GEOTECHNICAL PROBLEMS RELATED TO DESIGN AND CONSTRUCTION OF THE TAIPEI MRT

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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 in one of the lines, the majority of the routes in the Initial Network are located in soft ground. This paper describes some of the most significant geotechnical concerns associated with design and construction of this Initial Network. Discussions are made on the soil characterization, earthpressures on retaining walls and wall movements, ground settlements and problems encountered during tunneling.

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, however, 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 aspects in construction of rapid transit systems by using TRTS as an example. It covers the three major elements in underground constructions, i.e., site characterization, cut-and-cover construction, and tunnelling.

2 TAIPEI RAPID TRANSIT SYSTEMS

A system map of the Initial Network of the TRTS is depicted in Fig. 1. The Network comprises six lines, namely the Mucha, Tamshui, Hsintien, Nankang, Panchiao, and the Chungho Lines. At the present (February, 1999), the Mucha, Tamshuei and Chungho Lines have been completed and are open to revenue services. All these three lines are running in the north-south direction. The completion of the middle section of the east-west route at the end of 1999 will bridge these lines and form the first net. The entire Initial Network, except the extension to Neihu, is scheduled to be completed in the year of 2000.

Because this is the first rapid transit system constructed in Taiwan, the Department of Rapid Transit Systems (DORTS) of the Taipei Municipal Government realized the difficult situations to be encountered and 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 GESC to process the tremendous amount of field data in a systematic manner. The database has become a major resource of numerous research studies and has contributed tremendously to the advancement of technology.

3 GROUND CHARACTERIZATION

A geological zoning map is given in Fig. 2 (Lee, 1996) and an east-west and a north-south soil profiles across the Basin are presented in Fig. 3. As can be noted from Fig. 3 that at the surface is a thick layer of Sungshan Formation underlain by the Chingmei Gravels. In the central city area, where the Taipei Main Station is located, the six-sublayer sequence is evident. Toward the east, silty clay dominates and toward the west, the stratigraphy becomes rather more complex with silty sand and silty clay seams interbedded in these sublayers. The Chingmei Gravels 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.

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 in the Sungshan Formation is clearly identifiable. A phenomenon worthy of mentioning is that negative pore pressures were measured throughout the entire depths of Sublayers III and V 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 substantiate this hypotheses.

As can be noted from Fig. 4, the various soil strata can better be identified in the porewater pressure profile than tip resistance or local friction. 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 drastically improved as compared to those obtained at CPT-39.

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 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%. In addition, 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 the TRTS construction.

Drift woods were recovered in numerous excavations. Chunks of 1m or so in diameter were frequently encountered and they could 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 woods as long as they are not too large in size. However, as to be illustrated later, there was one occasion in which the presence of drift wood caused the ground to collapse as soft clay kept on moving into the earth chamber while the advancement of the shield was totally blocked.

4 CUT-AND-COVER CONSTRUCTIONS

For 2-level underground stations, excavations usually were carried out to depths of 17m or so and for 3-level stations excavations were carried out to depths of roughly 24m. The deepest excavation in the TRTS constructions was 36.6m and was carried out for the construction of Ventilation Shaft A in Contract CP262 of the Panchiao Line. In fact, all the three standing-alone ventilation shafts had excavations exceeding 30m in depth while there was only one station, i.e., Ching-An Station (O18) of the Chungho Line, at which the excavation exceeded 30m.

4.1 Methods of Excavation

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 rapid systems.

Because of the absence of a competent stratum within a reasonable depth for bonding ground anchors, all the excavations were strutted. The ventilation shaft in the Chungho Line was the only circular excavation and the rest of excavations were all rectangular. As ground conditions are poor and excavations were deep, diaphragm walls were exclusively used for station excavations. For shallower excavations, for example, at entrances, contiguous bored piles and sheet piles were sometimes used. 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 has become more popular nowadays. At a few stations, composite-wall system, in which the permanent wall is structurally connected to the diaphragm wall by dowels, is adopted. Because the diaphragm wall and the permanent wall form a single structural element, this reduces the thickness of the permanent wall, and thus saves some space. However, 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.

Groundwater was a major concern and pumping was carried out at three sites for lowering the groundwater table for maintaining sufficient factor of safety against blow-in (Hwang, et. el., 1996). Pumping rates ranged from 2,000 to 4,000 cubic meters per hour and pumping lasted for six months or longer. Detailed discussions on the measures taken against blow-in are available in Moh and Hwang (1999).

4.2 Earthpressures

It is a well known fact that earthpressure on a wall is a function of lateral deformation of the wall. Figure 6 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. 6(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 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., $\delta=0$. The results are shown in Fig. 7 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. 7, it is concluded that a displacement of 20mm is sufficient for the lateral earthpressure ratios to reach their lower-bound values. Ideally, earthpressure 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 thus suggested to start with a pressure ratio of 0.6 to allow for creep. In such a case, according to Fig. 7, an extra amount of wall deflection of 10mm is required for the pressures on cells to reduce to their lower-bound values.

In the TRTS constructions, a considerable quantity of earthpressure cells have been installed on diaphragm walls. Some of the results are shown in Fig. 8. Because all the deep cuts were retained by thick diaphragm walls and all the struts were preloaded, wall deflections were generally small and in very few cases the lateral pressure ratios reached their lower-bound values.

As indicated in Fig. 7 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. 9. As can be noted that the four points (solid triangles) for CPH Building 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. 9 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. 10, 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 some of the walls did settle. As a wall settles, the incident angle of earthpressure on the active side of the wall may drop. If the settlement of the wall is sufficiently large, the frictional forces on the active side may even reverse their direction. This could explain the fact that some of data points in Fig. 9 correspond to very low, even negative, friction angles.

Settlements of diaphragm wall were seldom monitored. Figure 11 shows the soil profile and configuration of excavation at Kungyang Station (BL16) where diaphragm wall settlements were closely monitored. The data obtained are shown in Fig. 12 and, as can be noted, a maximum settlement of 8 mm was observed. This magnitude, together with lateral deflections of, upto, 25mm, is believed to be sufficient to affect the distribution of earthpressures on the wall. In this particular case, excavation was carried out to a depth of 15m while diaphragm wall had a length of 32m, giving a penetration of 17m below the bottom of excavation at the final stage. Besides, the 3m grouted slab provided positive friction resisting settlement of the wall. It is thus expected settlements would have been much larger if the wall were shallower or if this grouted slab were absent.

The phenomenon that some of the data points, refer to Fig. 9, fall below the line corresponding to $\varphi' = \phi'$ is rather difficult to explain. It could have something to do with the fact that all the soils in the Sungshan Formation 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, it is possible for the full soil friction to develop on a wall. This is of course valid only 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 designs. As indicated in Fig. 9 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 based on engineering judgment.

4.3 Lateral Wall Deflections

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 geotechnical instruments. 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 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 13 shows the readings obtained for two neighboring inclinometers, installed in diaphragm walls, one to the toe level and the other to the bearing stratum. 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. When excavation reached a depth of 11.1m, Inclinometer SID4 which was anchored in the bearing stratum showed a maximum deflection of 45mm while Inclinometer SID6 which was installed only to the diaphragm wall toe 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 recorded by Inclinometer SID4. It is thus suspected that the toe of SID6 had moved by a similar amount, or even larger because the wall was thinner and shallower. After correcting the readings of Inclinometer SID6 for this anticipated toe movement, the two sets of readings were very close. This illustrates the fact that inclinometer readings shall be interpreted with great care. As a general rule, it is suggested that inclinometers shall always be anchored in a bearing stratum to eliminate doubts.

Wall deflections obtained in the TRTS constructions in the central city area (T2 Zone) are compared to those obtained previously (Woo and Moh, 1990) in Fig. 14 (Moh and Hwang, 1999). It is readily apparent that TRTS constructions out-perform the constructions carried out in the past as evidenced by the far less wall deflections observed. The factors affecting wall deflections can be summarized as follows:

a) depth of excavation
b) ground conditions, i.e., soil stiffness and ground water table
c) method of construction, e.g., top-down, bottom-up, or semi top-down
d) rigidity of retaining systems, including wall elements and strutting members
e) ground treatment, e.g., grouting
f) preloading of struts
g) workmanship, e.g., promptness of strutting

It will be desirable to quantify their effects individually. Unfortunately, since all these factors are affecting the magnitudes of wall deflections at the same time, they are coupled in mathematical formulations.

Of all the factors listed above, depth of excavation is the easiest to be quantified. Following the relationship proposed by Woo and Moh (1990), Moh and Hwang (1999) suggested that:

$$\delta_{\max} = \alpha H^2 \bullet 10^{-6} \quad \text{.......... Eq. (1)}$$

where

$\delta_{\max}$ = maximum wall deflection (in meter),
H = depth of excavation (in meter),
α = an empirical factor with an implicit dimension of m^{-1}.

The factor of 10^{-6} was introduced so α values will be whole numbers. Lines corresponding to α = 63, 125, 250, 500, and 1000 divide the plot into various zones, i.e., Zones I, II, III, etc, with wall deflections in each zone, on an average, equal to twice of those in the preceding zone. The majority of data points obtained previously in the T2 Zone in the Taipei Basin fall in Zone IV while data points for TRTS excavations mostly fall in Zone II and Zone I. Wall deflections in TRTS excavations are, on an average, only a quarter of what were obtained previously. Since depth of excavation and ground conditions have been explicitly considered in the plot, the difference in the two sets of data must be a result of the combination of the rest of factors listed above.

It is difficult to figure out the details associated with the previous excavations. In any case, it is believed that the superior performance of TRTS constructions are primarily a result of higher stiffness of the retaining systems and better workmanship. In the TRTS constructions, diaphragm walls are usually thicker than what were previously adopted by 200mm to 300mm. This, together with preloading of struts, gives much greater stiffness of the retaining systems. Furthermore, previous data were mostly associated with basement excavations which were frequently carried out using the top-down method while, as mentioned above, all the TRTS excavations, except one, were carried out using the bottom-up or the semi top-down method. In addition, stringent quality control and quality assurance procedures are followed, therefore, works were carried out more orderly and more carefully.

