

# UNDERGROUND CONSTRUCTION OF TAIPEI TRANSIT SYSTEMS

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# Underground Construction of Taipei Transit Systems

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**SYNOPSIS** With all the six lines in the full swing, the construction of the Taipei Rapid Transit Systems is reaching its peak in 1993. Deep excavations are being carried out in soft recent deposits for the construction of underground stations. Tunnels are to be bored through a variety of ground conditions. Without doubts, geotechnical engineering plays an important role in such a major civil work. Soil investigations have been carried out in different phases for clarifying the ground conditions along the routes and the characteristics of the soils are now better understood. Some of the new findings are documented herein for the purpose of serving as future reference.

Protection of adjacent structures has been a serious concern in all the phases and positive measures have been taken to reduce ground movements for ensuring the safety of these buildings. Some of such measures are presented with their effects discussed. Data show that with adequate design, tight supervision and good workmanship, it is possible to reduce ground settlements to a half of what was experienced before.

## 1 INTRODUCTION

As depicted in Fig. 1, the first phase of the Taipei Rapid Transit Systems (TRTS), the so-called Priority Network, consists of six lines, namely the Mucha, Tamshui, Hsintien, Nankang, Panchiao and the Chungho Lines. With all these six lines in the full swing, the construction is reaching its peak in 1993. Much has been learned from the works which has been completed so far and the experience is valuable to the future networks and works of similar nature.

Of the six lines, the Mucha Line (with its extension to Neihu in the near future) is for medium-capacity trains, which are capable of moving, up to, 30,000 passengers per hour in one direction, while the rest are for heavy-capacity trains with a capacity of 60,000 passengers per hour. The entire Mucha Line is above-ground. The Tamshui Line has a short underground segment in the central city area, an elevated section in the suburb and an at-grade section near its northern end. The rest of lines are all underground. In summary, not counting the extension of the Mucha Line to Neihu and the extension of the Panchiao Line to Tucheng, there are 22 km for the elevated sections with a total of 22 stations, 9.4 km for the at-grade sections with a total of 6 stations and 32 km for the underground sections with a total of 32 stations.

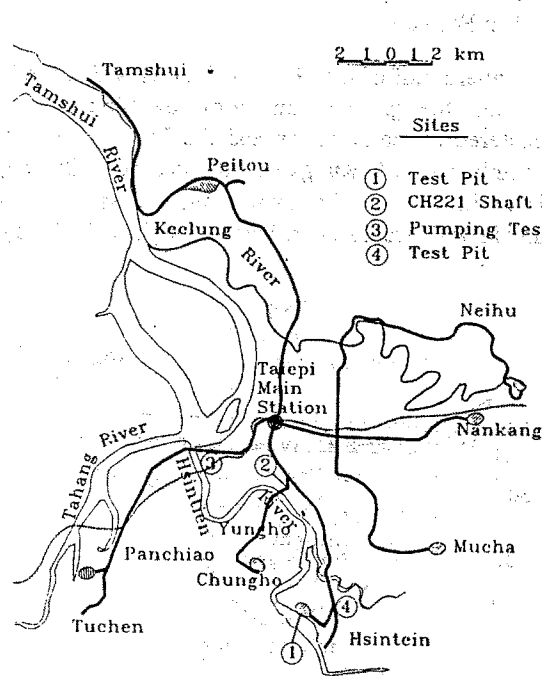


Fig. 1 Priority Network of Taipei Transit Systems

For the underground sections, all the stations, traction substations and crossovers are constructed by using the cut-and-cover method. Most of the tunnels linking the stations are constructed by shield tunneling, with a few segments constructed by using the cut-and-cover method and a short segment of 222 m in length constructed by using the NATM method.

Realizing the importance of geotechnical engineering in a major construction such as this, the Department of Rapid Transit Systems engaged Moh and Associates as Geotechnical Engineering Specialty Consultants (GESC) to assist in the review of designs and supervision of constructions. Numerous instruments have been installed for ensuring the safety of the constructions and obtaining information for guiding the future designs and it is one of the GESC's duties to perform system-wide evaluation of the data obtained.

## 2 GEOLOGY ALONG TRTS ROUTES

The geology of Taipei Basin has been extensively discussed by, for example, Moh & Ou, (1979), Moh & Chin (1991). Generally speaking, the Taipei Basin is underlain by a thick layer of alluvium, the so called Sungshan Formation, which is in turn underlain by the Chingmei Formation. The Chingmei Formation is an alluvial fan extending to a depth far beyond the influence of the construction. The Nankang and the Panchiao Lines, as depicted in Fig. 2, are constructed entirely within the Sungshan Formation.

The northern half of the Hsintien Line is within the Taipei Basin and the ground conditions are similar to those encountered in the Nankang and the Panchiao Lines. At the south end of Kungkuang Station (G7 Station), as depicted in Fig. 3, the twin tunnels pass by Chanchu Mountain and are to be driven through a thick layer of

