GEOTECHNICAL ISSUES OF SOME MASOR INFRASTRUCTURE DEVELOPMENTS IN TAIWAN

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1. INTRODUCTION

Geotechnical engineering plays a vital role in construction of any major infrastructure projects, particularly when the constructions are located in areas with soft ground conditions. The challenge in solving various types of problem enables geotechnical engineers to accumulate experience and sharpen their skills, also offers geotechnical engineers opportunities to contribute their knowledge and wisdom.

This paper illustrates the importance of geotechnical engineering in construction of three major infrastructure projects recently carried out in Taiwan. They include the Taipei Rapid Transit System (TRTS), Underground Tunnel Under An Airport, and the Taiwan High Speed Railway.

2. 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 the 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 15 billion US dollars).

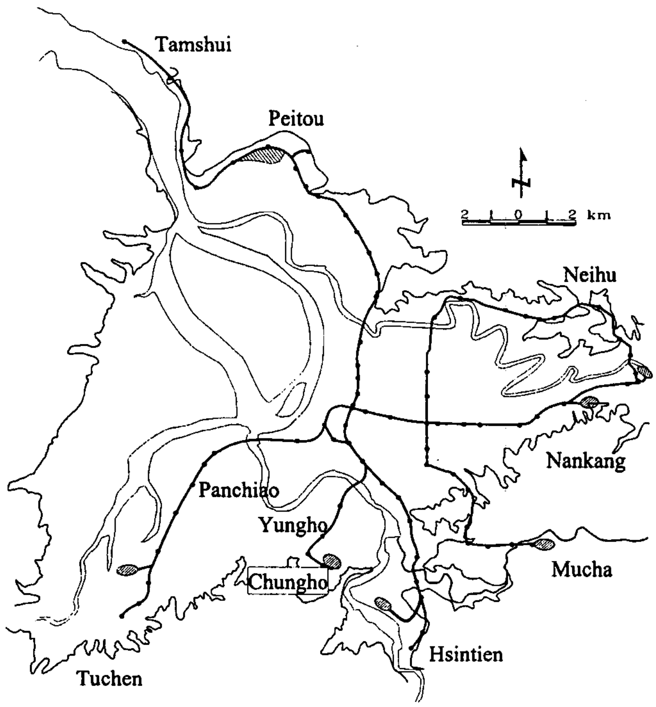


Fig. 1 Initial Network of Taipei Rapid Transit Systems

For the underground construction, about 45 km of diaphragm walls, with thickness varying from 80cm to 120cm, were constructed in cut-and-cover excavations whilst bored tunnels have a total route length of 22km.

Because this was the first rapid transit system constructed in Taiwan, the Department of Rapid Transit Systems (DORTS) of the Taipei Municipal Government engaged a Geotechnical Engineering Specialty Consultant (GESC) to assist in the design review and construction supervision right at the beginning of the project. This has been proved very fruitful as the design was optimized and many potential problems avoided. At the peak of construction, a total of 42 field stations were setup and managed by the GESC to assist the field staff of the DORTS in solving on-site geotechnical problems. This also enabled high quality geological and instrument data to be obtained to facilitate back-analysis for verifying the designs and the design assumptions (Moh and Hwang, 1996). A Data Center was established at the headquarters of the GESC to process the tremendous amount of field data in a systematic manner.

In this paper, some of the geotechnical issues associated with the design and construction of the Initial Network are presented. Discussions are made on ground characterization, groundwater problems, and earth retaining structures for cut-and-cover construction. More detailed discussions can be found in Moh and Hwang (1999).

2.1 Ground Characterization

In modern cities, in which rapid transit systems are to be constructed, usually numerous boreholes have already been sunk, for example, for the construction of foundations and basements of high-rise buildings prior to the implementation of the program. It is also quite common nowadays for city governments to compile borehole data into databases and these databases can be utilized in the preliminary assessment of construction methods. Figure 2 shows a geological zonation map of the Taipei Basin (Lee, 1996). Two general soil profiles, one in east-west direction and the other in north-south direction across the Taipei Basin are shown in Fig. 3. As can been seen, there exist a thick layer of young sediments, i.e. the so-called Sungshan Formation underlain by a stratum of gravels of various sizes. This gravel layer, the so-called Chingmei Gravel is water bearing and extremely permeable.

The boreholes drilled for building construction may not be sufficient in either quantity or quality for the design and construction of rapid transit systems. First of all, excavations for rapid transit systems are usually far deeper than for building basement. Secondly, the previous investigations generally lacked a unified quality control and may lead designers astray. Once a

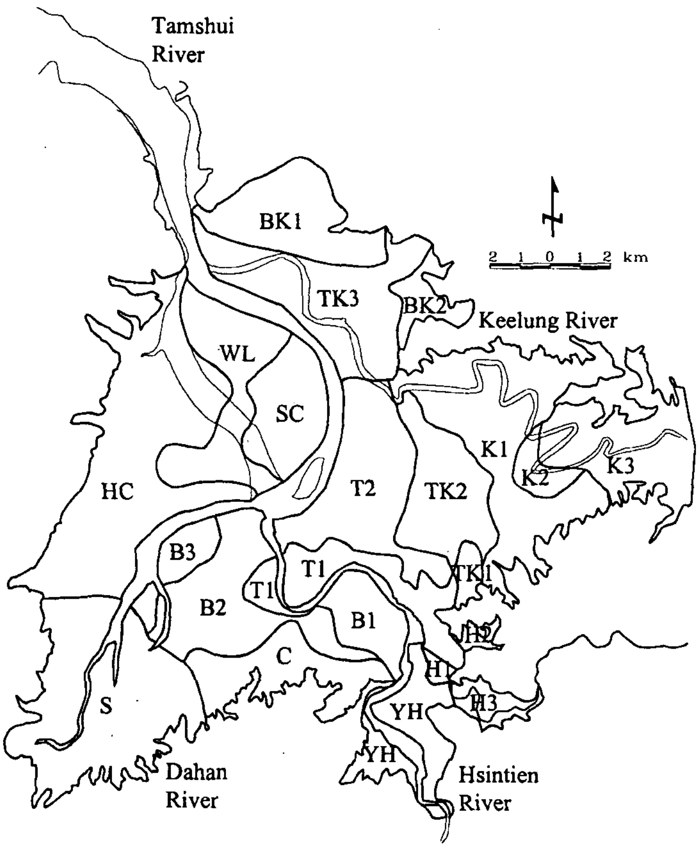


Fig. 2 Geological zonation of the Taipei Basin

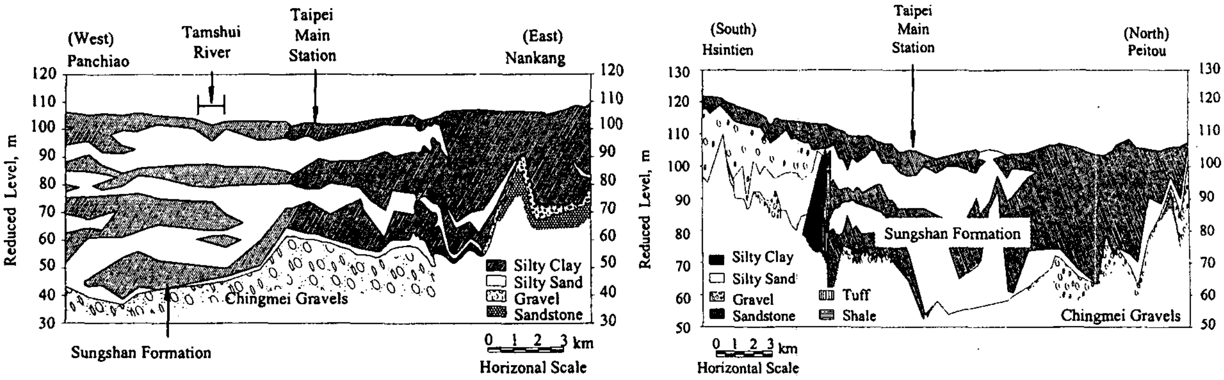


Fig. 3 Geological Profiles of the Taipei Basin

route is decided, it is a normal practice for the authority in-charge of the program to engage a qualified geotechnical consultant in the planning stage to compile all the available information to a consistent format and to perform additional investigations for calibrating such information on a unified basis. The designers awarded the job have to perform their own investigation to satisfy themselves subsequently and contractors will have to do the same.

There are no rules regarding the quantities of boreholes and tests to be performed in each phase of the investigation. For a system with stations and tunnels buried in uniform ground of the same geological formation, relatively low number of boreholes may be sufficient for structural design. However, additional boreholes may be required to optimize the design of retaining walls or pile foundations, or where blow-in and piping are potential threats.

For routes running through varying and complex geological formations, investigation have to be more detailed and sophisticated in-site and laboratory tests are sometimes needed to enable tenderers to judge the situation with a better accuracy and to reduce potential claims from contractors. The spending invested in soil investigations will be paid off as potential problems are eliminated beforehand. Unfortunately, such a viewpoint is seldom shared by the clients till it is too late.

It is a common clause in all the tender documents that the tenderers of a construction contract shall satisfy themselves by performing additional soil investigations before submitting their tenders. However, very few tenderers would do so not only because of the unwillingness in spending the money but also because of the short time generally available for preparation of the tenders.

During constructions, boreholes are often sunk and tests are performed by contractors for revealing ground conditions with a better accuracy. However, it should be pointed out that investigations performed by contractors during construction are usually, although many times

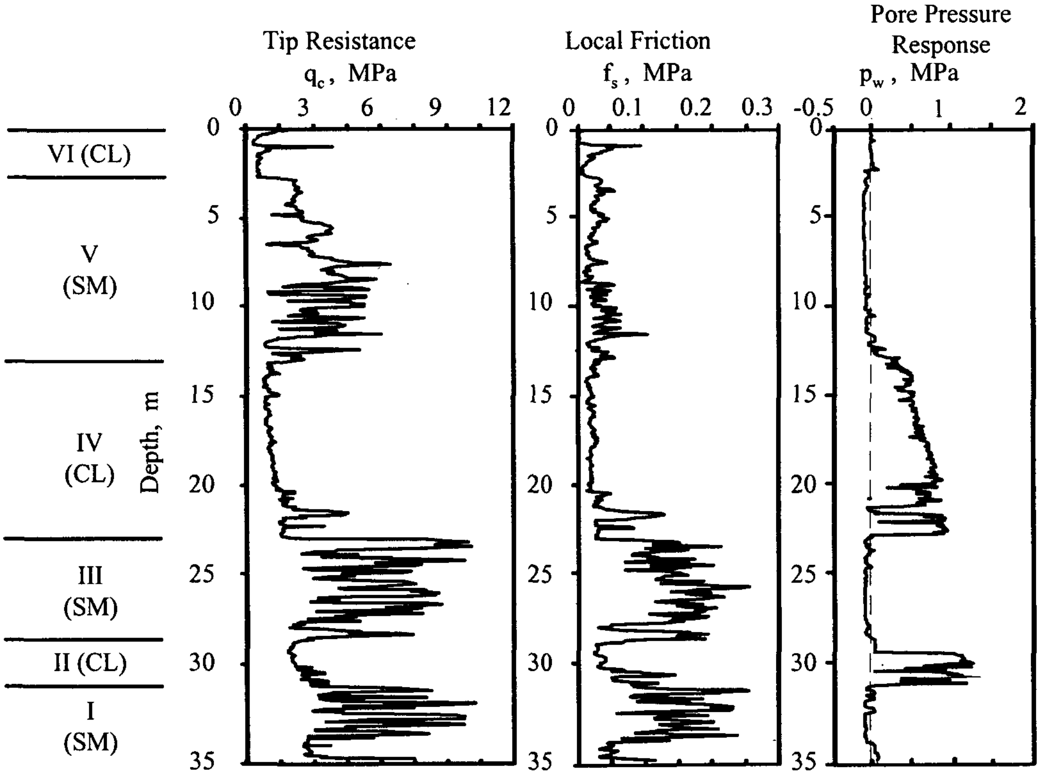


Fig. 4 CPT Profile in Central Taipei

greater in quantity, but often much less in sophistication than the investigations carried out by the designers or specialist consultants.

