SOFT GROUND TUNNELLING FOR SINGAPORE AND TAIPEI MRT SYSTEMS

by
Za-Chieh Moh,
Terence W. Hulme and Richard Nanhuei Hwang

Reprinted from Proceedings of
Bengt B. Broms Geotechnical Symposium
December 12~15, 1995, Singapore, pp.313-332

SOFT GROUND TUNNELLING FOR SINGAPORE AND TAIPEI MRT SYSTEMS

Moh, Za-Chieh
Moh and Associates, Taipei, Taiwan
Hulme, Terence W.
Mass Rapid Transit Corporation, Singapore
Hwang, Richard Nanhuei
Moh and Associates, Taipei, Taiwan

ABSTRACT: With the construction of the mass rapid transit systems in Singapore and Taipei, much has been learned about tunnelling in soft ground. It is the intention of this paper to compare the performance at these two places in an attempt to evaluate the influence of ground conditions on, primarily, the porewater pressure response to tunnelling operation and settlements over tunnels. Emphasis is on the use of earth pressure balancing shield machines which are becoming more and more popular in this region.

INTRODUCTION

Because of its complexity, the problem of ground response to tunnelling has not been fully understood. Numerous instruments have been installed for the construction of the Singapore Mass Rapid Transit Systems (SMRT) and the Taipei Rapid Transit Systems (TRTS) and valuable data have been obtained. As a result, much has been learned and our knowledge greatly improved. The limited space available for this paper does not allow for detailed discussions on all different types of tunnelling techniques, emphasis is thus placed on the use of earth pressure balancing machines (EPB) and their performance, particularly, porewater pressures induced around tunnels and ground settlements.

SINGAPORE MASS RAPID TRANSIT SYSTEM

A system map for the Singapore Mass Rapid Transit System (SMRT) is given in Fig. 1. Phase I construction covered the routes running from Yio Chu Kang to Outram Park and then to Clementi. Phase II construction extended the routes to Yishun in the north, Boon Lay and Choa Chu Kang in the west, Marina Bay in the south, and Pasir Ris in the east. The Phase III construction is to complete the loop linking Yio Chu Kang, Woodland and Choa Chu Kang. The details of the Phases I and II construction are available in Hulme, Potter and Shirlaw (1989). As shown in the insert in Fig. 1 that in the central city area, ie., between Toa Payoh and Marina Bay for the North-South Line and between Kallang and Redhill for the East-West Line, approximately 11 km route was formed in twin bored tunnels. The average drive between stations was 850m. Despite their relatively short length, most of these tunnel drives passed through a variety of ground conditions, from 150 MPa sandstones or granite to stiff clays, soft clays and to loose sands. The most common method of tunnelling was to use a Greathead type shield equipped with rotary backhoe, backacter, boom cutter or roadheader. For soft ground sections the shields were typically provided with face rams and grids, and face stability was maintained by compressed air. Soil grouting was used either as a substitute for or in conjunction with the compressed air. All of the tunnels driven by the Greathead type of shield were lined using precast concrete segments. Earth pressure balancing shields were used only in Contract 301 (C301) for driving the twin tunnels between Lavender and Bugis Stations.

The geology and soil conditions along the SMRT routes are well documented in Pok, Buttling and Hwang (1987). A soil profile along the route of Contract 301 in which earth pressure balancing shield machines were used is given in Fig. 2. The soil formation of primary interest is the young marine deposits, denoted as Type M in the profile, presented at shallow depths (20m or above) in the Kallang formation which infills the Kallang River Basin. The engineering properties of this marine clay are summarized in Table 1.

TAIPEI RAPID TRANSIT SYSTEMS

As depicted in Fig. 3, the first phase of the Taipei Rapid Transit Systems, the so-called Priority Networks, 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 in its peak now. Of the six lines, the Mucha Line (with its probable extension to Neihu in the future) is for medium-capacity trains, which are capable of moving upto 30,000 passengers per hour in each 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 section in the central city area, an elevated section in the suburb and an at-grade section near its northern end. The rest of the 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.

For the underground sections, all the stations, traction substations and crossovers are constructed by using the cut-and-cover method. Most the tunnels linking the stations are constructed by shield tunnelling, with a few sections constructed by using the cut-and-cover method and a short section of 222 m in length constructed by using the so-called NATM method. There are a total of 58 drives of which 4 were driven by using 2 slurry shield machines and the rest all by using earthpressure balancing shield machines.

The geology of Taipei Basin has been extensively discussed by, for example, Moh & Ou (1979), Moh & Chin (1991), Woo and Moh (1990). Generally speaking, the Taipei Basin is underlain by a thick layer of alluvium, ie., 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 too far to have any influence on the MRT construction.

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 eastern half of the Basin is divided into 7 geological zones: T1, T2 and T3 along the Tamshui River; K1 and K2 along the Keelung River; and H1 and H2 along the Hsintien River, as shown in Fig. 4, while the western half is yet to be mapped. The 6-layer stratigraphy of the Sungshan Formation is the most distinct in the central city area which is entirely within the T2 Zone. In K1 and K2 Zones, the sandy layers are rather thin and sometimes are totally absent.

The strength parameters for the various sublayers can be derived from the typical results of cone penetration tests shown in Fig. 5 and index properties for the sublayers relevant to the materials to be presented hereinafter are summarized in Table 1.

POREWATER PRESSURE RESPONSE TO TUNNELLING

Piezometers were installed in 7 sections along the TRTS routes and for each section pore pressures were closely monitored during the passing of shields in each of the two drives, giving a total of 14 cases. The results of three of these cases are presented hereinafter. The tunnels are lined by precast concrete segments of 250mm in thickness and 1m in length. The pressures in the earth chambers were about the same as the at-rest pressures.

The outer diameters of the shields, outer diameters of segmental linings and theoretical volumes of the tail voids (ignoring over-cut) for the three cases are:

Construction Contract	Outer Diameter of Shield (mm)	Outer Diameter of Lining (mm)	Volume of Tail Void (litres/m)
CN257	6040	5900	1312
CP261	6250	6100	1455
CN256	6050	5900	1408

The grout for filling the tail voids comprised cement/bentonite and sodium silicate and was injected as soon as each ring moved out of the shield. The line pressures were typically 350 kPa maximum.

Case 1: Contract CN257 Section T1 - K1 Zone (Clayey Ground)

Figure 6 shows the results obtained by the two piezometers installed in Section T1 of Contract CN257, located in K1 Zone in which clay dominates, of the Nankang Line. The data are replotted in Fig. 7 in a semi-log scale. Readings were taken at 5-min intervals. As can be noted that a maximum excess pore pressure of an order of 90 kPa was recorded by PP63 during the passing of shield in the Down-track tunnel and the dissipation of excess porewater pressure lasted for 60 days. This is typical for K1 Zone where the tunnels are embedded in a thick clay layer. When the shield in the Up-track tunnel passed this instrumented section, the maximum excess pore pressure recorded by the two piezometers was only 10 kPa, indicating that the lateral extent of pore pressure generation is approximately one diameter beyond the edge of the tunnel.

Figure 8 is a close-up of the data obtained during the passing of the shield. This figure can be read in conjunction with the progress of the operations depicted in Fig. 9. Shoving for each ring took about 40 min and back grouting for filling the tail voids was synchronized with shoving. The shield then stopped for 40 to 50 min for the installation of lining segments. Two distinctly different modes can be observed in Fig. 10. The pore water pressures responded to back grouting faithfully till the tail of the shield passed. In the period after the tail passed the instrumented section and before the ring (R238) was grouted, the pressures dropped as soon as the shield advanced, creating tail voids behind, and increased later as the voids were fully filled by the grout. Once Ring 238 was grouted, refer to the later part of curves in Fig. 8, the pore pressure response became insignificant subsequently. The different patterns in the responses between the two piezometers were most likely due to different positions of injection.

The excess pore pressures were small when the head of the shield passed and it is thus clear that excess pore pressures were induced as a result of grouting for filling the tail void, not a result of "pushing" of the ground by the shield machine.