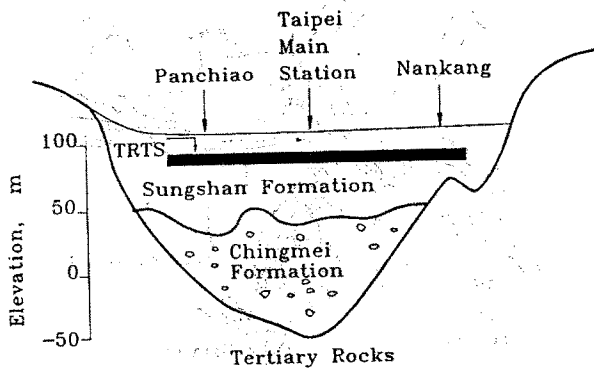


Fig. 2 Geology Along the Routes of Panchiao and Nankang Lines

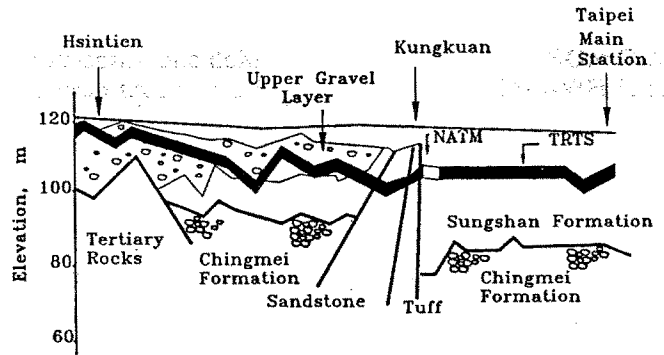


Fig. 3 Geology Along the Route of the Hsintien Line

sandstone which is the root of the Mountain. Further to the south, a layer of gravels is present at shallow depths below surface, and is underlain by a sandy layer and, further below, by the Chingmei Formation. Toward the very southern end of the Hsintien Line, this gravel layer is exposed at the surface and is underlain by tertiary rocks. A gravel layer is presented at the similar elevation along the Chungho Line as shown in Fig. 4. This Upper Gravel Layer is believed to be of the same origin as that encountered along the Hsintien Line but was deposited at a different time (Fu, et al, 1990).

### 2.1 Sungshan Formation

The Sungshan Formation consists of an alternation of 6 sublayers of silty sand (SM) and silty clay (CL) with varying thickness from place to place. Depending on the distribution of these sublayers, the Basin is divided into 7 zones as follows:

- Tamshui River Region : T1, T2, and T3 Zones
- Keelung River Region : K1 and K2 Zones
- Hsintien River Region : H1 and H2 Zones

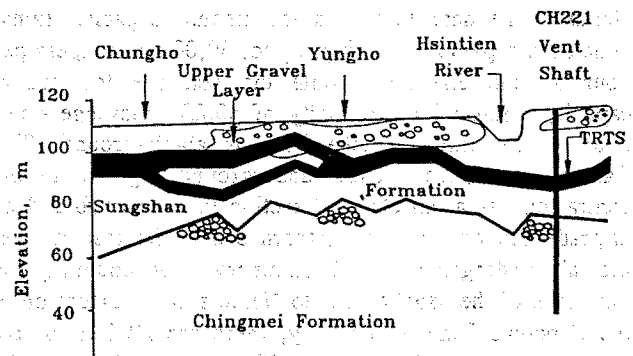
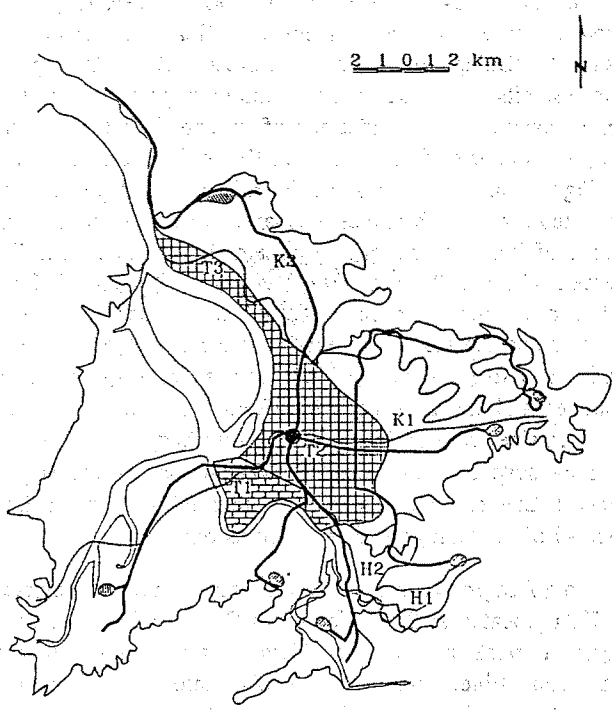


Fig. 4 Geology Along the Route of the Chungho Line



as shown in Fig. 5 (Moh and Associates, 1987a). The 6-layer stratigraphy is the most distinct in the central city area which is in the T2 Zone. In K1 and K2 Zones, the sandy layers are rather thin and sometimes are totally missing.

In addition to the soil investigation carried out during the design stage, an extensive soil exploration program was performed by the TRTS contractors using cone penetration tests for the purpose of clarifying the stratigraphy in the central city area. A typical soil profile obtained is given in Fig. 6. It can be noted that thin sand and clay seams in these sublayers are clearly identifiable in the porewater pressure profiles proving that cone penetration test is an effective tool for that purpose. Another interesting fact to note is that in Sublayers 3 and 5, which are classified as SM layers, the porewater pressure responses to the cone penetration are constantly below the atmospheric pressure which is normally taken as reference pressure and is assumed to be zero in engineering applications. For these tests, the piezometer is located above the shoulder of the cone, not right at the tip of the cone. It is postulated that as the cone advances, dilation of the surrounding soils at