Soil stratigraphy is conventionally determined by boring and standard penetration tests. However, cone penetration test (CPT) is gaining its popularity as a major tool for determing soil stratigraphy in soft ground primarily because of its consistency in results and also because of the fact that a continuous profile can be obtained, as illustrated in Fig. 4. Modern CPT apparatus allow the measurements of pore water pressures induced at the tip of the cone as the cone advances. This enables fine seams to be identified along the depth. Although CPT is considered as an excellent tool in characterizing soil stratigraphy, it should be noted that interpretation of the results requires local knowledge.

2.2 Groundwater Conditions

Without any doubts, groundwater conditions have dominating effects on the designs and constructions of temporary and permanent underground structures. Experience does indicate that majority of failures occurred during underground construction were directly related to

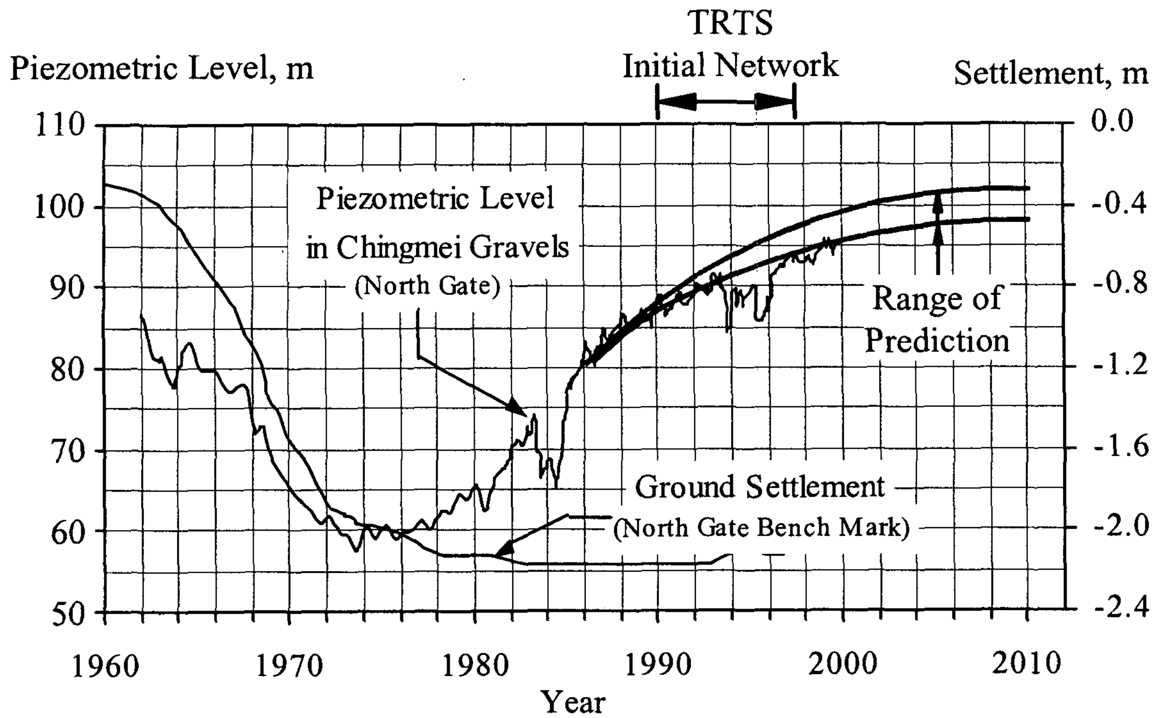


Fig. 5 Piezometric Level in the Chingmei Gravels Formation and Historical Ground Settlement in Central Taipei

water problems and some of them were disastrous and even fatal. To provide background information to enable designers to arrive at optimum designs, groundwater movements have to be monitored for a sufficient length of time before design starts. This is particularly important in cities where groundwater is known to experience large drawdowns, for example, Taipei and Bangkok. In the Taipei Basin, historical records on groundwater movements are available for the last 50 years or so. As can be noted from Fig. 5 (Wu, 1967; 1968), the piesometric level in the Chingmei Gravels was once lowered by as much as 40m due to excessive pumping. The use of groundwater as water supply was banned in the late 60's and the piezometric level in the Chingmei Formation gradually recovered subsequently.

Lowering of piezometric levels has been found to be beneficial to underground constructions. In the first place, it reduces active pressures and increases passive resistance on retaining systems Secondly, the ground in the central city area had settled by 2.2m prior to the construction of the TRTS and the preconsolidation effects greatly reduced consolidation settlement of soft ground during construction and, hence, reduced damaging potential to buildings nearby. To obtain data on the groundwater conditions in the Sungshan Formation along the routes of all the six lines in the Initial Network, GESC carried out monthly monitoring of piezometric heads at 798 locations, including 287 observation wells and 511 piezometers, since October 1987 till May 1993 when the duty was assumed be the DORTS.It was found that, as illustrated by Fig. 6, over the entire

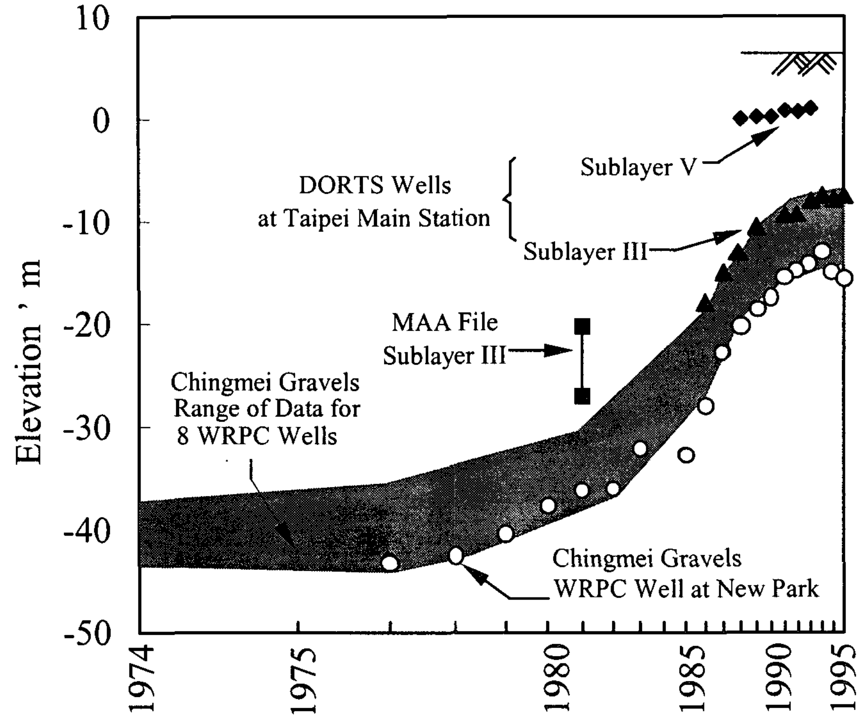


Fig. 6 Piezometric Pressures in the Sungshan Formation

basin, Sublayers III and V are indeed separated by two aquitards, i.e. Sublayers II and IV, and become two aquifers.

2.3 Retaining Systems for Cut-And-Cover Constructions

Thick diaphragm walls are usually chosen for maintaining the stability of the sidewalls of deep excavations in soft ground. Because of the poor ground conditions and the great depths of excavations, diaphragm walls were exclusively used for station excavations for the TRTS. For shallow excavations, for example, at entrances, contiguous bored piles and sheet piles were sometimes used.

There are basically three types of diaphragm wall: a) single wall, b) double wall, and c) composite wall. Due to concern on the water-proofing quality of diaphragm walls, single wall system is gradually phased out and double wall system is more used in construction of rapid transit systems. In a few stations in the TRTS, composite wall system, in which the permanent wall is structurally connected to the diaphragm wall by dowels, is adopted. This reduces the thickness of the permanent wall and saves some space because the diaphragm wall and the permanent wall form a single structural element. However, due to difficulties introduced by the dowels, which hinder the tremieing of concrete and require manual work for bending, composite wall system is to be discouraged.

For the structural design of retaining systems, the use of Peck's apparent earth pressure diagram is still a popular procedure. As pointed out by Prof. Peck that (Peck, 1974)

> "It has been found that, even in a single open cut in which the work has been executed in expert fashion, the loads in equally spaced struts at a given level vary over a wide range and, correspondingly, the pressure diagrams for struts in various vertical profiles differ from each other. Since it is not possible to predict which of apparently identically situated struts will experience the greatest loads, conservative use of the empirically derived pressure diagrams for design requires that each strut be proportioned as if it would be subjected to the maximum load indicated by any of the pressure diagrams. Hence, for design of struts, it is appropriate, to use a pressure envelop that encloses all the pressure diagrams derived from observations."

Peck's diagram was first published in the year of 1943. Fifty years since then, this situation remains unchanged. Although computer analyses are claimed to be able to handle complex ground conditions, variable construction procedures and different structural configurations, they are unable to produce the ranges of strut loads observed in field. Strut loads appear to be affected by minor construction details and environment factors which are far beyond the designers control.

It has to be admitted that, however, computer analysis does represent the state-of-the-art technology and, as time goes by, will eventually become a reliable design tool. The increasing popularity in the use of computer analyses for analyzing retaining structures, however, leads to the worry that inexperienced engineers may blindly rely on the results of computer analyses for designing structural elements. An even more serious worry is that some structural engineers may perform computer analyses using soil parameters from textbooks and commercial software packages, of which the use is subject to limitations, and design the walls without consulting a geotechnical engineer.

Studies on the TRTS are undergoing to see if Peck's diagram is applicable to the Sungshan Formation. Early findings are inconclusive for the reasons that the retaining systems were designed on the principle of limiting wall deflections so the ground settlements behind the wall would not be detrimental to structures adjacent to the excavation. Furthermore, all the struts were pre-loaded to at least 50% of their design loads. In other words, the designs are "stiff", as opposed to the "soft" design adopted in the case histories considered in developing the Peck's diagram. In such a case, soils would behave quite differently from what was assumed and it is doubtful that the diagram will be suitable without certain modifications.

It should also be pointed out that lateral earth pressure on a wall is a function of lateral deformation of the wall, wall friction, as well as wall settlement. Interpretation of earth pressure measurements recorded by earth pressure cells are further complicated by the effects of installation method and disturbance, and possible arching effect of the soil in contact with the pressure cells. Much judgments based on experience are often required.

2.4 Groundwater Control

Blow-in and piping are serious concerns in deep excavation wherever the Chingmei Gravels exists and provisions had to be made for excavations to be carried out safely. For example, excavations for constructing the three ventilation shafts of TRTS were carried out to depths exceeding 30m. These excavations could not be carried out safely unless special measures were taken to resist the groundwater pressures. For Ventilation Shaft A in the Panchiao Line shown in Fig.7, continuous pumping had to be carried out to reduce the piezometric level in the Chingmei Gravels 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 4,170cu m per hr. For constructing Ventilation Shaft B in the same line, refer to Fig. 8, pumping was carried out at a rate of 3,600cmh to lower the piezometric level in the Chingmei Gravels from RL. 89m to RL. 79.5m. However, groundwater drawdown has serious effect on ground settlement of the surrounding areas. As shown in Fig.9, the effect of groundwater drawdown for the above two cases extended over a distance of 5km.

