5	70~100	30~50

Fig. 21 Location Map of the SBIA Site

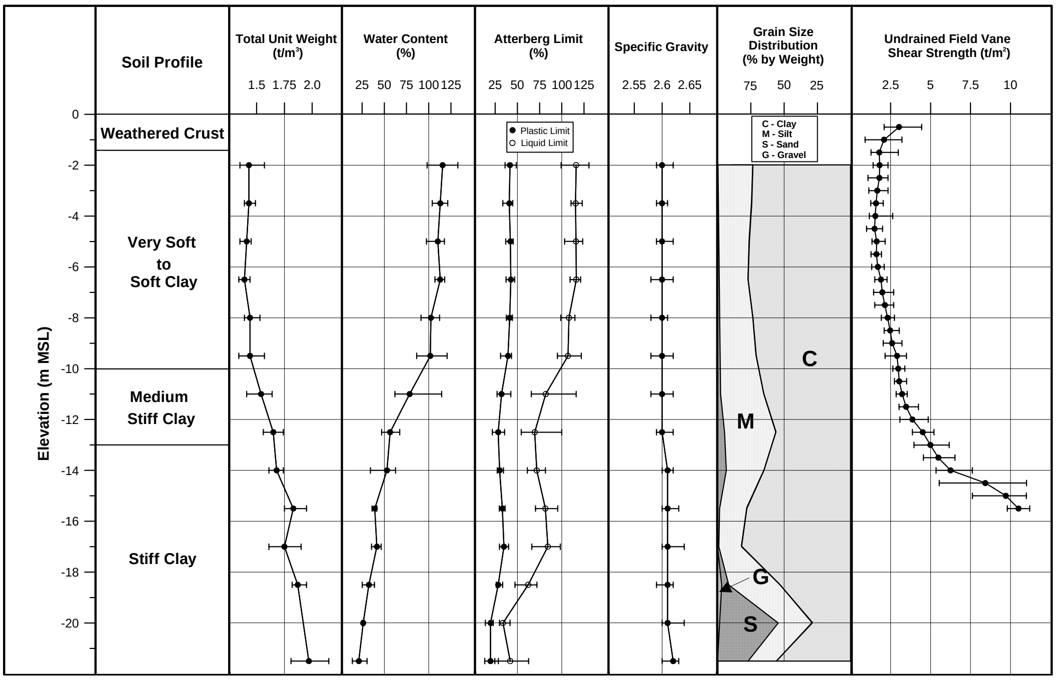


Fig. 22 Typical Soil Profile at SBIA site

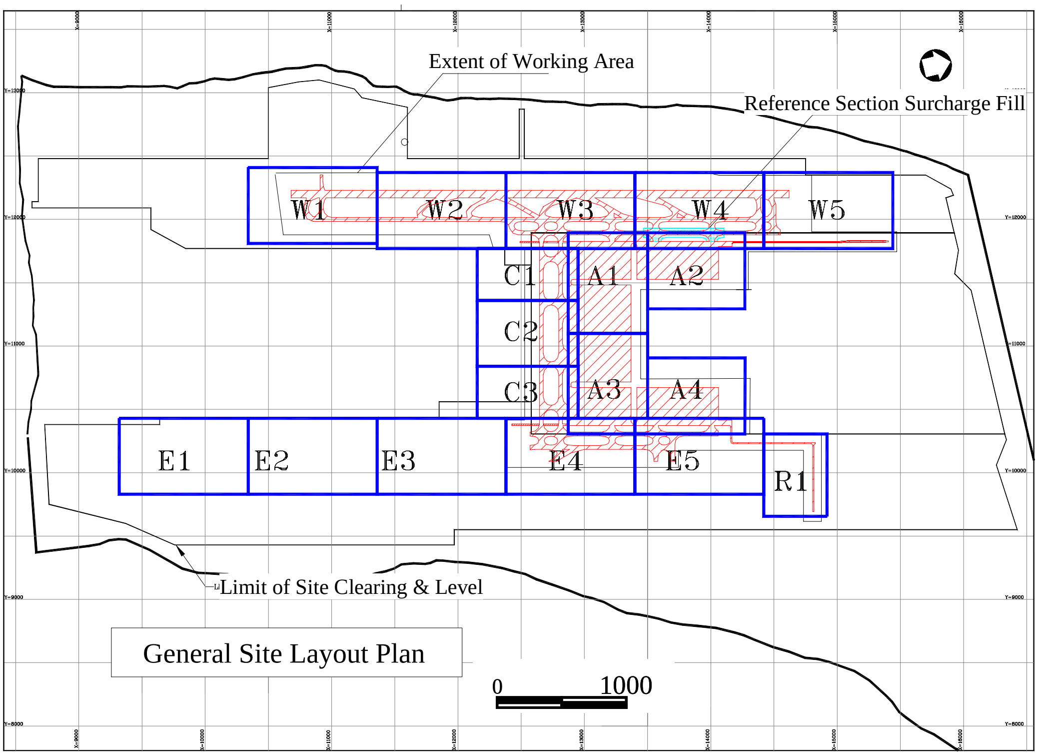


Fig. 23 Layout plan of ground improvement work for Airside Pavements, SBIA

Aprons and Landside Road System will be implemented in 2001 at an estimated cost of about 5 billion Bahts under a separate contract and will not be discussed in this paper.

