GEOTECHNICAL PROBLEMS RELATED TO DESIGN AND CONSTRUCTION OF THE TAIPEI TRANSIT SYSTEMS

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SYNOPSIS The Initial Network of the Taipei Rapid Transit Systems comprises six lines with a total of 88 km 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. At the present, two lines in the Network have been completed and open to operation. The remaining four lines are anticipated for revenue operation in the next two to four years. For the underground construction, about 45 km of diaphragm walls were constructed in the cut-and-cover excavations whilst bored tunnels have a total route length of 22 km. This paper describes some of the most significant geotechnical concerns associated with design and construction of the Initial Network. Discussions are made on the soil characterization, effects of groundwater movements, ground subsidence, earthpressures on walls and ground movements during construction.

1 INTRODUCTION

Geotechnical engineering plays a vital role in constructions for rapid transit systems in cities, in which usually the ground is soft and difficult to deal with, and the Taipei Rapid Transit Systems (TRTS) is certainly not an exception. The challenge in solving various types of problems 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 rapid transit systems by using TRTS as an example. It covers the three major elements in underground constructions, i.e., soil investigation, cut-and-cover construction, and tunnelling.

2 TAIPEI RAPID TRANSIT SYSTEMS

A system map of TRTS is depicted in Fig. 1. The Initial Network of the Systems comprises six lines, namely the Mucha, Tamshui, Hsintien, Nankang, Panchiao, and the Chungho Lines. Because this is 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

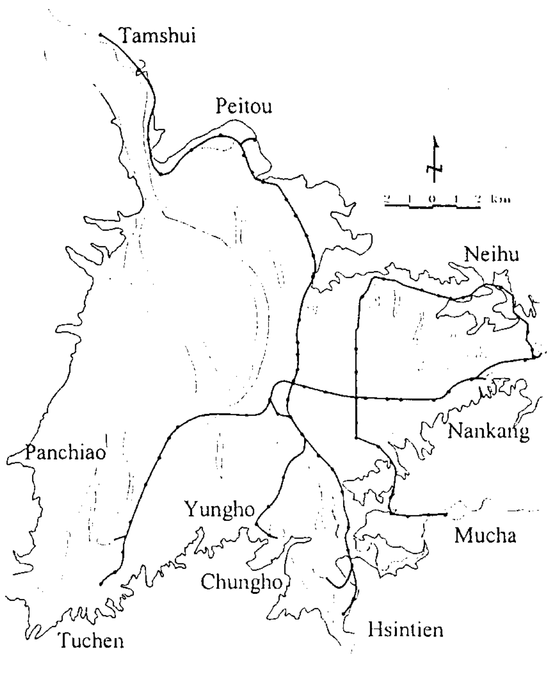

Fig. 1 Initial Network of Taipei Rapid Transit Systems

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 GESC to process the tremendous amount of field data in a systematic manner.

3 GROUND CHARACTERIZATION

Geotechnical engineers shall always have an appreciation of local geology and geological map shall always be the first piece of information to study whenever a project starts. Previously, the geological mapping for the Taipei Basin was limited to the east of the Tamshui and Hsintien Rivers. A recent study, refer to Fig. 2, extended the zonation westward beyond these rivers to cover the entire basin (Lee, 1996). The number of geological zones increased from 7 to 22. For the benefit of the readers, an east-west and a north-south soil profiles across the Basin are presented in Fig. 3. As can be noted that at the surface is a thick layer of Sungshan Formation. Toward the east, silty clay dominates while in the central city area, where the Taipei Main Station is located, the six-sublayer sequence is evident. Toward the west, the stratigraphy becomes rather more complex with silty sand and silty clay seams interbedded in these sublayers. The Sungshan Formation is underlain by the so-called Chingmei Gravels which contains gravels of various sizes and is extremely permeable. This gravelly layer is practically a reservoir and has been responsible for several major failures during the underground construction of TRTS.

3.1 Soil Investigations

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

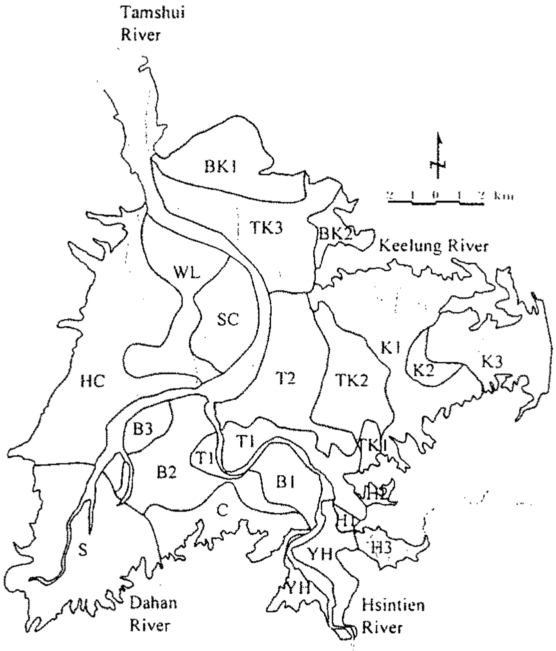

Fig. 2 Geological zonation of the Taipei Basin

and basements of highrise buildings prior to the implementation of the programs. 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. However, these boreholes 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 basements. Secondly, the previous investigations generally lacked a unified quality control and may lead designers astray. Once a route is decided, it is a normal practice for the authority in-charge of the program to engage a qualified geotechnical consultant in

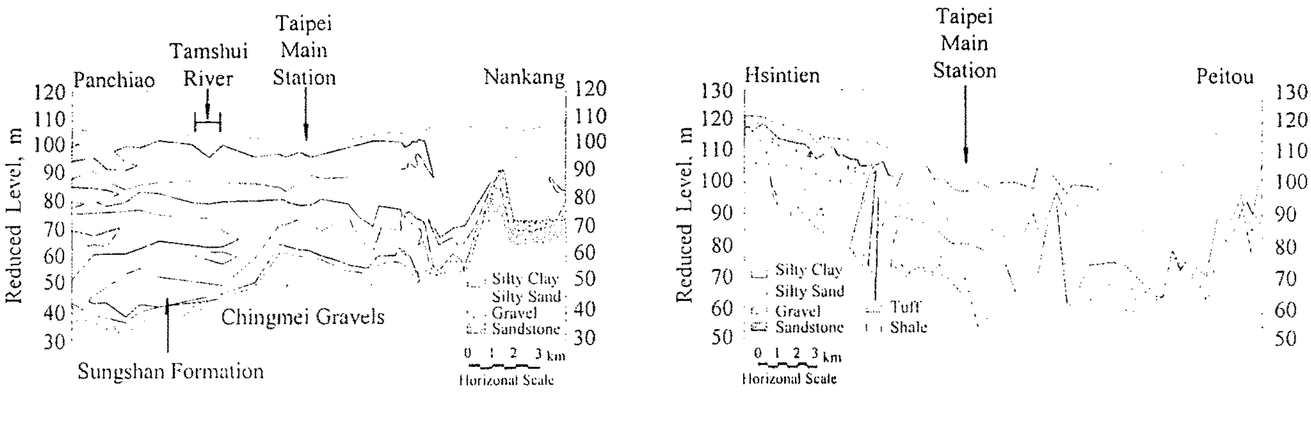

Fig. 3 Geological profiles of the Taipei Basin

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, say, six to eight boreholes are usually sufficient for a station as far as structural design is concerned. However, additional boreholes may be required to optimize the design of retaining walls or pile foundations if the top of the bearing stratum is erratic and/or the soil stratigraphy is complex. Additional boreholes are also required if blow-in and piping are potential threats and the thickness of the impervious clay blanket is critical to the safety of construction. For shield tunneling, boreholes at 50m intervals are sufficient for the design purpose but additional boreholes are required locally to ascertain mix-face conditions.

For routes running through varying geological formations, investigations have to be more detailed and sophisticated in-situ and laboratory tests are sometimes required 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 well paid off as potential problems are eliminated beforehand. Unfortunately, such a viewpoint is seldom shared by clients till it is too late.

It is a common clause in all the tender documents that tenderers of a construction contract shall satisfy themselves by performing additional 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 available for preparing the tenders.

During constructions, boreholes are sunk and tests are performed by contractors for revealing ground conditions with a better accuracy. Boreholes are also sunk for other purposes such as installing instruments, confirming the results of ground treatment, and solving specific problems, etc. While doing so, in-situ tests are usually performed in boreholes for revealing ground conditions. The investigations performed by contractors are usually many times greater in quantities but much less in sophistication than the investigations carried out by designers.

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 determining 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. Modern CPT apparatus allow the measurement of porewater pressures induced at the tip as the cone advances. This enables fine seams to be identified along the depth. Some cones can even measure shear wave velocities in soils but such an application is limited to specific purpose, for example, evaluation of liquefaction potential.

A typical CPT profile obtained in the central city area of Taipei is shown in Fig. 4. As can be noted that the six-layer subdivision is evident. Although CPT is considered as an excellent tool in characterizing soil stratigraphy. It however should be noted that interpretation of the results requires local knowledge. For example, it has been found that the subsoils in the Sungshan Formation in the Taipei Basin cannot be classified by using any of the charts available in hand and new charts have to be established (Wong, Chu and Yang, 1993).

One phenomenon worthy of mentioning is that negative pore pressures were measured throughout the entire depths of Sublayers 3 and 5 as the cone advanced. The implication of this finding deserves further studies. The phenomenon would have indicated a dilative nature of the silty sand in these two sublayers upon disturbance. However, the data available are insufficient to sustaintiate this hypotheses.

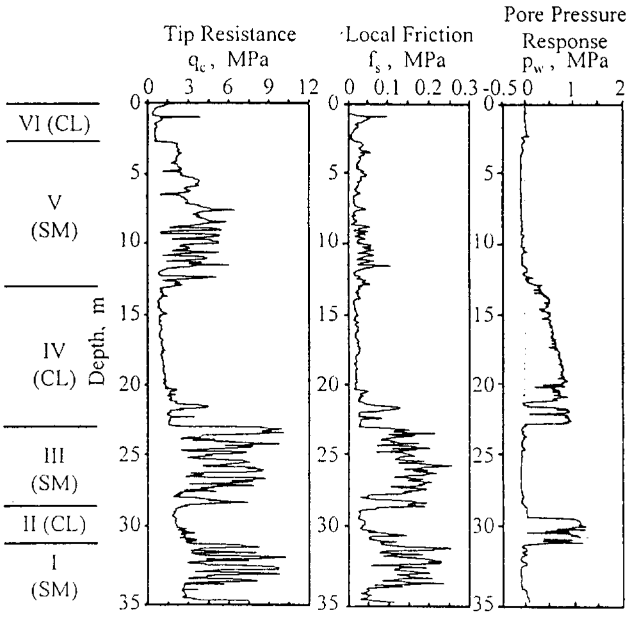

Fig. 4 CPT profile in Central Taipei

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As can be noted from Fig. 4, the soil stratigraphy is more clearly identifiable in the porewater pressure profile than the two other profiles. However, it should be noted that porewater pressures induced are dependent on the type of the cone used and, more importantly, are affected by the workmanship. Figure 5 shows a comparison of the results obtained at two neighboring locations. At CPT-39, the piezometer tip was simply submerged in water for 24 hours before the commencement of the test. The pore pressure response was poor and the boundaries between consecutive layers could not be identified. The test was repeated at CPT-39A, which was 2m away from the location of CPT-39, with the piezometer tip submerged in a water-glycerin mix and boiled for 10 minutes to drive air bulbs out. The results were much improved as indicated in the figure.

Theoretically, it is possible to determine the consolidation characteristics of soils by observing the dissipation of porewater pressures induced at the tip. A considerable quantity of dissipation tests have been performed in the central city area of Taipei and typical results are shown in Fig. 6. In silty clays, i.e., in Layers II, IV and VI, positive excess porewater pressures were obtained during the advancement of the tip and these excess pore pressures dissipated with time. In silty sands, i.e., in Layers I, III and V, the porewater pressures measured were smaller than the atmospheric pressure during the advancement of the tip and these porewater pressures increased with time till they equalized with the background pressures. The t_{50} values, which are the times for 50% of the excess pore pressures to dissipate, obtained for clays in Contracts CN251, CN253A and CN253B of the Nankang Line are plotted in Fig. 7(a). The times for half of the deficient pore pressures to recover in silty sand are plotted in Fig. 7(b). It is conceivable that the speed of dissipation of excess pore pressures in the case of clay and the speed of recovery of deficient pore pressures in the case of silty sand are somehow related to the consolidation characteristics of the soils in which the tip was buried. Mathematical formulations have been developed for interpreting the results of dissipation tests for partially saturated soils (Chin, et al., 1993). However, as a cone penetrates into the ground, the stress field in the surrounding soils will not only depend on the consolidation characteristics of soils, but will also depend on the geometry of the cone and the position of piezometer. The mechanism involved is far too difficult to be simulated by any mathematical model which inevitably involves assumptions and some of these assumptions may not be realistic at all.