TRTS data, supplemented by data obtained in some recent excavations, were grouped into 10 different sets according to the geological zone, method of construction and thickness of wall, and the α values for different sets of conditions are summarized in Table 1. Based on these results, the effects of various factors were evaluated and multipliers on the α values were proposed as given in Table 2. These multipliers can be used to derive α values for different sets of conditions for which data are unavailable based on the α values for cases with data available. It should be noted that in all the cases in which the bottom-up method was used, struts were preloaded while in all the cases in which the top-down method was used, floors served as struts and could not be preloaded. Therefore the comparison of these two sets of data includes both the effects of construction methods and the effects of preloading of struts. It should also be noted that the data in each set are insufficient to serve as a reliable basis for statistic analyses and the multipliers proposed shall be deemed as preliminary. Nevertheless, the ranking among these factors appears to be reasonable.

It is reasonable to doubt that Eq. (1) would work in all the cases. A generalized expression of deflection-depth relationship is as follows:

$$\delta_{\max} = \beta_{\theta} H^{\theta} \quad \text{.......... Eq. (2)}$$

Empirical factor β_{θ} has a dimension of $m^{(1-\theta)}$. It is a function of all the factors listed above and will depend on the θ value chosen. Worst of all, θ is also a function of all the factors listed above, including the depth of excavation, H.

It is hypothesized that θ will fall in a narrow range and analyses will be insensitive to the θ chosen. Equation (3) can be expressed in a logarithm form as follows:

$$\log(\delta_{\max}) = \theta \log(H) + \log(\beta) \quad \text{Eq. (3)}$$

then θ is simply the slope of deflection-depth relationship in a logarithm plot. Figure 15 shows how wall deflections vary with depth of excavation for individual inclinometers in recent excavations, including some TRTS constructions. The data for excavations shallower than 5m are rather widely scattering without a clear trend. However, in the range of 5m to 30m, with only one exception (T2-B-070), the slopes of curves (not the data points) indeed fall in a narrow range of $1 < \theta < 2$.

The case corresponding to the upper-bound value of θ, i.e., $\theta = 2$, has been discussed above. It is desirable to study how sensitive the results given in Tables 1 and 2 are to the θ value.. This can be achieved by studying the case for the lower bound value of θ, i.e., $\theta = 1$, and comparing the results obtained.

For $\theta = 1$, Eq. (2) reduces to:

$$\delta_{\max} = \beta_{\theta=1} H$$

or, simply

$$\delta_{\max} = \beta H \quad \text{Eq. (4)}$$

β is conventionally given in percents. Following the same procedures given above, β values were obtained for different conditions and are given in Table 1. Similarly, multipliers are proposed for β as given in Table 2.

As illustrated in Fig. 15, θ values fall in a narrow range for excavations in the Taipei Basin. It is concluded that depth of excavation is the primary factor affecting the θ values and the rest factors are secondary. A similar plot is given in Fig. 16 for excavations carried out in Singapore marine clay. As can be noted that for excavations ranging from 5m to 15m, the slopes of curves (not data points) fall in the same range of $1 < \theta < 2$ as those obtained in the Taipei Basin. Limited data indicate that for excavations retained by sheet piles, the α values vary from 1500 to 7000 and β vary from 1% to 5%. For excavations retained by diaphragm walls, typically

800mm in thickness, the α values vary from 200 to 400 and β vary from 0.2% to 0.4%. The quantity of data collected so far is insufficient for parametric study for computing the α and β values for different types of sheet piles or diaphragm walls with different thicknesses.

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 17 and 18 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).

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 19 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.

5 TUNNELLING

Tunnels linking subway stations are usually constructed by using: a) the 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 of 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 20 shows a cross section of the tunnels. The tunnels were bored through Sublayer V 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. 21, 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.

The consumption of compressed air was about 110 cubic meters per minute, refer to Fig. 22, 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. 23, 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 of 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.

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, 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. There was a 2,200mm diameter water main running across the two tunnels and supplying water to the entire Taipei City. Should this water main rupture, the consequence would be disastrous. Fortunately, the face had previously been stabilized by grouting for reducing the loss of compressed air, therefore, it was able to stand up. The contractor was able to replace the transformer promptly and the crisis was resolved without even minor damages.

Shown in Fig. 24(b) are the readings of air pressuremeter AP-908, which was buried at a depth of 14m. Figure 24(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 24(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 go through medical treatment. 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.

The second application of NATM Method in soft ground was in the Nankang Line for a 54m section of the Up-Track and a 44m section of the Down-Track. Figure 25 is a plan showing the layout of the tunnels and the locations of settlement markers. A longitudinal section of the Up-Track tunnel is given in Fig. 26. Jet grouting to the west of the underpass was carried out by the contractor of Contract CN256 and the shells of the two shield machines were left in place. Jet grouting underneath the underpass was carried out previously by the contractor of Contract CN256A when the underpass was constructed. Jet grouting to the east of the underpass was carried out by this contractor, i.e., the contractor of Contract CN256B.

Figure 27 shows the cross section of tunnel and the sequence of excavation. The progress of the Up-Track tunnel was quite satisfactory with ground settlements in general less than 50mm. Tunnel convergence and settlement of the crown were within 10mm. As shown in Fig. 28, settlements above the Down-Track tunnel were rather large, up to 240mm. The drastic difference in behavior between the two tunnel drives was due to the fact that ground treatment was difficult for the Down-Track tunnel because of the presence of utilities. The recovery of cores was 50% for the Up-Track tunnel and below 10% for the Down-Track tunnel. In an attempt to reduce ground settlement, sheaths of 3m in length were installed at 250mm pitches and dowels were inserted into the ground to support the invert as illustrated in Fig. 27.

Despite the fact that NATM tunnelling in soft ground is popular in Europe, this is the first time it was carried out for considerable lengths 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 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.

The first major obstacle was a 125mm diameter steel casing of a borehole left in place in a previous site investigation encountered in the Down-Track Tunnel in the Hsintien Line. Steel fragments choked the screw conveyor and had to be removed by sending a worker into the earth chamber. This was much more difficult than what one might expect because of the high groundwater table together with high permeability of sands in Sublayer V. Chemical grouting was attempted in vain in front of the face to stop water from entering the chamber. The operation was abandoned because of the fear that further grouting might glue the shield to the ground to the extent that driving would not be able to resume. Finally, pumping was carried out to lower the groundwater table to a level below the tunnel invert for the worker to be able to enter the chamber and to stay there safely for removing the steel fragments and repairing the damaged conveyer.

Steel fragments frequently appeared in spoil removed from tunnels, however, other than the one mentioned above, no serious problems were reported. This was due to the fact that modern shield tunneling machines have sufficient power and the cutters are strong enough to cut steel members as long as they are not too large in size. There were cases in which small RC piles and thin sheet piles were cut through. On the other hand, there were cases in which ground treated by jet grouting was too hard for shield machines to go through. Figure 29 shows a situation encountered during tunneling in constructing the Tamshuei Line. A sinkhole of roughly 3m in depth and 75 m^3 in volume was found in front of the shield machine as the specialist subcontractor looked for a missing settlement rod installed for monitoring ground settlement. The cavity was covered by the RC pavement which did not show any signs of subsidence. In this case, jet grouting had been used to treat the ground at the back of the diaphragm wall to prepare for launching of the shield machine. Because the treated ground was too hard, driving of the shield was difficult since the very beginning and chemical had to be injected into the earth chamber as lubricant. Even so, worker had to go into the earth chamber to free the cutter from time to time. It was reported that the temperature of the spoil in the earth chamber was as much as 60°C. It is postulated that, as the cutter reached the end of the treated zone, a mixed-face situation was encountered. As most of the face was still in the treated ground, the shield advanced rather slowly. On the other hand, portion of the face was already in the natural ground and soil could easily be excavated and "sucked" into the earth chamber. From what was observed and experience learned elsewhere, it may be concluded that if the treated ground (full face) has an unconfined compressive strength of 4 MPa (40 kg/cm^2) or above, shield driving is likely to encounter difficulties. This, however, certainly will depend on the capacity of the machine used and the uniformity of ground treatment. To avoid similar events from happening, it may be a good idea to have a transition zone with weaker strength at the end of the ground treated by jet grouting.

Large tree trunks, up to 1.5m in diameter and up to 5m in length, were often encountered, usually at depths of 10m to 20m, during deep excavations (Ju, Kung and Duann, 1997). During TRTS constructions, pieces of wood were frequently removed from the spoil during tunneling, however, few problems were reported. A large tree trunk nevertheless did stop the shield machine during excavation for the Up-Track tunnel of the Chungho Line. It was reported that the daily progress rate was reduced from 43 rings (1m per ring) to 4 rings on the day prior to the event. It appears that this tree trunk had been pushed by the shield machine by more than ten meters. As the advancement of the shield machine was obstructed, the earthpressure balancing mechanism was destroyed and soil was "sucked" into the earth chamber in a manner similar to what was observed. A sinkhole of 5m in diameter occurred right above the head of the shield machine. As illustrated in Figure 30, jet grouting was carried out in front of the shield and compressed air was applied for workers to enter the earth chamber to free the cutter. Two pieces of drift wood, 500mm and 400mm in length, were recovered in the earth chamber. To ensure that the remaining tunneling work can be carried out safely, cement/bentonite grout was injected by using the tube-A-manchutte method all the way to the arrival shaft located in Station O17.

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 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. 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.

In one case in which leakage occurred when the tunnel portal was enlarged for installing the flexible joint, ground subsided by several meters and a section of tunnel was damaged. A total of 23 rings were seriously distorted and had to be replaced (Hwang, et. al., 1998). In another case in which ground subsided by several meters when the shield machine was making the breakthrough on the diaphragm wall, 39 rings in one tunnel drive and 34 in the other were damaged and two shield machines were submerged (Ju, Duann and Tsai, 1998).

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 tended to lead to misjudgment because only good specimens were tested. Furthermore, in most of the incidents, strength of the treated ground was irrelevant.

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 they overlap, 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.

6. CONCLUSIONS

The above discussions lead to the following conclusions:

(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 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. Observations indicate that diaphragm walls with soft toes do settle. Settlement of the wall may affect the incident angle and the distribution of earthpressures on the wall.