Due to cost constrain and different design philosophy of the design consultants, details of the design for the two projects were somewhat different. Figure 24 shows typical cross-section of the highway embankment and the airfield pavement. Comparisons of the design parameters for the two projects are summarized in Table 3. It should be noted that in both cases, the ground improvement target has been set at 80% of estimated primary consolidation settlement.

Instrumentations including piezometers, surface settlement plates, deep settlement gauges, inclinometers and lateral movement stakes were installed at controlled sections. For the SBIA, a 500m long “Reference Section” with full instrumentations was constructed prior to the full scale ground improvement work for the purposes of: (1) to confirm the design assumptions and criteria for acceptance of improved ground, (2) to check the contractor’s working methods, and (3) to check the installation procedures and suitability of the instruments used.

The following sections give a brief summary of the extent of the ground improvement work in the two projects. Special discussions regarding problems encountered during construction and counter-measures undertaken are presented. More detailed discussion of the two projects were reported by Moh et al. (1998), Ruenkrairergsa et al. (2001) and Lin et al. (2000)

Performance of Ground Improvement

(A) Settlement

For the BCNH, settlement values along the entire route corresponded well with the thickness of the soft clay with the highest value occurring at sections of Zone 2 as depicted in Fig. 25. More than 250 cm of settlement was recorded at Stations 2A/1, 2A/2 and 2B/1. Figure 26 shows typical settlement profiles and lateral movement of the foundation soils. The observed vertical and lateral movement with time are illustrated in Fig. 27.

Since the field measured settlement values appeared to be much higher than the value originally predicted by the designer, Asaoka’s method (Asaokia, 1978) was used to predict the ultimate consolidation settlement by use of field monitored data in order to determine the 80% degree of consolidation criterion for removing surcharge load. The results are shown in Table 4. Post construction survey one year after the completion of construction at Section 2A/1 between Sta 24+800 and Sta 25+400 indicated additional settlement of 3.6 to 5.4cm (JICA, 2000). Adding these values to the observed settlement at the end of construction, 275.4 cm (average), it is still less than the total settlement value estimated by Asaoka’s method. Judging from the settlement-time curve, the highway appears to be continuing to settle, but at a slower rate.

For the SBIA project, comparison of settlement among field performance of the reference section, design estimation and data from AIT test embankment (AIT, 1995) are shown in Fig. 28. In Fig. 29, typical settlement and lateral movement profiles are shown. The field measured settlement, both the amount and trend of variation with time match well with that predicted. Comparing with the data in the highway project, the low predicated settlement values in the latter case could partly be contributed to the larger lateral movement of the subsoil, with maximum in the order of 500mm as comparing to 120mm in the SBIA case.

The effectiveness of PVD in accelerating consolidation of the Bangkok subsoils is well illustrated in Fig. 30. There was a difference of 50cm in settlement between PVD treated area and area without PVD one month after the second stage loading.

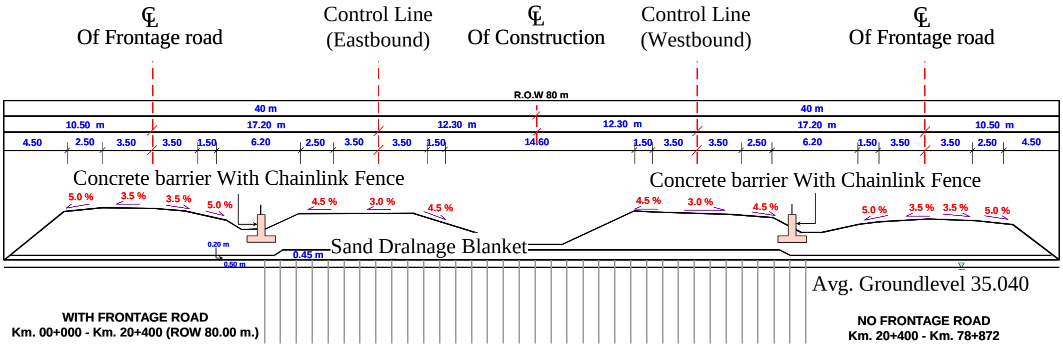
(B) Stability

Stability of embankment during the ground improvement period was one of the major considerations in controlling the rate of surcharge loading as well as removal of surcharge in both projects. Instrumentations have played an important role in the safety control. Besides a regular program, monitoring frequency of the instruments were adjusted according to the construction activity, to the rate of which the readings are changing, and to the requirements of data interpretation.