Case 2: Contract CP261 Section T2 - T1 Zone (Sandy Ground)

The story is quite different in other zones. For example, Fig. 11 shows the readings obtained for Contract CP261, located in the T1 Zone in which sand dominates, of the Panchiao Line. This figure can be read in conjunction with the progress of tunnelling in Fig. 12. The maximum excess pore pressure recorded was of an order of 40 kPa and it dissipated rather quickly. It took only three hours for excess pore pressures to fully dissipate at the end of each shift. During the pause after each ring was excavated, excess pore pressures dropped to one-third of their values. Fortunately, readings were taken in 10 minute intervals, otherwise, the results would not have been meaningful.

Unlike the case for clayey ground, the effects of passing of the tail were not noticeable.

Case 3: Contract CN256 Section T3 - K1 Zone (Silty Layer)

Figure 13 shows the instrument layout for Section T3 of Contract CN256 of the Nankang Line and Fig. 14 shows the results obtained (Hwang, Wu and Lee, 1995). Although the section is located in K1 Zone, piezometers appear to be buried in a silty lens. Readings were taken at 5-min intervals. As can be noted, during the driving of the Down-track tunnel, excess pore pressures exceeding 100 kPa were recorded by ELP6. Directly above the crown, however, the excess pore pressures were minimal all the times as indicated by the readings for ELP7. The

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

where δ = ground settlement at surface, v = ground loss , A = sectional area of the tunnel, x = offset from center. The distance to the point of inflection, i, can be calculated from the following empirical formula (Clough and Schmidt, 1981):

$$i = (\frac{D}{2})(\frac{z}{D})^{0.8} \tag{2}$$

where D = tunnel diameter and z = depth to center of the tunnel. In using the above equations, it is uncertain that how long the observation period should be.

Ground settlements over tunnels were closely monitored for Section B1, refer to Fig. 15 for the layout of instruments, of Contract CH218, of the Hsintein Line and excellent results were obtained as shown in Fig. 16. Based on these results, it was suggested that settlements be divided into three phases as shown in Fig. 17 (Hwang, Fan and Yang, 1995), such that:

Phase 1: settlement/heave due to face advancement; settlement due to overcutting and close-in of soils, sliding of shield against soils, and dissipation of excess porewater pressures

Phase 2: settlement due to closure of tail void; heave due to back-grouting for filling the tail void, and settlement due to dissipation of excess porewater pressures

Phase 3: settlement, primarily, due to dissipation of residual excess porewater pressures, and settlement due to lowering of ground water table

All these three components are governed by different mechanisms and thus have to be studied individually for identifying factors affecting their magnitudes. However, because settlement readings are usually taken weekly, or daily the most, it is difficult to differentiate Phases 1 and 2 settlements. It is thus recommended to combine them as immediate settlements to be included in calculating ground loss using Eqs. 1 and 2. Phase 3 settlements are rather easily identifiable and are recommended to be analyzed separately.

For the case shown in Fig. 16, heaves of a couple of milimeters were observed before the passing of the head. Heaves of an order of upto 25mm were observed for about 5% of the settlement records for TRTS. In fact, the true figure could even be greater because settlement readings, as mentioned above, usually were taken weekly and ground heaves could have disappeared unknowingly as the ground settled subsequently. Even in Phase 2, ground may also heave as a result of grouting for filling tail void, eg., refer to the readings taken at a depth of 14.5m in Fig. 16. This phenomenon is more pronounced immediately above the tunnel than at the surface.

For the case shown in Fig. 16, Phase 2 ended at an elapse time of 4 days after the passing of the head of the shield. The immediate settlements, ie. the combined Phases 1 and 2 settlements, recorded by rod extensometers (RE) at various locations are marked on the right-hand side of Fig. 18 and the settlements troughs at the surface and at a depth of 10m from surface drawn. It is noted that, as expected, the settlement trough become narrower and deeper with depth. Eqs. 1 and 2 have been found to be applicable to Taipei Basin for cases with $z > 15$m or so and the settlement trough at the surface corresponds to a ground loss of 1.3% for the case shown.

The Phase 3 settlements in 100 days are plotted on the left-hand side of the figure. Unlike the case for immediate settlements, Phase 3 settlements decrease with depth. This is perfectly reasonable as the consolidation settlement at surface is an accumulation of the settlements of all the underlying layers. The widths of troughs are insensitive to depth because as illustrated in the preceding sections that the generation of excess pore pressures is limited to one diameter beyond the edge of the tunnel for clays and somewhat farther for sands, and is not a function of depth to the tunnel axis. This, of course, does not refer to consolidation as a result of lowering of ground water table.

pore pressure response during the passing of the tail of shield was very unfortunately missed because of a mistake in locating the section in relation to the rings.

ELP6 was damaged as the head arrived during the driving of the Down-track tunnel and a similar event occurred to ELP10, which is at an equivalent position, during the driving of the Up-track tunnel. Even ELP5 which is 2m farther away from ELP6 was damaged as the head passed. ELP8 which was on the opposite side of the tunnel and was 0.75m farther away from the tunnel in comparison with ELP5, however, did not show significant response. The reason is not readily available.

In fact, in another section with a similar instrument layout, ie. Section T4 of the said contract, the piezometers installed at 1m and 3m away from edge were also damaged as the head passed during the driving of the Down-track tunnel in February of 1995. It was suspected the grouting pressures were too high. The contractor was asked to reduce the grouting pressures from a maximum of 450 kPa to 350 kPa when the Up-track tunnel was driven in June of 1995. This time, the piezometer at a distance of 3m away from the edge of the tunnel was spared, however, the one at a distance of 1m away was still damaged. The contractor resumed the use of 450 kPa after the section was passed and leakage of grout through fissures was observed at ground surface, presumably, as a result of hydro-fracturing. This leads to the suspicion that these piezometers, which were supposed to be adequate for pressures up to 150 psi (1 MPa), were in fact damaged not due to the high pore pressures, but due to the ingress of grout into the pores in the sensors.

Factors Affecting Pore Pressure Response

Comparing Fig. 8 with Fig. 11, the influence of grouting on pore pressures reached a far longer distance in sand (the case for CP261) than in clay (the case for CN257) . In the case for CN257, refer to Fig. 8, very little excess pore pressures were induced till the head arrived. At that time, grouting was conducted at 9 rings (9m) behind the piezometers. This, together with the fact that during the driving of the Up-track tunnel, minimal excess pore pressures were recorded by the two piezometers which were at a distance of one diameter away from the edge, indicates that in clay the generation of excess pore pressures is limited to 9m.

In sands, refer to Fig. 11, the pore pressure response was already very strong when the head was several rings behind the piezometers. The strong response obtained by ELP39 which was at a distance of 24m from the center, or 21m from the edge, of the tunnel, also suggests that the influence of grouting stretches to 21m, or even farther.

In the case of CN256, ELP5 and ELP6 are most likely buried in a layer of silt with low-plasticity (ML) and their performance is thus expected to be in-between that for clay and that for sand. The fact that all the 4 piezometers installed at a distance of 1m, and 2 out of the 4 installed at a distance of 3m, from the edge of the tunnel were damaged during the passing of the shield, or rather, the head of the shield, is quite a mystery and demands further investigation.

It has been suspected (Hwang et. al, 1995) that the clay in the Sungshan Formation has sufficient standup time to hold up the tail void, however, the relaxation of the surrounding soil mass leads to reductions in pore pressures. The hypothesis appears to be supported by the data presented in Fig. 10. It is even possible for the excess pore pressures to become negative in certain circumstances.

In sands, the permeability is so high that excess pore pressures dissipate rapidly. On the other hand, it has been suspected (Hwang et. al, 1995) that water also gets into the tail void quickly to equalize the external pressure preventing the wall from collapsing. This hypothesis appears to be supported by the fact that, refer to Fig. 11, the passing of the tail did not produce abnormal phenomenon.

GROUND SETTLEMENTS OVER TUNNELS

In analyzing ground settlements due to tunnelling, Peck's approach is usually followed (Peck, 1969) and settlement troughs are assumed to have normal distributions such that:

CONSOLIDATION SETTLEMENTS

In soft ground, Phase 3 settlements may be far more significant than immediate settlements and they may drag on for months. Unless the period of monitoring is well defined, ground losses back calculated based on observed settlement troughs may be misleading. To have a consistent definition for Phase 3 settlements to be evaluated, it is recommended to use the slope of the settlement curve in the Phase 3 as an index. In other words, the index of consolidation settlement, α, is defined as the settlement in a cycle in the log-t plot. It is more preferable to plot the elapse time after the passing of the head of the shield, instead of the tail, because the Phase 1 settlements occurred during the passing of the shield are of interest as well. The difference will be insignificant because the tail usually passes within a few hours after the head passes.