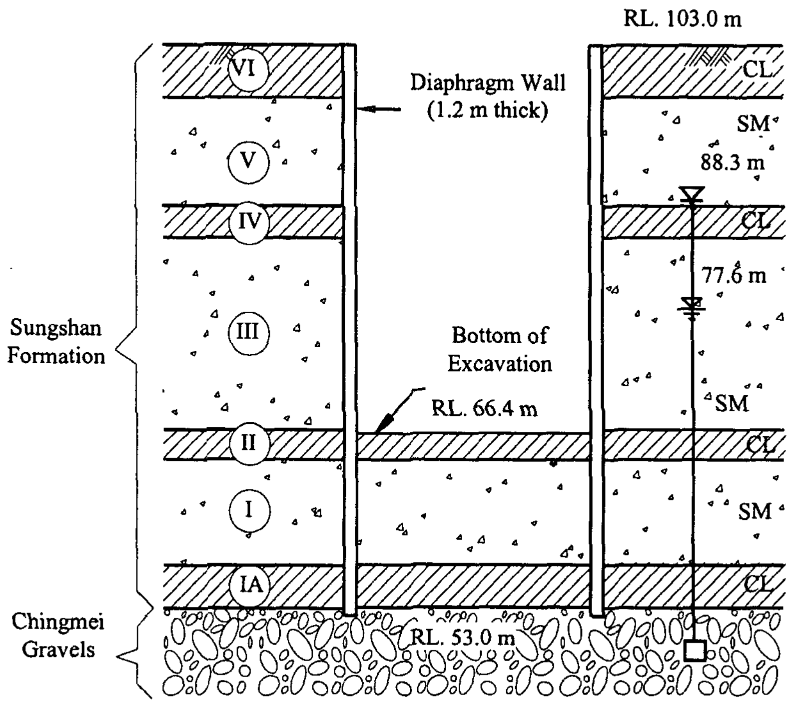


Fig. 7 Pumping at Ventilation Shaft A in Contract CP262, TRTS

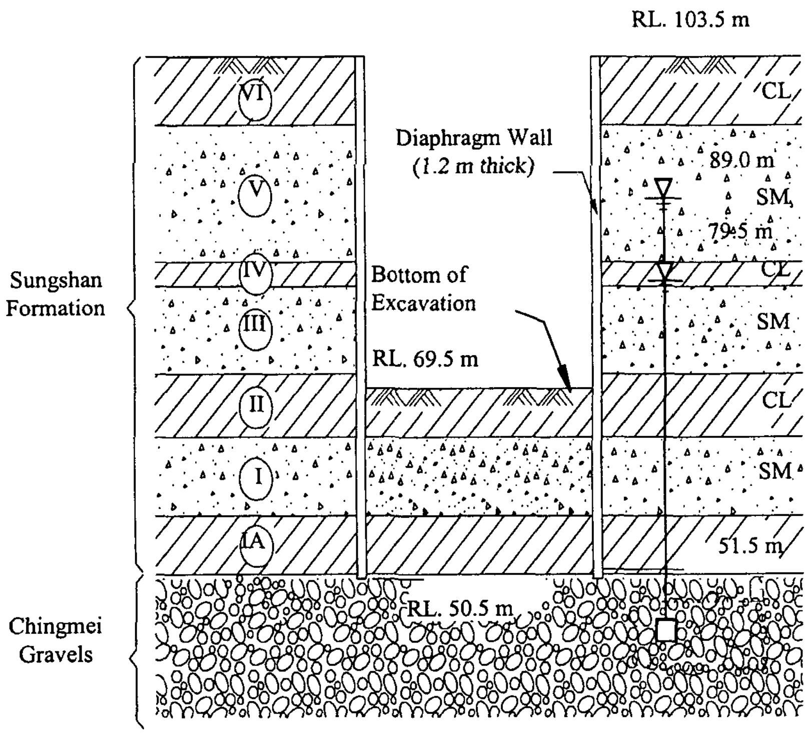


Fig. 8 Pumping at Ventilation Shaft B in Contract CP261, TRTS

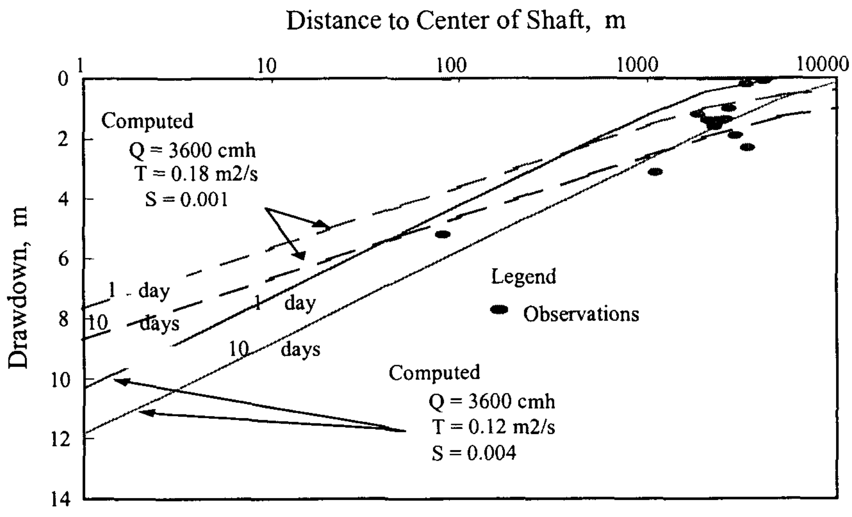


Fig. 9 Influence of Dewatering in the Chingmei Gravel

Other measures used in TRTS to increase the factor of safety against blow-in include the construction of grout slab in the permeable Chingmei Formation. Figure 10 illustrates such a case in the construction of ventilation shaft of Contract CH221 of the Hsintien Line.

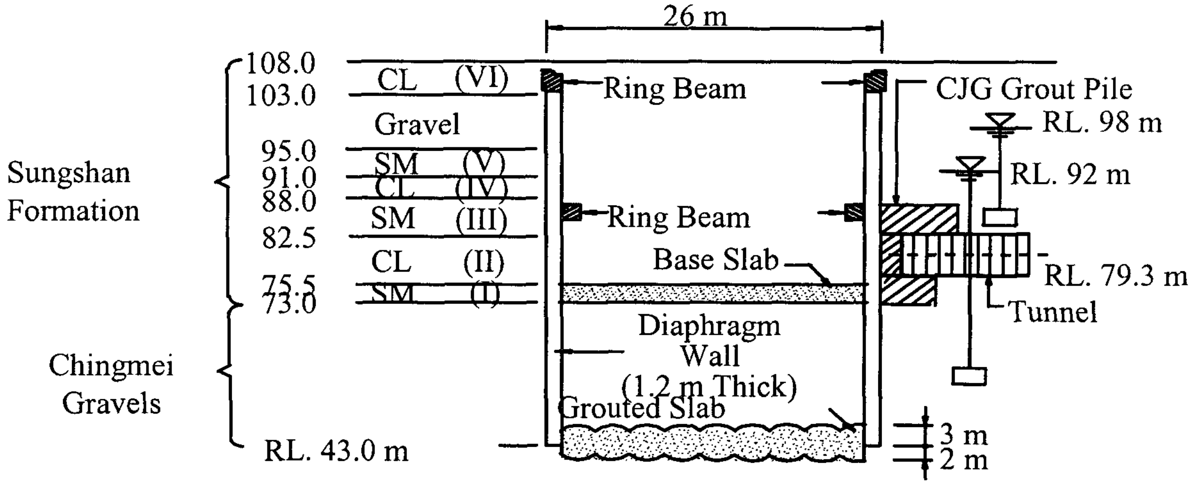


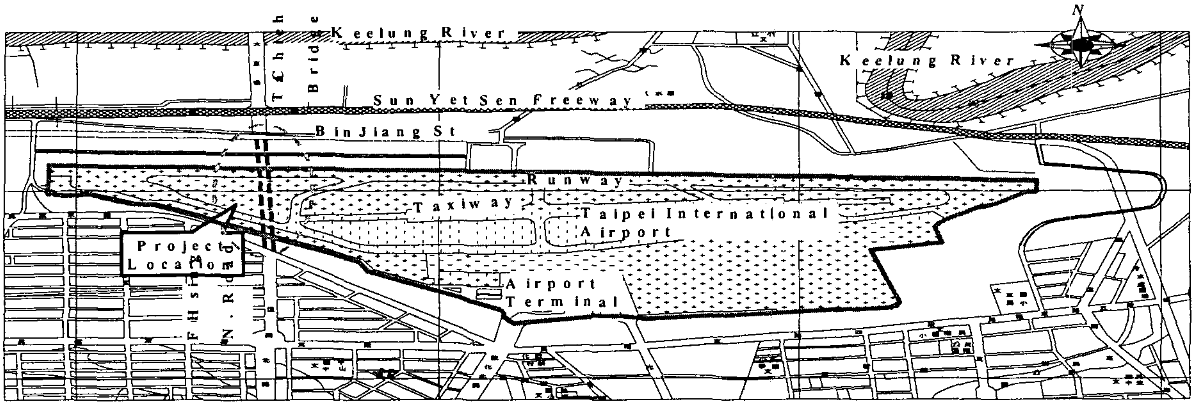
Fig. 10 Use of Grout Slab for Ground-Water Cutoff, Contract CH221, TRTS

3. UNDERPASS BENEATH TAIPEI INTERNATIONAL AIRPORT

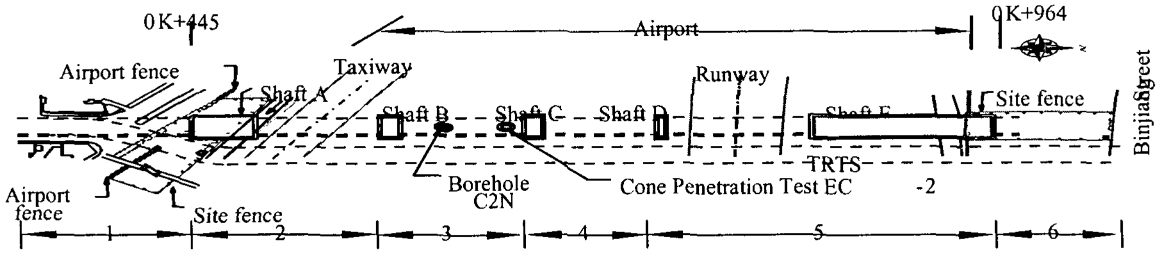
To ease the local traffic congestion in the northern part of Taipei City, an underpass is currently being constructed to extend Fuhsing North Road northward to connect to Tachieh Bridge which crosses the Keelung River. A major portion of this extension is underneath the field of the Taipei International 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. 11(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 shown in Fig.12, the underpass is a 4 lane highway tunnel and is to go underneath a 40m wide taxiway and a runway of 60m width. The twin-cell box is 22.20m in width and 7.80m in height.