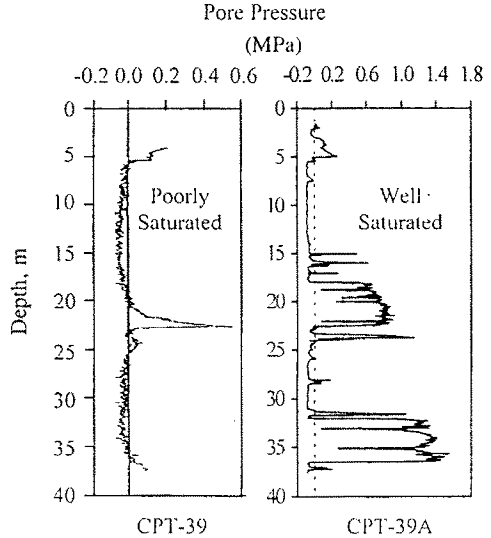

Fig. 5 Results of piezocone tests as affected by saturation of piezometer

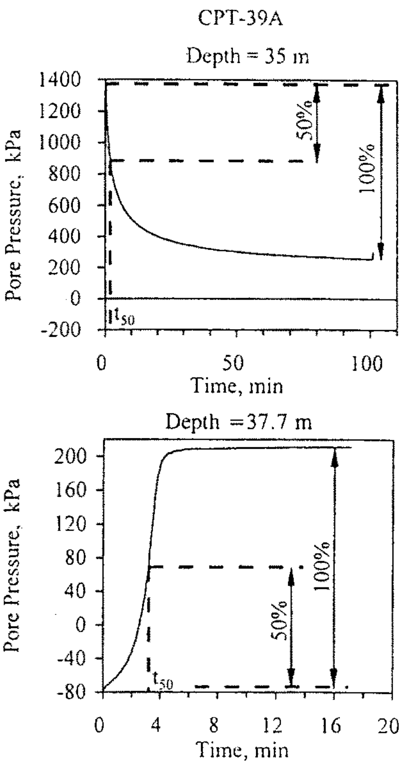

Fig. 6 Dissipation and recovery of porewater pressure

It is envisaged that, as more studies are conducted, consolidation properties of soils can be interpreted from dissipation tests with confidence. If so, dissipation tests are superior to other types of consolidation tests and permeability tests for its simplicity in procedure and the repeatability in results. Furthermore, tests can be conducted at very close intervals for obtaining a nearly continuous profile of whatever the property is along the depth.

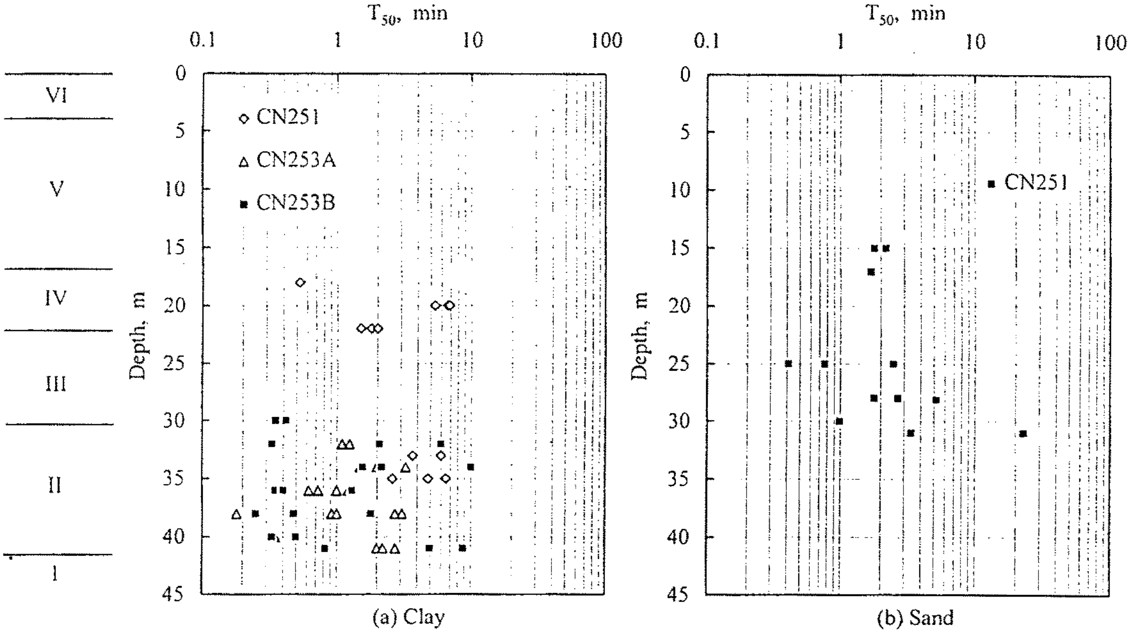

Fig. 7 Time required for porewater pressure to dissipate or to recover

3.2 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 the majority of failures occurred during underground construction were directly related to 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 true in cities where groundwater is known to experience large drawdowns, for examples, 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. 8 (Wu, 1967; 1968), the piezometric level in the Chingmei Gravels was once lowered by as much as 40m due to the excessive pumping. The use of groundwater as water supply was banned in late 60's and the piezometric level in the Chingmei formation gradually recovered subsequently.

Based on the records available prior to 1990, it was envisaged that, in the central city area, the piezometric level in the Chingmei Formation would have risen to a level of RL 97m (mean sea level: RL 100m) this year, i.e., the year of 1997. However, as the construction for TRTS started in 1990, the recovery of the piezometric level did slow down because pumping was carried out at many sites as a measure against blow-in. Large scale pumping, exceeding 2,000 cubic meters per hour in rate, was carried out for three deep excavations (Hwang, et al., 1996; Moh, Chuay and Hwang, 1996) and, as can be noted from Fig. 8, the current piezometric level is 4m below what was predicted. As nearly all the underground works will be completed this year, whether the recovery will return to its previous track remains to be observed. It shall be noted that before the turn of this century, the water in the Chingmei Gravels was in an artesian condition with the piezometric level above the ground level (Wu, 1967; 1968). Therefore, it is

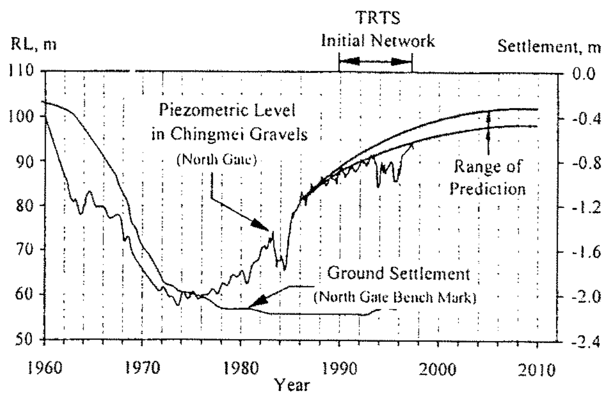

Fig. 8 Piezometric level in the Chingmei Gravels and historical ground settlement in Central Taipei

conceivable that the piezometric level will keep on rising for the many years to come.

The 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 implementation of the Initial Network and the preconsolidation effects greatly reduced consolidation settlements of soft ground during the construction and, hence, reduced damaging potential to buildings nearby. Since the Chingmei Gravels is relatively incompressible, settlement mainly came from consolidation of soft clays in the Sungshan Formation. To obtain data on the groundwater movements 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 by the DORTS. It was found that, as depicted in Fig. 9, over the entire basin, Sublayers III and V are indeed separated by two aquitards, i.e., Sublayers II and IV, and become separate aquifers. However, the piezometric levels in all the sublayers in the Sungshan Formation did move in phase with the piezometric level in the Chingmei Gravels, which implies moderate leakage in these two aquitards. With time, it is anticipated that the piezometric levels in the Sungshan Formation will eventually be equalized with the piezometric level in the Chingmei Gravels.

3.3 Local Geological Features

Every city has its own unique geological features which deserve special considerations. For examples, cavities in the limestone formation are common in Malaysia and have caused difficulties in pile foundations and large boulders are obstacles for underground works in Singapore and Hong Kong.

In the Taipei Basin, methane and drift woods are the two unique geological features. Methane was encountered in several boreholes during investigation. Its presence is related to the capture of gas in dooms encapped by impermeable clays (Lin, Chang and Chu, 1997). In one case, the eruption sent a jet of methane-water mix to a maximum height of about 10m into the air and continued for more than 3 days. Contractors have been warned of the possibility of encountering methane in tunnel drives and possible consequences. As a precaution, specifications require the concentration of methane be continuously monitored during tunnelling and shield machines be

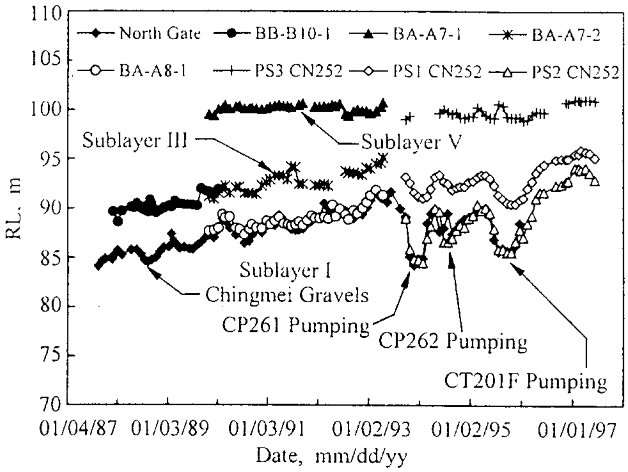

Fig. 9 Piezometric pressures in the Sungshan Formation

equipped with detective devices and alarm systems. The power supply shall be automatically cut off when the concentration of methane reaches 1.25% and resume only after the concentration of methane drops to 1%. The capacity of ventilation in tunnels was increased from 780 cubic meters per minutes to twice as much. Although methane was encountered at locations all over the Taipei Basin during investigation, the problems, to the authors' knowledge, were limited to the Chungho Line during TRTS construction.

Drift woods were recovered in numerous excavations and were responsible for a few accidents encountered during tunnelling (Lin, Chang and Chu, 1997; Ju, Kung and Duann, 1997). Chunks of 1m or so in diameter were frequently encountered and they may be as long as 5m. A piece of wood found at a depth of 9m in Observation Well 2 (W-1797) in Panchiao dated back to 6,760 years and that found at a depth of 23m dated back to 7,950 years (Liew, 1994). The presence of drift woods was well recognized and has been well reported, however, it is still very difficult to predict the exact locations and depths of drift woods beforehand. For cut-and-cover sections along the routes, problems were usually localized and were relatively easy to solve. The problems were much more serious for tunnelling sections. All the shield machines adopted in TRTS have sufficient strength and power to cut through the woods as long as they are not too large in size. However, there was one occasion in which the presence of drift wood caused the ground to collapse as the movement of the shield was hampered and soft clay kept on moving into the earth chamber. Finally, a sinkhole of 5m in diameter occurred right above the head of the shield machine. Workers entered the earth chamber and recovered two pieces of drift wood which were 500mm and 400mm in length.

A gravelly layer exists at the tunnelling depth along the Hsintien Line, refer to Figs. 1 and 2 for location and profile. The shield machines used were capable of handling cobbles of 200mm or less in size. There were no reported cases which required the removal of cobbles inside the earth chambers. The presence of this gravelly layer, however, did lead to leakage of compressed air in the NATM tunnel for Contract CH221. Although it was possible to maintain the pressure in the tunnel by increasing air supply, the air did travel to a far distance causing damages to underground facilities which would not have been damaged otherwise. Therefore, it is important to investigate ground conditions more thoroughly if compressed air is to be used.

4 CUT-AND-COVER CONSTRUCTIONS

For cut-and-cover constructions, the major fields of interest are: a) method of construction, b) design of retaining system, c) groundwater control and d) ground movements. In TRTS, the deepest excavation was 36.6m and was carried out for the construction of Ventilation Shaft A in Contract CP262 of the Panchiao Line. Although this depth of excavation is not surprisingly large in today's standard, the presence of the Chingmei Gravels was a potential threat to the safety during construction and called for precautionary measures. A similar situation was faced in some other contracts and different solutions were worked out.

4.1 Methods of Excavation

Basically, deep excavations in soft ground can be carried out by using: a) the bottom-up method, b) the top-down method, and c) the semi-top down method. As always, time and money are the two elements to be considered in the evaluation. As far as the construction period is concerned, the top-down method has the advantage that super-structure can be constructed while excavation is being carried out, and is hence favored for the construction of, for example, highrise buildings with deep basements. For rapid transit systems, however, it is usually necessary to complete the base slabs for the rails to be laid as soon as possible. Therefore, the bottom-up method is often the choice. On the economy side, it may be argued that the use of floor slabs as struts in the case of top-down construction leads to savings. However, on the other hand, earthworks are more efficient in the case of the bottom-up construction. Besides, to support the upper floors while excavation is proceeding in top-down construction does require additional efforts.

From a geotechnical point of view, studies indicate that the use of top-down method may lead to larger lateral deflections of the walls in comparison with the bottom-up method, hence, larger ground settlements behind the walls and greater damaging potential to adjacent buildings. The reasons are:

(i) The spans between floors in top-down constructions are usually larger than the spans between struts in bottom-up constructions, thus, it will take longer time to excavate for each level in the former case than the latter.

(ii) The walls will also have less number of levels of lateral support in the former case than the latter.

(iii) Casting of floors is more time consuming than strutting. More wall deflection will be induced as soils creep.

(iv) Struts can be preloaded to minimize lateral deflections of walls but not the floors.

It is possible to construct the first floor first using the top-down method and proceed with the remaining excavation using the bottom-up method. This becomes the so-called semi-top-down method. Of the 34 underground stations in TRTS, 23 were constructed by using the bottom-up method, 10 by using the semi-top down method but only 1 by using the top-down method. It is clear that the top-down method was not favored in construction of the TRTS systems.

4.2 Retaining Systems

Thick diaphragm walls are usually chosen for maintaining the stability of the side walls of deep excavations in soft ground. However, other types of retaining structures such as contiguous bored piles, soil-mix wall, and heavy sheet piles are also frequently adopted. Because of the poor ground conditions and the great depths of excavations, diaphragm walls were exclusively used for station excavations for TRTS. For shallower 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. In the old days, the single wall system was quite common because the diaphragm wall served as the structural wall as well. There is a growing concern on the poor water-proofing of diaphragm walls, and as a result, single wall system is gradually phased out and the double-wall system is more popular nowadays. In a few stations in 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. In hindsight, composite walls appear to be a nuisance. First of all, the provision of dowels makes the tremieing of concrete difficult. Secondly, these dowels have to be manually exposed and bent for the permanent wall to be cast. Unless space is crucial, the use of composite walls shall be discouraged.