Based on many observations, it is found that in clayey media or in sandy media with considerable silt/clay contents, Phase 3 settlements usually begin in the period between 3 to 10 days after the passing of the tail of shield. For all practical purposes, it will be reasonable to consider the settlements on the 10th day, denoted as δ_{10}, as immediate settlements and the corresponding settlement trough as ground loss, denoted as v_{10}. In such cases, as illustrated in Fig. 17, the index of consolidation settlement will simply be the difference in settlements between the 10th and the 100th day after the passing of the tail.

TRTS Experience

Surface settlements at the center of tunnels in the K1, T1 and T2 Zones are plotted in Figs. 19, 20 and 21, respectively, and a summary of the α values is available in Table 2. For CP262, the shield for driving the Up-track tunnel arrived in 30 days after the shield for the Down-track tunnel passed the instrumented section. It is intended to limit the scope of this paper to the performance of single tunnels only and ignore interaction effects between tunnels, therefore, the later part of the data was truncated.

As can be noted that, although the immediate settlements, ie. $\delta_{10,}$ vary in a very wide range, the indices of consolidation settlements for the T1, T2 and K1 Zones, ie. the α values, for different zones are fairly consistent and fall in a narrow range of 5mm to 14mm per log cycle. This presumably is due to the fact that the presence of sand lenses in clays and clay lenses in sands reduces the contrast among different zones.

SMRT Experience

The data obtained during the driving for the west-bound tunnel of Contract 301 of SMRT are plotted in Fig. 22 (Hulme, Shirlaw and Hwang, 1990; Shirlaw and Doran, 1988; Shirlaw and Copsey, 1987) and the α values are listed in Table 2. The shield for the East-bound track arrived in 20 days after completion of the West-bound drive and the subsequent data were truncated. The α values vary from 6 to 20. Unfortunately only 4 sets of data are available and α values obtained may not be representative.

Factors Affecting Consolidation Settlements

The TRTS and SMRT tunnels discussed herein are of similar sizes and all of them were driven in soft ground using earthpressure balancing shield machines. Although the data available for SMRT are too few, the fact that greater α values were obtained for SMRT than for TRTS does not come as a surprise. Singapore marine clay is two to three times more compressible than the silty clay in the Taipei Basin, larger consolidation settlements are thus expected.

Presumably the α values are predominantly affected by soil conditions. It is hypothesized that for tunnel drives in uniform media:

$$\alpha = f(D, C_V, p_g) \tag{3}$$

where D = tunnel diameter, C_v = coefficient of consolidation, p_g = pressure of back grouting for filling tail void. The effects of depth will be minimal, if any. For layered strata, the length of drainage path will certainly play an important role. Efforts are being made in evaluating the influences of each of the above-mentioned factors in a hope to have a better understanding of the mechanism of consolidation settlements so tunnelling technique can be improved.

CONCLUSIONS

The foregoing discussions lead to the following conclusions:

1. For shield tunnelling using earthpressure balancing machines, so long as the chamber pressures are near the at-rest earth pressures, the predominant source of excess pore pressures induced in the ground is the grouting for filling the tail void.

2. In clays, excess pore pressures accumulate till the tail passes. As soon as the tail passes and a void is created, the excess pore pressures drop immediately.

3. In sands with a high fine contents, water gets into tail void rapidly to equalize external water pressures preventing the wall from collapsing.

4. To have a common base for comparisons, it is recommended to consider the settlements occurred in 10 days, ie. δ_{10} , after the passing of shield as "immediate settlements" and use these settlements in computing "ground loss", v_{10}. Subsequent settlements are considered as "consolidation settlements" and the indices of consolidation settlements, α, defined herein can be used in comparative studies.

5. Unlike immediate settlements, long term consolidation settlements are less affected by factors such as workmanship, earthpressure in chamber, speed of advancement, timing of grouting, etc. and are thus, relatively speaking, fairly consistent and predictable. Indices of consolidation settlements for similar ground fall in a narrow range and representative values are available in Table 2.

With the foregoing discussions, it is suggested that the earthpressures in chamber be appropriately increased to deliberately heave up the ground before the passing of the shield in order to reduce the total settlements at the end. The worry of inducing high excess pore pressures, as a result, leading to large long-term consolidation settlements is unfounded.

ACKNOWLEDGMENTS

The authors are indebted to the Department of Rapid Transit Systems of the Taipei Municipal Government and the Mass Rapid Transit Corporation, Singapore for granting the permission to publish the data presented herein, to Continental Engineering Corporation, Tekken Construction Corporation, Pan Asia Corporation and Chizaki Kogyo Co., Ltd, for the co-operation and assistance provided during the monitoring, and to Obayashi Corporation and Fu Tsu Construction for providing valuable data.

REFERENCES

Adachi, K., Todo, H. and Mizuno, H. (1981) "Quality of samples of soft cohesive soil", Proc. 10th ICSMFE, Stockholm

Chin, C.T. , Crooks, J.H.A. and Moh, Z.C. (1994) "Geotechnical properties of the cohesive Sungshan deposits, Taipei", Proc. Geotechnical Engineering Journal, December, Bangkok, Thailand

Clough, W. and Schmidt, B. (1981) "Design and performance of excavations and tunnels in soft clay", Soft clay engineering, Elsevier, Amsterdam

Elias, V.H and Mizuno, A. (1987) "Tunnelling with earth pressure balance shields", Proc. 5th Intl. Geotechnical Seminar, Nanyang Technological Institute, Singapore

Hulme, T.W., Shirlaw, J.N. and Hwang, R.N. (1990) "Settlements during the underground construction of the Singapore MRT", pp521-526

Hulme, T.W., Potter, L.A.C., Shirlaw, J.N. (1989) "Singapore Mass Rapid Transit System : construction", Proc. Instn Civil Engineering, Part 1, 86, Aug., 709-770

Hwang, R.N., Fan, C.B. and Yang, G.R. (1995) "Consolidation settlements due to tunnelling", Proc. the South East Asian Symposium on Tunnelling and Underground Space Development, Jan. 18-19, Bangkok, Thailand, pp79-86

Hwang, R.N., Wu, D.J. and Lee, C.J. (1995) "Pore pressure response to shield tunnelling in soft clay", Proc. the South East Asian Symposium on Tunnelling and Underground Space Development, Jan. 18-19, Bangkok, Thailand, pp33-40

Hwang, R.N., Ju, D.H., Tsai, M.S. and Fang, Y.S. (1995) "Soft ground tunnelling in Taiwan", Proc. U.S./Taiwan Geotechnical Engineering Collaboration Workshop, Taipei, Taiwan, January 9-11

Kobayashi, Y., Todo, H., Weerasinghe, W.A.Y., and Chandra, P. (1990) "Comparison of coastal clays found in Singapore, Malaysia and Indonesia", 10th Southeast Asian Geotechnical Conference, 16-20 April, Taipei

Moh, Z.C. and Ou, C.D. (1979) "Engineering characteristics of the Taipei silt", Proc. 6th Asian Regional Conference on SMFE, v1, pp 155-158, Singapore

Moh, Z.C. and Chin, C.T. (1991) "Geotechnical considerations for underground mass rapid transit systems", Proc. 9th Asian Regional Conference on SMFE, Thailand

Peck, R.B. (1969) "Deep excavations and tunnelling in soft ground", Proc. 7th ICSMFE, State-of-the art volume, Mexico City, Mexico, pp225-290

Poh, K.B., Buttling, S. and Hwang, R. N. (1987) "Some MRT Experiences of the Soils and Geology of Singapore", Proc. Singapore Mass Rapid Transit Conf., Singapore

Shirlaw, J.N. and Copsey, J.P. (1987) "Ground movement and settlements caused by tunnelling for the Singapore mass rapid transit system", Proc. 5th Intl. Geotechnical Seminar, Nanyang Technological Institute, Singapore, pp59-72