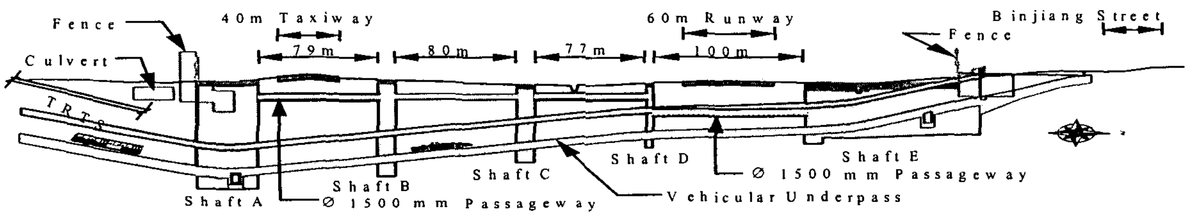
Since the Taipei International Airport is the only airport in the vicinity of Taipei metropolis, the air traffic must be maintained all the time and there are several constraints which must be considered in the design and construction of the underpass. Major constrains include: (1) construction activities 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 inclinometer, installed at the site. Majority of these instruments can be read automatically at any desirable frequencies. Data are transmitted



(a) General layout



(b) Site Plan



(c) Longitudinal section

Fig. 11 Underpass Beneath Taipei International Airport

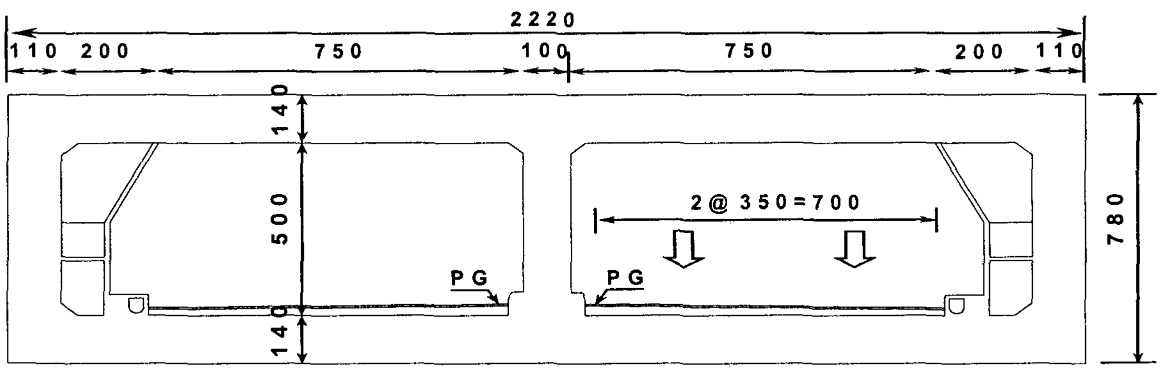


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Fig. 12 Typical Tunnel Section of the Airport Underpass from data loggers to the central control room at the site office through cables for immediate processing and analyses. An automatic alert system is adopted for prompt actions to ensure operational safety of the airport.

3.1 Ground Conditions

Before commencement of the design, a total of 18 boreholes, of which 5 are located within the airport, were drilled to a maximum depth of 75m. As shown in Fig. 13, the subsoil profile consists of a thick layer of silty clay from the ground surface to a depth of about 25m. The standard penetration SPT value of the soil varies from 2 to 5 and the water content is very close to the liquid limit. The undrained shear strength of this clay increases linearly with depth with an overage value of 100kPa. The sensitivity of the clay varies from 4 to 10.

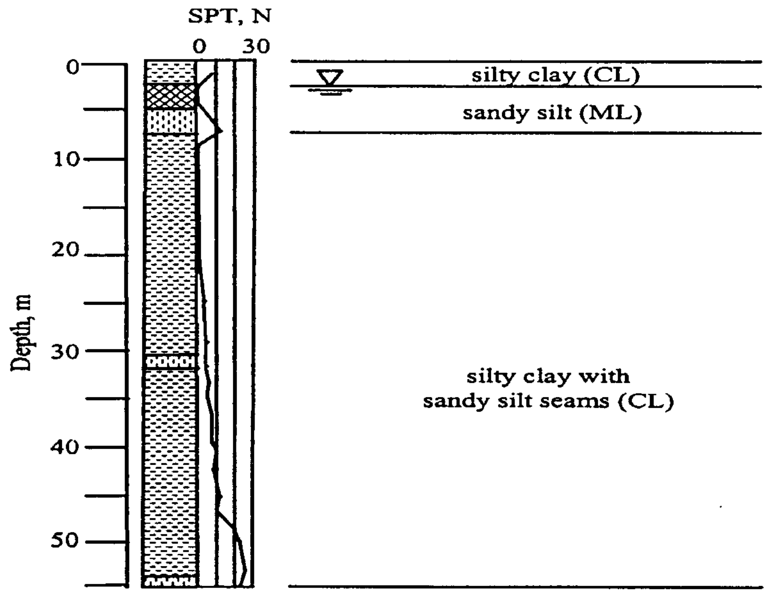


Fig. 13 Typical Subsoil Profile at the Taipei Airport

Because the site is very close to the Keelung River and is protected from flooding by a levee located only a couple hundred meters away, the groundwater table at the site is high and is affected by the water level in the river. The groundwater level rises to levels very near the ground surface during typhoon seasons.

3.2 Design Consideration and Construction

The requirement that air traffic shall be maintained all the time eliminates the possibility of using cut-and cover construction method in the sections where the taxiway and runway are present. Circular tunnels by using shield tunneling method were ruled out because very large diameter, i.e.

11m, will be required. Furthermore, large ground settlement would occur which would endanger the safety of flight operation.

After a through study and extensive evaluation of all the options, it was decided that the sections of the underpass within the boundary of the airport be constructed by tunneling in a protective shelter formed by interlocked steal pipes of 812.8mm in diameter (Fig. 14). For sections directly underneath the taxiway and runway, a so-called Endless Self Advancing (ESA) method was adopted to construct the tunnel box. Full-section segment of each ESA tunnel box is 10.5m and the head segments are 6.5m in length. The working shafts were constructed by using cut-and-cover method.

In the design, ground settlement was the primary concern. Not only the retaining systems for the shafts must have sufficient rigidity and water tightness, it is more important that the pipe-shelter satisfy these requirements. Extensive ground improvement work by means of high pressure and low pressure grouting was carried out to strength the soft subsoils and to cut off seepage paths.

The five working shafts, refer to Fig. 11, have been sunk and are inter-connected by a 1,500mm passageway for transporting materials and crew so work could be carried out underground without surface activities. The section of the passageway under the runway was in the way of the tunnel box and has to be removed, piece by piece, while the box was being jacked into the shelter. There were five guild tunnels, as depicted in Fig. 15, in which rails were laid for tunnel segments to rest. In addition, there were 12 cable ducts. The basic concept of the ESA method is that each time a segment is being driven, whether by jacking or pulling, the majority of the reaction comes from the frictional resistance acting on the rest of segments. For example, refer

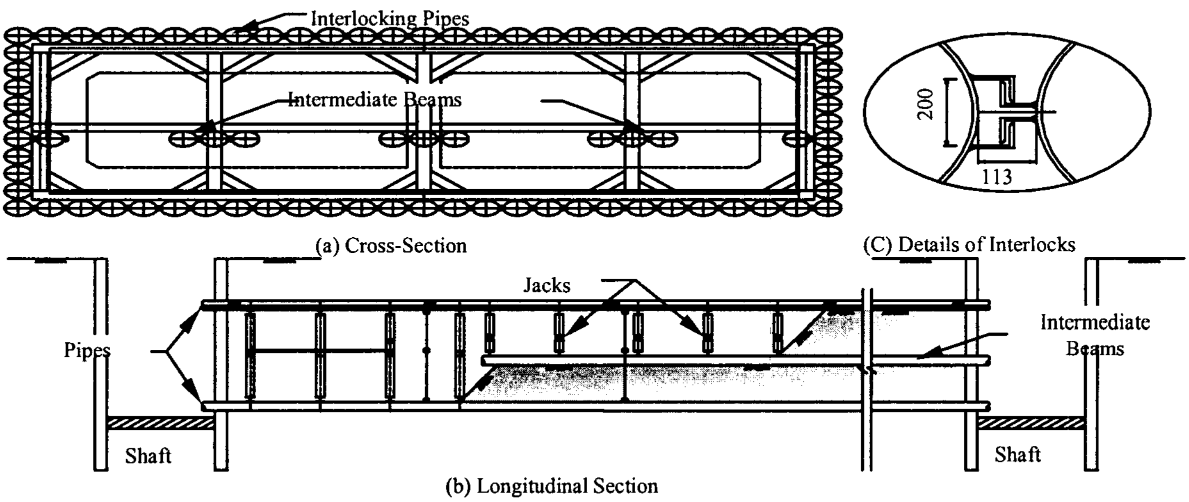


Fig. 14 Pipe Shelter and ESA Sections of the Underpass

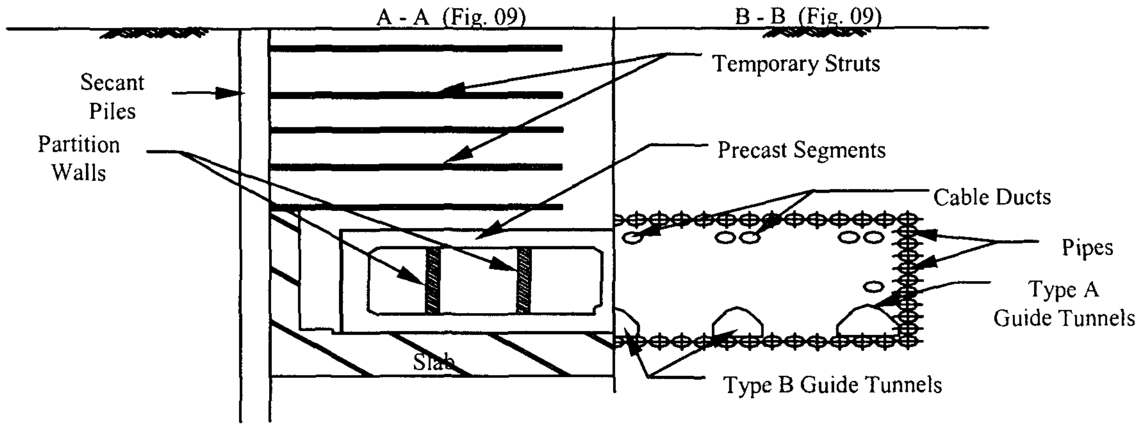


Fig. 15 Cross-Section of ESA Layout of the Airport Underpass

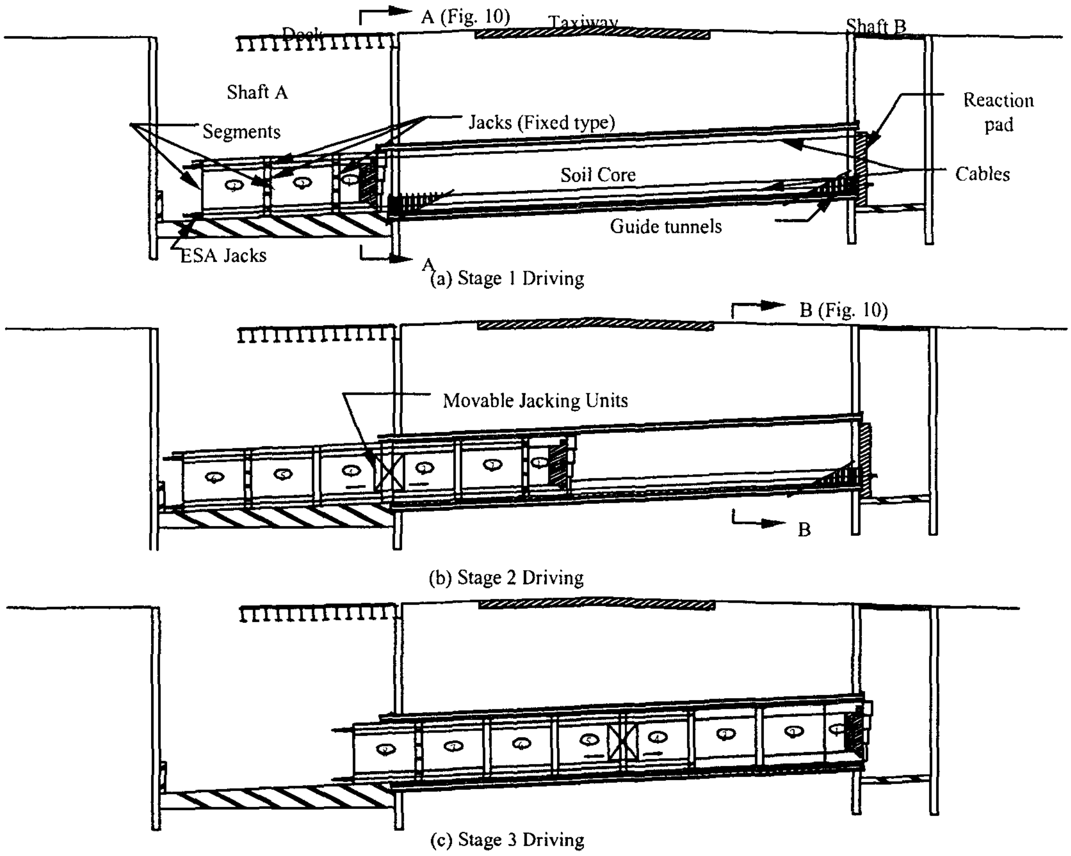


Fig. 16 ESA Tunneling in Section 2 of Underpass

to Fig. 16, 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. 15, 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 was 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 (March 2003), all the sheltering pipes have been installed and the ESA segments were satisfactorily constructed. 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.