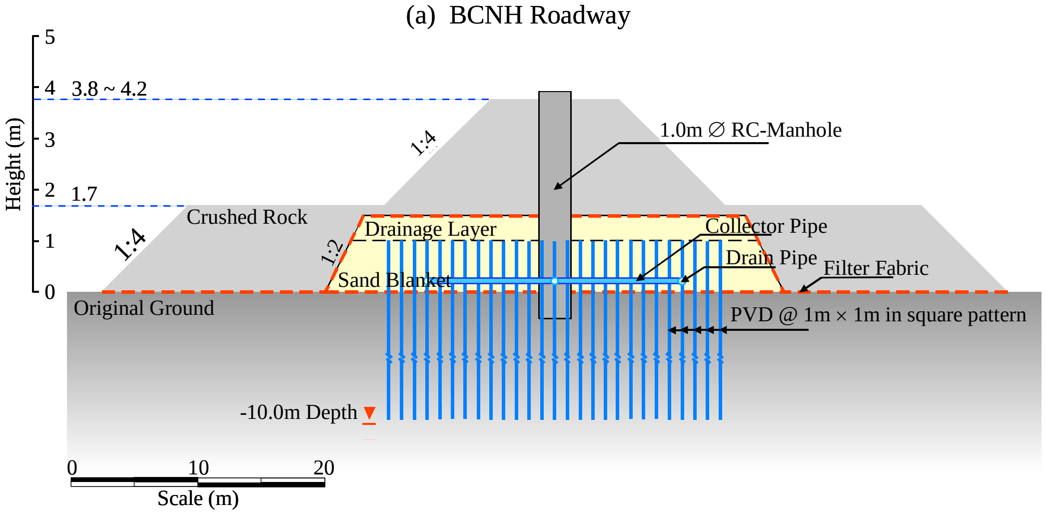
In the design, the calculated total settlement consisted of consolidation settlement only, without considering the effect of shear strains caused by lateral displacement of the soft subsoils. Ladd (1991) has reported that the ratio of maximum lateral movement to maximum settlement for embankment fill on normally consolidated clay could be as high as 0.9±0.2 in the initial stage of loading and drops to 0.16±0.07 when consolidation settlement predominates. A value of 0.33 was adopted as the criteria for stability control in the two projects. Special attention was however given to the control of construction rate (i.e., loading by fill) when the ratio reached 0.25. For the BCNH, the amount of lateral movement of the soft subsoil was quite large and the value is closely related to the thickness of the soft clay, as shown in Table 5.

(C) Changes in Soil Properties

In general, consolidation of soil should cause change in soil properties, including decrease in water content, void ratio and permeability and increase in dry unit weight and shear strength. Significant improvement in the properties of the subsoils were achieved in both projects due to the preconsolidation by using ground improvement. Figures 31 and 32 show comparison of soil properties before and after ground improvement work. The figures clearly indicate that in both projects there were significant decrease in the water content and increase in undrained shear strength. These changes of properties became very small beyond the depth of the PVD. Figure 31 further shows that the effect of PVD was most obvious during the first year, maybe within even shorter period, after the ground improvement work was started



(a) BCNH Roadway



(b) SBIA Airside Pavements

Fig. 24 Typical cross-sections of ground improvement

Table 3 Comparison of Ground Improvement Work between BCNH and SBIA

	BCNH	SBIA
Average Thickness of Soft Clay	8~12m	9~11m
Embankment Fill Thickness	2.55~3.00m	3.80~4.20m
Thickness of Drainage Sand Layer	40 cm (PVD portion) 20 cm (berm)	1.5m
Filter Fabric	none	2 layers
PVD Length	6m ~ 12m	10m
Preloading Material	sand	crushed rock
Surcharge Load	50~60 kPa	75 kPa & 85 kPa
Stage Loading	3 stages	2 stages
Required Waiting Period after Final Stage Loading	9 months	6 & 11 months
Required Settlement Rate before Surcharge Removing	≦2 cm/month	≦4% or 2% of settlement ratio
Required Percentage of Primary Consolidation	80%	80%

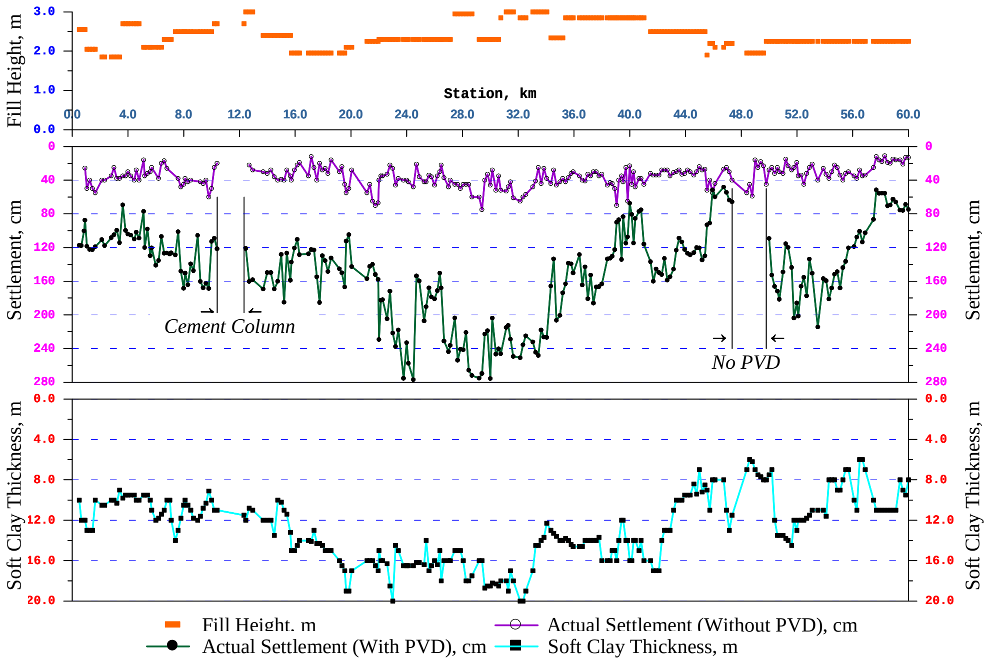


Fig. 25 Settlement profile (Up to Sept, '97) with fill height and soft clay thickness