Shirlaw, J.N. and Doran, S. (1988) "Ground movement and settlements caused by tunnelling for the Singapore mass rapid transit system", Proc. Tunnelling, 1988, Instn of Mining and Metallurgy, London. pp295-314

Todo, H., Sagae, T. and Adachi, K. (1993) "Compressibility model for highly sensitive Singapore marine clay", 11th Southeast Asian Geotechnical Conference, 4-8 May, Singapore

Woo, S.M. and Moh, Z.C. (1990) "Geotechnical Characteristics of Soils in the Taipei Basin", Proc., 10th South East Asian Geotechnical Conference, Taipei, Taiwan

TABLE 1 Soil Properties at Tunnel Levels

Soil Type / Properties	Singapore Lower Marine Clay	Taipei Silt in T2 Zone		
		Sublayer 5	Sublayer 4	Sublayer 3
	CH	SM/ML	CL/ML	SM/ML
Water Content %	50 - 90	26	32	24
Unit Weight kN/m^3	16 - 18	18 - 20	18 - 19	19 - 20
Liquid Limit %	60 - 95	-	34	-
Plastic Limit %	25 - 35	-	22	-
Plasticity Index	70 - 75	-	12	-
Liquidity Index	0.5 - 1.0	-	0.8 - 1.0	-
Sensitivity	5 - 12	-	3.5 - 4	-
OCR	1 - 1.5	-	1.6	-
Particle Sizes				
Sand %	0 - 5	75	8	60
Silt %	30 - 50	19	60	34
Clay %	50 - 70	5	30	6
Gravel %	0	1	0	0
Su kN/m^2	50	20 - 40	50	50 - 70
C_c	1.2	-	0.3 - 0.4	-
Coef. of Secondary Consolidation	0.02 - 0.05	-	0.02	-
C_v $m^2/year$	1 - 5	-	20	-
References	(1) Todo et al, 1993 (2) Kobayashi et al, 1990	(1) Woo and Moh, 1990 (2) Chin, Crooks and Moh, 1994		

TABLE 2 Indices of Consolidation Settlements

Area	Zone	Contract	Instrument	Ring No.	Depth (m)	α value (mm)
Taipei Basin	K1	CN256	SM143	DN 56	12.2	12
	K1	CN256	SM156	DN 143	12.7	5
	K1	CN256	SM175	DN 291	12.9	14
	K1	CN256	SM14	UP 364	18.9	7
	K1	CN256	SM27	UP 279	17.7	6
	K1	CN256	SM39	UP 199	16.5	6
	T1	CP262	SM35	DN 23	13.6	8
	T1	CP262	SM52	DN 43	13.6	10
	T1	CP262	SM59	DN 63	13.8	12
	T1	CP262	SM66	DN 83	14.2	6
	T2	CH218	SM34	DN 169	17.3	10
	T2	CH218	SM35	DN 200	18.2	5
	T2	CH218	SM36	DN 223	18.9	7
	T2	CH218	SM37	DN 256	19.8	4
	T2	CH218	SM39	DN 301	21.2	7
Singapore		C301	CH32600		18.9	6
		C301	TA151		18.9	20
		C301	TA177A		18.9	20
		C301	TA158		18.9	14

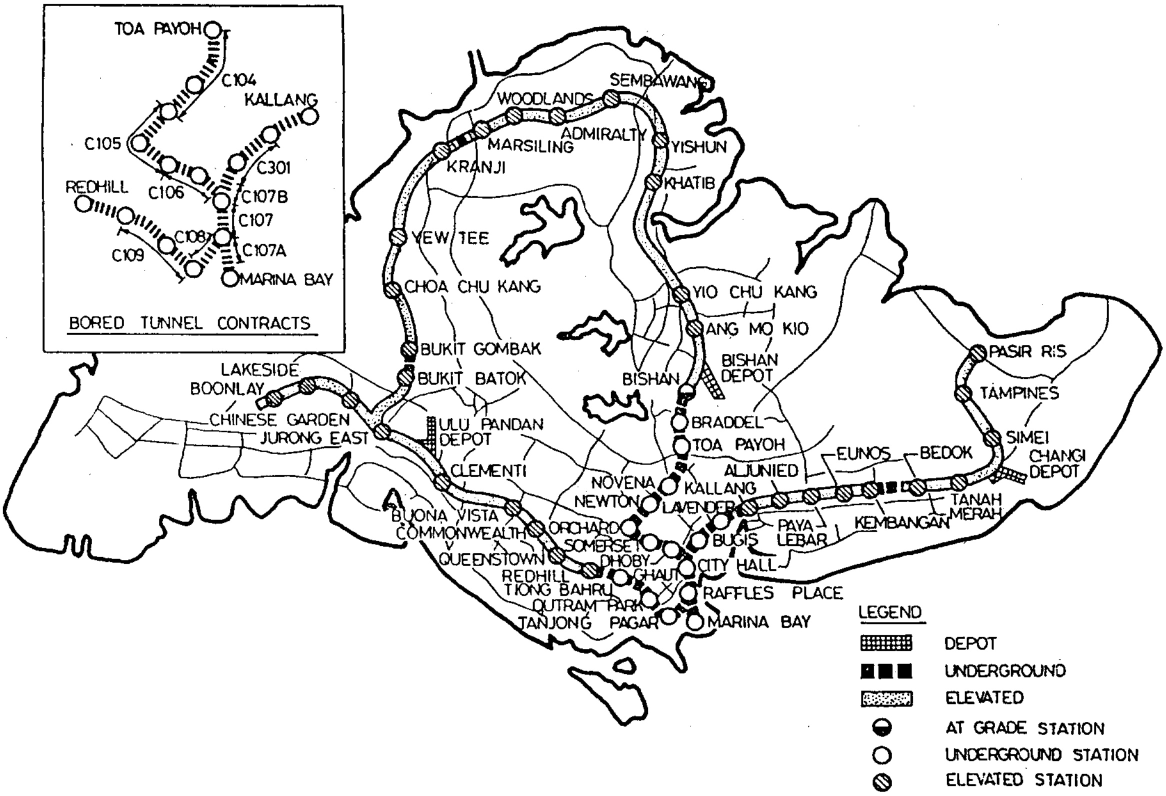


Fig. 1 Singapore Mass Rapid Transit System

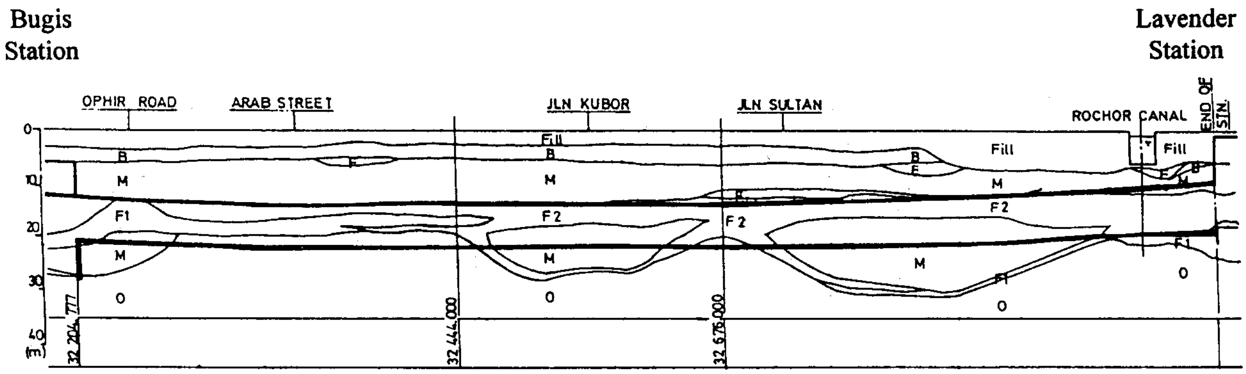


Fig. 2 Geological Section for C301, SMRT
(after Elias & Mizuno, 1987)