Jacking of the pipe shelter was commenced on 01 July 1998 and completed on 30 June 2000. The total length of pipes jacked into the ground was 27, 375m at an average rate of 9.5m per 24 hour day per machine. When holidays and other obstactes were not included, the effective rate of pipe installation was about 14m per day per machine. Installation of the precast tunnel box under the runway by ESA method was started on 01 July 2002 and completed on 01 November 2002 at an average rate of 0.9m (2 cycles) per day.

3.3 Instrumentations

Because of the utmost importance of the air traffic safety, a comprehensive instrumentation program has been implemented at the site. In addition to temporary works, some navigation facilities, such as radar towers, are also being monitored to ensure that any ground movements will not affect their performance. The most important item to be monitored among all is certainly the settlements of the runway and the taxiway. Settlement points are arranged in a 7 by 23 grid in the primary area above the tunnel on the pavement of the taxiway and in a 13 by 15 grid above the runway. They are spaced at 5m intervals. Intermediate points will be added as needs arise. Since survey is not allowed to be carried out on either the taxiway or the runway

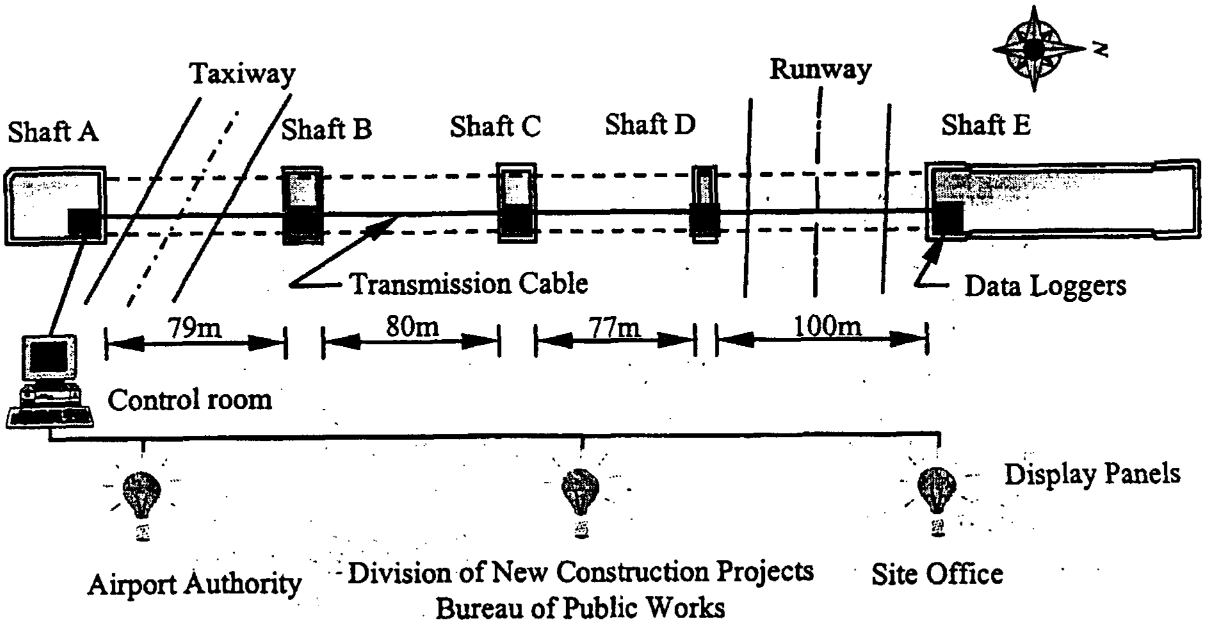


Fig. 17 Transmission of Instrument Readings at the Airport Underpass Project

during the period in which the airport is open to services, inclinometer casings are installed in two of the roof-pipes under the taxiway and two of the roof-pipes under the runway and deflections of these pipes are monitored by using horizontal inclinometer probes. Deflections of pipes are measured at 1m intervals every morning and every night during ESA works.

What is worthy of mentioning is the automatic alert system adopted at the site. There are more than a thousand pieces of instruments which can be read automatically at any desirable frequencies. As illustrated in Fig. 17, a total of 14 data loggers are located in shafts and data are transmitted to a control room located at the site office where data are processed and analyzed. In addition, there are more than 500 settlement points and many other instruments of which readings must be taken manually and keyed in to the databases. For each instrument, two trigger values are defined and the current reading is compared with these trigger values. Once the first-level trigger value is exceeded, the contractor is alerted of potential risk and is urged to prepare contingency measures. Once the second-level trigger value is exceeded, contingency measures shall be taken unless it is proved that the situation posts no immediate danger.

The status of instruments is displayed on three panels, one at the site office, one at the office of the project owner, i.e., the Division of New Construction Projects of Bureau of Public Works of the Taipei Municipal Government and the third at the office of the Airport Authority. A green light will indicate that the current reading of the corresponding instructment is within the first-level trigger value and a yellow light will indicate that the reading has exceeded the first-level trigger value but is still within the second-level trigger value. A red light will indicate that the second-level trigger value has been exceeded.

Upon the completion of all the pipes running from shaft E to shaft D, including passageway, the settlement of the runway was within 10mm, as illustrated in Fig. 18.

In order to ensure safety of the airfield operation, detailed settlement analyses were carried out to portray the effect of construction sequence, particularly during the ESA construction. Two-dimensional as well three-dimensional finite difference analyses by employing the software FLAC were carried out. Figure 19 shows the estimated ground surface settlement of the runway surface by 2-D analysis and Fig. 20 presents results of the 3-D analysis with full ground improvement. In the analysis, various degrees of ground improvement were considered. For the actual construction, full ground improvement was carried out. The actual measured values of the surface settlement are shown in Fig. 21 which indicate that with full ground improvement, ground settlements are well within the tolerable limit.