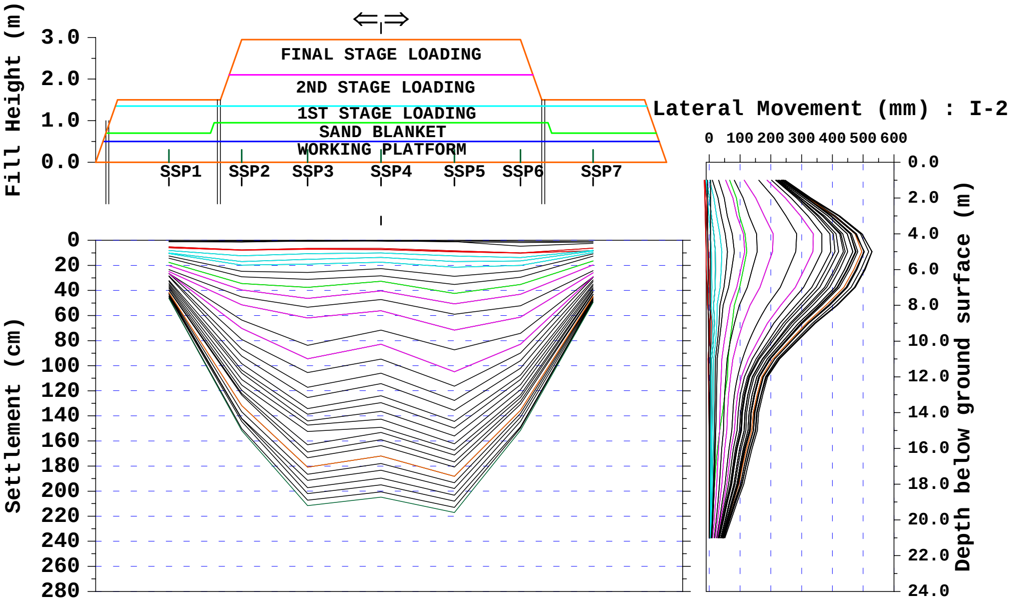


Fig. 26 Vertical and horizontal movements at Section 2A/2 (Sta. 28+200), BCHN

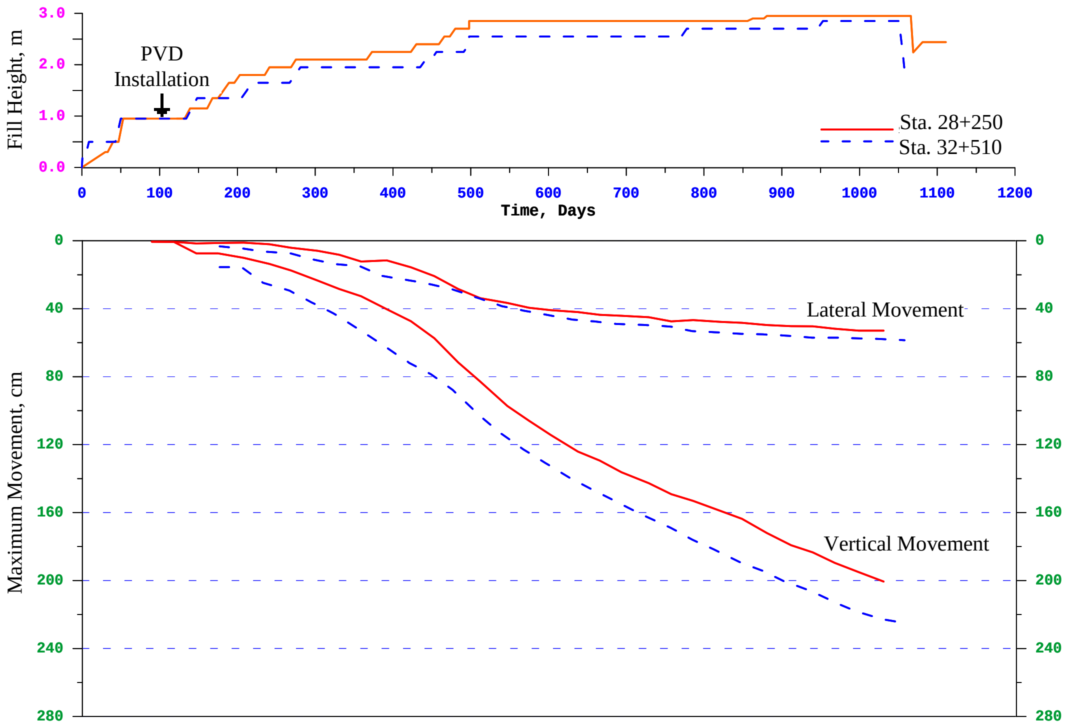


Fig. 27 Observed vertical and lateral movements vs. time with fill height at sta.28+250 and 32+510, BCNH