Fig. 5 Subzones in Sungshan Formation

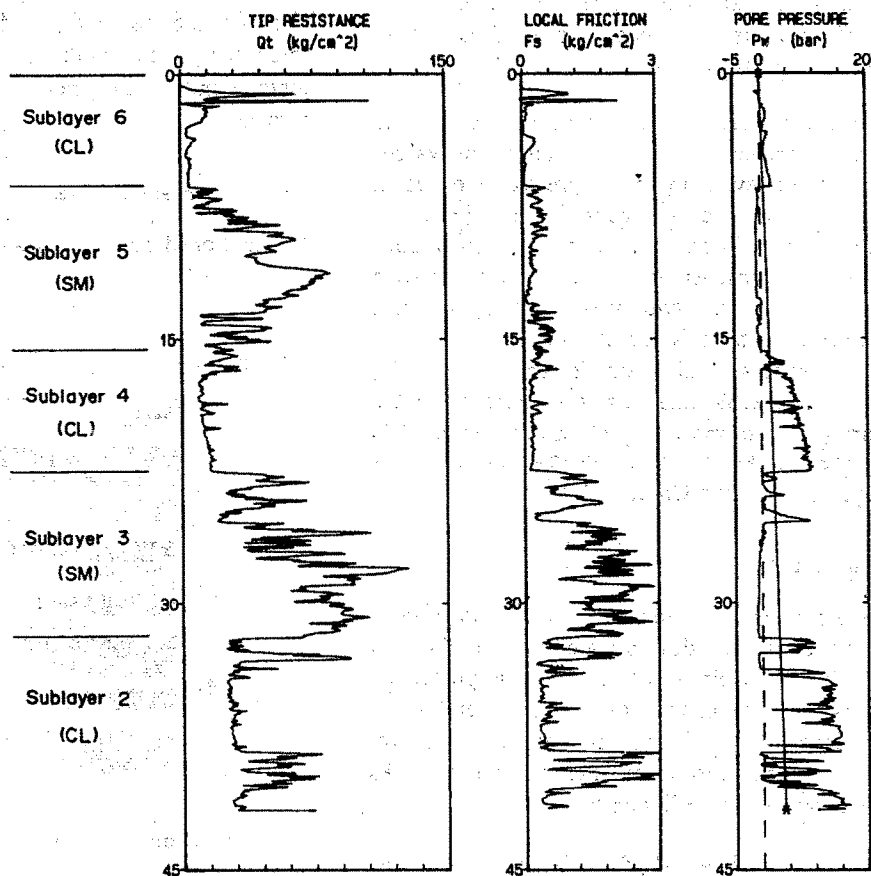


Fig. 6  
Typical Soil Profile  
in Central Taipei  
City Area

where the piezometer is resulted in negative porewater pressures.

For most seepage analyses for obtaining pressure distributions and flow nets, relative permeabilities of subsoils will be sufficient and absolute permeabilities will not be required. Dissipation tests were performed in conjunction with the cone penetration tests and relative permeabilities of the sublayers in the Sungshan Formation were determined from the rates of dissipation of excess porewater pressure in the clayey layers and the rates of recovery of porewater pressure in sandy layers. Assuming that the permeability of the least permeable layer, ie. Sublayer 2, is a unit, the relative permeabilities of the sublayers in the Sungshan Formation are as follows:

Sublayer	Soil Type	Relative Permeability
5	SM	200
4	CL	2
3	SM	50
2	CL	1
1	SM	500

Data on Sublayer 6 are insufficient in quantity for a meaningful statistical analysis. The permeability of Sublayer 6 is of little interest anyway because it is seldom required in practical problems.

It should be realized that the permeability of a layer is governed by the presence of seams. Though the values listed above are believed to be representative, local variations could easily be as much as an order of magnitude, if not two. Fortunately, pressure distributions and flow nets are insensitive even to the relative permeability as long as the two interfacing layers have permeabilities differing by a factor of, say, 10 or greater. For practical purposes, Sublayers 2 and 4 can be considered as impermeable and this is evidenced by the fact that the drawdown of more than 40 m in water head in Sublayer 3 in the past did not result in lowering of water table in Sublayer 5 (Moh and Chin, 1991).

## 2.2 Chingmei Formation

The Chingmei Formation was formed by the deposition of sands and gravels carried downstream by the Hsintien River after the Taipei Basin was formed. It is postulated that large rocks were deposited near the rim and small rocks were carried by floods further downstream. Therefore, it is reasonable to expect that the sizes of rocks decrease with distance from Hsintien.

The nature of the Chingmei Formation in the central area

of the Taipei Basin is relatively little known because of the great depth at which it is buried. Near the rim of the basin, the Chingmei Formation is present at shallow depths and test pits were dug in the planning stage of the project for determining the gradations of its constituent. In a pit dug on the bank of the Hsintien River, i.e. Site 1 shown in Fig. 1, about 100 m away from Hsintien Depot, it was reported that the Chingmei Formation was encountered at a depth of 2m and the gravel content was greater than 70 percent (Moh and Associates, 1987b). Gravels of 30 to 50 mm in size accounted for 40 to 50 percent of the volume and the matrix materials were mainly course sands with little fine contents. The gravels are subrounded to rounded and appear to be interlocked. Boulders as large as 1 m in size were exposed on the river bed. However, it is now suspected that this pit was in fact in the Upper Gravel Layer, which is to be discussed in Section 2.3, instead of the Chingmei Formation as reported.

The diaphragm walls for the ventilation shaft of Contract CH221, located at Site 2 shown in Fig. 1, which is on the northern bank of Hsintien River at the crossing of the Chungho Line, penetrate deeply into the Chingmei Formation as depicted in Fig. 4. Boulders as large as 700 mm in size were encountered during the installation of panels. The materials removed from excavations for Panels 5 and 13 were logged and the results are given in Figs. 7 and 8, respectively, for information. As can be noted that, except in the top portion of Panel 5, the gravel contents in the Chingmei Formation are far greater than 50 percent.