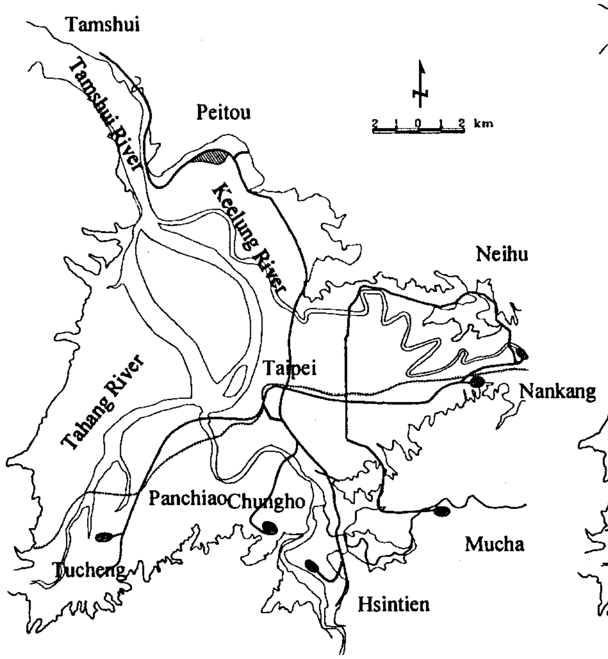


Fig. 3 Priority Network of Taipei Transit Systems

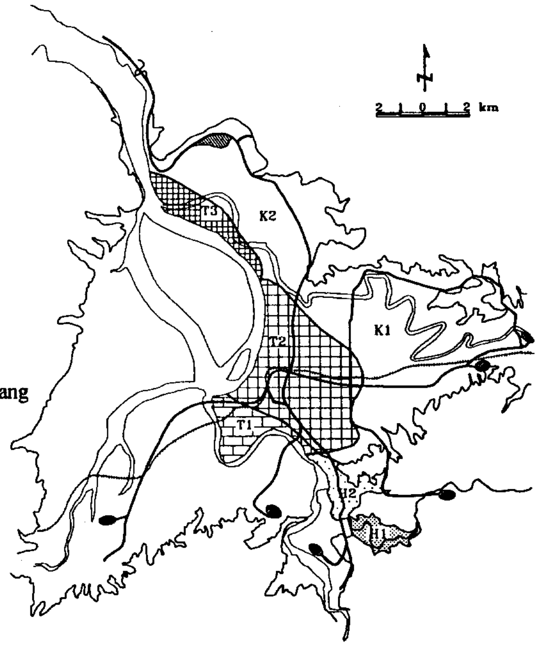


Fig. 4 Subzones in the Sungshan Formation

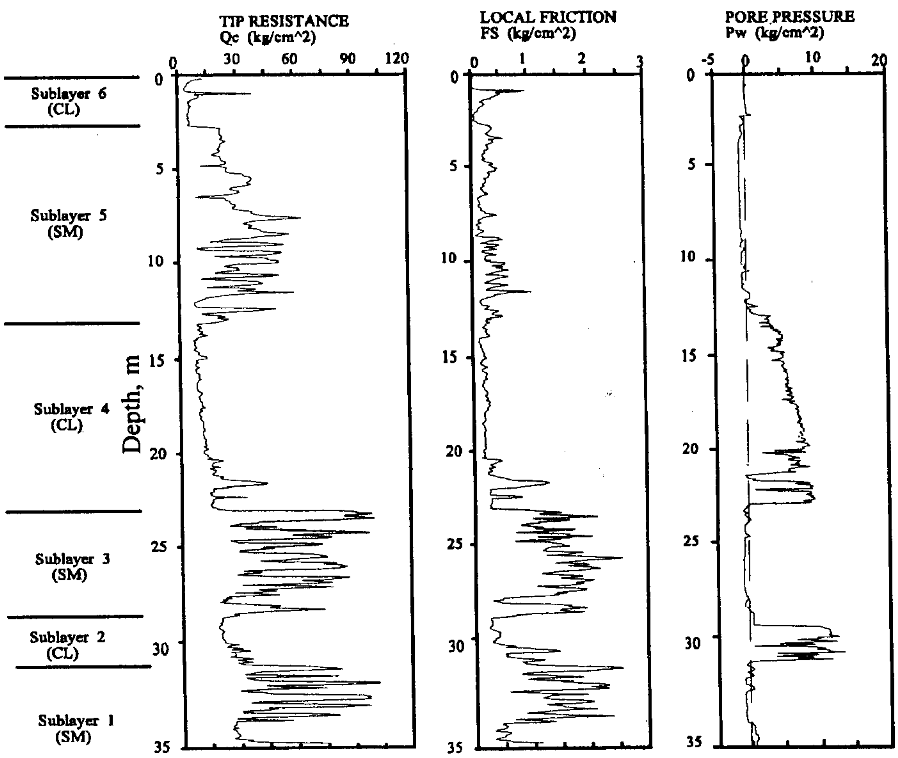


Fig. 5 Typical Soil Profile in the Central Taipei City Area (T2 Zone)

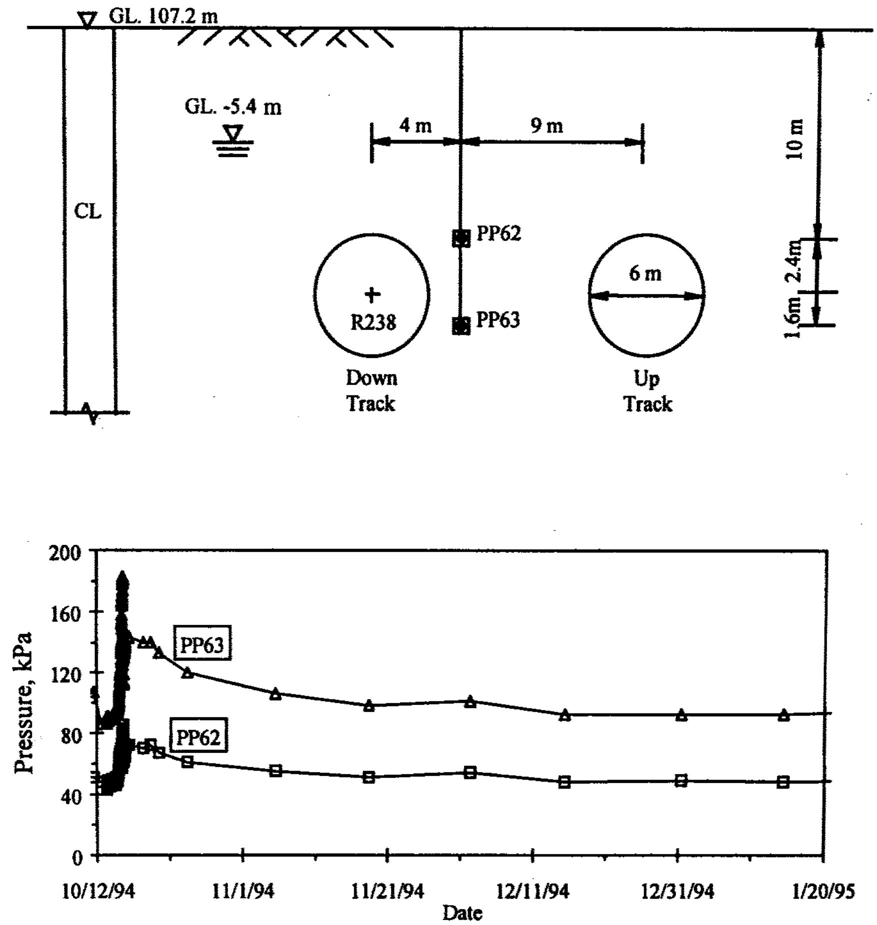


Fig. 6 Instrument Layout and Pore Pressure Readings during Driving of Down Track Tunnel, Section 257T1