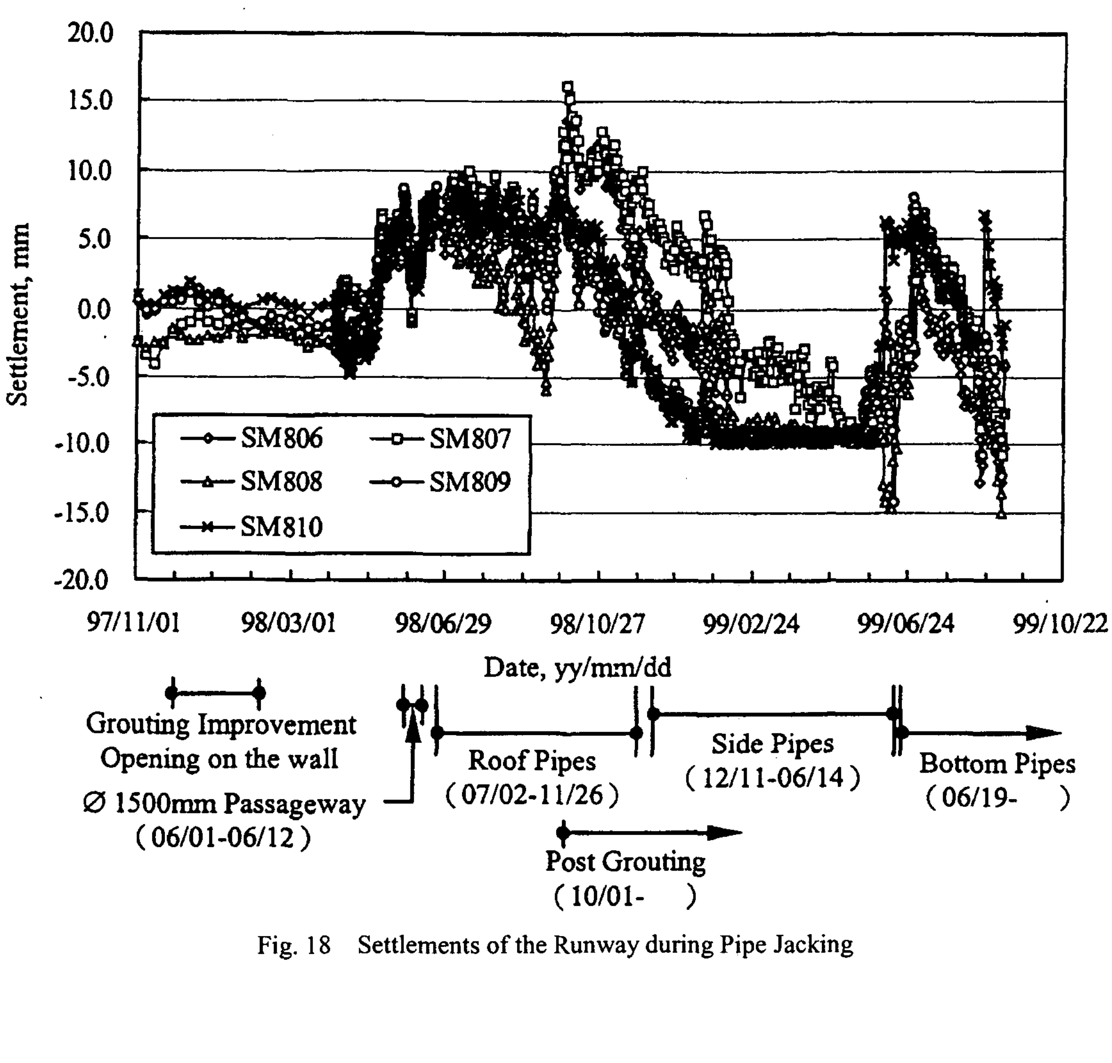


Fig. 18 Settlements of the Runway during Pipe Jacking

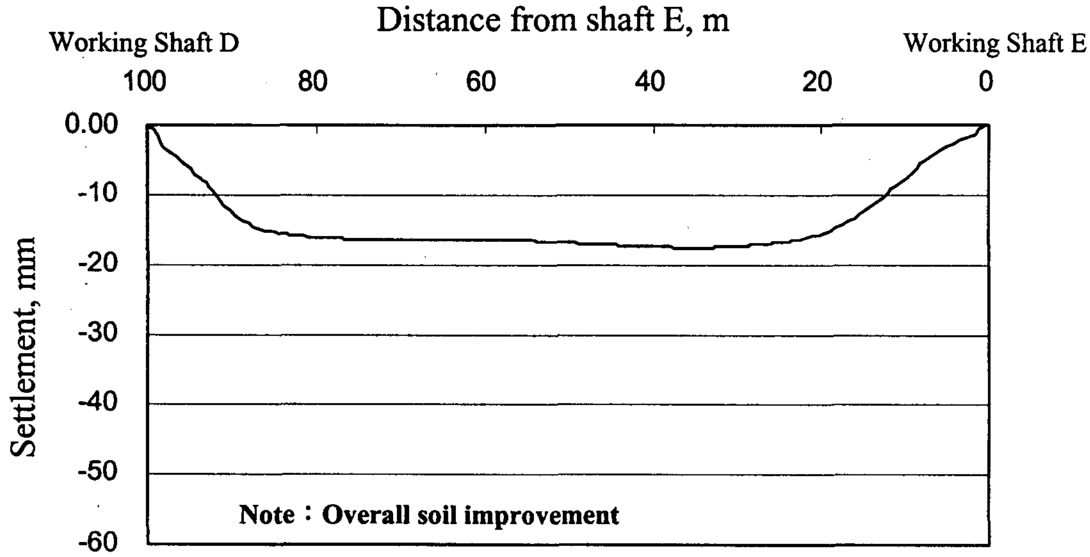
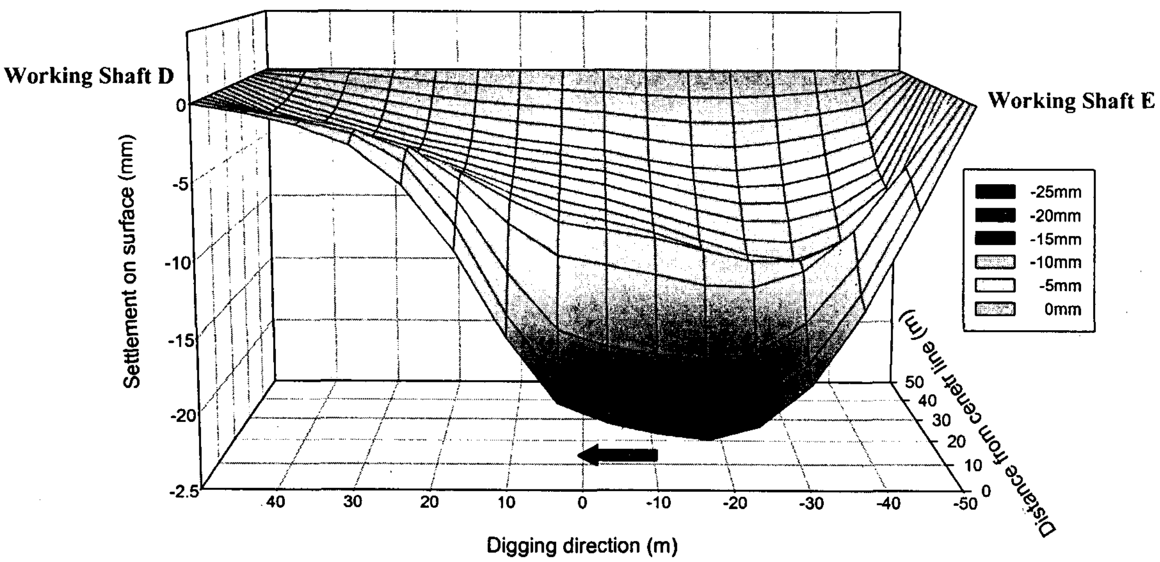


Fig. 19 Ground Surface Settlement by 2-D Analysis



Note : (1) The tunnel section is advanced to half way
(2) Overall soil improvement

Fig. 20 Ground Surface Settlement by 3-D Analysis

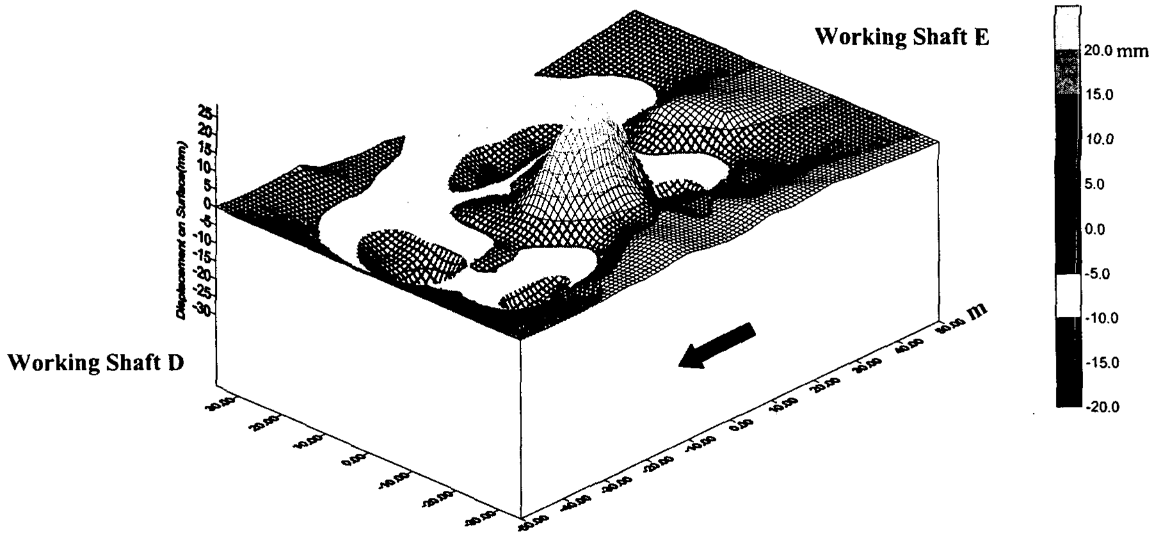


Fig. 21 Measured Ground Surface Settlement After Completion of the ESA Construction Under Runway

4. HIGH SPEED RAIL PROJECT

In view of the anticipated fast growing demand for intercity travel in the future and the need to balancing of regional development, the government of Taiwan, ROC decided to construct a high speed rail system along the west corridor of the island in the early nineties. In the mid-nineties, the government decided to invite private investment in this important project. The first major transportation infrastructure BOT (Built-Operate-Transfer) project was awarded to the Taiwan High Speed Rail Corporation (THSRC) in July 1998. The HSR line runs approximately 345 km from Taipei in the north to Kaohsiung in the south along the western corridor of Taiwan, passing 14 major cities and counties, and 77 townships and regions (Fig. 22). There will be 12 stations, 1 main workshop, 3 stabling yards and 5 maintenance bases. Government concessions awarded to the THSRC include construction and operation of the project, management of subsidiary commercial enterprises, and development of five major station areas for a period of 35 years. After commissioning, the Taiwan HSR system is expected to reach a normal speed of up to 300 km/hr, and ultimately 350 km/hr. At an estimated total construction investment of US$15 billion, this project not only represents one of the most challenging infrastructure projects in the world to date, but also is the largest private sector invested public construction project.

To meet the target date for passenger service in October 2005, i.e. a total of 5 years is available for the construction of this mega size project, the entire civil construction work was divided into 12 turnkey (design-build) contracts carried out on fast track basis. All the contracts are proceeded simultaneously. The civil work includes about 50 km (14%) tunnels, 265 km (77%) viaducts and bridges, 32 km (9%) cut and/or embankment. To date, more than 60% of the civil

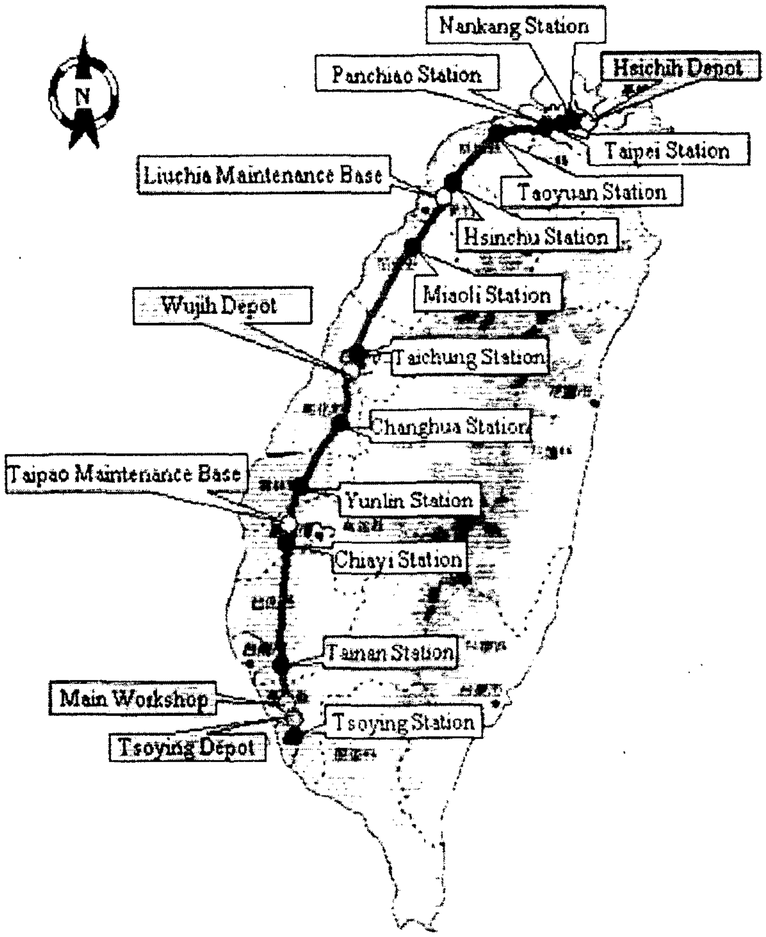


Fig. 22 Route Map of the Taiwan High Speed Rail

work has been completed.

The HSR line carries very heavy load and can tolerate very limited settlement, particularly differential settlement. It passes terrain with variable and complex geological conditions. There are many geotechnical issues which must be carefully considered and evaluated for the design of the civil works in order to achieve a safe but economical solution. This paper presents short discussion on three of the many geotechnical issues, including soil liquefaction, land subsidence, and pile foundation.

4.1 Soil Liquefaction

Taiwan is located at the Circum-Pacific earthquake belt of the boundary of Eurasian plate and Philippine Sea plate. The southern part of the High Speed Rail Route is through alluvial plain which is composed of interbeded soft sandy soil and clayey soil. According to the records of the Central Weather Bureau for the last contrary, there were 6 earthquakes which had induced soil liquefaction with blowing sand in the southwestern part of Taiwan. The design life for the High Speed Rail is 100 years, and design earthquake level is the ground acceleration corresponding to a return period of 950 years, which has a 10% probability of exceedance in 100 years. During strong earthquakes, the pore water pressure in saturated soils will increase due to the application of reversing shear stresses induced by ground motions. This may cause a partial or total loss of shear strength of soils with serious consequences such as loss of foundation support, flow slides, slumping, lateral spreading, ground subsidence, etc. The soil deformation accompanied may be unlimited (liquefaction) or limited (cyclic mobility). Liquefaction of foundation soils is critical to the safety of the structures built on them. Along the route and in the near vicinity of the HSR project, the contractors have to investigate, to evaluate the possibility, and to take into consideration on foundation design for the important issue of soil liquefaction.

4.1.1. Evaluation Method of Liquefaction Potential

Basically, there are two different approaches, the simplified empirical method and the analytical method, that may be used to evaluate the liquefaction potential of level ground. The first method (empirical method) utilizes field performance data to establish the liquefaction strength of a given soil by SPT blowcount data. The average earthquake-induced shearing stress is estimated from the expected peak ground acceleration at the site using a simple equation. The second method (analytical method) utilizes laboratory cyclic triaxial or simple shear tests to establish the liquefaction strength of a given soil.

The commonly used simplified methods for liquefaction evaluation in Taiwan are the Seed empirical method (Seed et al, 1985), the Tokimatsu & Yoshimi (1983) empirical method, the NJRA method (Japan Road Association, 1996), and the NCEER method (National Center for Earthquake Engineering Research, 2001). According to the soil investigation and liquefaction evaluation report prepared by the National Center for Research on Earthquake Engineering (Taiwan) for the Chi-Chi Earthquake in 1999, which has caused a large number of soil liquefaction in Taiwan, the NCEER method indicated a good correlation on the evaluation of soil liquefaction as compared with the actual site condition.