## 2.3 Upper Gravel Layer

As mentioned above, a gravel layer is present at shallow

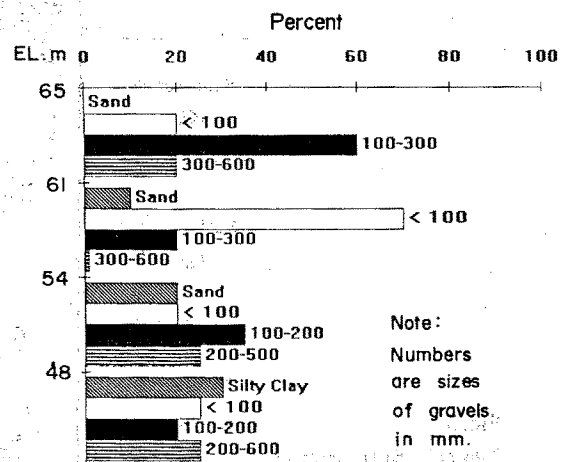


Fig. 7 Gradation of Chingmei Formation - Panel 5

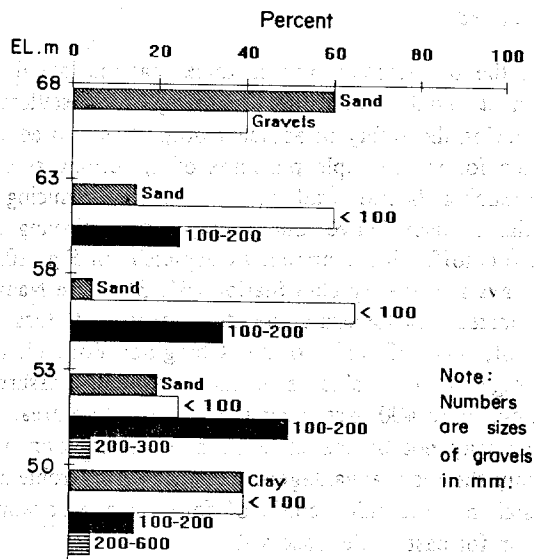


Fig. 8 Gradation of Chingmei Formation - Panel 13

depths in the middle section of the Chungho Line and the southern section of the Hsintien Line. In the preliminary soil investigation carried out in the planning stage, a test pit was dug at an outcrop near the junction of the Hsintien Line and the spur track to Hsintien Depot, i.e., Site 4 in Fig. 1. Excavation reached a depth of 8 m and it was reported (Moh and Associates, 1987b) that rocks, as large as 600 mm in size, count for 70 percent of the materials removed from the pit. During excavation for the ventilation shaft for Contract CH221 in this gravel layer, see Figs. 1 and 4 for location and profile, boulders of 700 mm were encountered. However, it was reported that the rocks, in general, are smaller than those encountered in the Chingmei Formation at the bottom. Besides, the loss of slurry was heavy during diaphragm walling indicating the looser nature of this layer in comparison with the Chingmei Formation.

It was reported by Fu, et al, (1990) that this gravel layer is underlain by Sublayer 4 at places and indeed belongs to Sublayer 5 of the Sungshan Formation.

### 3 CUT-AND-COVER CONSTRUCTIONS

#### 3.1 Methods of Construction

The underground portions of the TRTS were awarded to 12 Detailed Design Consultants (DDC). Bottom-up construction is favored by the majority with only three DDC adopting the top-down or semi-top-down method. This may be due to the demand for early release of tracks at the bottom level. The bottom-up construction is also

more flexible in traffic diversion which is a major problem in congested city areas. Since almost all the stations are under existing roads, steel decks are provided for maintaining the surface traffic and are supported on pin piles which in most of cases are bored piles installed by using the reverse-circulation technique. One deficiency observed is the presence of slime at the bottom because of the difficulty in cleaning the sediments. This will result in soft toes and the absence of tip resistance. It is therefore suggested to discount the tip resistance unless the toe is grouted.

#### 3.2 Diaphragm Walling

Diaphragm walls are adopted to retain the side wall practically for all the deep excavations exceeding 8m in depth and the double-wall system is favored with limited cases of single wall and composite walls. The diaphragm walls normally range from 1,000 to 1,200 mm in thickness. However, for Chungshan Station (R14 Station), diaphragm walls are 1,500 mm in thickness due to the poorer-than-usual ground conditions and the greater depth of excavation, i.e. 26 m. Panel excavations are usually limited to 6 m in width and stability appears to be satisfactory. In one case, an excavation of 8 m in width was attempted unsuccessfully and it had to be abandoned due to the cave-in of the ground.

The diaphragm walling for the ventilation shaft for Contract CH221 can be considered as the most difficult because of the presence of large boulders in the Chingmei Formation at the bottom as shown in Fig. 9. The circular shaft is formed by 16 panels of 1,250 mm in thickness. The diaphragm walls penetrate into the Chingmei Formation by 35 m. During the work, boulders of various sizes were encountered. The smaller ones were broken up and removed by the diaphragm walling machine and the larger ones had to be chiseled. The soils and gravels removed from the excavations for Panels 5 and 13 were logged and the results have been discussed in Section 2.3. The excavations for the male panels are 3 m in width and those for the female panels are 6 m in width. Because of the presence of boulders in the Chingmei Formation, the progress of excavation was very slow. On an average, it took 7 days to excavate for a male unit and 14 days for a female panel without counting machine breakdowns and repairs. A more powerful machine equipped with rock cutters would have done a better job.