(iv) Performance of retaining walls can be expressed in terms of α and β values, which are the two empirical parameters to be established based on observations. These two parameters are functions of ground conditions, rigidity of retaining systems, preloading of struts, method of construction, and workmanship.

(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) The New Austrian Tunneling Method was successfully used for tunneling in the Taipei Silts. This can be deemed as a milestone in the history of soft ground tunneling in Taiwan.

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Table 1 Alpha and beta values for deep excavations in the Taipei Basin

	α (Moh and Hwang, 1999)			β, % (This Study)		
Set	T2 Zone	TK2 Zone	K1 Zone	T2 Zone	TK2 Zone	K1 Zone
B-120	59(3)			0.14(3)		
B-100	94(3)		160(7)	0.15(3)		0.30 (7)
B-090						
B-080	106(3)			0.18(3)		
B-070		140(2)			0.30(2)	
S-120	43(1)			0.10(1)		
S-100			189(3)			0.31(3)
S-090						
S-080						
S-070						
T-120		163(2)			0.43(2)	
T-100						
T-090			265(3)			0.52(3)
T-080	352(2)			0.61(2)		
T-070						

Notes:
S: seimi-top down construction method
B: bottom-up construction method
T: top-down construction method
The numbers after "-", e.g., -120, -100, etc are wall thicknesses in centimeters
The numbers in paratheses are number of cases in the category

Table 2 Multipliers for extrapolating α and β values from observed values

Parameter	Representative Condition	Multiplier on α (Moh and Hwang, 1999)	Multiplier on β (This Study)
Ground Conditions	T2	1	1
	K1	2	2
	TK2		1.5
	Singapore marine clay	5	
Retaining Structures	1.2m diaphragm wall	1	1
	1.0m diaphragm wall	1.5	1.1
	0.8m diaphragm wall	2	1.3
	0.7m diaphragm wall		1.5
	sheet pile	4	
Strut	preloaded	1	1
	without preloading	2	1.5
Method of Construction	bottom up	1	1
	top down	2	2
Ground Treatment	treated	1	
	untreated	2	