Table 4 Comparison of Field Measured Settlements with Estimated Values

Section	Station	Waiting Period After Final Stag		Design Settlement, cm	Measured Maximum Settlement, cm	Asaoka Estimated Total Settlement, cm
		Planned	Actual			
		Month	Month			
1-A/1	0+000~5+100	9~12	under construction	124~150	122.50	160.0
1-A/2	5+100~10+100		12~14		168.00	178.0
1-B	10+100~12+400		under construction	140~160	106.70	130.0
1-C/1	12+400~15+700		10~13	152~185	185.00	190.0
1-C/2	15+700~19+600		12~14		173.70	210.0
1-D	19+600~22+000		9~11	152~169	169.20	190.0
2-A/1	22+000~26+400		14~15	170~183	275.40	285.0
2-A/2	26+400~30+700		15~17		272.70	290.0
2-B/1	30+700~35+200		15~17	155~165	260.00	300.0
2-B/2	35+200~38+600		12~15		186.10	230.0
2-C	38+600~41+500		9~13	100~117	134.20	160.0
2-D	0+000~1+000		12~13	118~126	166.10	170.0
2-E	41+500~15+450 (main road)		9~13	95~135	160.20	165.0
	1+000~2+880 (access road)				182.30	195.0
2-F	45+450~47+675		10~11	96~106	106.70	110.0
3-A/1	47+675~52+000		12~13	114~214	181.50	210.0
3-A/2	52+000~57+600		10~11		201.50	215.0
3-A/3	57+600~64+500		9		76.30	80.0

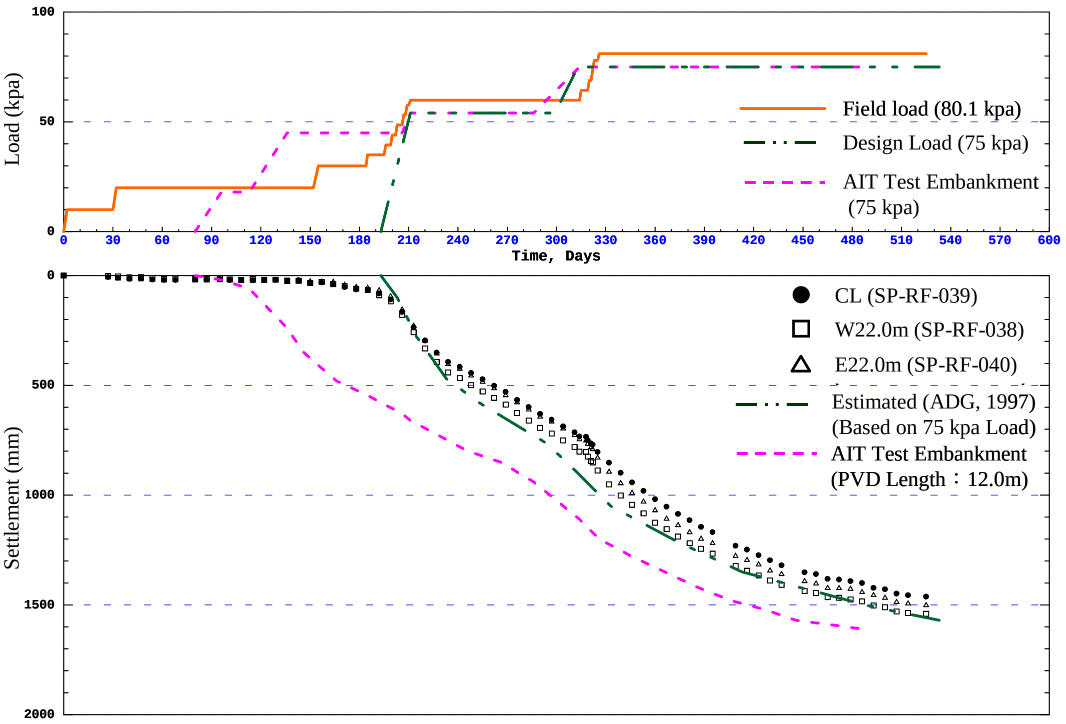


Fig. 28 Settlement comparison among the SBIA Reference Section, design prediction and AIT test embankment