Strutting is a common practice. However, ground anchors are sometimes used if an adequate hard pan is available at shallow depths. At launching and arrival shafts where struts may become obstacles to shield machines, anchoring is frequently the choice. It should be noted that in quite a few cases, ground anchoring led to serious inflow of water through holes made on walls and resulted in excessive ground settlements and damages to adjacent structures. Therefore, so long as possible, ground anchoring shall be avoided if the ground consists of sandy soils. Secondly, the consequences of failure, if so happens, will be more serious for anchored walls. Failure of an anchor may soon progress to the rest of anchors as soils are disturbed and lose their strengths. On the other hand, failure of a strut usually is localized because the capacities of struts do not rely on soil strength. It will also be much easier to remedy a failed strut than a failed anchor because the failed anchor might be several meters above the bottom of excavation and might not be accessible by machines. For all these reasons, preference shall always be given to strutting, rather than anchoring.

For the structural design of retaining systems, the use of Peck's apparent earthpressure diagram is till 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 envelope that encloses all the pressure diagrams derived from observations"

Peck's diagram was first published in the year of 1943 (Peck, 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 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 to a geotechnical engineer.

With proper judgment, Peck's earthpressure diagram appears to work well for various types of soils in Singapore (Hwang, Quah and Buttling, 1987; Hulme, Potter and Shirlaw, 1989). Studies on the TRTS are undergoing to see if Peck's diagram is applicable to the Sungshan Formation as well. 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' 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 modifications.

It is a well known fact that earthpressure on a wall is a function of lateral deformation of the wall. Figure 10 shows a typical plot frequently appearing in textbooks (Terzaghi, 1954). As can be noted that earthpressure on the wall also depends on the soil properties and direction of wall movement. Figure 11 shows a case history reported by Moh and Hwang (1993). The site, i.e., CPH Building, is located in the central city area of Taipei. Excavation was carried out to a depth of 17.4m using the top-down construction method and a maximum lateral deflection of 110mm was observed. Presented in Fig. 11(c) are the ratios of effective lateral earthpressures to the effective overburden pressures, denoted as Ra values, interpreted from the readings obtained by four earthpressure cells mounted on the active side of the diaphragm wall. As can be noted that the initial conditions for all the earthpressure

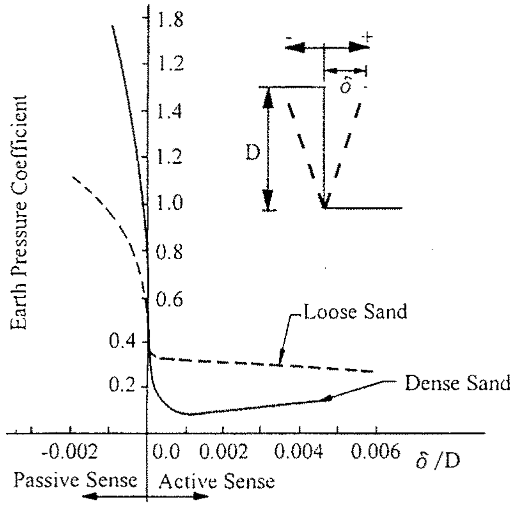

Fig. 10 Earthpressures as affected by wall movement (after Terzaghi, 1954)

cells were quite different with Ra values varying from 0.2 to 0.7. During the installation of these earthpressure cells, it was necessary to extrude the two loading plates of each cell by jacking laterally to make a good contact with the soils at the interfaces with the diaphragm wall on both sides. The initial horizontal earthpressures recorded by the cell were thus a result of jacking and do not necessarily represent the in-situ pressures.

The true in-situ horizontal earthpressures at the soil-wall contacts are difficult to ascertain even for the simplest type of soil, i.e., normally consolidated clay, because the ground has been disturbed in the process of installing the diaphragm wall. For all practical purposes, an initial Ra value of 0.5 can be assumed. Three of the four cells recorded pressures with initial Ra values greater than 0.5, presumably, because of over-jacking. The initial lateral pressures sensed by these cells, i.e., A, B and D, were in the passive state and corresponded to certain outward movements of the jacking plates. It is reasonable to correct the readings by shifting the three curves laterally to yield a Ra value of 0.5 at zero displacement, i.e., δ=0. The results are shown in Fig. 12 and, as can be noted that, after adjustments, the three curves are surprisingly consistent. The fourth curve with an initial Ra value of 0.2 is difficult to be corrected because the lateral pressure was too low to start with. The initial lateral pressure on this cell was already near its lower-bound, obviously, because the jacking plate did not have a good contact with the soil.

Based on Fig. 12, it is concluded that a displacement of 20mm is sufficient for the lateral earthpressure ratios to reach their lower-bound values. Ideally, earthpressure

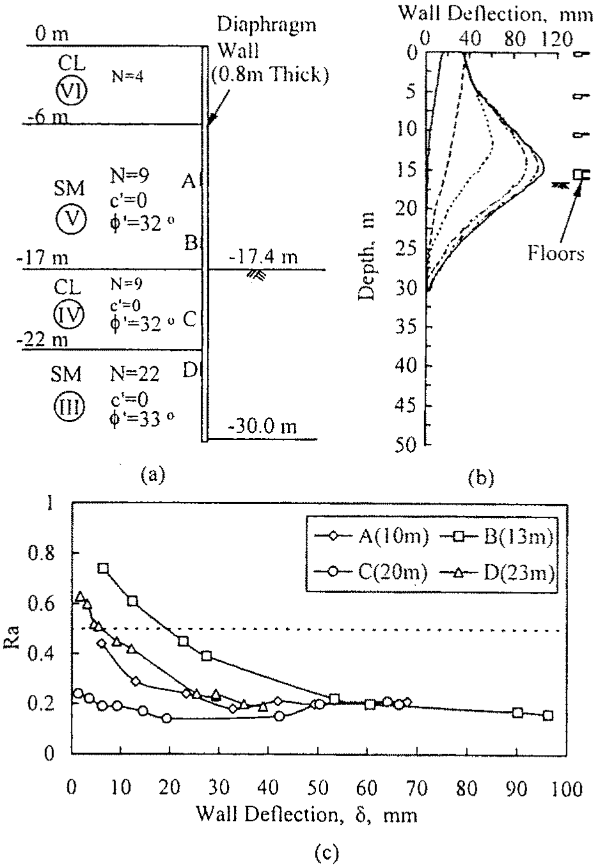

Fig. 11 Lateral earthpressure ratios at CPH building

cells shall be loaded to yield a lateral pressure ratio of 0.5, as assumed, during installation. However, this cannot be guaranteed as soils tend to creep with time. It is suggested to start with a pressure ratio of 0.6 to allow for creep. In such a case, according to Fig. 12, an extra amount of wall

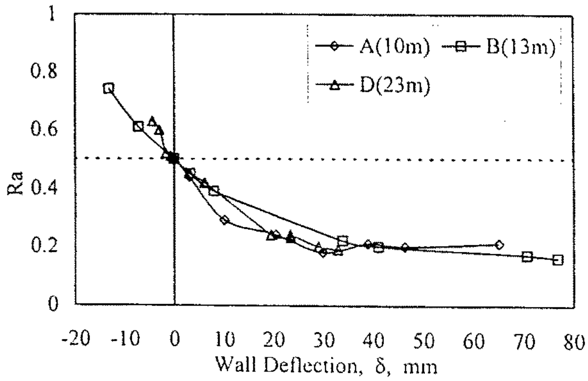

Fig. 12 Lateral earthpressure ratios after adjustment- CPH Building

deflection of 10mm is required for the active state to fully develop.

In the TRTS constructions, a considerable quantity of earthpressure cells have been installed on diaphragm walls. Some of the results are shown in Fig. 13. Because all the deep cuts were retained by thick diaphragm walls and all the struts were preloaded, wall deflections were generally small and only in few cases the lateral pressure ratios reached their lower-bound values.

It is a fact that the magnitudes of lateral pressures on walls are a function of wall friction. The angle of wall friction is often assumed to be a material property but this is incorrect. It depends on the directions of soil and wall movements, the amounts of movements and the properties of soils. Moreover it may vary along the wall. Hence, it is a response and not a property. Schofield (1961) reported observations of passive pressure on a model wall rotating into sand and found a substantial variation of wall friction during a test. A comprehensive series of large passive model tests have been described by Rowe and Peaker (1965) who explored in details the mobilization of wall friction as a function of relative density, wall movement and mobilization of shearing resistance. It is of interest to note that maximum mobilization of wall friction is not necessarily coincident with maximum shearing resistance. The practical significance of knowing more about the movements of a retaining wall and their influence on wall friction is illustrated by the failure of a high crib wall discussed by Tschebotarioff (1965). In this case the wall settled substantially causing the direction of the wall friction to reverse its usual attitude. This effect was sufficient to reduce the factor of safety against sliding along its base from 2.82 to an unsafe value. of 0.95 and hence account for the distress which was apparent in the wall.

As indicated in Fig. 11 that in all the four cases shown the lateral earthpressure ratios dropped to a value of 0.2 or so. This value is considerably low for the type of soil involved. The data are compared with the theoretical values proposed by Caquot and Kerisel (1948) in Fig. 14. As can noted that the four points (solid triangles) are below the line corresponding to $\varphi'=\phi'$, in which φ' = angle of wall friction and ϕ' = angle of shearing resistance of soil. The TRTS data, however, scatter over a wide range. There are cases, similar to the case of CPH Building, in which the data points fall below the line corresponding to $\varphi'=\phi'$, but there are also many cases in which the data points fall above the line corresponding to of $\varphi'=0$, implying negative friction angles on the active side of the walls.

The implication of Fig. 14 is unclear at this moment. The scatter of data points could have something to do with the vertical movement of the wall. As shown in Fig. 15, as excavation proceeds, the soils on the active side of a wall tend to drag the wall down. The downdrag and the self-weight of the wall are resisted by the frictional force on the passive side of the wall and the reaction at the toe. It is a fact that sludge is frequently present at the bottom of trench and the resistance at the toe of diaphragm walls is usually minimal, if any. As the excavation goes deeper and deeper, the frictional resistance to wall settlement on the passive side becomes less and less. It will not be a surprise if

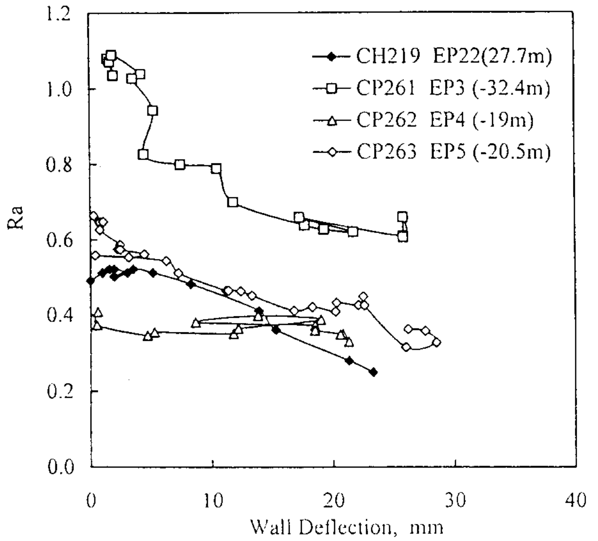

Fig. 13 Earthpressure measurements for TRTS

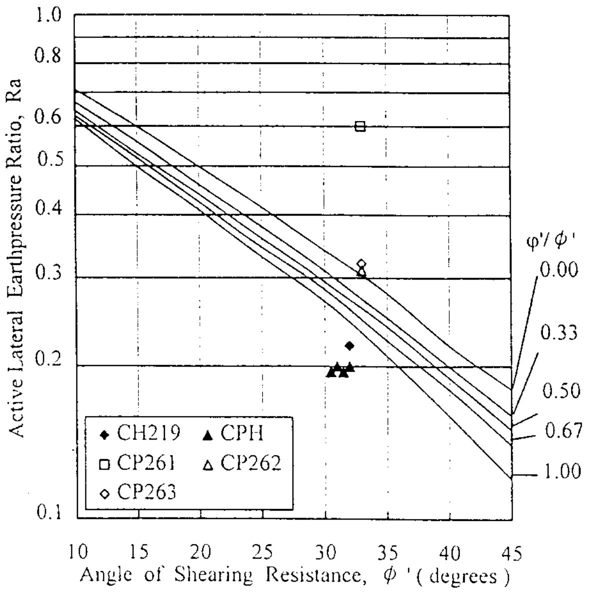

Fig. 14 Lateral earthpressure ratios for Taipei Silts

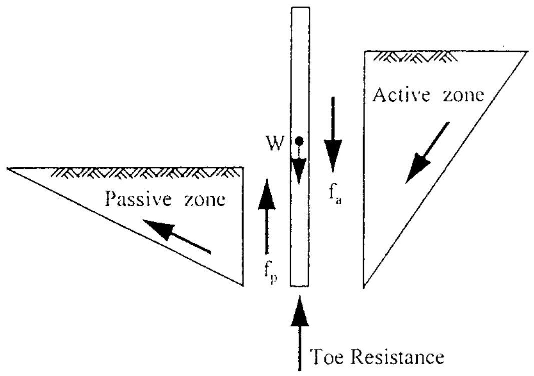

Fig. 15 Force equilibrium on diaphragm wall

some of the walls did settle. As a wall settles, the friction angle on the active side of the wall may drop. If the settlement of the wall is sufficiently large, the friction on the active side may even reverse its direction. This could explain the fact that some of data points in Fig. 14 correspond to very low, even negative, friction angles.