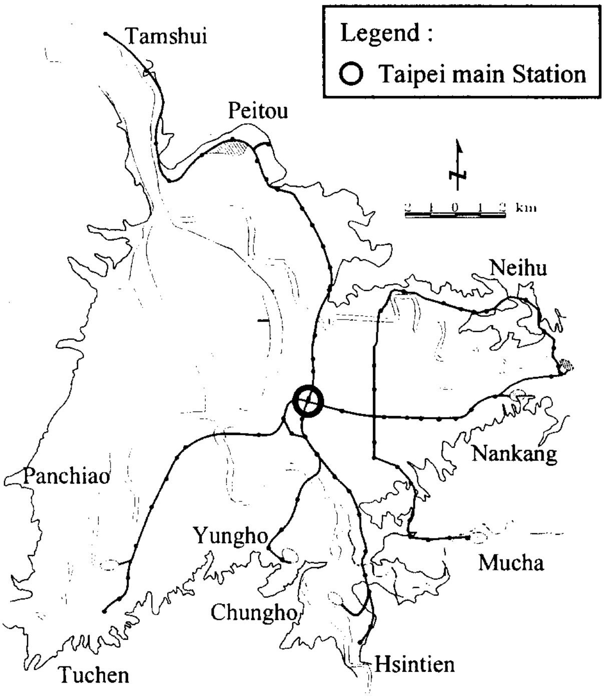


Fig. 1 TRTS Initial Network of Taipei

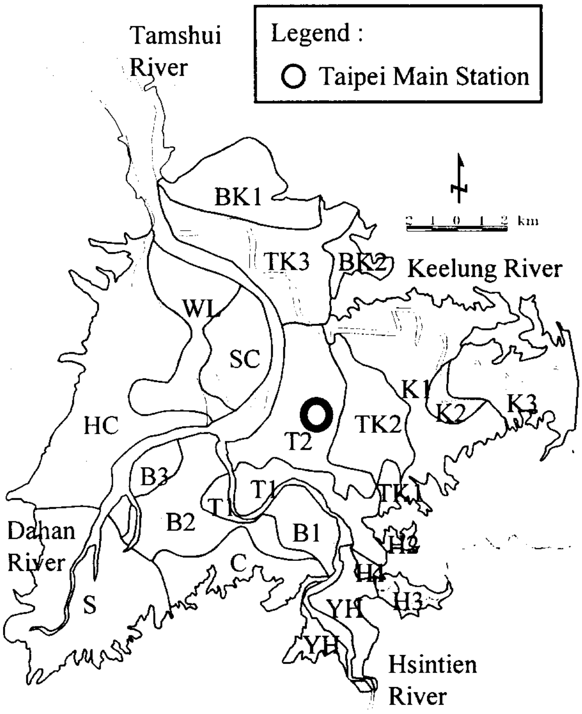


Fig. 2 Geological zoning map of the Taipei Basin

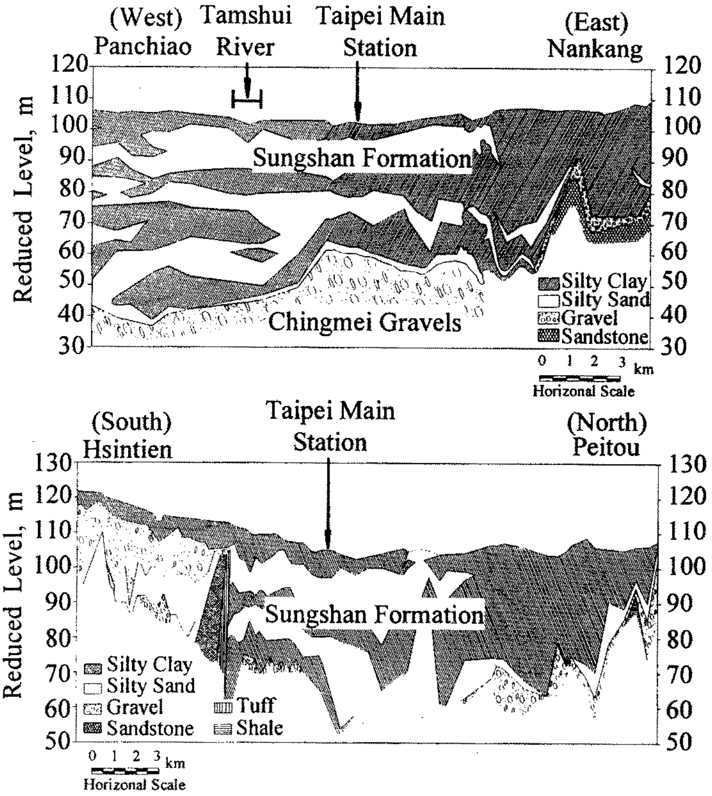


Fig. 3 Geological profiles of the Taipei Basin

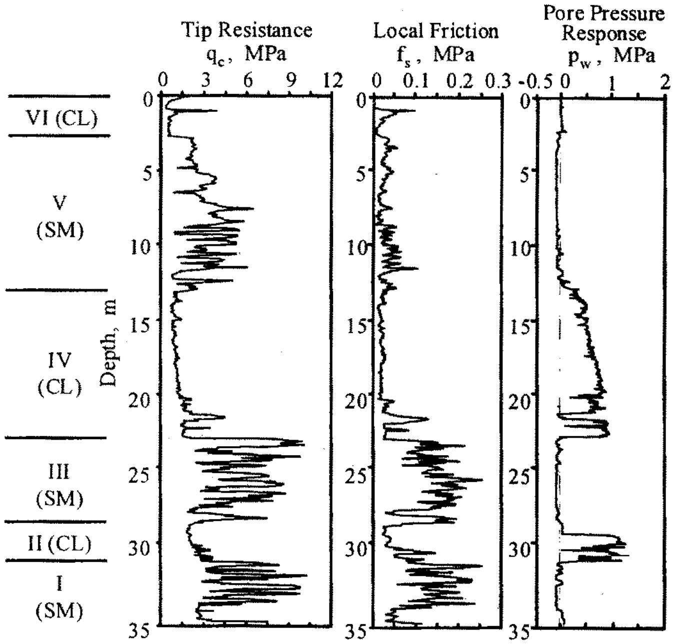


Fig. 4 CPT profile in central Taipei

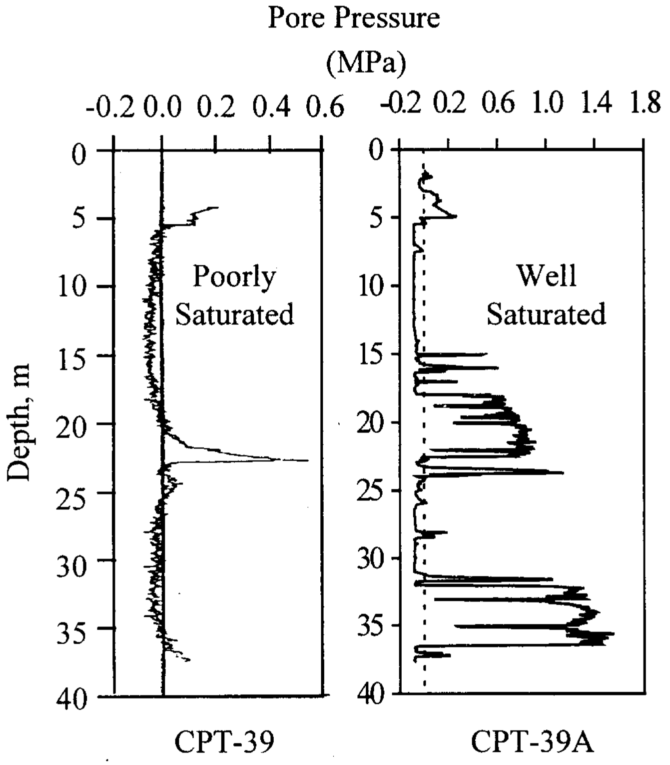
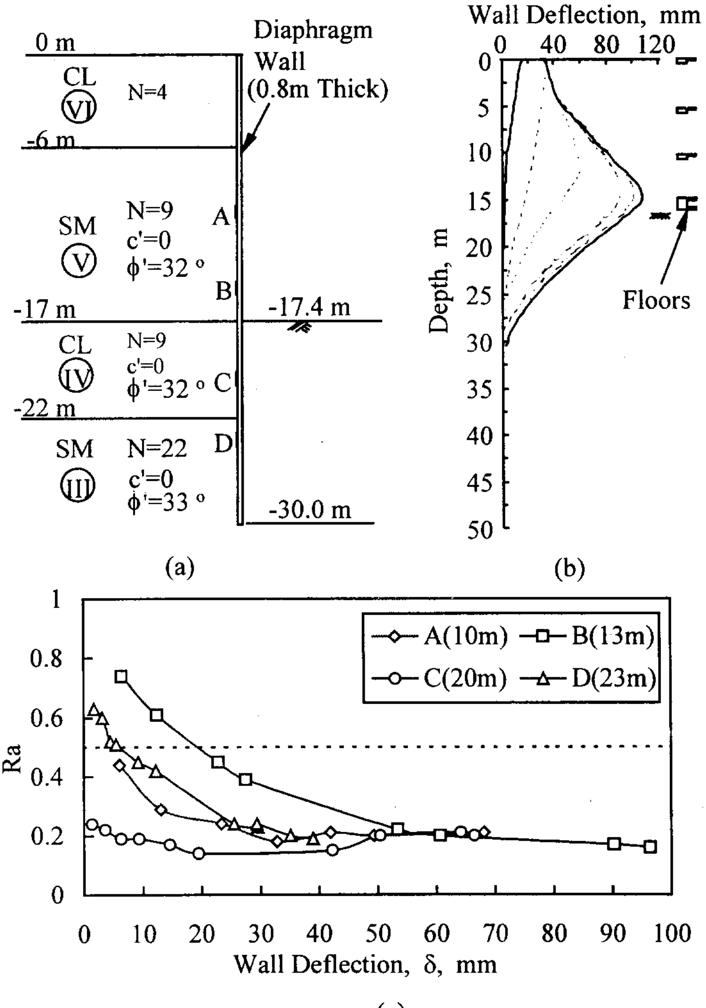


Fig. 5 Results of piezocone tests as affected by saturation of piezometer



(a)

(b)

(c)