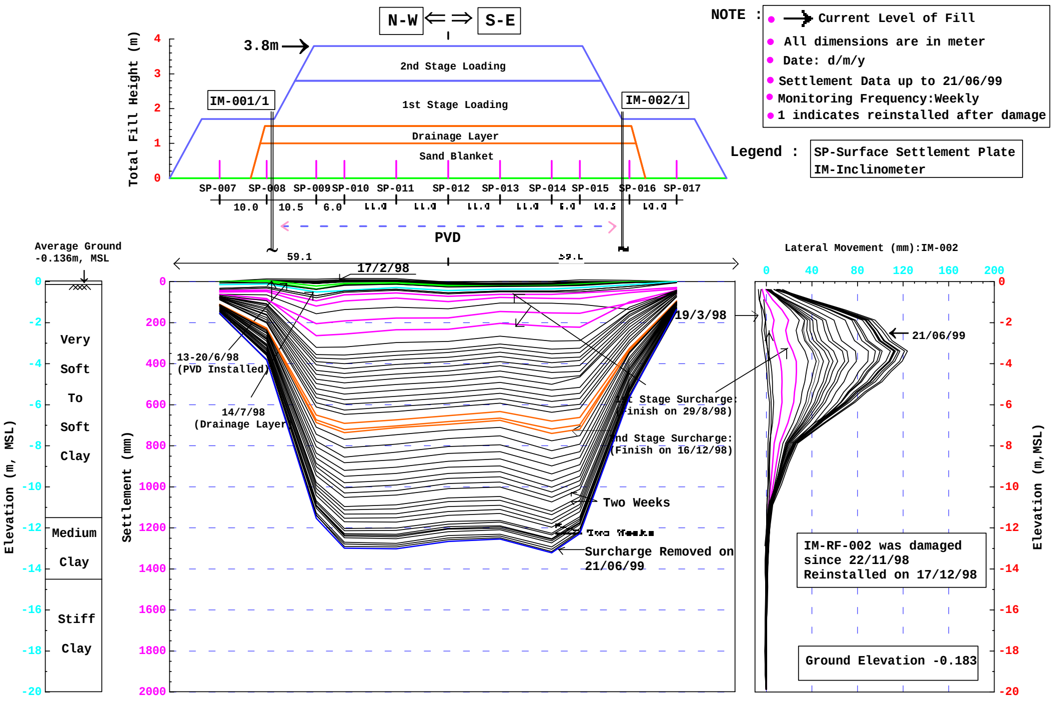


Fig. 29 Profile of settlement and lateral movement at SBIA Reference Section

Table 5 Summary of Observed Maximum Settlement and Maximum Lateral Movement in BCNH

Zone	Thickness of Soft Clay, m	Actual Preloading, m	Design Settlement cm	Observed max. Settlement cm	Observed max. Lateral movement cm
1	4.3~12.5	2.70~3.00	124~185	122~185	9.5~22.7
2	11.7~18.0	2.85~3.00	152~185	169~275	15.1~58.6
3	10.9~14.8	2.85	100~165	134~186	33.3~34.2
4	7.0~16.4	2.55~2.85	95~135	107~182	13.1~34.4
5	5.5~10.2	2.55~2.70	104~214	76~181	13.5~27.8