The NJRA method was adopted for Taiwan High Speed Rail design, which incorporated the experience from the Hyogoken-Nanbu Earthquake in 1995. The advantage of this method is to provide a code for determination of reduction factor when the soil layer is vulnerable to liquefaction under specified earthquakes.

4.1.2. Design Soil Properties affected by Soil Liquefaction

Two levels of earthquake are considered in the THSR design. They are:

(1) Design for repairable damage (Type I Earthquake)
This earthquake level is the ground acceleration corresponding to a return period of 950 years, which has a 10% probability of exceedance in 100 years. The ground peak acceleration is 0.22g to 0.40g depend on the zone in Taiwan.

(2) Design for safe operation at maximum speed and no yielding (Type II Earthquake).
The design ground acceleration in the horizontal direction for Type II earthquakes is equal to one-third of that due to Type I earthquakes.

Where potential for liquefaction exists under the excitation associated with either the Type I or Type II earthquake, the following measures are to be undertaken:

- the liquefiable soils shall be removed, or
- soil improvement techniques shall be used, or
- deep foundations such as piles or caissons shall be used, and shall be designed by using the reduced soil properties depending on the value of liquefaction resistance factor F_L and the depth of soil susceptible to liquefaction..

For pile design, the reduction of soil properties will induce decrease of bearing capacity. In the mean time, the reduced soil stiffness will induce higher lateral movement and higher moment when the pile is subjected to lateral force due to earthquake. Consequently, the pile length, reinforcement, and even the pile diameter have to be increased.

4.1.3. Negative Skin Friction Caused by Soil Liquefaction

The design of piles need to take into consideration the effect of negative skin friction which may result from dewatering or liquefaction. When ground subsidence occurs due to dissipation of excess pore pressure within liquefied soil layer after a specified earthquake, the negative skin friction are evaluated. If it exist, it is treated as an addition to the working load as required by design specifications. However, according to the Chinese Building Code issued by the Ministry of Interior, Taiwan, ROC, there is no need to consider the negative skin friction when short term forces are applied (e.g. earthquake force, wind force, impact force and traffic force.). It means that there is no need to consider negative force with the other short term forces at the same time, so check are undertaken separately. In general, negative skin friction does not govern the design of piles.

4.2 Land Subsidence

Due to over utilization of groundwater for agricultural and aquacultural use, regional and local land subsidence became a serious problem along the western coast of the Taiwan island. According to the Bureau of Water Resources' report (2000), the center of regional subsidence in the Yunlin County has moved from coastal area in early years to the inland in recent years. The average annual subsidence has reached more than 7cm. Alignment of the proposed Taiwan High Speed Railway passes through this area as well as other areas south of Zhosui River having over pumping of groundwater and land subsidence problems. Their potential effect on the safety and operation of the high speed railway is one of the key issue being considered. A special study to evaluate the effect of land subsidence on structures along the Taiwan High Speed Railway (south of Zhosui River) was carried out (Moh and Associates, 2002). This section summarizes the results of study in the Yunlin area (HSR Station TK218+000 to TK237+000).

4.2.1 Geology and Geotehcnical Conditions in Yulin Area

The topography along the HSR in Yunlin area is generally plain without major variation. It is covered by unconsolidated sediment of the Holocene Epoch of the Quaternary Period (Ho, 1986), varying in thickness from 750m to 3,000m. This area is one of the major groundwater resources in Taiwan. As shown in Fig. 23, within 200m depths along the HSR line, generally speaking, there are three aquifers and two aquitards. However, due to complexity in the sedimentary environments, considerable local variations were found in the area. Information shown in the figure were obtained from wells drilled in the area. Since the wells were drilled primarily for

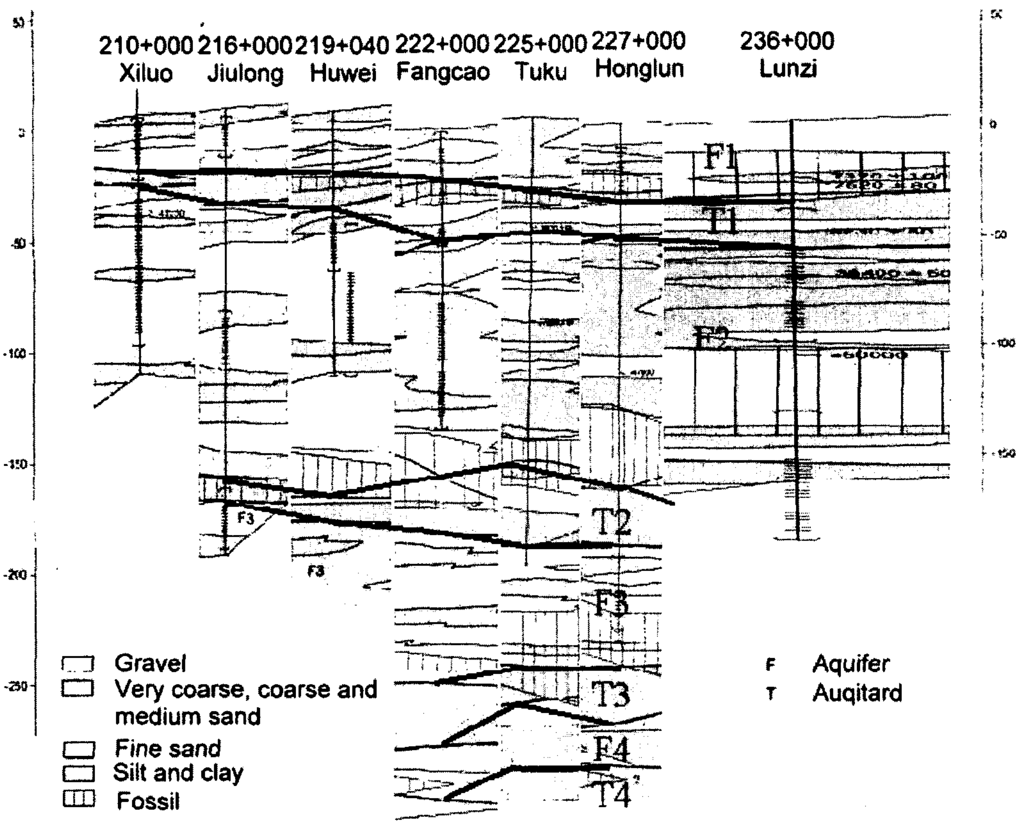


Fig. 23 Hydrogeological Section in the Zhosui River Alluvial Fan

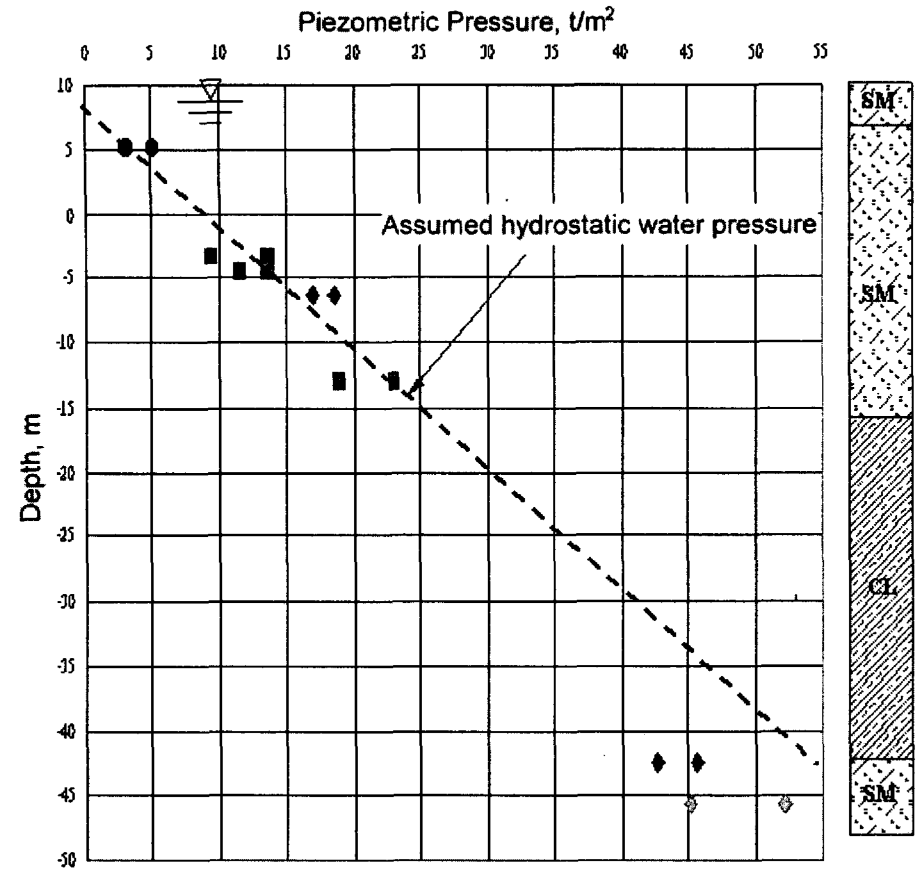
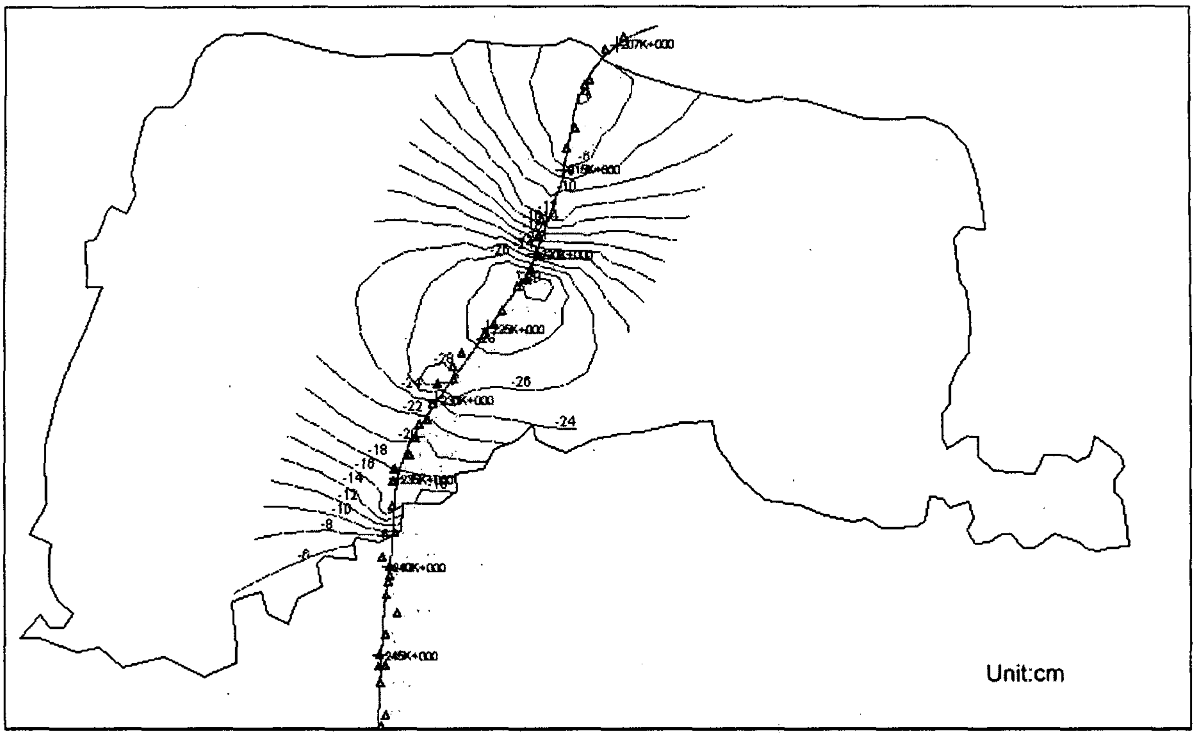


Fig. 24 Variation of Piezometric Pressure with Depth

the purpose of identifying water resources, there were no engineering data available, such as soil strength, compressibility. Furthermore, depth of the wells was limited to less than 250m.
For the HSR project, shallow brings, less than 70m deep, were drilled along the alignment. Based on information obtained from monitoring of piezometers and observation wells installed for the HSR project, Fig. 24 was prepared which shows the distribution of piezometric pressure of the groundwater. It should be noted that the piezometric pressures in the shallow strata are less than hydrostatic which indicate that there were withdrawal of groundwater in those areas.