Fig. 6 Earth pressure measurements at CPH building

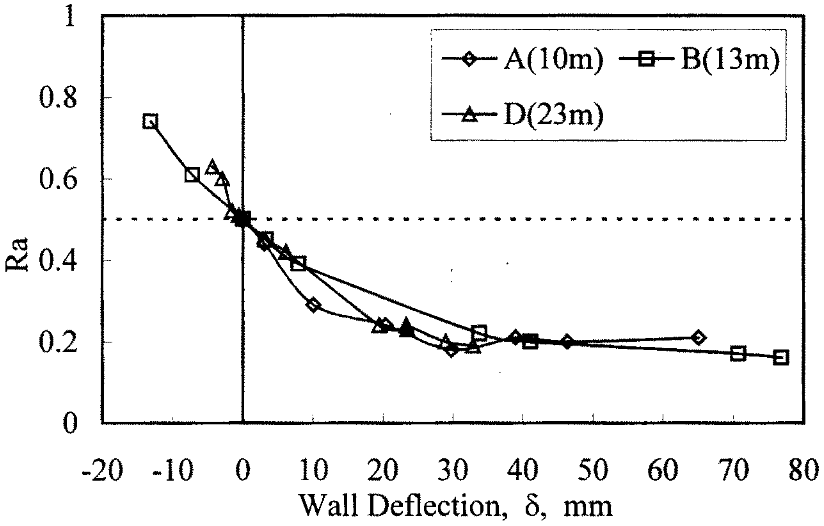


Fig. 7 Lateral earthpressure ratios after adjustment - CPH Building

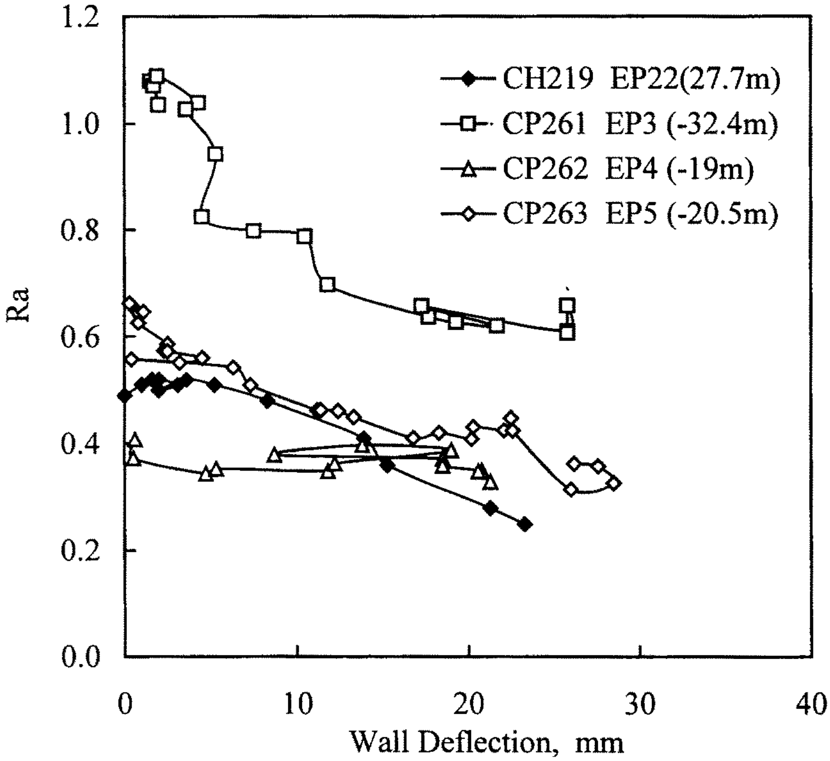


Fig. 8 Earth pressure measurements at TRTS excavations

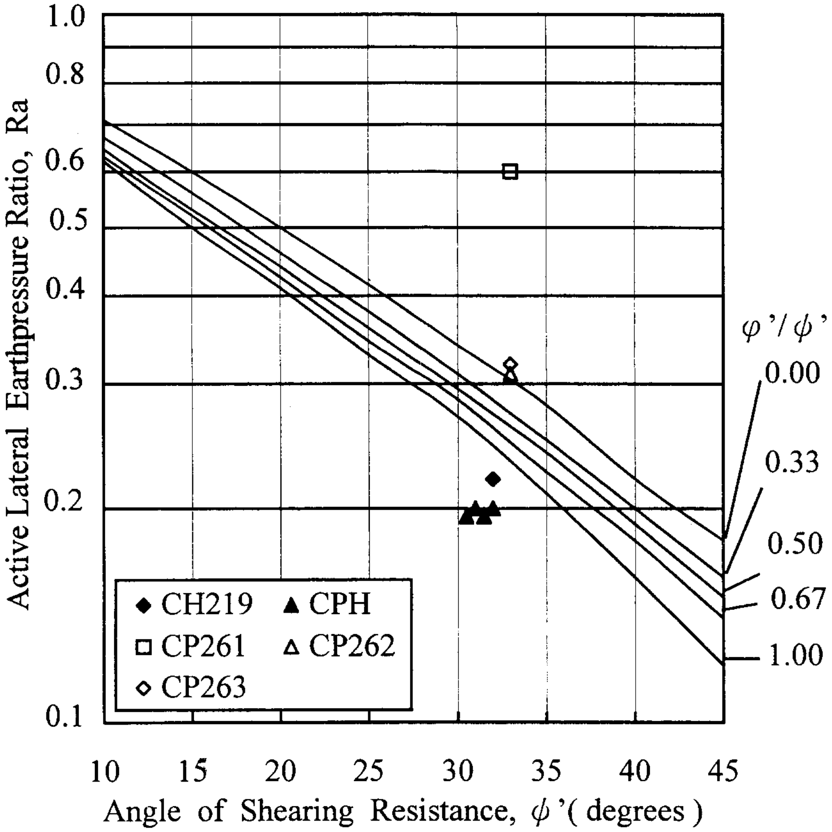


Fig. 9 Lateral earthpressure ratios for Taipei Silts

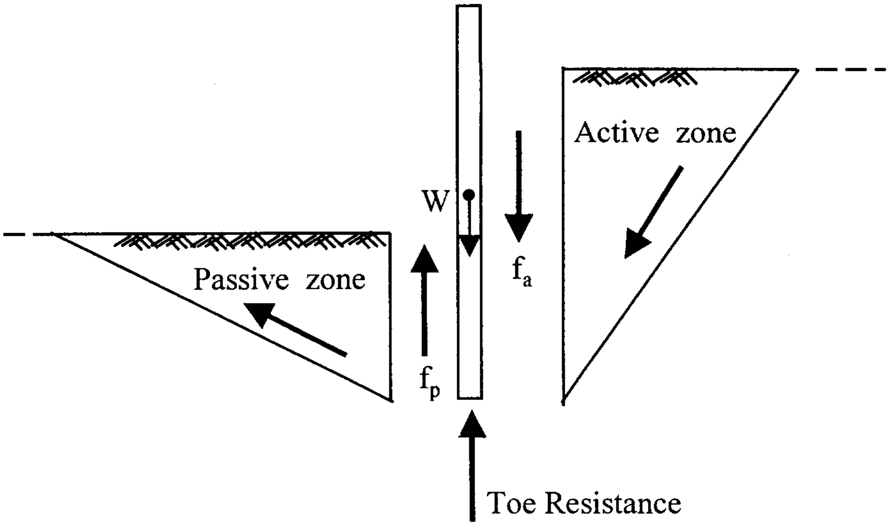


Fig. 10 Force epuilibrium on retaining walls

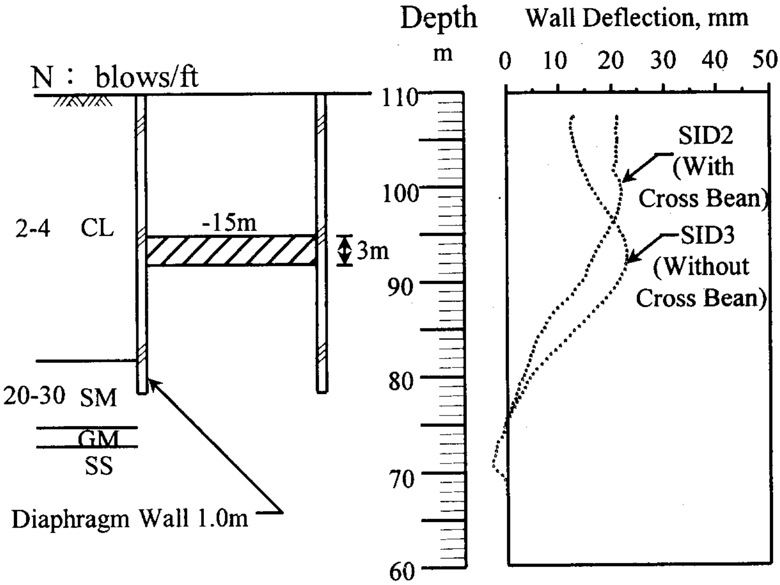


Fig. 11 Profile and inclinometer readings at Station BL16

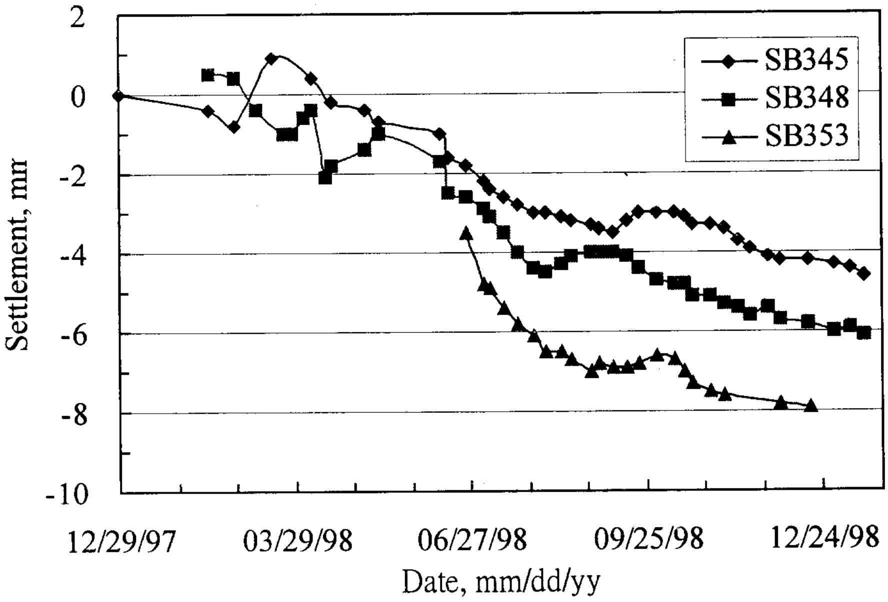