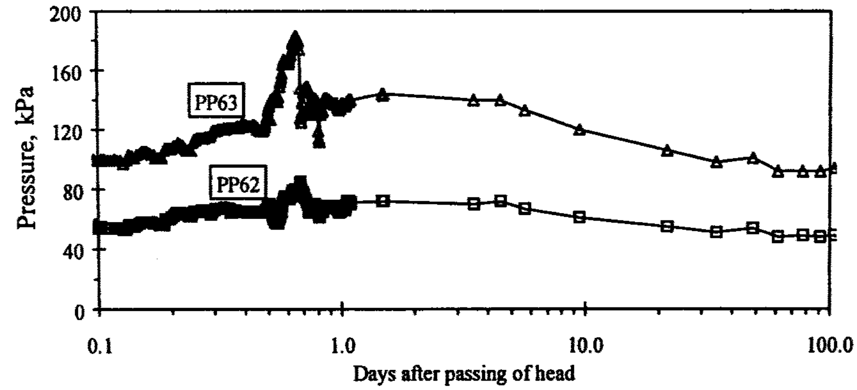


Fig. 7 Pore Pressure Response, Section 257T1

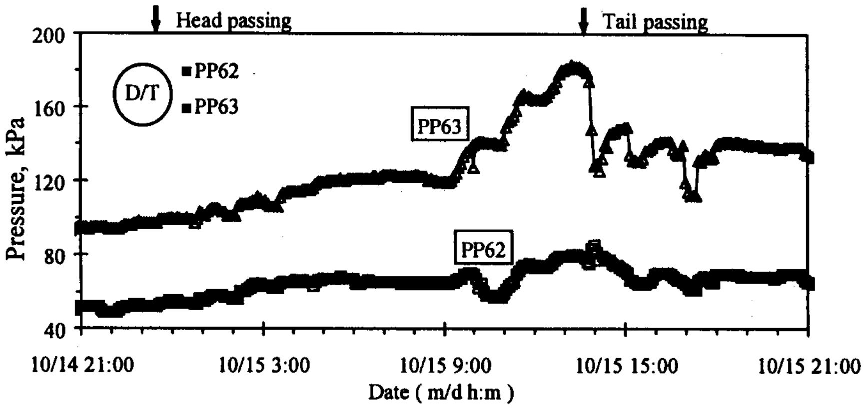


Fig. 8 Pore Pressure Response during the Passing of Shield, Section 257 T1

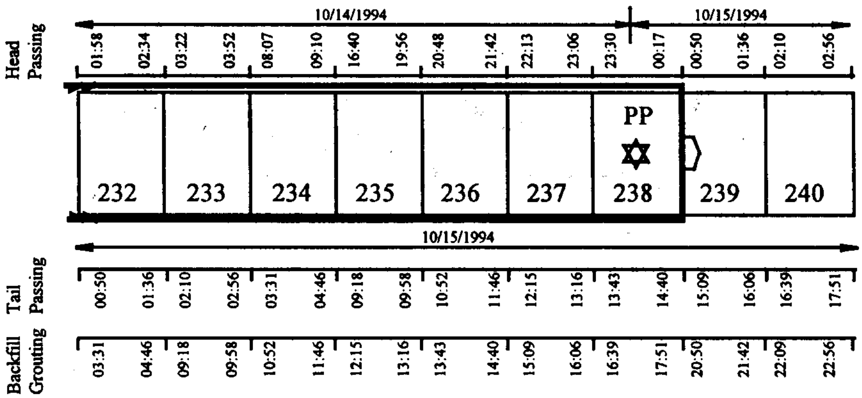


Fig. 9 Progress of Tunnelling, CN257

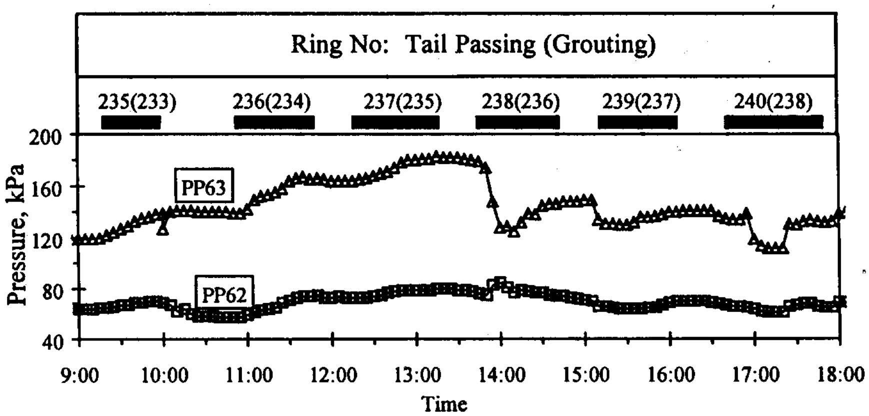


Fig. 10 Effects of Grouting on Pore Pressures, Section 257 T1

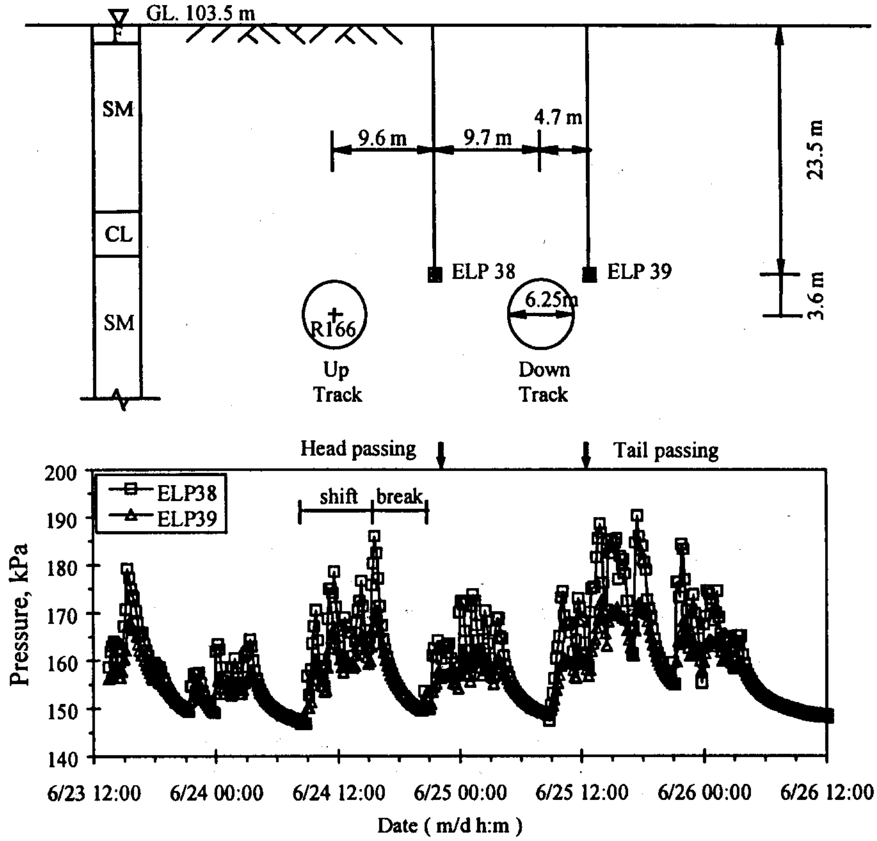


Fig. 11 Profile and Pore Pressures Readings during Driving of Up Track Tunnel, Section 261 T2

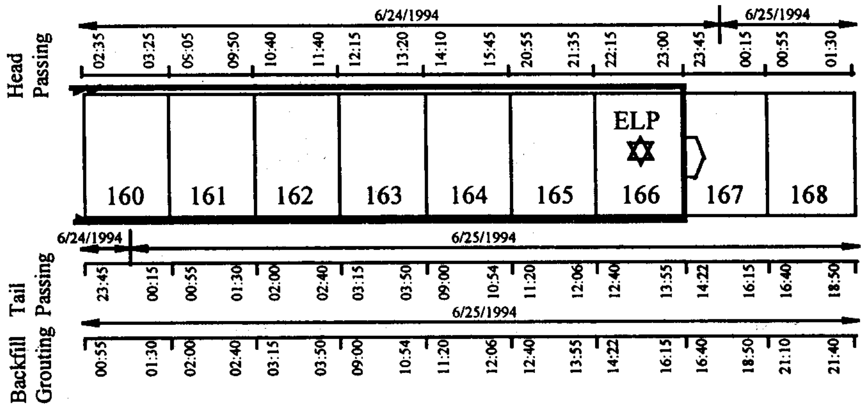
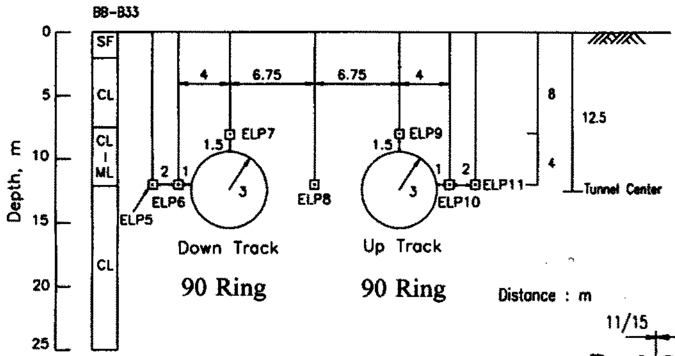