Figure 25 shows the settlement contours of the land subsidence along the HSR route measured between January 1998 and September 2001. Settlement varied from 4 to 30cm with annual rate of 1 to 8cm. Further comparison of the settlement data with those reported by other government agencies indicate that there is a tendency of increasing in land subsidence with time.

In order to fully understand the nature of subsidence and for the purpose of establishing controlling or prevention mechanism, satellite images were collected to identify the land use. Satellite images can also be used to evaluate land subsidence problem. (Moh and Chin, 1994)



source：THSRC

Fig. 25 Measured Settlement Along the THSR from January 1998 to September 2001

4.2.2 Land Subsidence in Yunlin Area

On the basis of information and data collected by the Department of Water Resources, the Central Geological Survey, and the Taiwan Water Supply Company as well as measurements carried out by the HSR Corp., land subsidence in the Yunlin Area from TK218+000 to TK237+000 reached 0.5 to 16cm between year 1999 and 2000. The regional subsidence could be attributed to:

(a) Withdrawal of groundwater for farmland and aquaculture use,

(b) Presence of large thickness of clayey soil,

(c) Withdrawal of groundwater from the deep aquifer for domestic water supply.

4.2.3 Effect of Land Subsidence on HSR Structures

Assessment of potential effects of land subsidence on the safety of HSR structures includes:(a) bridge foundations, (b) vertical alignment and (c) negative skin friction of foundation piles. According to the design specifications, allowable angular distortion between any two points along the HSR should not exceed 1/1,000. Based on measurements carried out by the HSR Corp. in 1998-2000, the estimated angular distortions, except near St. TK245+000, were all below the allowable value.

The HSR Design Specifications also stipulate that "...Vertical curves need not be provided in the structure or embankment if the difference in gradient is less than 1.0‰ in full speed sections, or if the calculated vertical curves mid ordinate is less than 10mm in lower speed sections....". Since the calculated angular distortions along the HSR route at the present are all less than the specified limit, no adjustment of the vertical alignment appears to be needed.

According to the design standards for Japanese National Railways (Japan Civil Engineering Society, 1986), design of foundation piles should consider effect of negative skin friction when the rate of land subsidence exceeds 2cm/year. When the subsidence rate is more than 4cm/year, 100% negative skin friction should be considered.

For the section of the HSR considered, between TK218+000 and TK237+000, the average rate of land subsidence is between 0.5 and 16.0 cm/year, development of negative skin friction is a potential problem. However, it is important to identify the source of land subsidence due to withdrawal of groundwater. If the land subsidence as measured by the settlement of ground surface, is caused by consolidation of deep soil strata, negative skin friction may not need to be considered. On the other hand, negative skin friction could be a serious problem in the design of foundation piles, when land subsidence is caused by consolidation of shallow soil layers.

Although the assessments carried out in the study were based on short term measurements, potential effects of land subsidence on the safety and maintenance of the HSR are obvious. Since effects of land subsidence caused by groundwater withdrawal are long term, irreversible and extend over a large area, proper control of groundwater pumping is a "must" in areas where the HSR passes through.

4.3 Pile Load Tests

From the mid to south section of the Taiwan High Speed Rail pile foundations were adopted for elevated structures in the plain area due considerations of subsoil conditions, environmental impact, safety, economy, construction practice and etc.. This elevated section is 155km in length, and over 200,000 piles were placed as foundations. Taiwan is located at the Circum-Pacific earthquake belt, large earthquake forces have to be taken into consideration in pile design, and foundation pile need to have high bearing capacity. For economy and fast construction, pile diameters adopted are between 1.5m and 2.2m, mainly 1.8m to 2.0m, and pile lengths are 50 to 60m.

Due to the huge amount of piles to be constructed as foundations, pile load tests are very important to verify the design assumptions and parameters used, for ensuring safety and economy of the project. The adopted full-scale pile load test methods included Conventional Pile Load Test Method with 4,400 ton as maximum capacity, Osterberg Cell (O-Cell) Load Method, Dynamic Pile Load Test Method and Stadynamic Load Method。 The piles for compression and tension tests were instrumented with rebar stress transducers at different depths together with surface movement measurements via displacement transducers. From these instrumentation data, the skin friction distribution and end bearing of the piles were estimated. Dynamic load test and Stadynamic load tests were chosen as effective testing methods for verifying the pile capacity through comparison with the static pile load tests and to be used for testing working piles during construction.

The followings are some valuable findings for the design and construction of pile foundation for the THSR project based on pile load test results.

4.3.1 Construction Methods

In this series of pile load tests, the construction methods of bored pile installation were improved to a great extent through the use of stabilizing agent (polymer), shorter construction time, and multi-stage toe grouting. The pile load test results clearly demonstrated increase in pile capacity with these adjustments in construction procedures. Test results indicated that a 50% lower in skin friction of sandy soils could be caused if polymer was not used and the construction duration was long (Fig. 26). High skin friction was achieved through reduction in caking effect of the sand layer with polymer as stabilizing agent. Shorter construction duration reduces the softening of clayey soil due to swelling. Special attention should be paid during construction to ensure that proper construction procedure has been followed. Multistage toe grouting ensures that any soft toe caused by poor toe cleaning or sedimentation of soil at the bottom of excavated holes

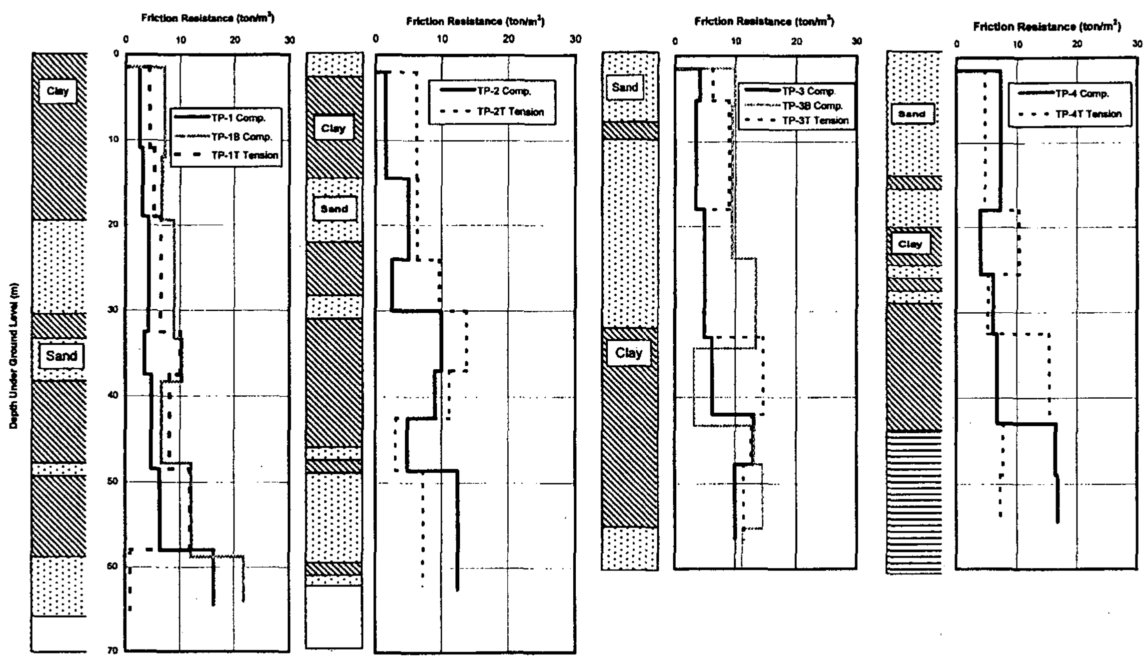


Fig. 26 Skin Friction Profiles of Compression and Tension Piles

can be improved via injection of high-pressure grout in stages to strengthen the weak zone within the pile toe area. The performance of the end bearing is greatly affected by the construction method as well. In the current adopted construction method, it can be seen that stiffer response in the end bearing was achieved through multi-stage toe grouting provided that the bearing layer is not clayey soil.

4.3.2. Pile Capacities in Tension and Compression

From the results of pile load tests, the ratio of ultimate tension capacity to compression capacity without end bearing ranges from 0.62 to 1.05 (62 to 105%), having a mean of 81% with a standard deviation of 16%. The ratio is much higher than the allowable limit of 40% stated in the design specifications. It may be worthy the effort to reconsider the limit if tension capacity is the controlling factor on pile design due to earthquake force. If the allowable limit of 40% is raised to 50 or 60%, then the governing condition of most of the pile foundation design will be reversed. This means that compression capacity will control the pile design except for river bridges. Based on this finding, it is advisable to modify the stated allowable limit in the design specifications for an optimized design from economical point of view.

4.3.3. Osterberg Cell Pile Load Test

In addition of conventional static compression pile load tests, an alternative method was used by means of Osterberg cell. A two-level test arrangement of the Osterberg cells was adopted for measuring the end bearing as well as skin friction along a section of the pile shaft. The first Osterberg cell level was installed close to the tip for measuring the end bearing capacity. The position of the second Osterberg cell level was determined with respect to the soil conditions in such a way that skin friction could be tested separately for the upper and lower parts of the pile. The testing plan was handled by a specialist subcontractor. The Osterberg cell method is good for measuring the end bearing capacity of full-scale pile. However, some test results indicated that the induced movement at some sections of the pile did not fully mobilize the skin friction, which might be due to underestimation of the skin friction during the test planning stage and refinement of adjustment of the cell position was based on previous results. Pile load test results indicate that when the Osterberg cell results are used as the basis for comparison with the static pullout tests, the ratio of ultimate tension capacity to compression capacity ranges from 1.0 to 1.14 (100 to 114%).

5. CONCLUDING REMARKS

In the three projects described, the geotechnical consultant plays in three different modes. For the Taipei Rapid Transit Project, the geotechnical consultant served as the geotechnical engineering specialty consultant to the owner and being responsible for review, advice and monitoring of all geotechnical related activities from the planning to the construction stage. For the Taipei Airport Underpass Project, the geotechnical engineer is a part of the design and construction management team in a conventional design and construction project. The consulting engineering team is responsible for the design during the design stage, and management/quality control during the construction. In the High Speed Rail Project, the geotechnical engineer is also a part of the design team but is responsible to the constructors in the turnkey contracts. Although technically, the geotechnical engineer should always provide safe, sound and economic design no matter how his service is provided. In reality, the methods of approach and ways of thinking are different. For example, in conventional design and built work, the designer has lots of freedom to carry out his design and to select possible method of construction provided that the owner/client is satisfied with the solution. In the case of design-build turnkey contract, the consultant/designer must always work closely with his client, i.e. the constructor, in terms of selection of construction methodology or sometimes even materials. Value engineering becomes very important. Nevertheless, no matter which mode of operation is being adopted, it is the geotechnical engineers' responsibility to provide safe, sound and economical solution. It should also be emphasized that for a successful project, particularly in difficult site, close coordination and cooperation among the owner, geotechnical consultant and constructor is a "must".

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