Nearly all the formulas imply a triangular distribution of earthpressures. However, this is far from being true in reality even for the most ideal situation of rigid walls rotating about their toes. Preloading of struts does add to the complexity of the problem. The way the lateral earthpressure ratios varied with wall movement in Fig. 13 is certainly different from what is shown in Fig. 11. Preloading created loops in the Ra-δ curves. Because of the fact that measurements were taken only weekly or even less frequently, what happened in-between readings could have been missed. Therefore, the curves are not as ideal as expected. It is reasoned that more wall deflections are required for the Ra values to drop to their lower bounds. This could be another reason that the Ra values for TRTS excavations are on the high side in Fig. 14.

The phenomenon that some of the data points fall below the line corresponding to $\varphi'=\phi'$ is rather difficult to explain. It could have something to do with the fact that the soils in the Sungshan Formation all have very high fine contents and possess apparent cohesion. The soil parameters obtained in drained tests may not be representative of the soil behavior during excavation. In any case, there are sufficient number of data points to indicate that if conditions are right, the full soil friction can develop on the wall, i.e., $\varphi'=\phi'$. This is of course limited to the cases of soft to medium stiff ground. It has been argued that the presence of bentonite cake at the soil/wall interface tends to reduce wall friction and a range of wall friction angle of $\varphi'=\phi'/3$ to $\varphi'=\phi'/2$ has been proposed by various researchers. The authors are of the opinion that the desiccation as concrete hardens is able to reduce the water content in the bentonite cake to the extent that the strength of the cake equals to the soil strength.

The above discussion does not necessary lead to the conclusion that $\varphi'=\phi'$ shall be assumed in design. As indicated in Fig. 14 that the wall friction varied from one case to another and it is conceivable that it may vary from one depth to another in any particular case. The amount of wall settlement is unpredictable, therefore, a relatively conservation attitude may be justifiable.

It must be pointed out that, although the above discussion serves the purpose of illustrating how complex the problem could be, earthpressure readings must be interpreted with care because measurement of earthpressure is an extremely difficult task, particularly, on diaphragm walls. Apart from the uncertainties associated with the installation, it may also be questionable that the readings obtained by pressure cells indeed represent the pressures on the wall because of the arching effects. In fact, in TRTS constructions, many earthpressure cells failed to give reasonable response even during installation and many became malfunctioned shortly afterward. Those readings which appear reasonable must be examined with care and frequently require corrections.

4.3 Groundwater Control

As mentioned previously that water was responsible for a great majority of failures occurring during excavations. As illustrated in Fig. 16, in the central city areas of Taipei, the ground surface is roughly at an elevation of RL 102.6m (mean sea level = RL100m) while the top of the Chingmei Gravels is at RL 53m or so. The piezometric level in the Chingmei Gravels was at RL 88m, for example, in 1990 when the construction for TRTS started and excavation was able to be carried out to a depth of 26.6m safely if a factor of safety of 1.25 is applied against blow-in. As indicated in Fig. 8 that the piezometric level may rise to RL 98m in 1999 when the construction of the Hsinchuan Line is scheduled to start. By that time, excavation can only proceed to a depth of 20m, instead of 26.6m, without additional measures for controlling groundwater. It should be noted that it has been assumed in the above discussion that a clay blanket is present right at the contact between the Sungshan Formation and the Chingmei Gravels. Such a blanket may be at a higher level at places and the safe excavation depth will be less in such cases.

There are several ways to increase the factor of safety if blow-in is indeed a concern. In constructing the

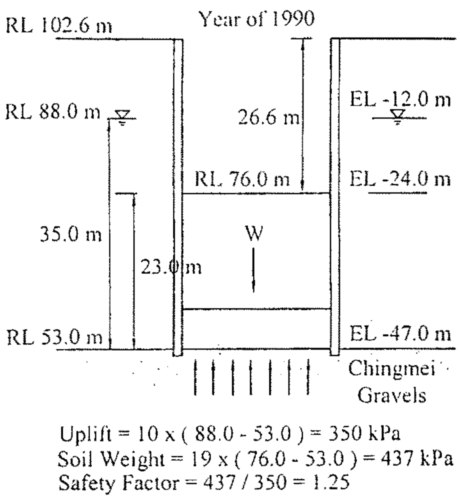

Fig. 16 Analysis for blow-in

ventilation shaft of Contract CH221 of the Hsintien Line, as depicted in Fig. 17, excavation was carried out to a depth of 35m below surface. The shaft is 23.6m in its inner diameter and is formed by 16 diaphragm wall panels of 1.2m in thickness. To prevent blow-in from happening, diaphragm walls were extended by 31m into the Chingmei Gravels, giving a total length of 65m, and a grout seal was formed at the toe level to increase the length of the soil plug. Because of the extreme difficulty in penetrating into the Chingmei Gravels, it took half a year to complete these diaphragm wall panels.

The effectiveness of this grout seal in cutting off seepage flow was confirmed by pumping tests carried out in the soil plug. A 30 Hp submersible pump was installed at RL 60m. Pumping continued for 19 hours and a quantity of 204 cubic meters of water was pumped out. The water level in the soil plug dropped from RL 92m to RL 67m. It rose to RL 72.5m in 6 hours after the pump was turned off. Subsequently, it rose by 1.14m in 32 hours. Based on these data, the permeability of the grout seal was estimated to be roughly 4×10^{-7} m/sec.

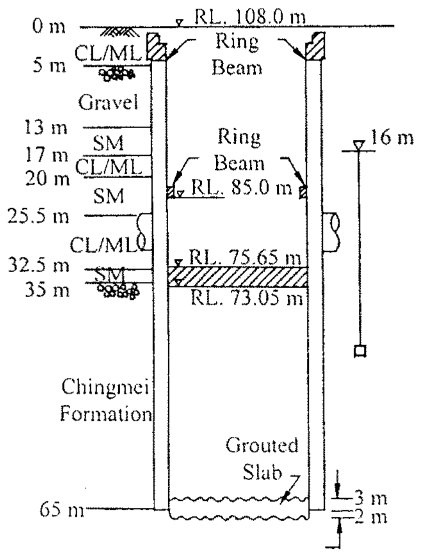

Fig. 17 Grout plug at Ventilation Shaft in Contract CH221

The wall was supported by two ring beams without internal bracing. Lateral deflections of the wall were within 10mm, which were very low in comparison with the deflections observed for box-shape braced excavations with similar depths, indicating that circular shafts outperformed rectangular ones as far as wall deflection is concerned.

In constructing Ventilation Shaft A (Contract CP262) and Shaft B (Contract CP261) in the Panchiao Line, refer to Figs. 18 and 19, respectively, excavation was carried out to depths of 36.6m and 34m. Pumping was carried out to lower the piezometric levels in the Chingmei Gravels by 10.7m and 9.5m in the two cases and the rates of discharges reached maxima of 4,170 and 3,600 cmh (cubic meters per hour), respectively (Hwang, et al., 1996). Similarly, in constructing the Taipei Main Station, refer to Fig. 20, excavation was carried out to a depth of 28.9m and the piezometric level in the Chingmei Formation was lowered by 5.2m by pumping at a maximum rate of 2,450 cmh.

During the peak of construction, the zone of influence with a drawdown of 2m or greater in the Chingmei Gravels stretched to a radius of 10 km. In carrying out pumping at such a scale, it is important to ensure that ground settlements are limited to within tolerances so the adjacent

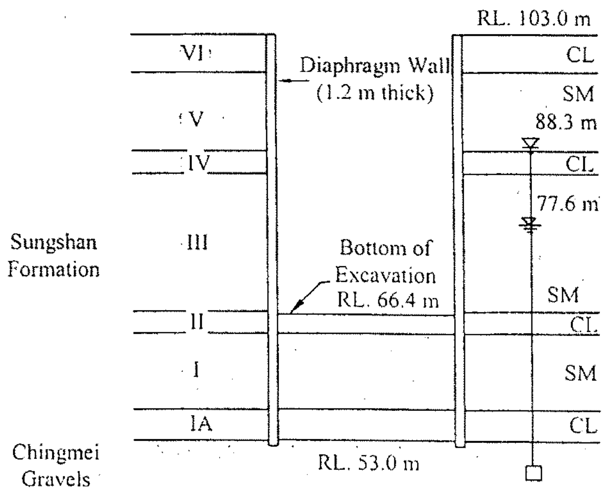

Fig. 18 Pumping at Ventilation Shaft A in Contract CP262

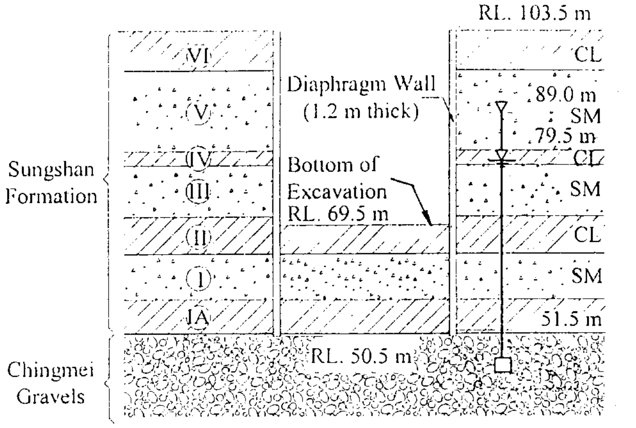

Fig. 19 Pumping at Ventilation Shaft B in Contract CP261

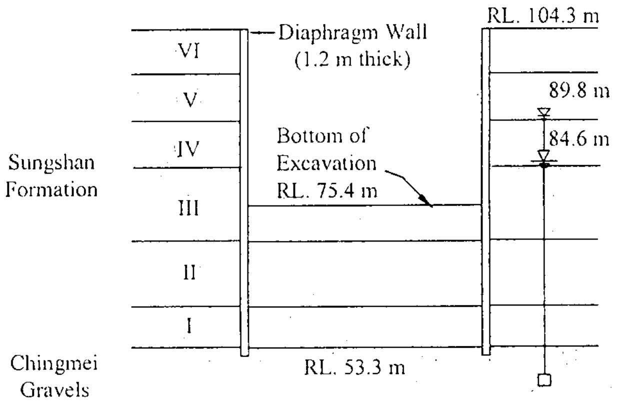

Fig. 20 Pumping at Taipei Main Station in Contract CT201F

buildings will not be damaged. Fortunately, the piezometric levels in Sublayers III and V in the Sungshan Formation, refer to Fig. 9, were not much affected and ground settlements induced as a result of pumping were, in general, well within 10mm.

It may be of interest to mention that in Contract 310 of the Singapore Mass Rapid Transit System, an innovative scheme was adopted to deal with the problem of blow-in in a different way. As illustrated in Fig. 21, excavation was first carried out to a depth of 7m with two levels of struts installed (Hulme, Potter and Shirlaw, 1987; Clark and Prebaharan, 1987). The cofferdam was then flooded with a water level in the cofferdam 2m higher than the ground and the remaining excavation was carried out underwater by dredging. When the final excavation level was reached, bored piles (drilled shafts), 1m in diameter, were installed to act as the station foundation. A 1.5m thick concrete slab was then poured using the tremie method before the cofferdam was dewatered. This unreinforced slab acted as bottom strut and were anchored down by these bored piles. They resisted base heave pressures once the cofferdam was dewatered. The cofferdam was dewatered and the station box constructed. In fact, the entire offshore section of tunnel box between Marina Bay Station and Raffles Place Station was completed in a similar manner.

This scheme was adopted because of the presence of very poor soils to a depth of 40m or so. The site was located in a piece of land reclaimed not too many years before the commencement of construction and ground settlements were continuing when the construction started. The deep excavation, 18m, would require very thick retaining walls and very many levels of struts. Although it was not meant for solving groundwater problem, this method can indeed serve the purpose.

4.4 Lateral Ground Movements

In addition to the safety of retaining structures, a major concern in deep excavations is ground movement outside the excavation. Excessive ground movements may become detrimental to adjacent buildings. One of the major contributors to ground settlement is the lateral deflection of retaining walls. For deep excavations, the lateral deflections of walls are routinely monitored by using inclinometers. Inclinometers are amazingly accurate and can be considered as one of the most reliable types of instruments for geotechnical engineering. However, this does not mean that inclinometers always faithfully report wall deflections. In quite a few cases, inclinometers were not anchored in a stable stratum and toe movements were

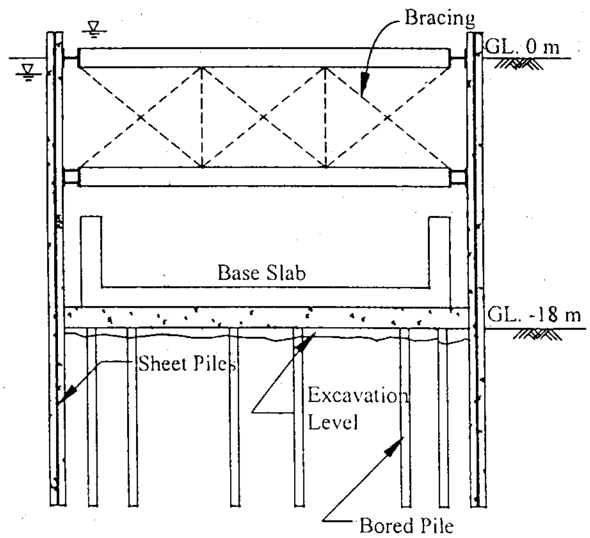

Fig. 21 Flooding at Marina Bay in Contract 310 of SMRT

large. This is particularly true for inclinometers which are cast in diaphragm walls. In such cases, usually, inclinometers are only installed to the toe levels of the walls. The toe of an inclinometer is normally assumed to be fixed and the movements at all other depths are computed in relation to the toe. Although sometimes specifications do require the movement at the top be measured for calibration. In reality, this is difficult to be done. Figure 22 shows the readings obtained for two inclinometers, SID4 (at Sun Yat Sen Memorial Hall Station) and SID6 (at Subway to Chungshan Science Park) of Contract CN256 of the Nankang Line, installed in diaphragm walls, both to the toe levels. Because of the different excavation depths in the two cases, 16.2m and 11.1m, respectively, the diaphragm walls were different in thickness, i.e., 1,000mm and 800mm, and in length, i.e., 53m and 26m. The two sites were both located in the K1 Zone, refer to Fig. 2, and the ground conditions at the two places are apparently similar. When excavation reached a depth of 11.1m, Inclinometer SID4 showed a maximum deflection of 45mm while Inclinometer SID6 showed a maximum deflection of only 20mm. At a depth of 26m, which corresponds to the toe level of Inclinometer SID6, a movement of 30mm was observed by Inclinometer SID4. It is thus suspected that the toe of SID6 would have moved by a similar amount. After correcting the readings of Inclinometer SID6 for this anticipated toe movement, the two sets of readings were very close. The slightly smaller deflection for Inclinometer SID4 could be due to the thicker wall and deeper penetration of the wall.