#### 3.3 Grouted Plug

It is worth mentioning that for the ventilation shaft shown in Fig. 9, a grouted plug of 5m in thickness was formed at the bottom to prevent blow-in and piping. Silicalizer (SL)

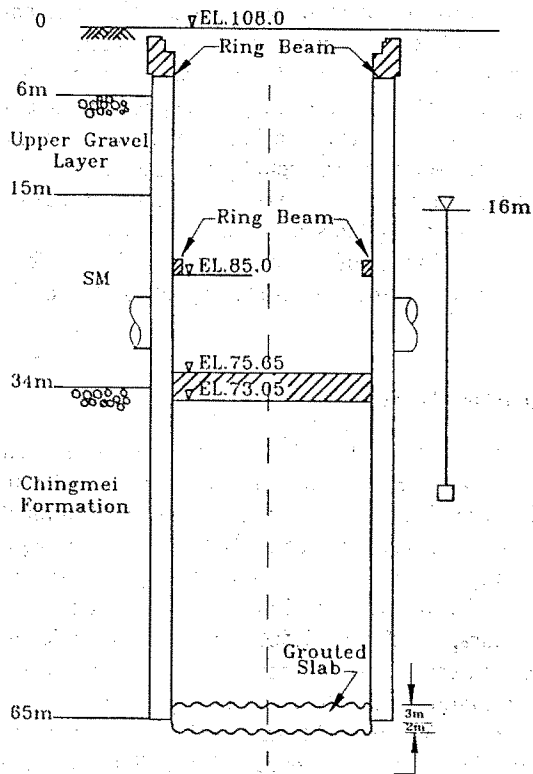


Fig. 9 CH221 Ventilation Shaft

was injected into the Chingmei Formation by using the Sleeve Grouting (double-packer) method. Pumping test was carried out subsequently for confirming the effectiveness of the slab in reducing seepage flow. The rate of rising of water level was 2.75 cm/hour at the end of 2 days with a difference of 18 m in water heads between the inside and outside of the shaft. This rate corresponds to a coefficient of permeability of  $4 \times 10E-5$  cm/sec which is deemed satisfactory for the purpose.

The diaphragm wall panels in this case are structurally connected at joints. The circular shaft practically behaved as a compression ring during the excavation. It is braced by ring beams at two levels as depicted in Fig. 9. The lower ring beam is a provision for the breakouts of diaphragm wall panels for tunnel drives, two toward the north to join the Hsintien Line and two toward the south to Chungho. The verticality of diaphragm wall panels was excellent and the excavation has been carried out to the final depth of 35 m successfully with only a few millimeters of lateral movements of the diaphragm walls as measured by inclinometers. The water inside the shaft was continuously pumped, at a rate of a few cubic meters per day to maintain a level of a couple of meters below the bottom of excavation. Pumping was discontinued once the base slab was cast.

### 3.4 Buried Slab

Where the soft deposits are so thick that the lateral wall movement could lead to much ground settlements endangering the safety of adjacent properties, buried slabs are used for the multiple purposes of a) serving as struts and reducing lateral wall movements, b) reducing the potential of base heave and piping and c) serving as a seepage cutoff. For example, as depicted in Fig. 10, for the excavation of City Hall Station (BL13) of the Nankang Line, instead of lowering the diaphragm wall toes to a depth of, say, 52 m into the Chingmei Formation for anchorage, a 4 m slab is formed by high-pressure jet grouting, up to 400 bars, over the entire station area. The scheme may not be the most economical solution but is certainly the most advantageous and the least problematic. The slab has the side benefit of forming a neat working platform for casting the base-slab.

### 3.5 Ground Movements

Building protection has been emphasized and it has been realized that the most effective measures for reducing the damaging potential is to reduce ground movements at sources. The progress of excavation is under strict supervision and over-excavation is prohibited. The retaining systems are stiffer than those usually adopted and all the struts are preloaded to 50 percent, or even greater, of their design loads. Struts are installed promptly and

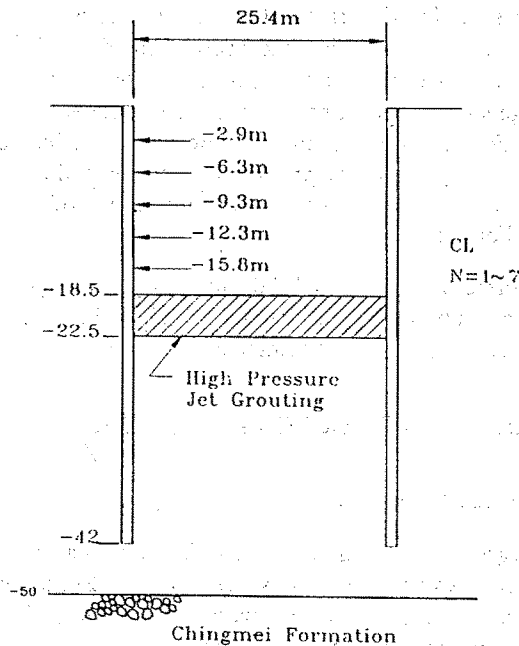


Fig. 10 Buried Slab at City Hall Station

delays are disallowed. Furthermore, it is realized that the bad practice of pumping ground water was responsible for large ground settlements observed in the past, pumping of groundwater outside the excavation is therefore strictly banned.