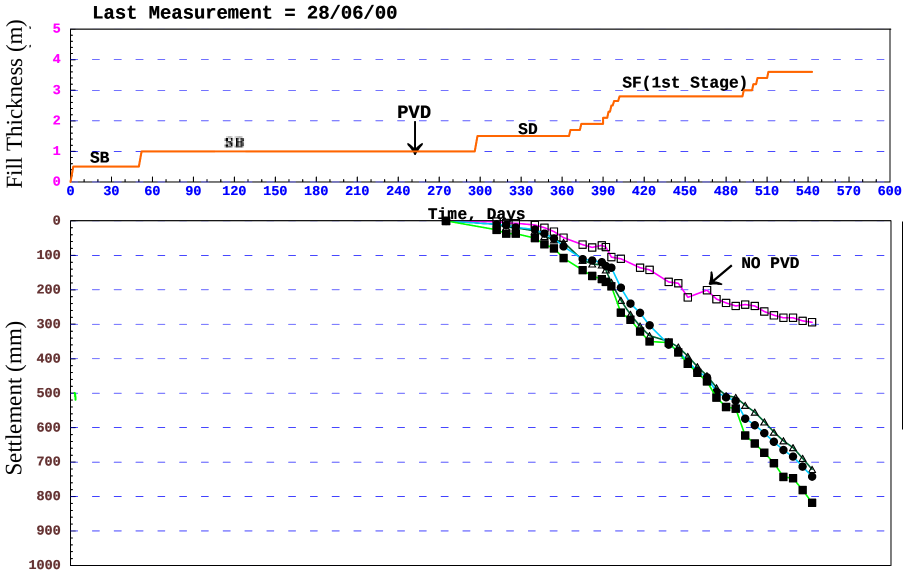


Fig. 30 The comparison of settlements in areas with PVD and without PVD

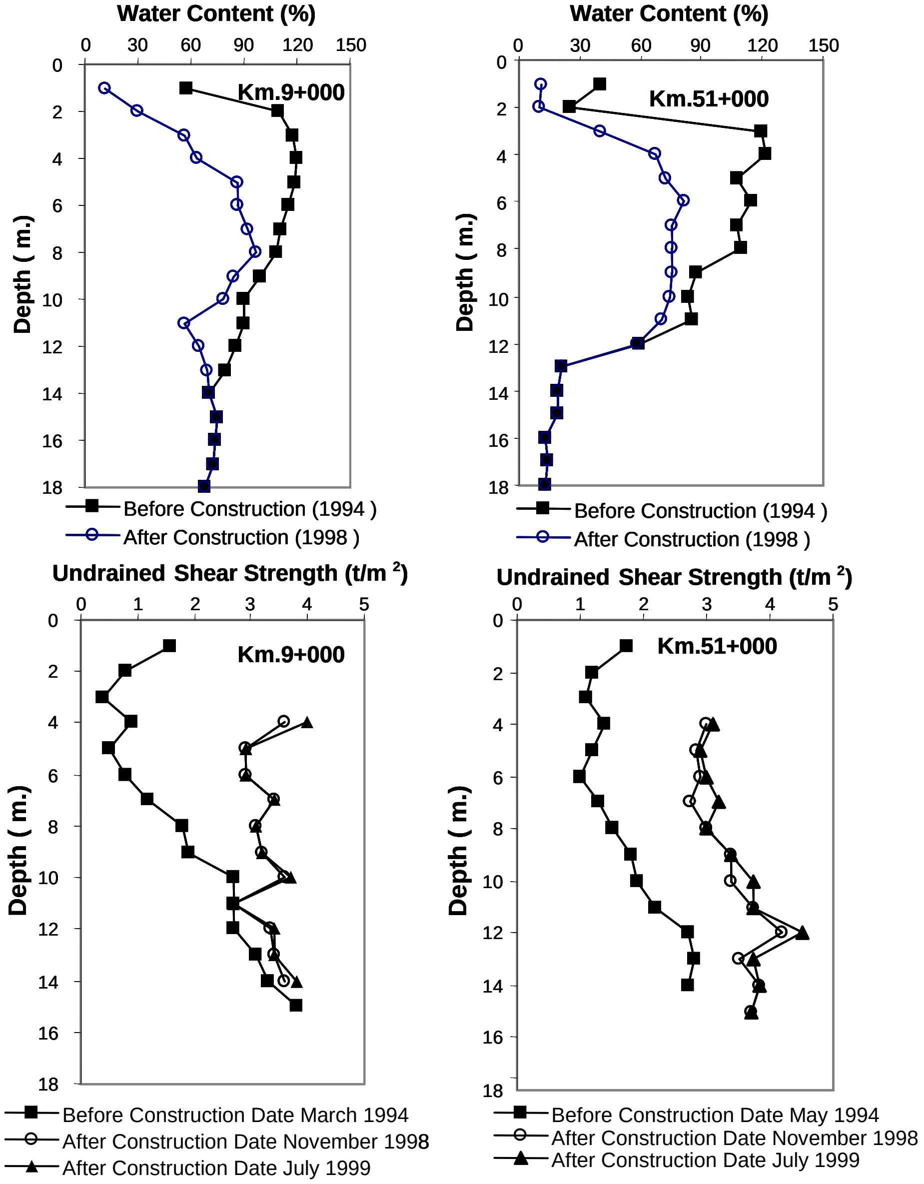


Fig. 31 Soil Properties before and after ground improvement in BCNH

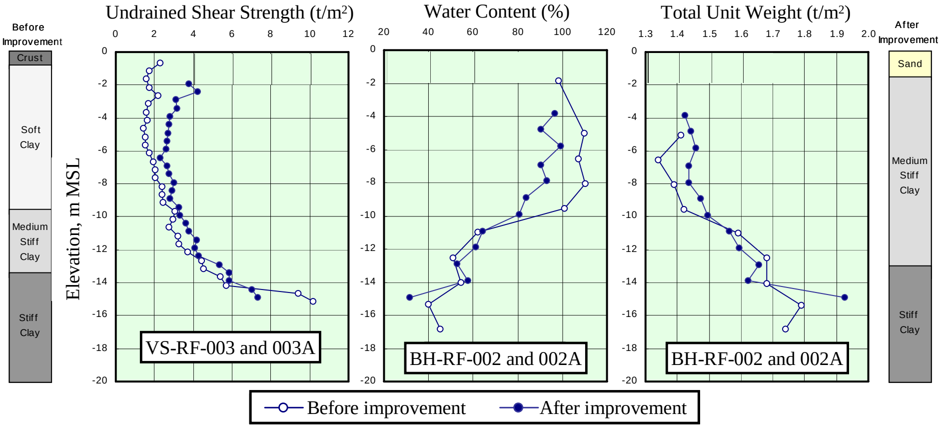


Fig. 32 Soil properties before and after ground improvement at SBIA

Problems Encountered and Countermeasures

(A) Inadequate Under-drainage

In the original design of the ground improvement work for the BCNH, a total of 45 cm thick sand drainage layer was placed above a 50cm thick sand-filled working platform over the original ground. There was no other under-drainage facilities. During the initial review of the design prior to undertaking the construction work, the supervising consultant has noticed this problem since the amount of settlement under the embankment load would be very large, the sand drainage layer together with the working platform would sink into the ground and shear off at junctions between the main embankment and the berm where no PVD were installed. This phenomena would impede proper flow of water discharged from the PVD to the sand drainage layer. High excess porewater pressure and slow rate of dissipation were indeed observed after the second stage loading. Additional drainage trenches traversing the side slope of the road embankment and pumping wells were installed to accelerate the groundwater dissipation, Fig. 33 (Moh, et al., 1998). This problem did not occur at the SBIA site because the designer had incorporated adequate underdrainage facilities including filter fabtric, sub-drainage pipes, collector pipes and manholes in addition to the sand drainage layer (Lin et al., 2000)