Whenever a large outward movement is observed, as is the case for the top of Inclinometer SID6 in Fig. 22, an immediate question to ask is "Is the toe moving?". This however does not preclude the possibility of having outward wall movement. Outward movement, if any, could be due to (a) different ground conditions between the two sides, (b) unbalanced excavation, (c) excessive preload

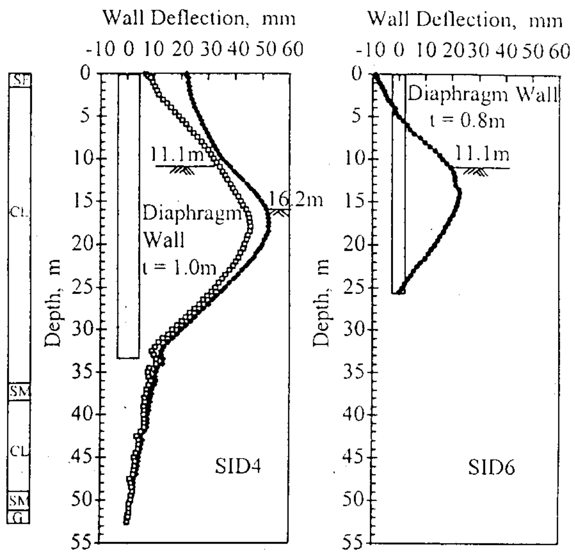

Fig. 22 Effects of toe stability on inclinometer readings

of struts, and (d) bulging in of the lower portion of the wall. Figure 23 shows a case in which the site is underlain by an inclined hard pan with a differential depth of 20m between the two sides. Excavation was carried out to a depth of 23.3m and six levels of struts were used. When the first level struts were installed, the deflections of the two inclinometers installed in diaphragm walls on the two sides were practically the same with maximum deflections of about 40mm at the top. However, at the final stage, Inclinometer SI5, refer to Fig. 23(a) on the side with a deeper depth to the bedrock bulged in to give a maximum deflection of 200mm while Inclinometer SI3, refer to Fig. 23(c) on the other side with a shallower depth to the bedrock was pushed outward with a net outward movement of a few millimeters at the top. This was obviously a result

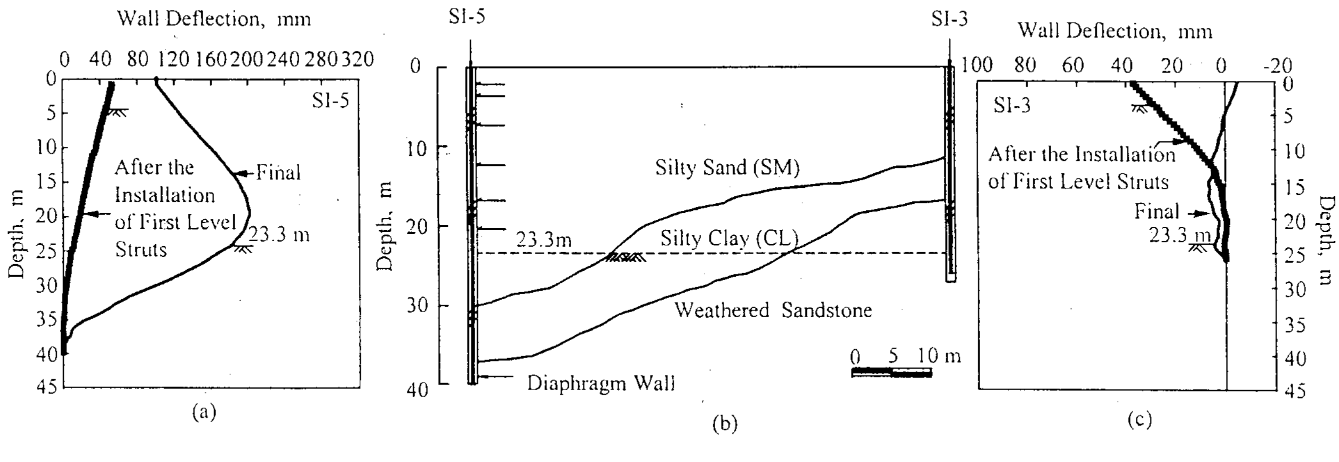

Fig. 23 Effects of geology on wall movement

of imbalance of pressure on the two sides.

To assist the interpretation of inclinometer readings, it is suggested to install inclinometers in pairs, one at each end of the same strut so the wall deflections can be double-checked. It is not difficult to show that once a strut is preloaded to, say, 50% of the design load, the subsequent shortening shall be very minimum. Therefore, the two ends of a strut shall move by the same amount subsequently but in the opposite directions.

Because of the stringent limitation on ground settlements for the purpose of limiting damaging potential to adjacent buildings, the diaphragm walls used in the TRTS are, in general, thicker than the walls used previously in Taipei by 200mm or so. This, together with the heavy preloading and strict supervision, reduced the lateral movements of retaining walls in TRTS to only 30% to 50% of what were reported in the old days. Because damages to a building may result in disputes, court injunction, and many undesirable consequences, in comparison, the extra 200mm thickness of diaphragm wall is indeed cost-effective.

4.5 Ground Settlements

Ground settlements induced during deep excavations have been extensively studied by many researchers, and for this reason, it is not intended to discuss ground settlements in detail herein. Recent studies indicate that the geometry of the pit has profound influence on the pattern of ground settlements. Figures 24 and 25 show the ground settlements as a function of distance to the corner of excavation, the so-called corner effects (Wong and Patron, 1993). In the former figure, settlements were normalized to the maximum settlements in individual sites and the distances were normalized in respect to the lengths or widths of individual excavations. In the latter figure, the distances were normalized in respect to the excavation depths. Both figures indicate that ground settlements at corners are about 20% of the maxima and as the location moves away from the corner and towards the mid-span of excavation, ground settlement increases. This finding is in line with those obtained by others (Wong, 1987; Ou and Chiou, 1993).

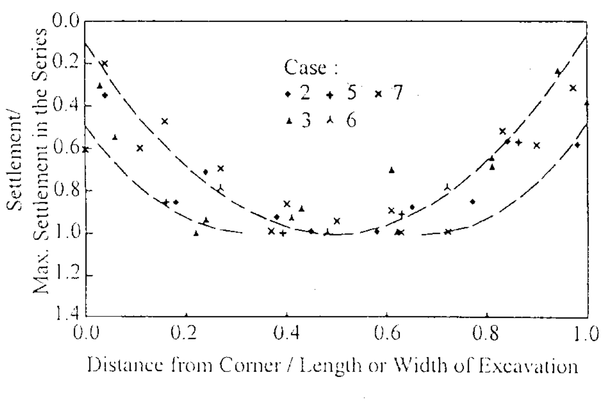

Fig. 24 Corner effects on ground settlement

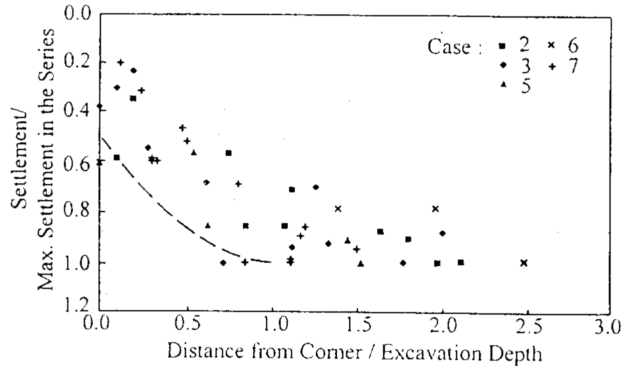

Fig. 25 Corner effects for long excavation

It should be noted that presumably the data presented in these two figures include only settlements induced by excavation and excluded settlements induced by diaphragm wall installation. Figure 26 shows a case in which the settlements at the corner of a building induced as a result of installation of diaphragm wall panels for a subway station and the entrance accumulated to 30mm. The influence of each panel was found to stretch to a distance of 10m and there were a total of nine panels within this distance. In a few incidents, digging trenches, usually 2m or so in depth, in very poor ground, or in poor backfill, for constructing guidewalls led to settlements of 30mm or so. It is therefore suggested that monitoring of building settlements shall start before the installation of diaphragm walls.

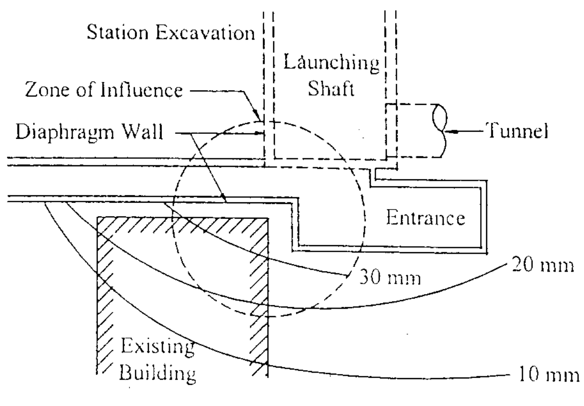

Fig. 26 Settlement due to diaphragm walling

5 TUNNELLING

Tunnels linking subway stations are usually constructed using: a) cut-and-cover method, b) NATM tunnelling method and c) shield tunnelling method. The choices are governed by tunnel configuration, ground conditions, traffic conditions, etc. The cut-and-cover constructions for tunnels are not different from the cut-and-cover constructions for stations and thus will not be further discussed.

5.1 NATM Tunnelling

In this part of the world, the so-called "New Austrian Tunnelling Method" (NATM) appears to have deviated from its original context of being merely a concept in the spirit of the observational method for tunnelling and have been extended to mean all types of tunnelling without using a shield. Such a deviation does solve the dilemma of lacking a proper name for tunnelling in soft ground other than shield tunnelling.

The New Austrian Tunnelling Method was used in the TRTS to drive two adjoining sections, one of 225m in length and the other of 487m in length in the Mucha Line. The two sections are separated by a short cut-and-cover section of only 32m in length. The twin tunnels have a horse-shoe shape with heights varying from 6.09m to 7.14m and a base varying from 9.17m to 9.48m in width.

The twin tunnels run through highly fractured shale and were lined with shotcrete of 100mm in thickness and wire mesh. Steel ribs were installed only as necessary. Rock bolts, 29mm in diameter and 4m in length, were installed at 1m intervals along the longitudinal direction and at 2m intervals along the transverse direction. Secondary lining was made of 300mm in-situ concrete. Excavation was carried out without major problems except that the roof caved in accidentally when an abandoned passageway was encountered above the crown. This passageway was once used for coal mining and was abandoned years before the TRTS construction (Guo, Yeh and Cheng, 1992). As a result, a volume of 50 cubic meters of debris fell into the tunnel.

The convergences of sections were closely monitored and they varied between 10mm to 20mm except that a maximum of 40mm was observed in one of the sections. Settlements of the crown were generally less than 40mm while a maximum of 63mm was observed in one of the sections.

A section the Hsintien Line was also driven using the NATM tunnelling method (Huang, 1997; Yang, et al., 1997). The section is 222m in length and is too short for shield tunnelling. Figure 27 shows a cross section of the tunnels. The tunnels were bored through Sublayer 5 of the Sungshan Formation with the crowns at depths of 8m to 11m below surface.

Excavation was carried out in two headings in each tunnel drive. The upper heading was kept at a distance of 2m to 4m ahead of the lower heading. Lattice girders were installed at 1m intervals and the tunnels were protected by shotcrete, 250mm in thickness, and wire mesh as primary lining. For maintaining the stability of the headings, steel lagging sheets, 6mm in thickness, 200mm to 300mm in width and 2m in length, were closely spaced to make a canopy. The tunnels were finally lined by 350mm reinforced concrete as permanent lining.

The soft ground called for the use of compressed air to a maximum of 1.35 bar. Construction was carried out in such a way that the two tunnels were inter-connected, as shown in Fig. 28, by a cross drift so that both tunnels were able to be pressurized by using a single set of compressed air facility. Also shown in the figure is the sequence of excavation. Excavation was carried out in five stages. Stage 1 excavation was carried out in free air for providing a space to house the compressed air plant. The rest of excavation was carried out in compressed air. Air pressure was not released till both tunnels were fully excavated and primary lining was completed.