At the time this paper was prepared, the excavations of 5 stations have been completed or nearly completed. Wall movements for excavations of 15 to 20 m in depths were usually 30 to 70 mm and ground settlements were commonly less than 30 mm. These values are in general only a half of those experienced before for deep basement excavations in the Taipei City.

## 4 BORED TUNNELING

### 4.1 Method of Tunneling

Except the section linking the crossover at the south end of Taipower Station (G9 Station) and Kungkuan Station (G7 Station), see Fig. 3 for location, all the tunnels are bored by using shield machines. This said section which is too short to be economical for shielding is to be bored in compressed air by using the NATM method.

The contractors are given the liberty of choosing their own methods of tunneling. Of all the contracts, slurry type shield machines are adopted in Contract CH221 only and earthpressure balancing shield machines are used in the rest of contracts. The former performs better in sandy soils and the latter performs better in clayey soils. The Sungshan Formation consists of sand and clay alternations and most of the tunnels are to be bored in both types of soils, therefore, as far as soil conditions are concerned, the two types of machines are competitive. Although slurry machines are slightly cheaper than earthpressure balancing shields, the high cost for the slurry treatment plant makes them less attractive. Besides, the lack of space for the treatment plant makes it almost impossible to use slurry shields in the central city areas. However, because of better control of pressure at face, leading to less ground movements, slurry shield remain to be a better choice for tunneling underneath existing buildings wherever possible. For Contract CH221, the ventilation shaft shown in Fig. 9 is to be used as the launching shaft. It is located in an open space nearby Hsintien River, therefore there is ample space for the slurry treatment plant. To deal with sands, all the earthpressure balancing machines adopted in TRTS have the facility of adding water or lubricating agents into the earth chamber in case needs arise.

The progress of tunneling is quite impressive. It was reported that a maximum daily production of 18 rings of 1m each in width was achieved and a rate of 6 rings per

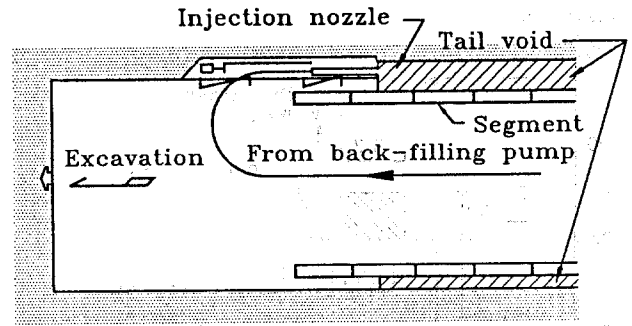


Fig. 11 Tail Void Grouting

day would be a reasonable average for the entire period of tunneling including the learning period. In fact, for one of the drives, the tunneling had to be slowed down toward the later stage because of the late completion of the arrival shaft.

### 4.2 Grouting of Tail Voids

It is realized that closure of tail void is responsible for the major portion of ground settlements, the need of automatic grouting of tail voids is thus emphasized by specification. Fig. 11 shows one type of the grouting facility provided on the shield machines. A grouting pipe is attached to the crown of the shield and pressure is maintained in the tail void all the times. Grouting is automatic as the shield advances. Although a piston is provided to clean the pipe in case the pipe is plugged by hardened grout, it is reported that operation is problematic and sludge sometimes ingressed through the pipe into the shield during cleaning.

The importance of prompt grouting is emphasized and specification requires that back grouting through the holes provided on the tunnel lining be carried out as soon as each ring leaves the shield. Only cement milk, with necessary additives to increase its workability or strength, is used as grout and sands are disallowed.

### 4.3 Ground Treatment

Jet grouting is used in front of launching and arrival shafts to provide stability and water tightness. Conventionally, the length of treatment is longer than the shield by a couple of rings so the tail void can be properly sealed before the cutter breaks through the treated ground. This is not only important for preventing loss of ground water but is also important for keeping the pressure in the earth chamber. However, in most of TRTS contracts, a sealing gasket is provided at the mirror for the above-mentioned purposes and the lengths of the treated zones are much reduced.

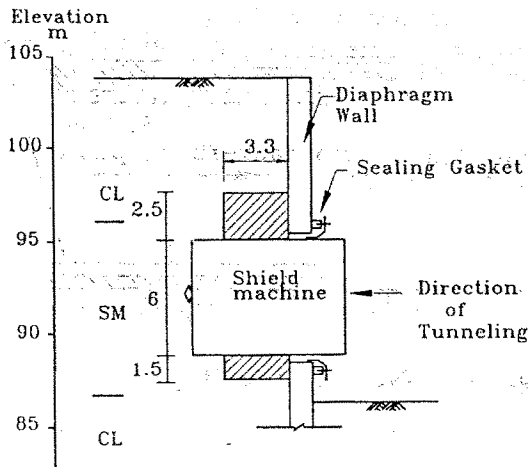


Fig. 12 Ground Treatment for Launching Shaft

Figure 12 shows that the treatment for Contract CH218, for example, is only 3.3 m in length. There are a couple of launching shafts for which the lengths of treatment are even shorter. It appears, at least for the tunnel drives which have been launched, the shorter zones of ground treatment did not lead to serious problems.

#### 4.4 Ground Settlements

At the time this paper was prepared, three tunnel drives had been completed. Ground settlements induced by tunneling are generally small for the ground conditions encountered. Most of the designers assumed a ground loss of 3 percent and because of the poor performance experienced before, it was thought this number was an optimistic one. Observations have indicated that the ground loss is generally much less than this limit.