Fig. 12 Settlements of diaphragm wall at Station BL16

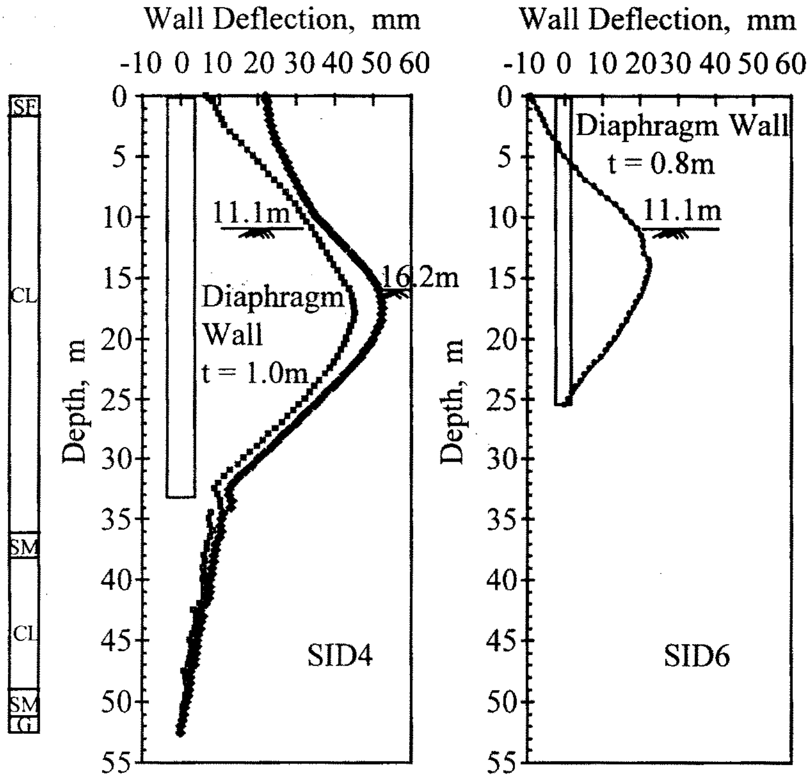


Fig. 13 Effects of toe stability on inclinometer readings

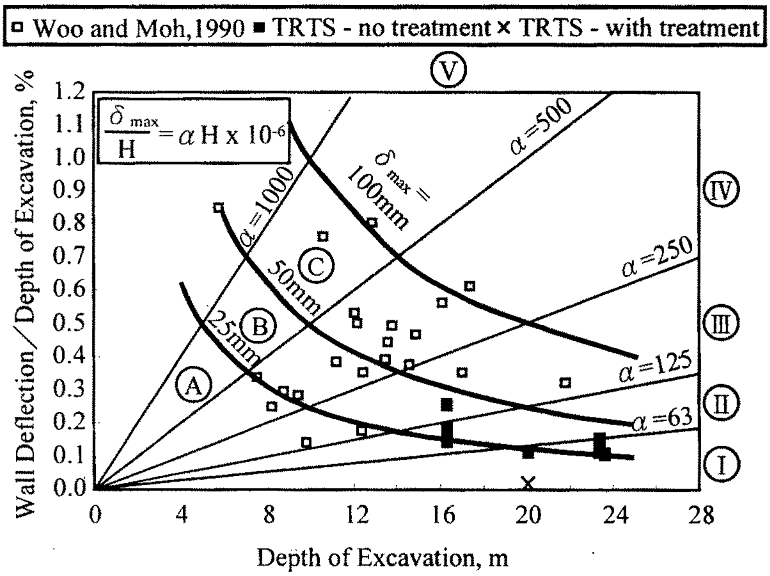


Fig. 14 Wall deflections in the T2 Zone of the Taipei Basin

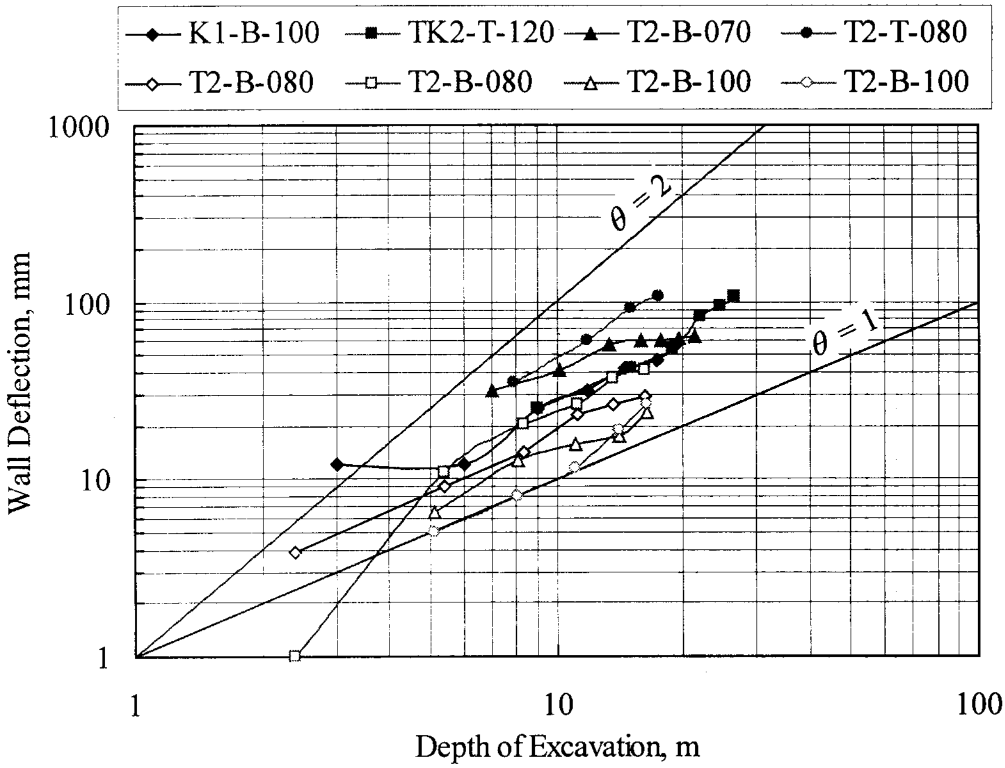


Fig. 15 Wall deflections in Taipei Basin

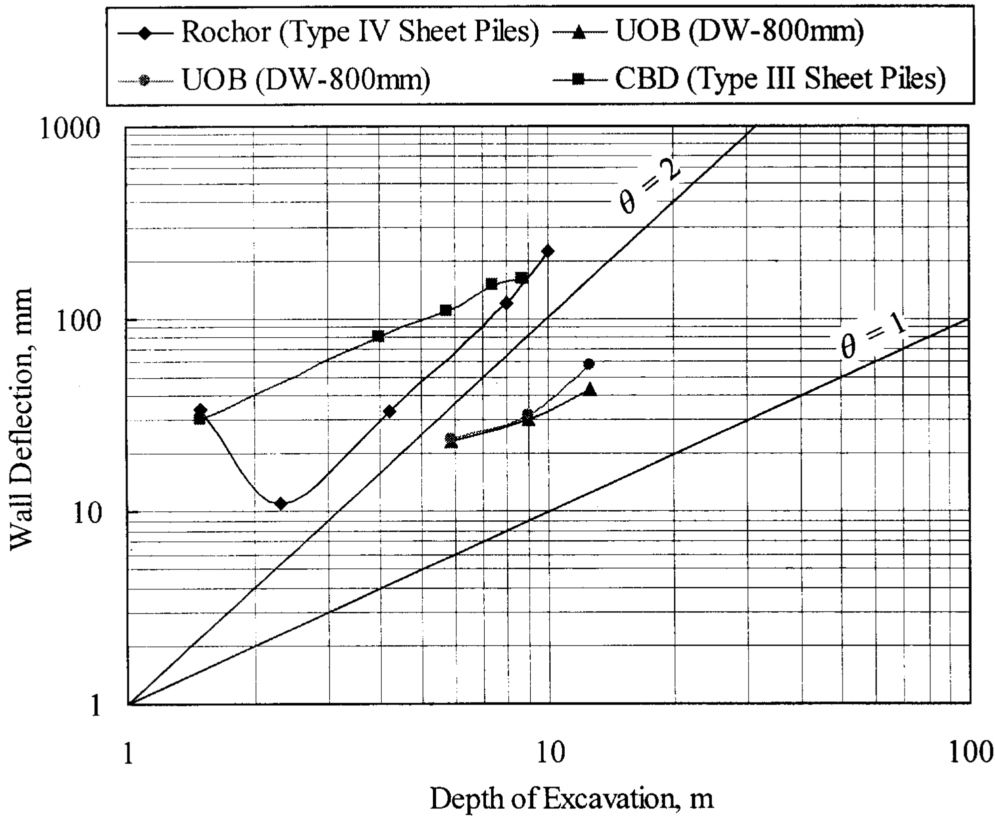


Fig. 16 Wall deflections in Singapore Marine Clay

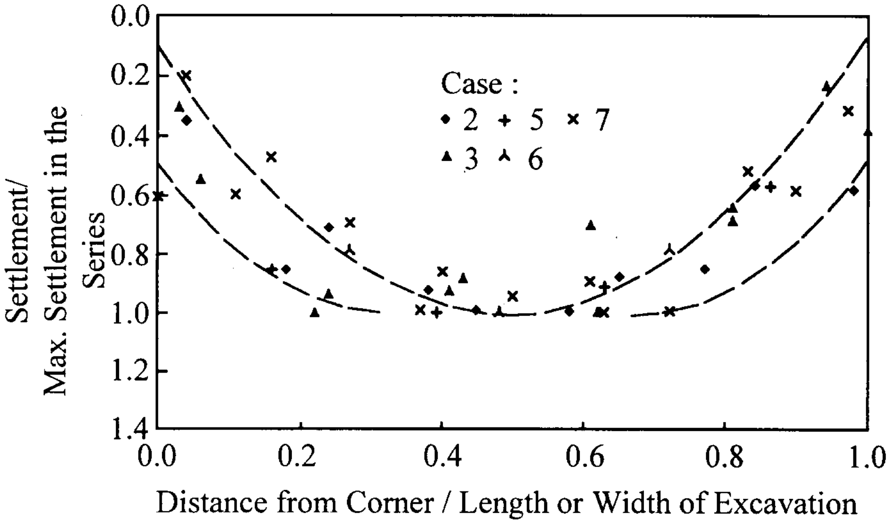


Fig. 17 Corner effects on ground settlement

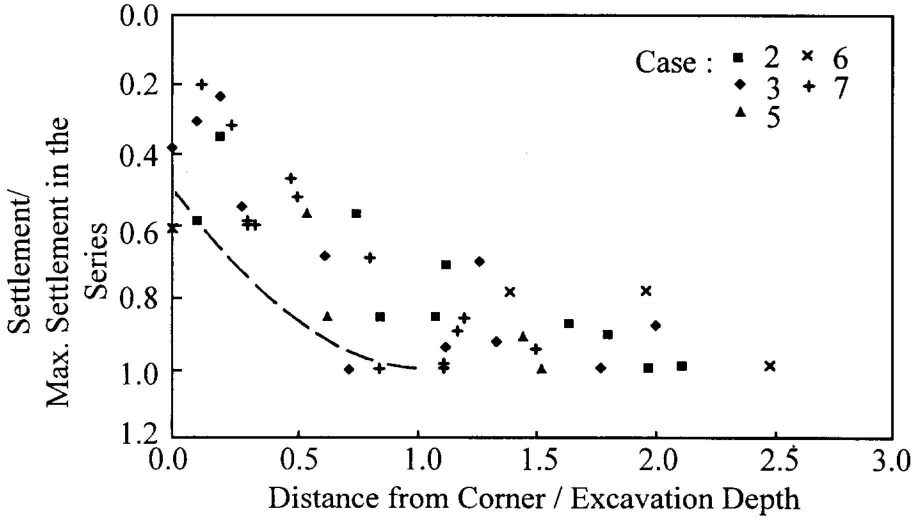


Fig. 18 Corner effects for long excavation