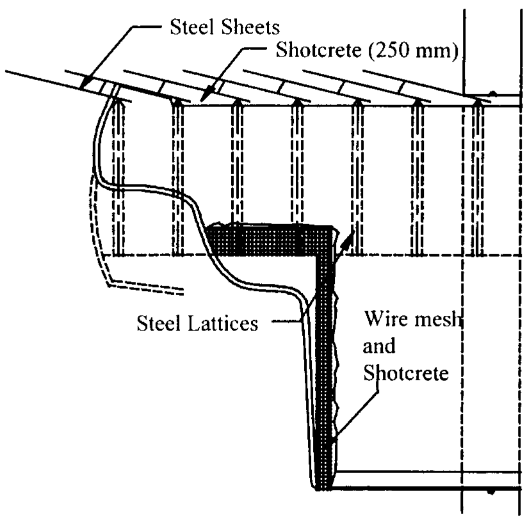

Fig. 27 Profile for NATM tunnels in Contract CH221

The consumption of compressed air was about 110 cubic meters per minute, refer to Fig. 29, when tunnelling was carried out in the Up-Track tunnel in the Stages 2 and 3 excavation before a layer of gravel was first encountered at the face at the halfway of the drive. It increased to 270 cubic meters per minute by the time the heading reached the end of drive. It was maintained at 170 to 190 cubic meters per minutes during the Stage 4 excavation for the Down-Track tunnel. Again, as the gravel layer was encountered, the air consumption increased to a maximum of 280 cubic meters per minute and the four compressors, with a power of 340 kilo-watts each, was fully loaded. As the tunnels were fully lined, the air consumption dropped to 140 cubic meters per minute.

Tunnelling was completed not without problems. Pressurized air traveled to a distance of as much as a couple of hundred meters and escaped to the ground through fissures and/or poorly backfilled utility trenches. An emergency situation was encountered on 29 March, 1994 when air escaped through the fissure left in place after sheet piles were withdrawn and the cracks in the base slab of a pedestrian underpass, refer to Fig. 30, and carried much water and solids into the underpass. As a result, a sinkhole of 70 cubic meters in volume was created at ground surface. Five days later, a minor explosion, presumably, due to the ignition of gas leaking from a gas line, shook nearby houses and broke window glass. The gas could have accumulated in a covered box culvert to a sufficient concentration for ignition. In fact, the explosion occurred at a location quite far away from the tunnel alignment and the steel covers of a couple manholes were blown off as a result of explosion. Although damages were minimal, it did cause panic of local residents. As a precaution, the pressure of compressed air was lowered from 1.2 bar to 0.4 bar and maintained at that level for about half a month. The excavation was suspended and the method of construction was carefully examined. It was resumed 4 months later after the situation was judged to be stable and the safety of the works was assured.

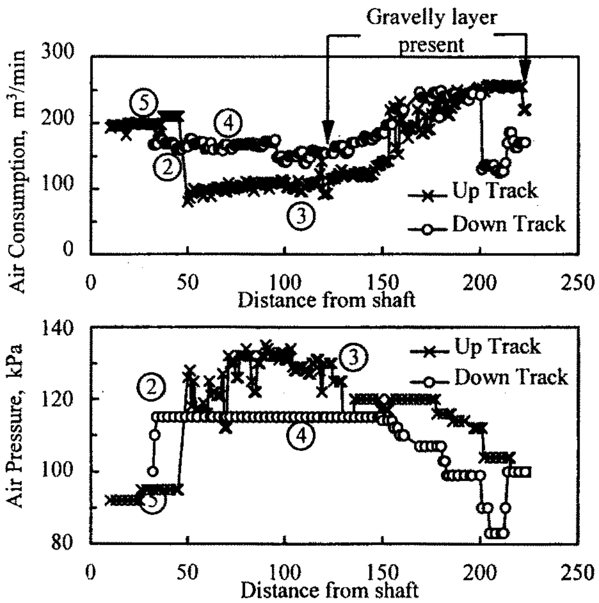

Fig. 29 Pressure and air consumption for NATM tunnelling

The most scary incident occurred on 22 March, 1995 when a transformer malfunctioned and disrupted the electrical supply. Air pressure dropped to 0.2 bar in 12 hours before the transformer was replaced and the electrical supply was back. At that time, refer to Fig. 28, Stage 3 excavations had already been completed and the heading was at Ch 0+180 of the Down-Track tunnel. In other words, there was a length of 352m of tunnel already been driven. Although the primary lining was designed to hold the tunnels even without compressed air, the face did rely on compressed air to stand up. A 2000 mm diameter water main runs across the two tunnels and supplies water to the entire Taipei City. Should it rupture, the consequence would be disastrous. Fortunately, the face had been stabilized by grouting for reducing the loss of compressed air. The contractor was able to replace the transformer promptly and

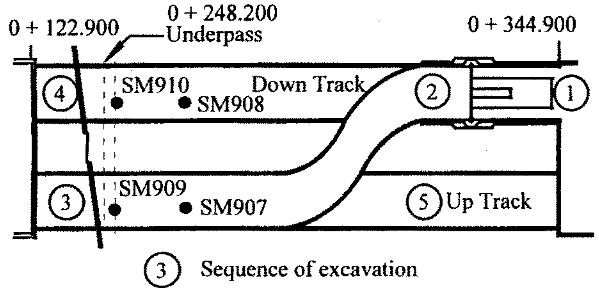

Fig. 28 Plan of NATM tunnels in Contract CH221

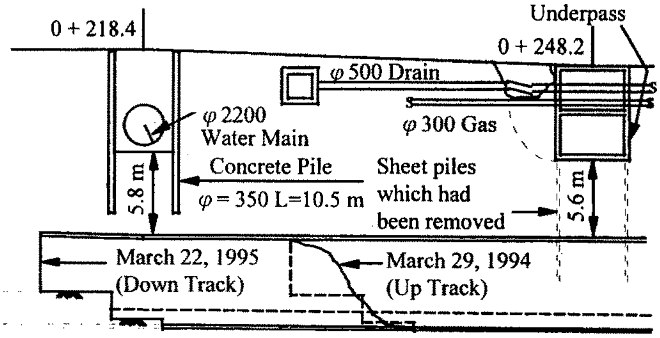

Fig. 30 Incidents occurred along the route of NATM tunnels

the crisis was resolved without even minor damages.

Shown in Fig. 31(b) are the readings of air pressuremeter AP-908, which was buried at a depth of 14m. Figure 31(c) shows the variations of water levels in two observation wells, i.e., OW-905 and OW-906, of which the tips were buried in Sublayer V of the Sungshan Formation. In general, the water levels moved in phase with the air pressures. The compressed air was totally released on July 25, 1995. The Up-Track tunnel was lined in the period of February to October, 1995 and the Down-Track tunnel was lined in the period of July, 1995 to February, 1996. As can be noted that as the tunnels were lined, the water levels in the Sungshan Formation gradually returned to their natural level of RL 102m.

Figure 31(d) shows the time histories of ground settlements. Settlement Markers SM-909 and SM-910 were located on top of the pedestrian underpass and showed much smaller settlements in comparison with others. The final ground settlements were between 120mm to 170mm which are about 2 to 3 times of the settlements induced by shield tunnelling without compressed air. Whether the ground settlements would have been the same had the two incidents not occurred is arguable. The two incidents did result in two unexpected cycles of compression-and-decompression. However, at the times when these incidents occurred, the excavated tunnel drives had already been supported by steel lattices and lined by using shotcrete and wire mesh. This situation was not any different from the situation when the compressed air was totally released at the end. It is unlikely that the final settlement would have been much less than what was observed.

A few workers who worked in the tunnels for a considerable duration, suffered from diver's disease (aeroembolism) due to improper decompression and had to be treated. This has raised serious concern by the Labors' Commission and the City Council of Taipei. The use of compressed air in another contract which was still ongoing when the problem surfaced was banned. Therefore, it is doubtful that compressed air tunnelling would gain a broad acceptance in Taiwan in the future.

Despite the fact that NATM tunnelling in soft ground is popular in Europe, this is the first time it was carried out in Taiwan. The method cast serious doubt when it was first proposed. Notwithstanding all the problems, purely from a technical point of view, the method was proved to be a success. The twin tunnels were driven with less disruption to traffic and less construction cost in comparison with the cut-and-cover method. However, it has to be admitted that the operation is highly risky and any major accident would

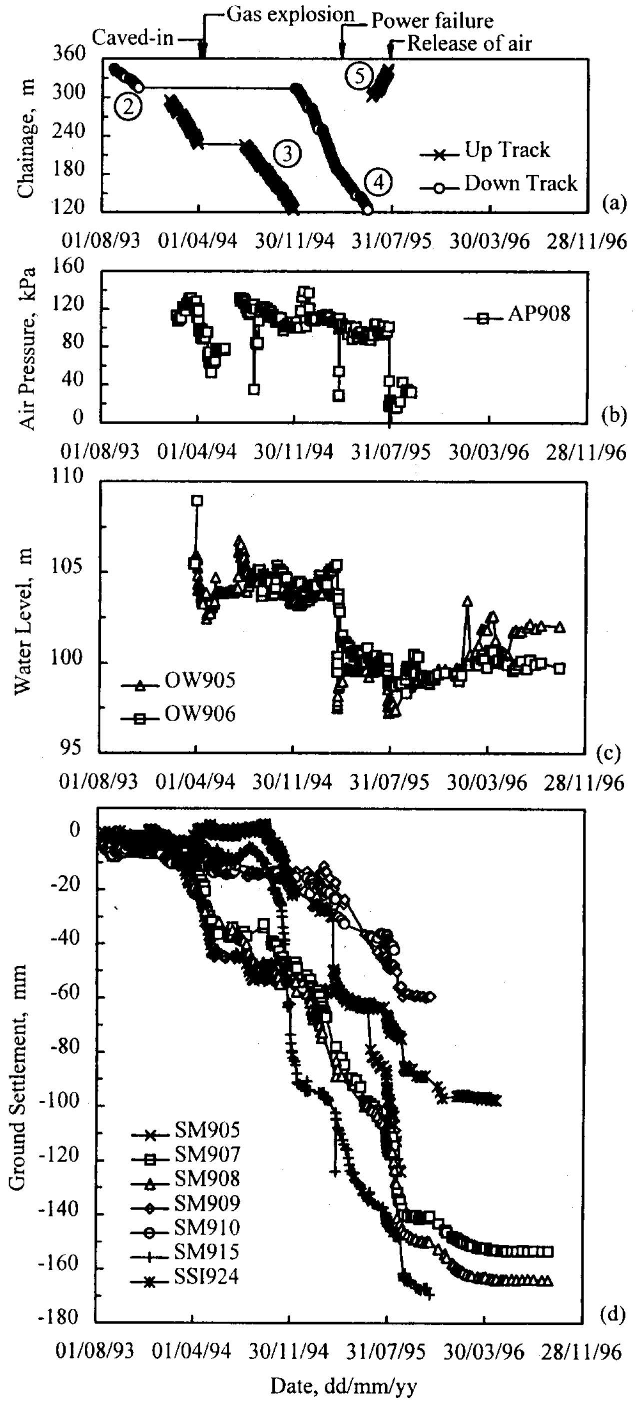

Fig. 31 Progress of NATM tunnelling and instrument readings

put the designer in an extremely difficult position defending himself.

5.2 Shield Tunnelling

Up to this year, there are a total of 58 drives in the Initial Network of TRTS excavated by shield tunnelling and 30 shield machines have been adopted. Except the two slurry shield machines, which were used to drive the four tunnels in the Contract CH221 of the Hsintien Line, all other contractors used earthpressure balancing shield machines for the job. Notwithstanding, all the earthpressure balancing shield machines were equipped with facility of injecting slurry or chemicals to deal with difficult ground.

Apart from the problems with methane and drift woods, the ground conditions in the Taipei Basin are ideal for shield tunnelling. The silty sand and silty clay in the Sungshan Formation can be handled by either earthpressure balancing shields or slurry shields with ease. The progress was in general satisfactory and a daily production rates of 10 to 20 rings, 1m in length each, were quite normal. A maximum daily rate of 47 rings was achieved in one of the tunnel drives in the Chungho Line. This impressive rate was achieved without any particular measures taken other than the incentive given to the crew.

There were a few incidents which led to serious consequences during tunnelling (Lin, Ju and Hwang, 1997; Moh, Ju and Hwang, 1997). Nearly all of them occurred either upon launching or upon arrival of shield machines and groundwater was a major source of problem. As openings are made on diaphragm walls, refer to Fig. 32, to prepare for launching or arrival of shields, water tends to leak into the shafts from gaps behind the retaining walls. The Taipei Basin was once a giant lake and was infilled by sediments not long ago. Sublayers 3 and 5 in the Sungshan Formation are very permeable and the Chingmei Formation underlying the Basin is an ideal aquifer with ample water reserve. If an opening is made at a location very close to the Chingmei Gravels, chances are, ingress of water may soon become uncontrollable.

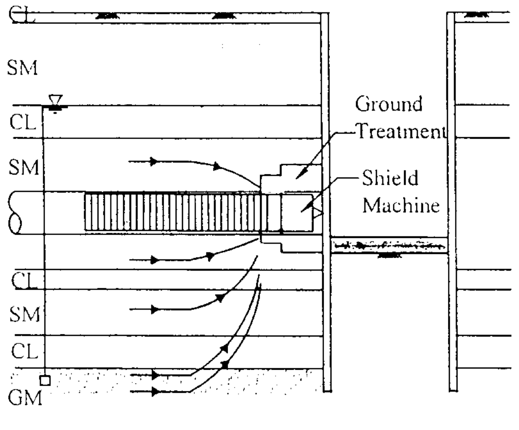

Fig. 32 Ground treatment and potential problem at portal

To prepare for launching or arrival of shield machines, it is a common practice to form protective annular shelters for the shield machines to temporarily stay by treating the ground with high pressure jet grouting. The quality of ground treatment is certainly of great importance and has to be confirmed by coring and tests. Strength and permeability of the treated ground are the two parameters frequently specified. However, it has been found that check borings did not reveal all the potential problems. Particularly, the results of unconfined compression tests tend to lead to misjudgment because only good specimens are tested. Furthermore, in most of the incidents, strength of the treated ground is irrelevant. In fact, too-high strength of the treated ground was responsible for a few incidents in which the advancement of shields was hampered and sinkholes were formed in front of the faces as soils kept on moving into the earth chambers. One way to prevent such a problem from occurring is to add a 2m to 3m extension at the end of the block with weaker strength.