Fig. 12 Progress of Tunnelling, CP261



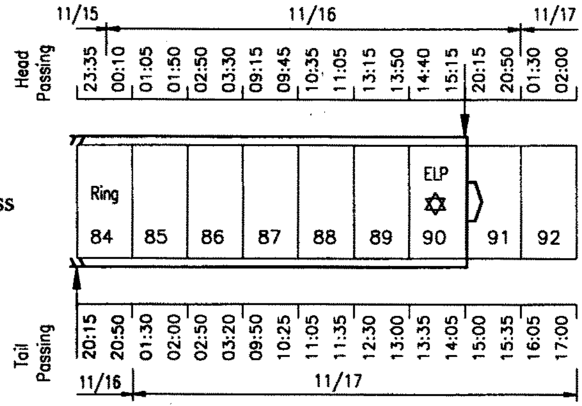


Fig. 13 Soil Profile Instrument Layout, Progress of Tunnelling, Section 256 T3 (after Hwang, Wu & Lee, 1995)

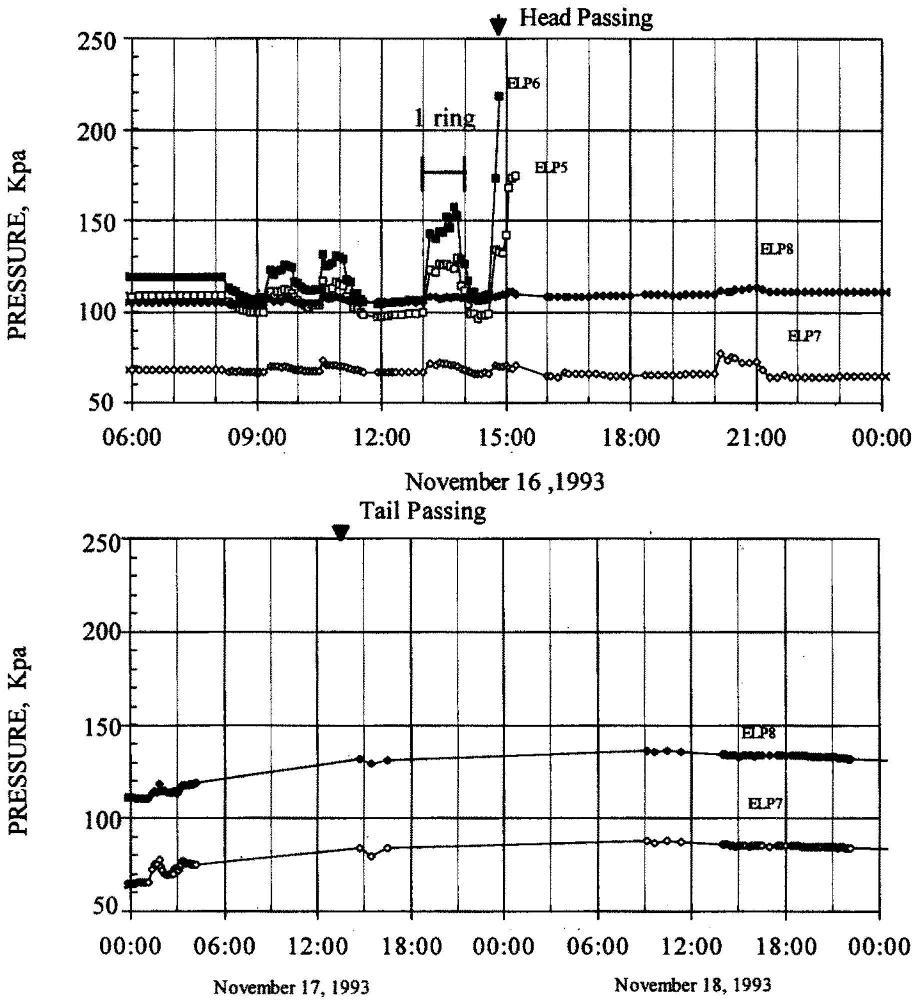


Fig. 14 Pore Pressures during Driving of Down Track Tunnel of CN256

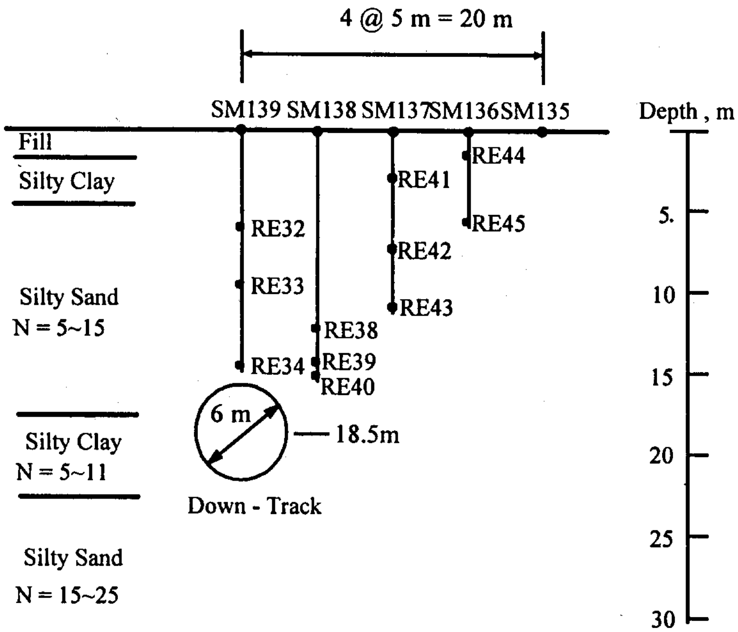


Fig. 15 Soil Profile at the Site and Instrument Layout, Section 218 B1

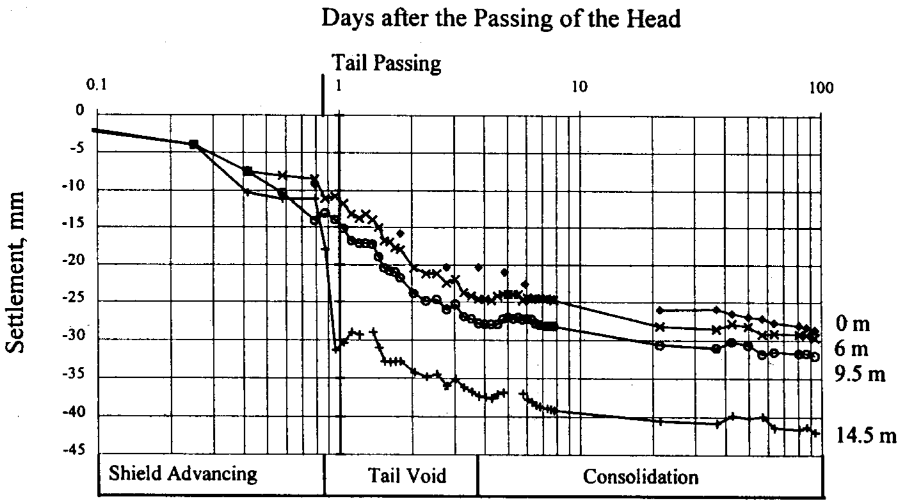


Fig. 16 Settlements at Center, Section 218 B1