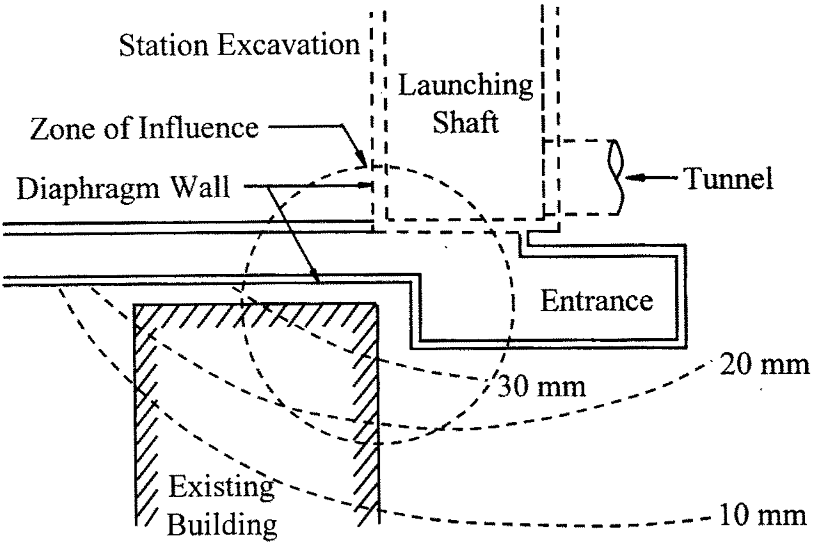


Fig. 19 Settlement due to diaphragm walling

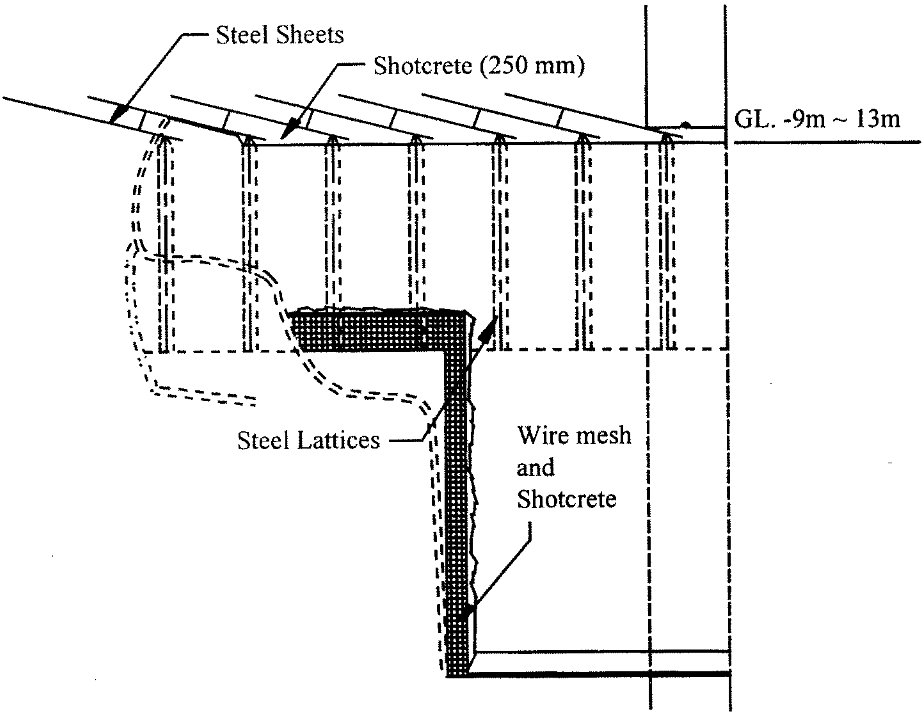


Fig. 20 Profile for NATM tunnels in Contract CH221

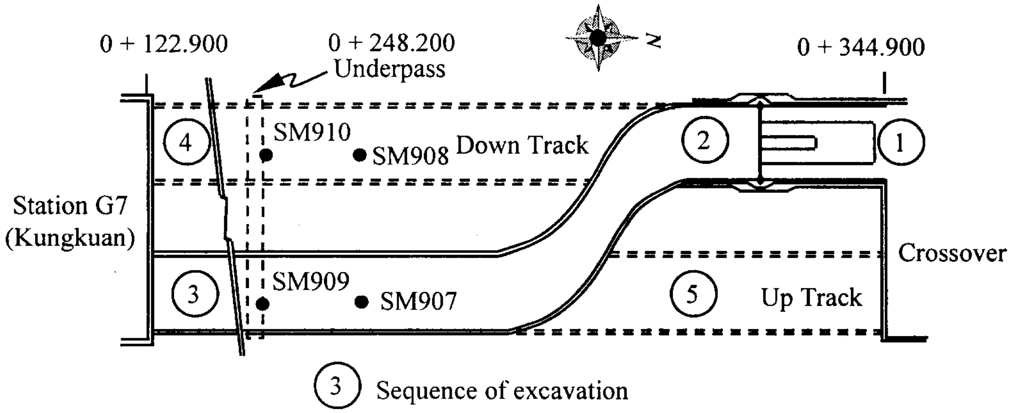


Fig. 21 Plan of NATM tunnels in Contract CH221

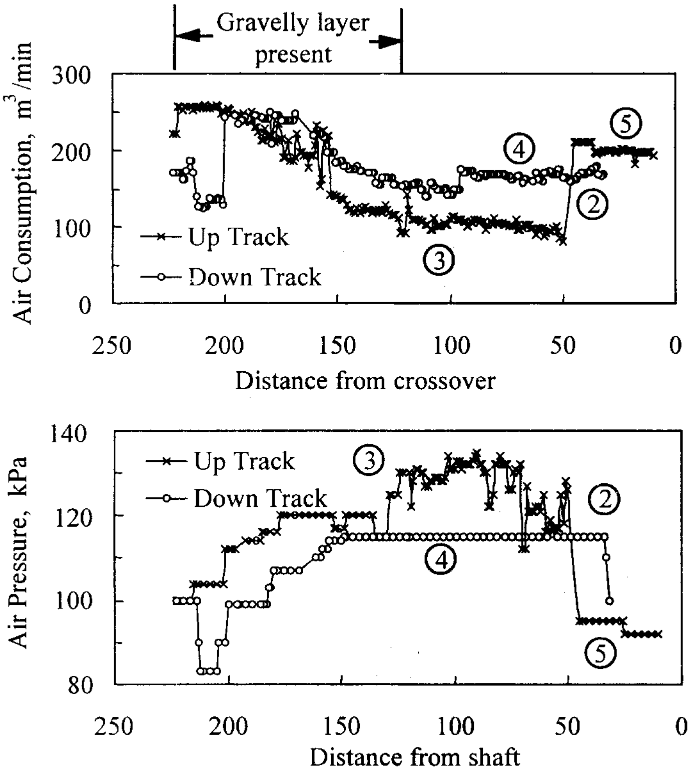


Fig. 22 Air pressure and consumption for CH221 NATM tunnelling

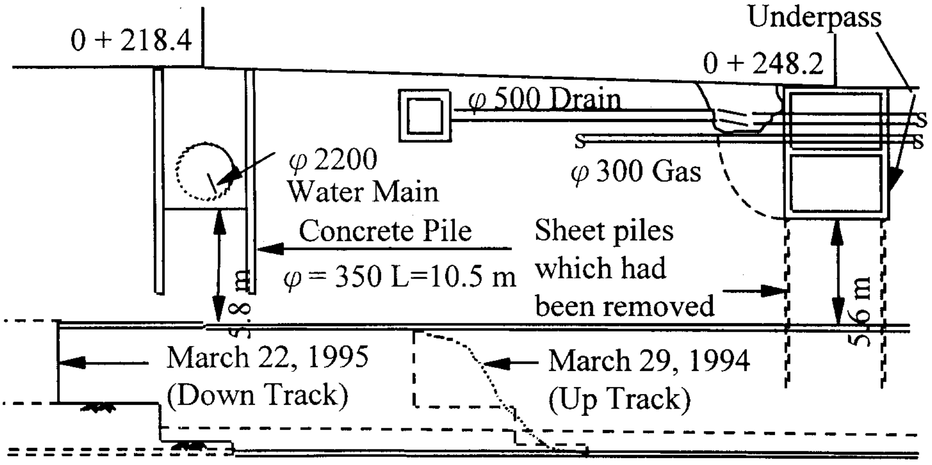


Fig. 23 Incidents occurred along the route of CH221 NATM tunnels

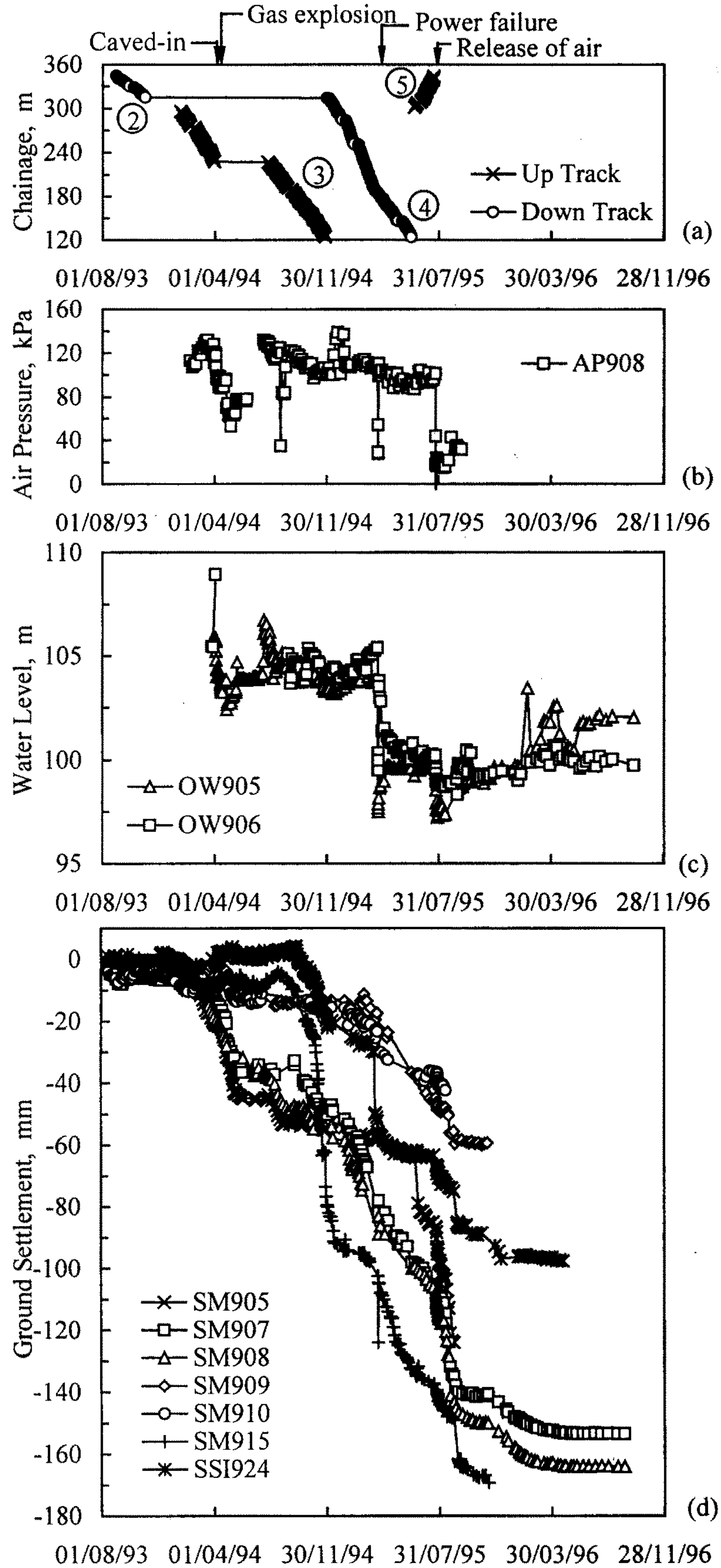


Fig. 24 Progress of CH221 NATM tunnelling and instrument readings