It is the permeability of the treated ground that matters. The conventional type permeability tests are not useful at all. Even Lugeon tests do not necessarily provide meaningful results. In the authors' opinion, the integrity is a better indication of the quality of treated ground and the integrity of treated ground is better represented by core recovery ratio and rock quality designation (RQD). Unfortunately, criteria have not been established using these two indices. In the lack of precedents, the authors wish to suggest a core recovery ratio of 90% and a RQD of 60% as the minimum requirements for ground treated using jet grouting. This is subject to discussion and suggestions are certainly welcomed.

For reducing the risk associated with seepage flow, it is more effective to increase the length of water path than to reduce the permeability of the treated ground. It is thus very necessary to have extra length and extra thickness of the ground treatment whenever the tunnel-station connection is very close to a water-bearing stratum. As a general rule, never rely on a single row of jet columns, no matter how large the diameter is, to serve as a curtain for cutting off seepage flow. The verticality of drilling is difficult to ascertain and it could well be gaps between columns. A minimum of two rows is absolutely necessary and, at locations where consequence of seepage could be serious, three rows are suggested.

As an excavation proceeds, the wall at the tunnel-station connection tends to move inward. Because the block of treated ground is relatively rigid, gap may occur behind the wall to become a water path. It is therefore necessary to drill a few holes on the wall to see if water does flow. It will be a good idea to apply sealing grouting anyway. The cost is really minimum and many problems can be avoided beforehand.

5.3 Ground Settlement over Tunnels

Settlements over tunnels are usually analyzed by using the Peck's method by assuming that settlement troughs are in a shape of normal distribution (error function), refer to Fig. 33 (Peck, 1969):

$$\delta = \frac{vA}{2.5i}\exp\left(\frac{-x^2}{2i^2}\right) \tag{1}$$

$$v = \frac{2.5i\delta_{max}}{A} \tag{2}$$

where

δ	= ground settlement
δ_{max}	= maximum ground settlement
x	= distance to tunnel center
i	= width parameter
v	= ground loss
A	= sectional area of the tunnel

Performance of tunneling is usually evaluated in terms of ground loss which is the ratio of the area of settlement trough to the sectional area of tunnel. For soft ground, long-term consolidation settlements could be significant. Therefore, it is important to differentiate immediate settlement from long-term settlement because they have different mechanisms and are governed by different factors. Furthermore, since long-term settlement may drag on for months, settlement troughs in different cases cannot be compared fairly unless a common criterion is adopted in determining the portion of settlements to be included in the evaluation. Figure 34 shows an idealized time history of settlement in a semi-log scale with the elapsed time after passing of the shield in the abscissa. For simplicity, the straight portion of the plot toward the end can be considered as long-term settlement which may be attributed primarily to consolidation and the preceding settlement can be considered as immediate settlement which is primarily a result of the closure of tail void. Studies showed that the transition occurred in four to ten days after the passing of the shield machine. For all practical purposes, it is appropriate to take the settlement in 10 days as immediate settlement and denoted as δ_{10} (Hwang, Moh and Chen, 1996; Hwang, Sun and Ju, 1996). Consolidation settlement is represented by the slope of the curve toward the end, and for all practical purposes, can be taken as the settlement occurring between the 10^{th} day and the 100^{th} day after the passing of the shield.

Ground loss can be computed by using Eq. (1) based on settlement readings. However, settlement readings do not necessarily fall on a nice curve as expected and it is frequently necessary to find the best-fit curves by using a trial-and-error process. This may serve practical purposes but the results may be subjective. Furthermore, the process does take time if the quantity of data is large. A more rational and less time-consuming method is thus desired. First, Eq. (1) is transformed to a linear equation,

$$t = ms + b \tag{3}$$

by letting

$$t = \ln\delta \tag{4}$$

$$s = X^2 \tag{5}$$

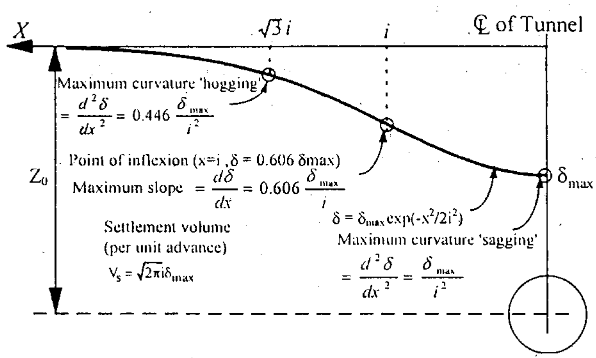

Fig. 33 Idealized settlement troughs above tunnels

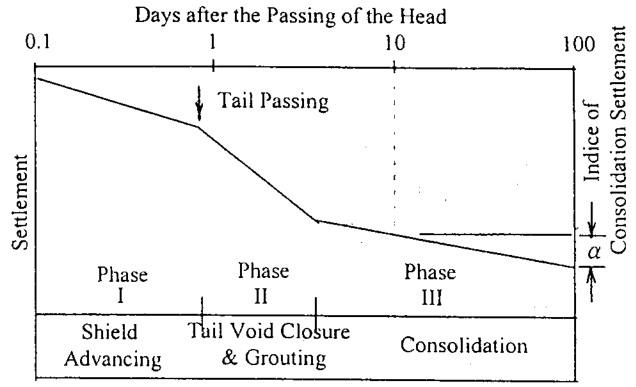

Fig. 34 Idealized time history of settlements above tunnels

The two parameters, m and b, which are the slope and the zero-intercept of the line, are readily obtainable from regression analyses using spreadsheet computer software, such as EXCEL or Lotus 123. The width parameter, i, and maximum settlement, δ_{max}, can be computed as follows:

$$i = \sqrt{\frac{1}{m}} \tag{6}$$

$$\delta_{\max} = \exp(b) \tag{7}$$

It has been noted that most of the settlement troughs are not symmetrical to the center of the tunnel as the theory assumes. The deviations may be due to spatial variation in soil stratum, imbalance of earthpressure in front of face, back grouting, and secondary injection, etc. It is also logical to expect such deviations in sections involving horizontal curves because of the use of copy cutters. In any case, such eccentricity shall be adjusted for the regression analyses to be correct. First, let

$$X = x + \varepsilon_0 \tag{8}$$

where

ε_0 = deviation of the center of trough from the tunnel center

and choose the ε_o value which gives the greatest "correlation coefficient" by the trial-and-error process.

Figure 35 shows the settlement trough obtained at Ring No. 224 of the Down-Track tunnel, driven from NTU Hospital Station toward CKS Memorial Hall Station, of Contract CH218 of the Hsintien Line. At this section, a deviation of ε_o =2.15m gave the best fit of data. Part of this deviation was due to the use of copy cutter for making the turn. The radius of gyration in this section is 380m and a copy cutting of 40mm to 60mm was used. For an outer diameter of 6,050mm for the shield and 5,900mm for the segment, a theoretical eccentricity of 0.5m can be expected from over-cutting alone. The remaining eccentricity could be due to different ground conditions on the two sides of the tunnel alignment and other factors which are yet to be quantified.

Ground loss, v, and width parameter, i, are the two parameters representing a settlement trough. They are needed for estimating ground settlements in the design. These two parameters can be correlated with ground conditions and other factors such as tunnel depth, method of tunnelling, etc. Empirical relationships in various types of soils have been proposed by many researchers. However, their validity to local soils has to be examined.

Figures 36 shows the soil profiles for Contracts CH218 and CH223 of the Hsintien Line and CP261 of the Panchiao Line. These three contracts are located in Zones T2, T1 and B2, refer to Fig. 2 for geological map, respectively. The trough width parameters of these three contracts are compared with the charts proposed by Peck (1969), Clough & Schmidt (1981) and O'Reilly & New (1982) in Fig. 37. To be honest, although all the data points fall in the ranges proposed by these researchers, it would have been difficult to predict what has been observed based on the simple soil classifications given. This is quite understandable because,

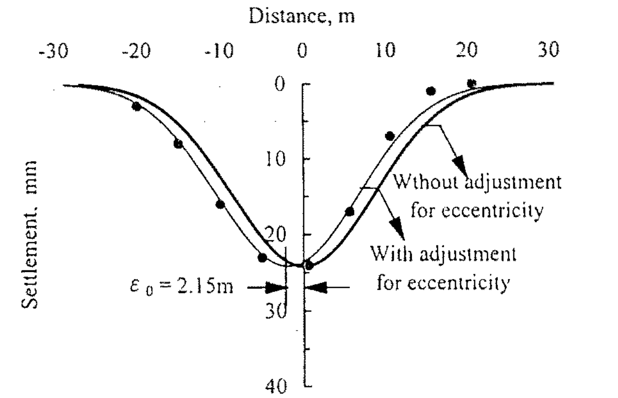

Fig. 35 Adjustment for eccentricity in settlement troughs

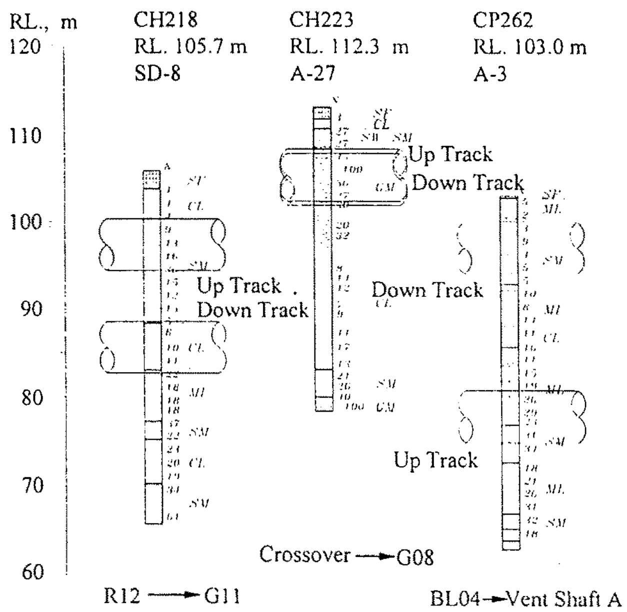

Fig. 36 Soil profiles for CH218, CH223 and CP262

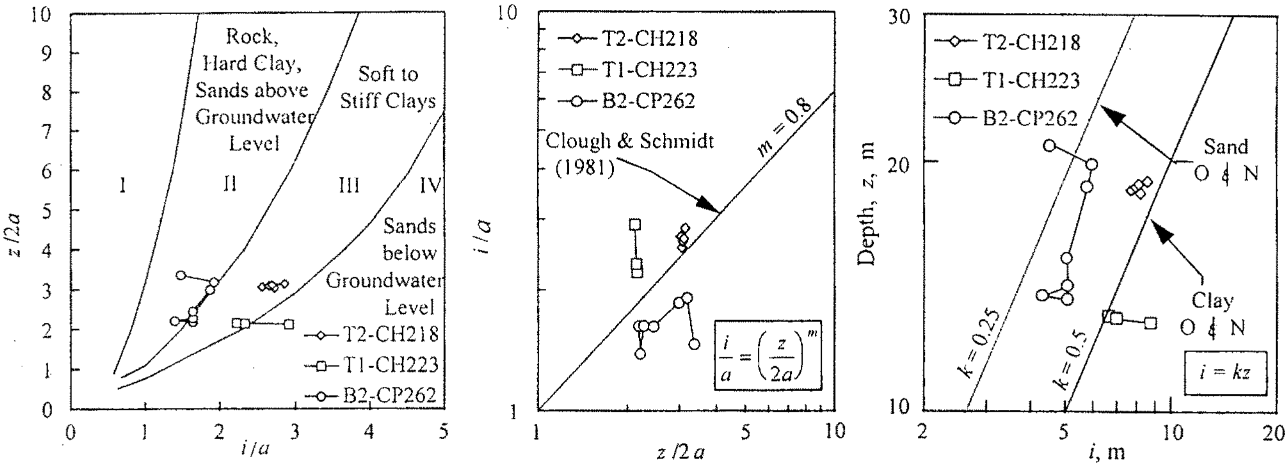

Fig. 37 Estimations of width parameters using Peck, Clough & Schmidt and O'Reilly & New's relationships

apart from the obviously different geological origins, very few of the case histories included in these previous studies involved the use of earthpressure balancing shields. Furthermore, considerable amount of consolidation settlement could have been included in the previous studies while only immediate settlements occurred in 10 days are included in this study.

Ground loss in TRTS was studied by quite a few researchers. Table 1 is a summary of the results. It should be noted that in many of these studies the width parameters were estimated by using the relationship proposed by Clough & Schmidt and the results, strictly speaking, may not necessarily be accurate.