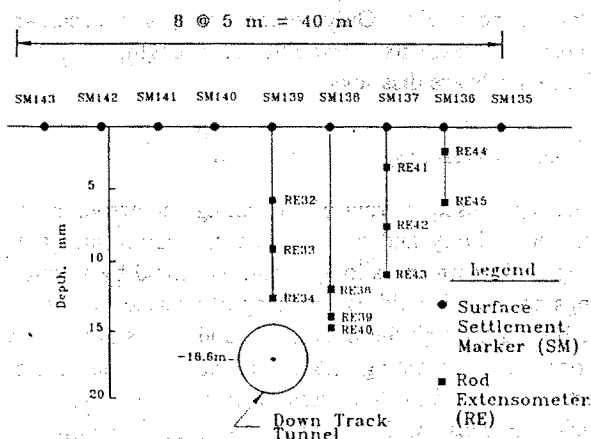


Fig. 13 Instruments at Section B1 of CH218

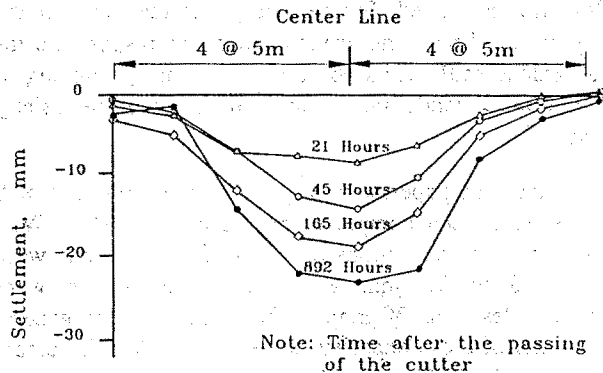


Fig. 14 Surface Settlements Over Tunnel - CH218, Section B1

The settlements observed at Section B1 of Contract CH218 are presented herein for illustration. Figure 13 shows the instruments installed and the settlement troughs obtained at various times after the passing of the cutter are shown in Fig. 14. Figure 15 shows the time histories of the settlements recorded by the surface marker (SM) and the settlements of the ground measured at depths of 6m, 9.5m and 14.5m by rod extensometers (RE) at the center of the tunnels. As can be noted from Fig. 15 that major

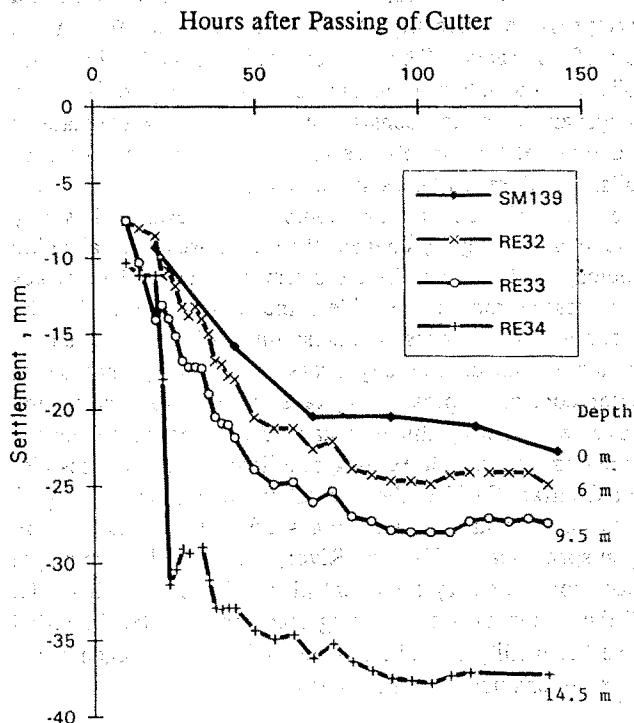


Fig. 15 Settlements Over Center of Tunnel - CH218, Section B1

settlements occurred after the passing of the tail of the shield. A sudden subsidence of 20mm was observed for sensor RE34 which is located at a depth of 14.5m, or 1m above the crown, immediately as the tail passed. The sensors at shallower depths, i.e. RE32 and RE33 did not respond accordingly, conceivably, due to the arching effects. The shield machine used does have the grouting facility illustrated in Fig. 11, however, at this location it was not functioning due to mechanical problems.

Back-grouting through grouting holes after the segment rings left the shield heaved up RE34 by a couple of millimeters but settlements continued subsequently. The time histories of settlements are presented in a semi-log form in Fig. 16 and three stages can be identified, namely,

- Stage I: shield advancement
- Stage II: closing of tail void
- Stage III: consolidation

Ground settlements due to the passing of the shield and the closing of tail void are usually referred to as ground loss and are usually assumed to have normal distributions as follows:

$$\delta = \frac{vA}{2.5\lambda} \exp\left(\frac{-x^2}{2\lambda^2}\right) \quad \text{Eq. 1}$$

where  $v$  = ground loss,  $A$  = sectional area of tunnel, and  $x$  = distance to the center. Most of DDC adopt the

relationship suggested by Clough and Schmidt (1981) for estimating the spreading of troughs:

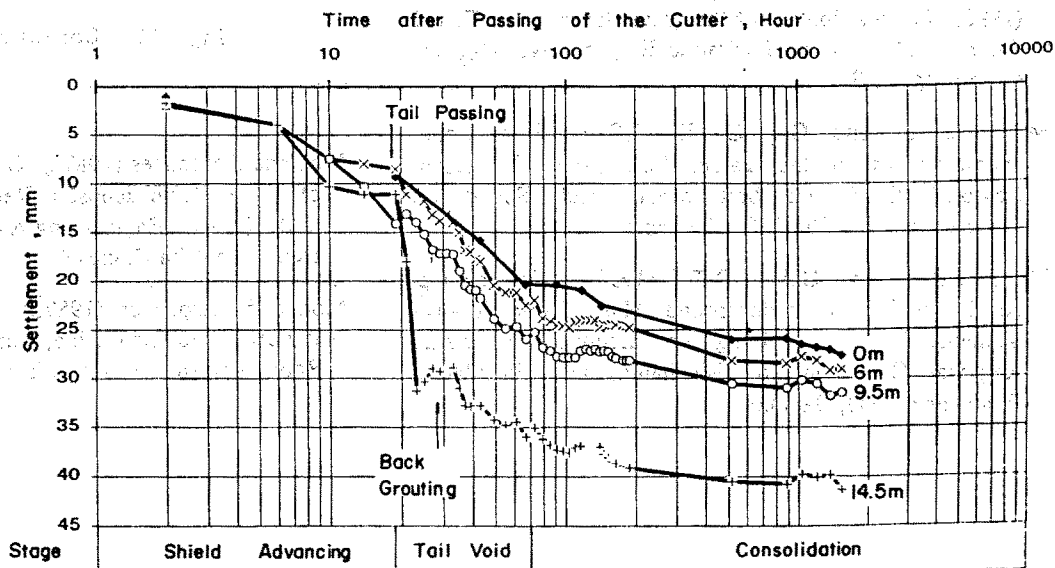
$$\lambda = \frac{D}{2} \left(\frac{z}{D}\right)^{0.8} \quad \text{Eq. 2}$$

where  $D$  = diameter of tunnel and  $z$  = depth to the springline of the tunnel.

Stages II and III can not be clearly distinguished because the two events were occurring concurrently. It is usually assumed that the point at which two straight portions of the settlement curves meet is the point separating the two stages. Accordingly, as can be noted from Fig. 16, the settlements occurred within 70 hours after the passing of the cutter can be attributed to ground loss and those occurred subsequently can be attributed to consolidation. The surface settlements obtained at 70 hours after the passing of the cutter correspond to a ground loss of 1.3 percent. The theoretical settlements computed by using the above equations for various depths are compared with those recorded by rod extensometers in Fig. 17. As can be noted that the use of the above equations overestimates the settlements at depths. For example, at the depth of 9.5m, a settlement of 35 mm was computed while the settlement recorded by RE33 is only 26mm. This results in over-estimation of differential settlements of the foundations of adjacent buildings with basements.

It can also be noted that consolidation settlements in

Fig. 16 Stages of Settlement



general decrease as depth increases. This trend is a reverse of that for the settlements due to ground loss. It is logical to be so because the consolidation settlement at any depth is the sum of the reductions in thicknesses of the subsoils below.

## 5 CONCLUSIONS

For cut-and-cover constructions, the lateral movements of walls and ground settlements observed so far are far less than the values predicted previously and the construction has been carried out with few technically related problems. The excellent performance is attributed to the adequate design, better workmanship and tight site supervision.

The ground conditions in the central city area have been found to be suitable for shield tunneling and the progress has been quite impressive. For the first drive of Contract CH218, the ground loss was limited to 1 to 2 percent and consolidation settlements are generally within 10 mm.

### Acknowledgments

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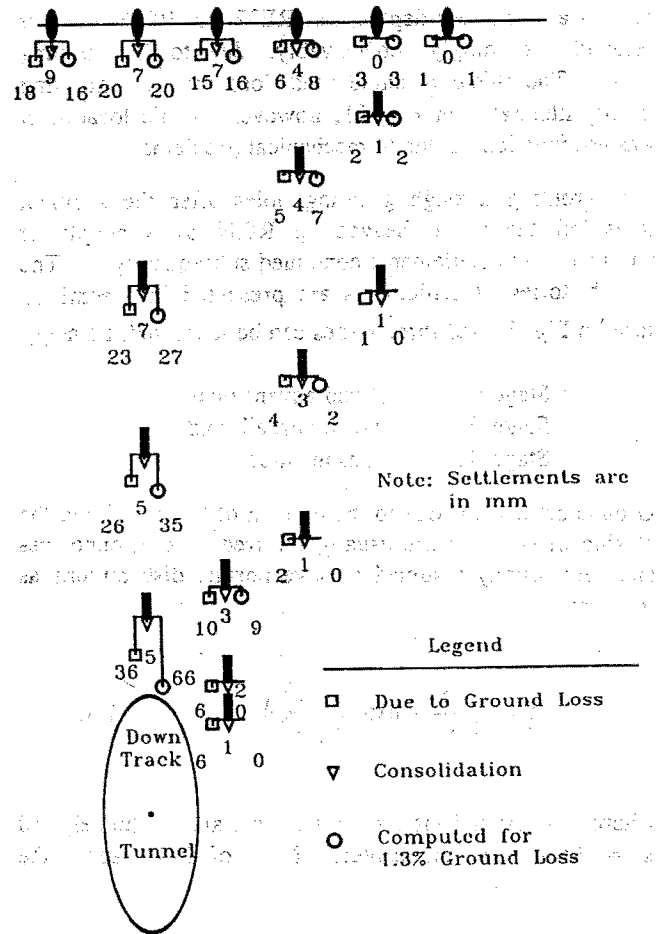


Fig. 17 Comparison of Settlements

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