(B) Cracks in Embankment

Due to consideration of cost, PVD were installed only under the main embankments in both projects. Large differential settlements occurred between the embankment section with PVD and the berms where no PVD were installed. This led to development of cracks along the boundaries of the two sections and sometimes even along centerline of the main embankment. In order to reduce potential of stability failure, actions were taken during the preloading period including reduction of preloading height, extending counterweight berms and improving under-drainage for relieving the excess porewater pressure. In fact, all these measures either prolonged the waiting period or directly increased the construction cost. It is advisable to install PVDs under the berms as well but the depth of installation could gradually decrease from the main embankment towards the toe of the berm.

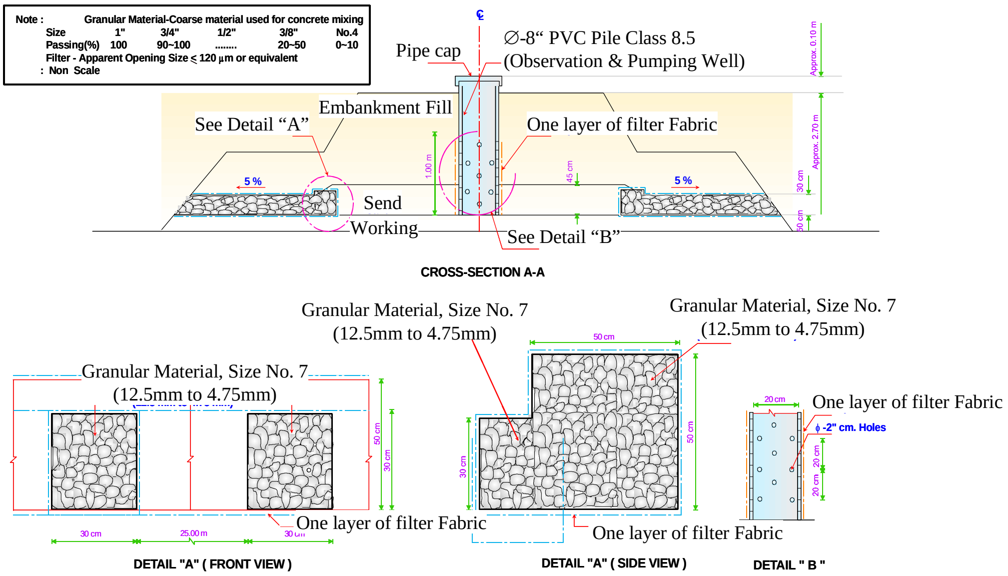


Fig. 33 Typical Cross – Section of additional trenches and pumping wells in BCNH

(C) Embankment Erosion during Raining Season

In Thailand, heavy rains often occurred during raining seasons. When sand or clay were used as the embankment fill, special protections of the side slopes of the embankments against erosion are needed. Top soils are commonly used for this purpose. However, precautions must be exercised in placement of the top soil on the side slope so that it would not block the drainage outlet for the dissipated water to flow out. Blocking of water outlets actually accounted for several cases of failure or unsuccessful of ground improvement with PVD.

(D) Problems with Right of Way

In the BCNH project many road embankments were bounded by fishponds and canals with limited right-of-way resulting in inadequate counter-weight berms. Local slope failures occurred at a number of places. To counter this problem, special attention was given to the rate of fill by observing the ratio of lateral movements to vertical settlement. At a few locations, other stabilizing measures such as cement/lime columns were installed at the toe to enhance the stability of the embankment.

CONCLUSIONS

Geotechnical engineering plays a critically important role in any infrastructure development, in terms of safety and economy. Due to the vast complexity of materials and techniques involved, geotechnical engineering up to the present time, is still not an exact science. It is a combination of engineering principles with observed experience and sound engineering judgment. The four case records reported herein clearly indicate several important factors in carrying out a geotechnical design in addition to the normal considerations in an engineering design. They include adequate and reliable subsurface information, appropriate selection of analysis principles and construction methodologies/details, ability to cope with variations in ground conditions, and timely interpretation of field performance data.

A good analogy is comparing geotechnical engineering to the medical practice, as shown in Table 6. It is appropriate to say that a geotechnical engineer is not just to compute numbers accurately but to make judgment soundly.

ACKNOWLEDGEMENTS

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Table 6 Comparison between Geotechnical Engineer and Medical Practitioner

Geotechnical Engineer	Medical Practitioner
Understand project	Understand illness
Reconnaissance and data collection	Medical history and background
Site investigation & testing	Physical examination and tests
Soil, rock mechanics, engineering geology	Physiology
Experience	Experience
Simplifications, analyses, models	Diagnosis
Recommendation and design	Prescription
Monitoring, comparing and modifying	Observation and further examination

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