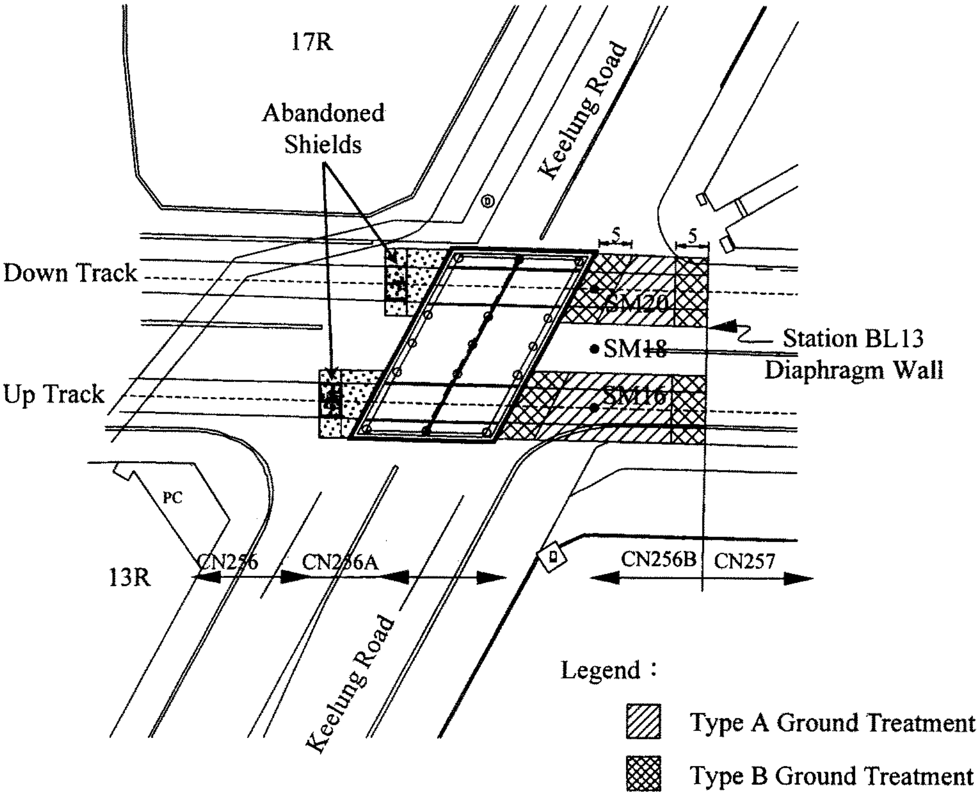


Fig. 25 Layout of CN256B NATM tunnelling

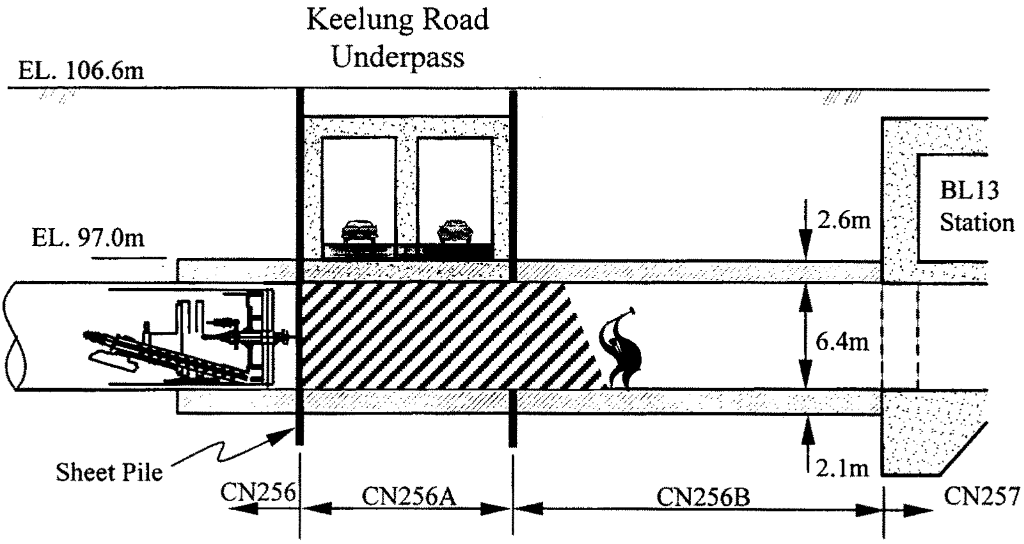


Fig. 26 Schematic view of CN256B NATM tunnelling

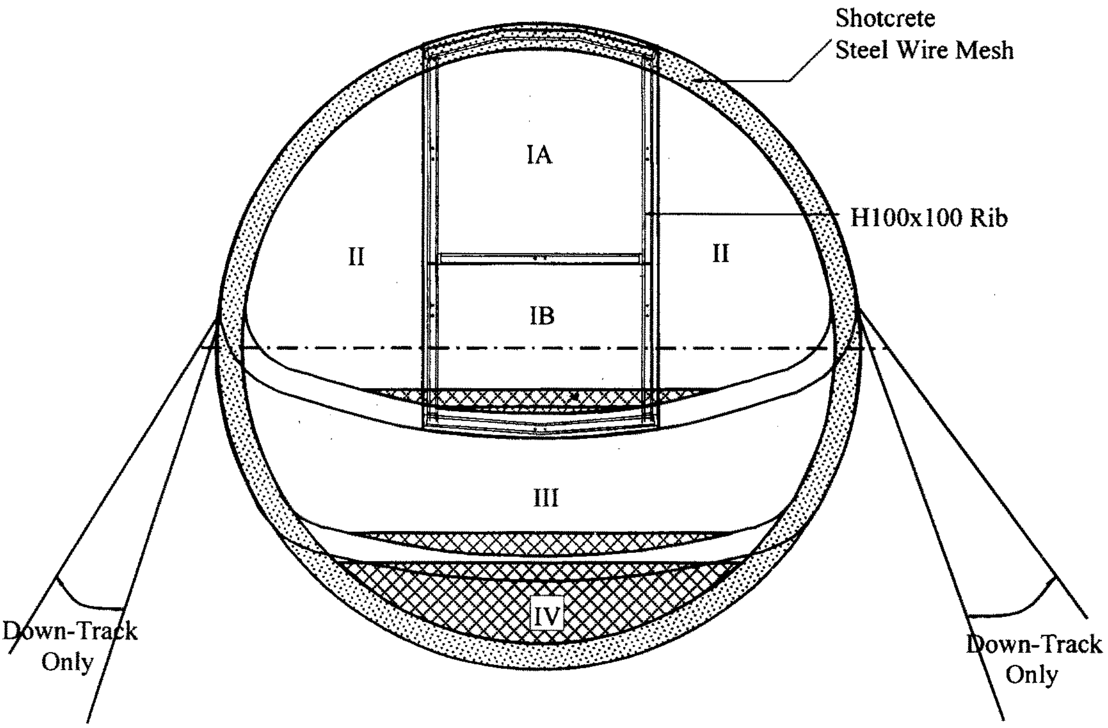


Fig. 27 Sequence of excavation for CN256B NATM tunneling

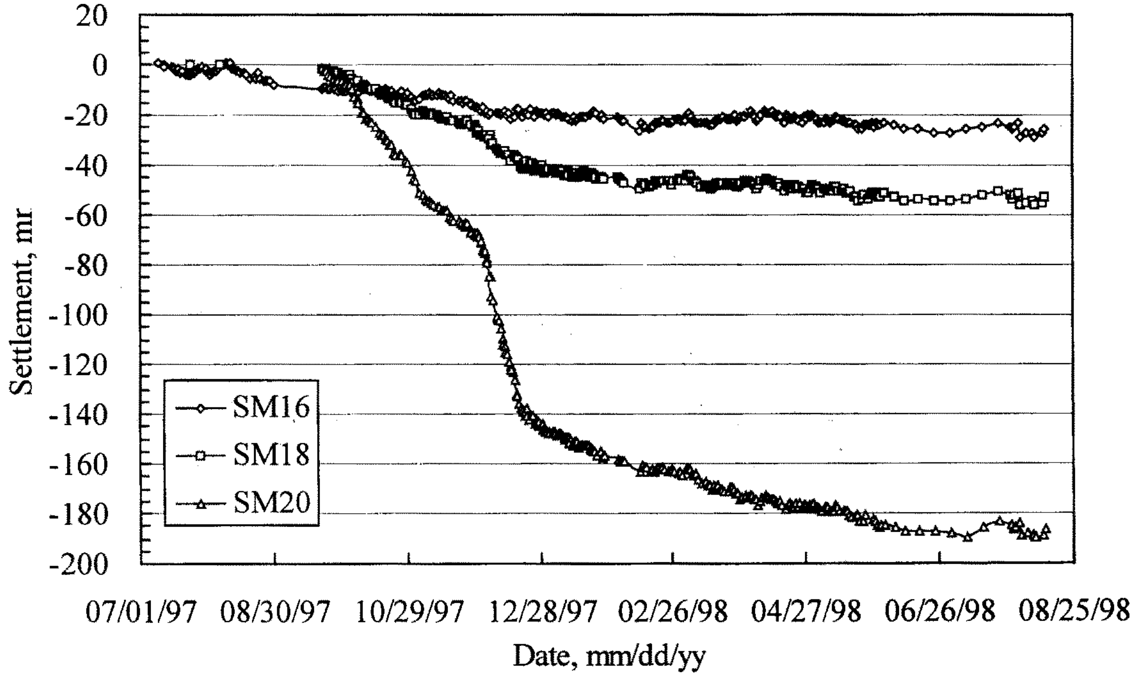


Fig. 28 Ground settlement for CN256B NATM tunnelling

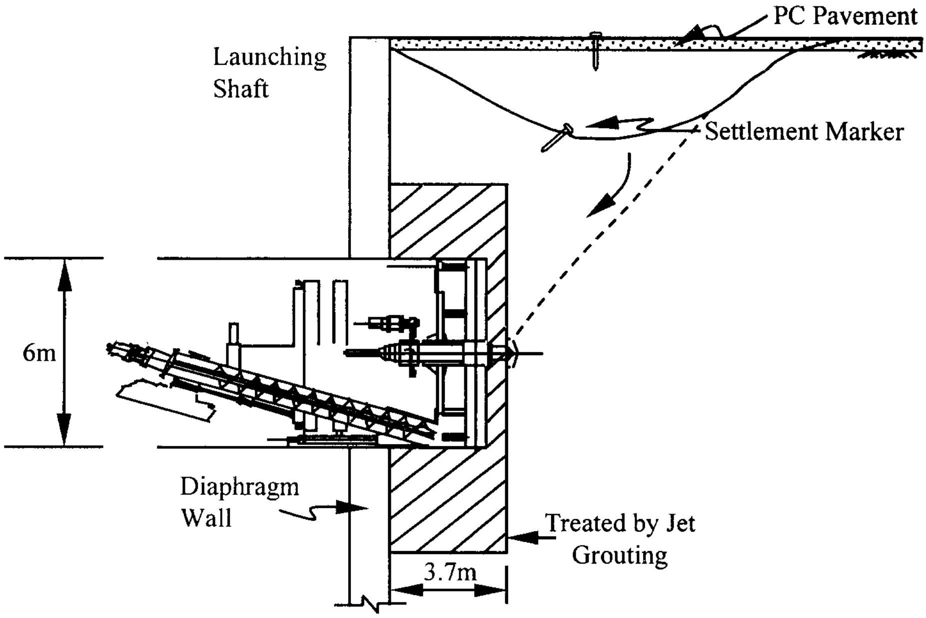


Fig. 29 Tunneling problem during launching in Contract CT201A

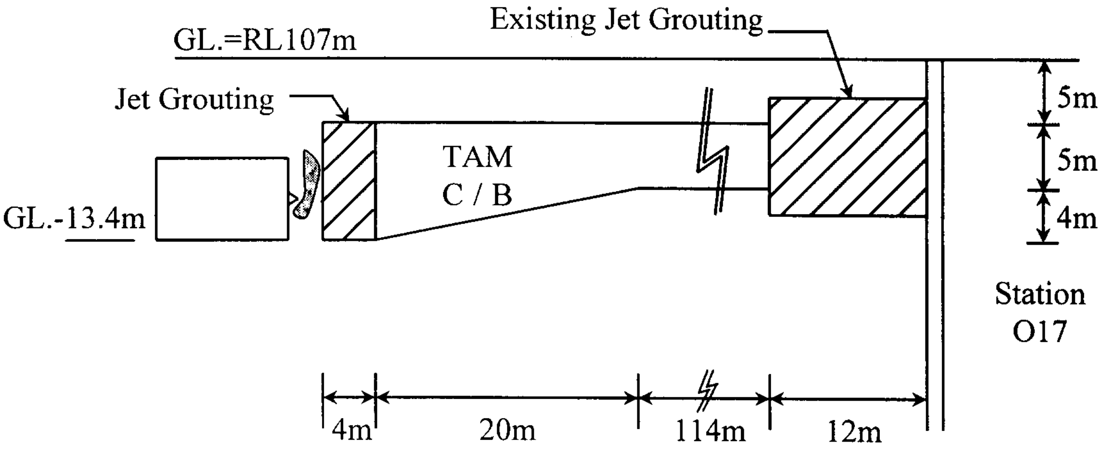


Fig. 30 Ground treatment at Up-Track Tunnel of Contract CC277