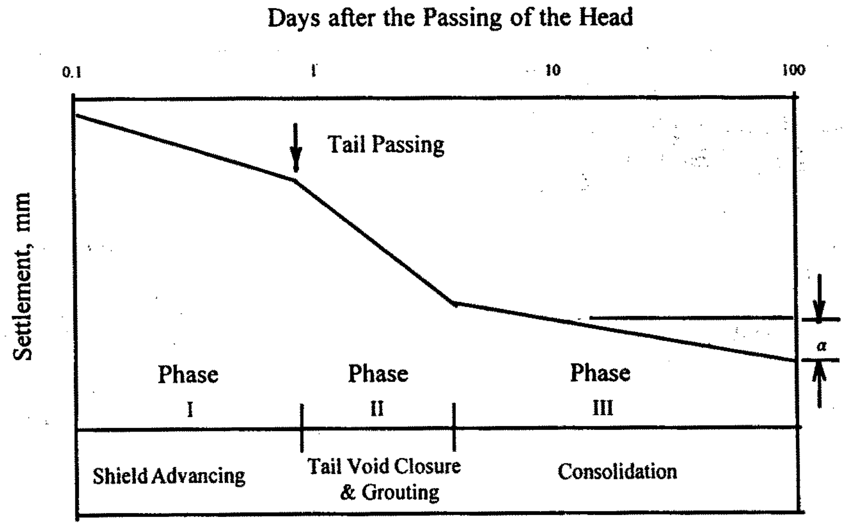


Fig. 17 Idealized Ground Settlement Curve

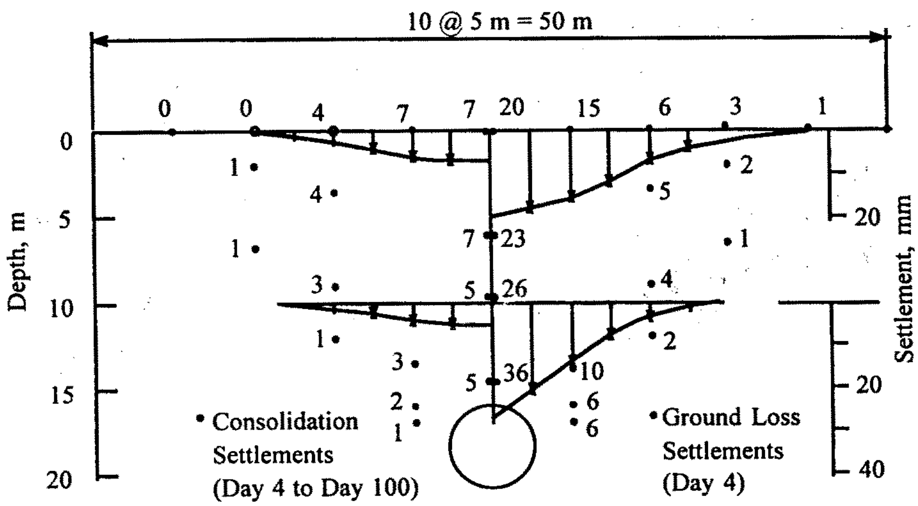


Fig. 18 Settlements and Settlement Troughs

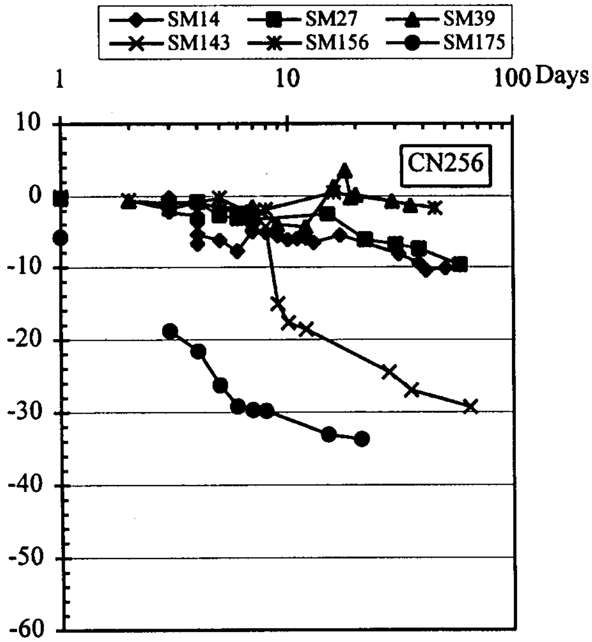


Fig. 19 Settlements in K1 Zone

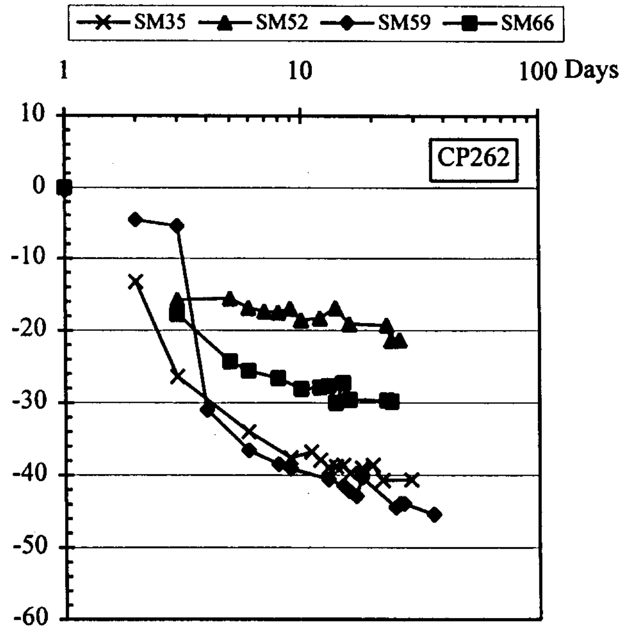


Fig. 20 Settlements in T1 Zone

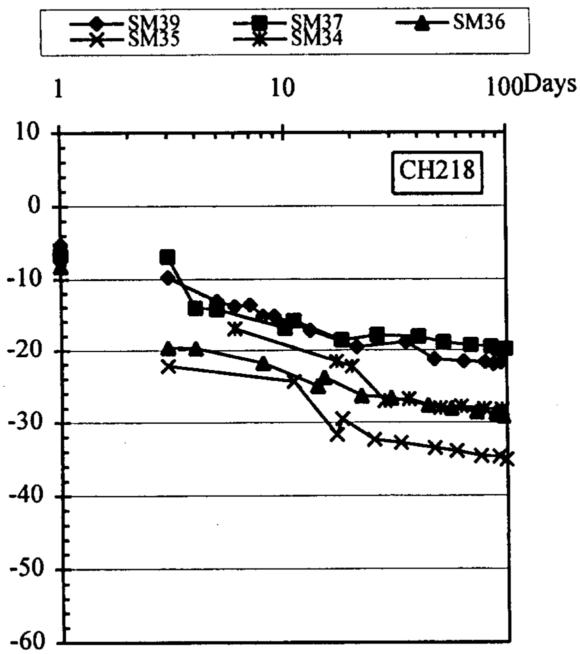


Fig. 21 Settlements in T2 Zone

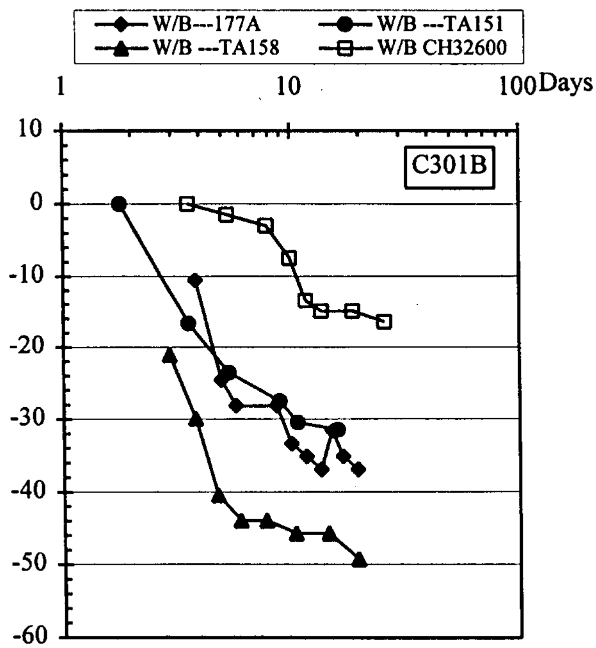


Fig. 22 Settlements for SMRT