Table 1 Ground Loss and Indices of Consolidation Settlement

Contract	v (%)	α (mm)	Reference
CC275	0.5-1.9	1-4	Lin, Chang and Chu (1997)
CC276	0.7-2.3	6-18	Lin, Chang and Chu (1997)
CC277	sand: 0.3-1	3-16	Lin, Chang and Chu (1997)
	clay: 1-1.6	3-10	Lin, Chang and Chu (1997)
	1.4-2.6	9-15	Hu and Hsieh (1997)
CH218	1.3-2.0	4-6	Yang, Wang and Fan (1995)
	1.7-1.9		Hwang, et al. (1997)
	1.1-1.6		Hwang, Sun & Ju (1996)
CH223	0.6-1.1	1-3	Yang, Wang and Fan (1995)
	0.9-1.0		Hwang, et al. (1997)
CN256	0.2-1.5	5-14	Hwang, Sun & Ju (1996)
CN258	0.5-2.0	4-14	Wu, Zhuang and Wang (1997)
CP262	0.5-1.6		Hwang, et al. (1997)
	0.9-2.0		Hwang, Sun & Ju (1996)

5.4 Ground Heave

Ground heaves were observed at quite a few locations during tunnelling. In the past, heaves were generally attributed to the excess pressure on the face (Yi, Rowe and Lee, 1993). Recent evidences indicate that grouting for filling tail voids might have played a by far more important role than face pressures in inducing ground heaves in tunnelling using earthpressure balancing shields. To start with, face pressures (rather, the pressures on bulkheads) are usually kept at, or attempted to be kept at, levels corresponding to at-rest lateral earthpressures and a review of literatures confirms this practice. In such cases, it is highly unlikely for face pressures of such magnitudes to induce ground heaves of meaningful amounts. On the other hand, line pressures for back grouting usually vary from 2 bars to 4 bars regardless of the overburden pressures. It is quite conceivable that, as illustrated in Fig. 38, for shallow tunnels, ground heaves could well be a result of grouting.

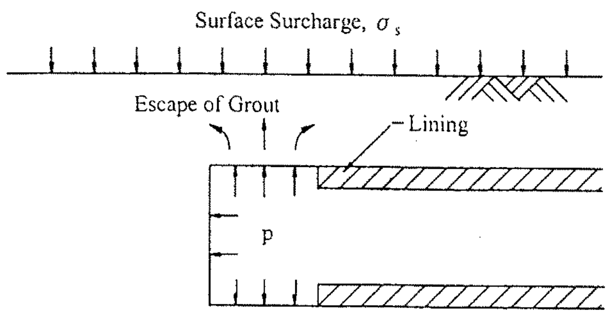

Fig. 38 Effects of back grouting on ground heave

Figure 39 shows the soil profile at Section T3 of Contract CN256 of the Nankang Line. The twin tunnels, with horizontal alignments, are embedded in the Sungshan Formation at a depth of 12.5m and are surrounded by highly silty clays and clayey silts with a representative water content of 32%, a unit weight of 19 kN/m^3, and an OCR value of 1.3. The two tunnels were driven by using a spoke type earthpressure balancing shield with a diameter of 6,050mm and a length of 7,180mm. The outer diameter of the segmental linings is 5,900mm.

Three settlement markers, i.e., SM51, SM52 and SM53, were available at a section corresponding to Ring No. 98 and the readings are shown in Fig. 40. Also shown in the figure is the progress of tunnelling so the ground movements can be correlated with the tunnelling activities. Ground was quite stable during the passing of the head of the shield in the Down-Track tunnel. Field records indicate that in the period in which the face passed through the space occupied by Ring Nos. 94 to 99, the total thrust varied from 810 to 900 tonnes and the average quantity of slurry injected into the earth chamber was 1,718 liters per ring and the average quantity of sludge removed was 28 m^3 per ring.

Earthpressures on the bulkhead were monitored by 4 pressure cells and the readings varied from 170 kPa to 190 kPa during shoving while the overburden pressure at the springline was about 238 kPa for a unit weight of 19 kN/m^3 and a depth of 12.5m. Ignoring the excess pore pressures

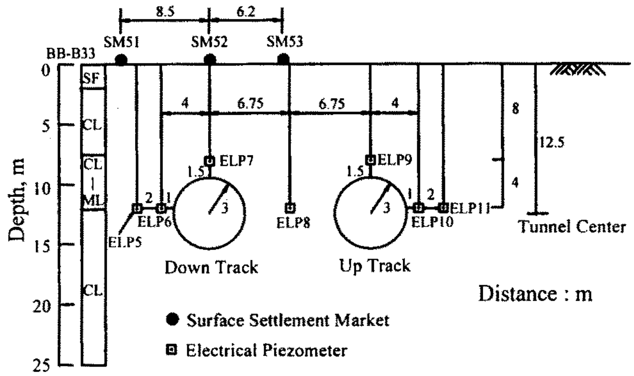

Fig. 39 Profile for Section 256T3 and instrument layout

induced, the coefficient of lateral earthpressure would be between 0.47 and 0.63 for a hydrostatic pressure of 110 kPa at the springline corresponding to a groundwater table of 1.5m below the surface.

When the tail of the shield passed Ring 98, heaves of 3mm to 5mm were observed, presumably, as a result of back grouting. Grouting was automatic and synchronized with shield advancement with line pressures varying from 250 kPa to 400 kPa. A pressure of 400 kPa, if indeed developed in the grout, was more than sufficient to cause hydraulic fracturing at the crown for an overburden pressure of 180 kPa and a laboratory undrained shear strength, s_u, of 40 kPa. The average take of the grout for Rings 92 to 99

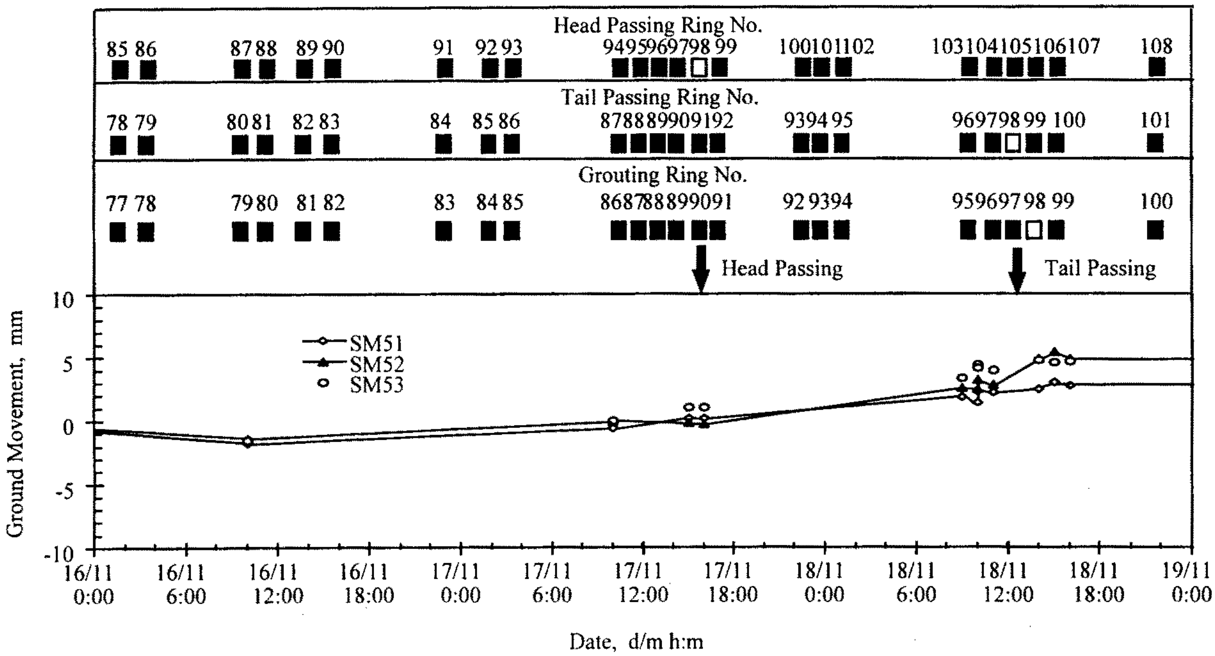

Fig. 40 Ground heave at Section 256T3

was 1,810 litres while the theoretical volume of tail void was 1,598 liters per ring of 1m in length (assuming a 10mm over-cut). It is thus conceivable that some of the grout did penetrate into the ground and cause the ground to heave up. In fact, grout escaped through fissures at Section T4, refer to Fig. 41, at a distance of about 600m to the east of Section T3 during grouting carried out in the Up-Track tunnel.

The overload factor at the tunnel crown is of an order of 4.5 for an overburden pressure of 180 kPa and a s_u value of 40 kPa, therefore the ground was expected to close in at the tail as the shield advanced leading to ground settlement. This did not happen and, on the contrary, heaves were observed. It is thus postulated that the grout pressurized the tail void all the time and prevented the closure of tail void. Back grouting synchronizing with shield advancement and carried out at deliberately high pressures essentially functions as compensation grouting. The long-term settlements can be read from Fig. 42. Notwithstanding the high grouting pressures, the subsequent consolidation settlements were pretty small. Indices of consolidation settlement, refer to Fig. 34, were only 3mm to 5mm which, as indicated in Table 1, are on the lower side of the values given therein.

6. CONCLUSIONS

The above discussions lead to the following conclusions:

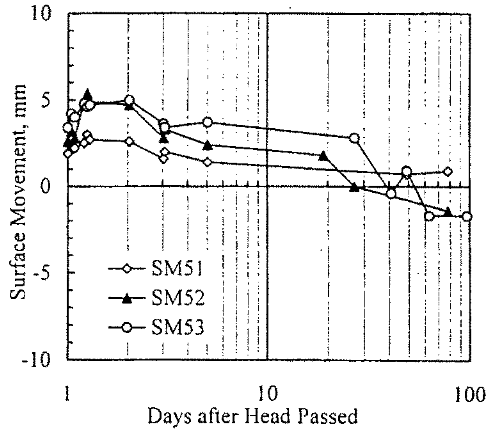

Fig. 42 Ground movements at Section 256T3

(i) Groundwater plays a dominating role in underground constructions and groundwater conditions have to be fully explored before design starts. Long-term monitoring of groundwater movements is necessary in areas experiencing groundwater drawdown.

(ii) Geological features which are unique to an area have to be clearly described in tender documents as international designers and contractors may not have sufficient local

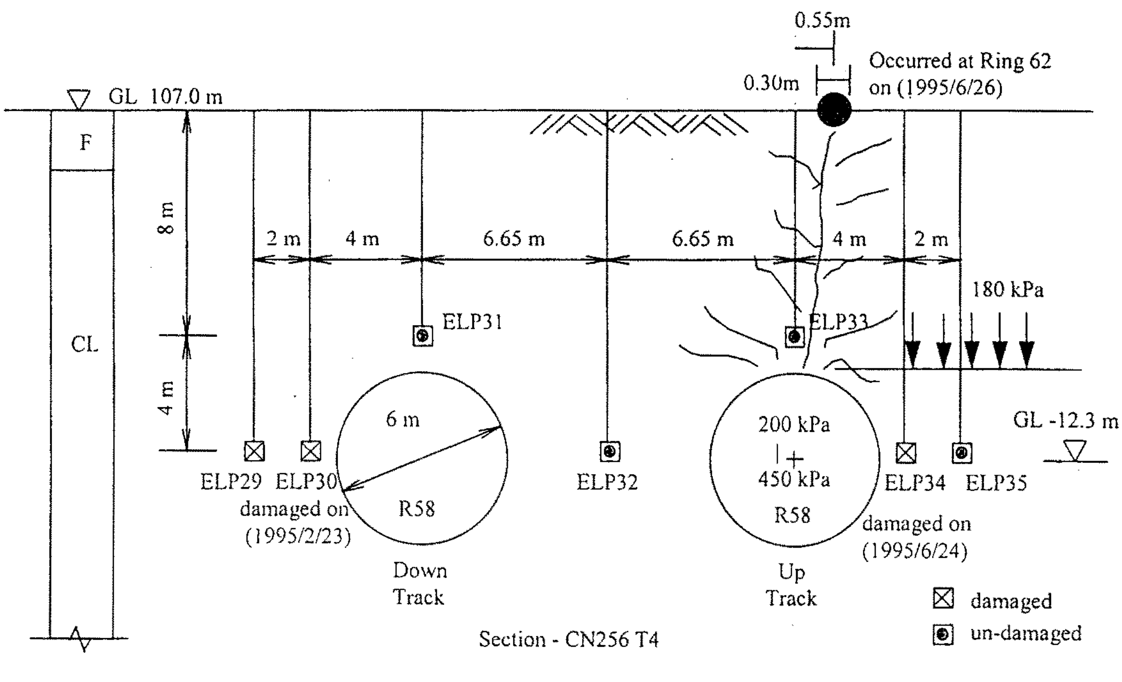

Fig. 41 Escape of grout at Section 256T4

knowledge and may misjudge ground conditions.

(iii) Earthpressures on retaining walls of braced excavations are dependent on wall movements and are difficult to be ascertained.

(iv) For deep excavations, blow-in may become a major problem if there exists a water bearing aquifer under the bottom of excavation. Three measures are presented herein, i.e., lengthening soil plug, lowering water head and flooding the cofferdam.

(v) Both slurry shields and earthpressure balancing shields are suitable for tunnelling in soft ground. Major problems in tunnelling are associated with making openings on diaphragm walls to prepare for launching and/or arrival of shields. Cautions must be exercised if such openings are close to water-bearing strata.

(vi) Peck's approach of approximating settlement troughs by error functions is suitable for analyzing settlement troughs over tunnels. However, local experience is needed for correlating ground loss and width parameter with ground conditions.

(vii) Ground heave over tunnels may be a result of grouting for filling the tail void. It is most likely to occur to shallow tunnel drives overlain by soft clay.

In summary, quality of soil investigation, instrumentation and ground treatment is of primary importance and experienced geotechnical engineers must be engaged in supervision of the field works and interpretation of the results.

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