GEOTECHNICAL OBSERVATIONS OF CONSTRUCTION OF A LARGE SHALLOW SCL TUNNEL IN SOFT GROUND

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ABSTRACT: The Fort Canning Tunnel is the first road tunnel in Singapore to be built using the sprayed concrete lining (SCL) method. The major technical challenge of this was to construct a 15m wide tunnel by mining in soft ground under a shallow overburden of 3m to 9m. This paper describes the geotechnical investigations and monitoring controls for the safe and progressive execution of the works, such as soil investigations, trial forepoling works, surface settlement monitoring, tunnel settlement monitoring, face movement monitoring, and the observational approach to construction. The monitored field data showed the volume loss to range from 0.4% to 2.1%, and the observed surface settlement trough was found to agree well with the theoretical Gaussian trough. Other observations made include substantial surface settlements induced by the stress relief at and ahead of the tunnel face in spite of the forepoling umbrella, and the higher volume losses associated with higher overburden. Tunnel face movements were observed during installation of forepoling. These observations are of interest to engineers planning future SCL tunnels in similar conditions.

Keywords: Soft ground, construction, settlement

INTRODUCTION

The Fort Canning Tunnel is the first road tunnel in Singapore built using the Sprayed Concrete Lining (SCL) method. It is a 350m long tunnel developed by the Land Transport Authority (LTA) as part of a 500m new road in the city centre of Singapore. To protect the tranquility of the historical Fort Canning Park, 180m of the road tunnel underneath the park was constructed using the mining and sprayed concrete lining (SCL) method. This method has enabled tunnel construction to progress without any impact on the natural serenity of the park environment above and minimized the felling of trees along the tunnel route. The tunnel was constructed by Sato Kogyo (S) Pte Ltd, and IL-Laabmayr and Partner Consulting Engineers was engaged by LTA to supervise the SCL tunnel construction. The SCL tunnel is about 15m wide and 11m high to meet the operational requirements of a 3-lane vehicular traffic width and headroom. With a soil overburden ranging from 3m to 9m above the tunnel crown, this is one of the largest soft ground SCL tunnel under shallow cover in the world to date (Shani, 2006). The tunnel cross-section is shown in Fig 1.

Hor et al. (2006) provide a detailed discussion of the method of constructing the SCL tunnel. Figure 2 is a schematic of the excavation and support system. The top heading excavation was preceded by a pre-support forepoling umbrella, and then followed by bench and invert excavation.

The tunnel support system was provided by G40 shotcrete with lattice girders and wire mesh reinforcement for good constructability. A pre-excavation support system was provided by means of 40 grouted steel pipes forming a forepoling arch above the tunnel crown. Each steel pipe with a diameter of 114mm and a thickness of 6mm was formed by joining together four pieces (each 3m-3.5m long) to form a 12.5m long section. These pipes were installed at an incline before a resin grout (urethane) was injected to fill the pipes. The pipes were perforated and so the pressurised injection also allowed the resin grout to fill any loose soil pockets in the vicinity of the pipes. This created a protective umbrella above the tunnel crown during excavation to provide overhead support for workers.

Each round of top heading excavation was 1m in length, and was supported by a dumpling to enhance the face stability during each advance. The shotcrete lining was 300mm thick and applied with each advance. To increase the bearing of the top heading and reduce tunnel settlements, an enlarged 600mm wide footing ("elephant's foot") was included at the sides of the tunnel, and a rapid ring closure of the top heading was ensured using a 150mm thick temporary invert. Figure 3(a) shows the face dumpling during top heading excavation, and Figure 3(b) shows the temporary invert and enlarged footing exposed during bench / invert excavation.

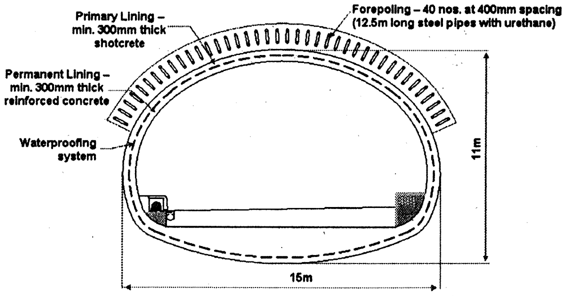

Fig. 1 SCL Tunnel Cross-section

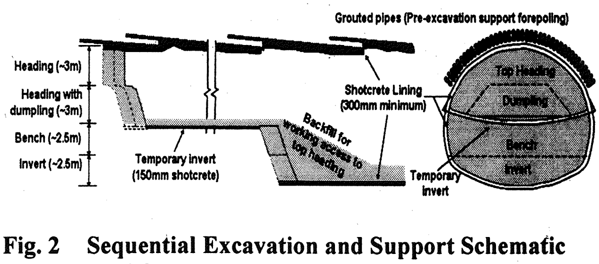

Fig. 2 Sequential Excavation and Support Schematic for SCL tunnel construction

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Bench and invert sections of the tunnel were excavated together in 2m advance lengths, and full ring closure was provided using a 300-350mm thick shotcrete lining.

To increase production rate and to provide adequate working space for the forepoling and shotcreting operations in the top heading, a targeted bench length in the range 18m-27m was proposed originally,but this was increased during construction based on monitoring results; this is discussed later in the paper.

GEOTECHNICAL INVESTIGATIONS

As it was the first time in Singapore where such a large tunnel was excavated using the SCL method under a shallow overburden in soft ground, an extensive soil investigation and a trial of the fore-poling works on site were carried out prior to commencement of the tunnelling works.

The soil investigation programme was carried out with boreholes done on both sides of the tunnel alignment staggered at 10m c/c spacing.

From the borelogs and interpreted soil profile shown in Fig 4, the tunnel was expected to be excavated in the Fort Canning Boulder Bed (FCBB) geological formation. As reported by Shirlaw et al. (2003), the FCBB is a colluvial deposit of

Pleistocene age and typically consists of boulders in a hard sandy clayey silt or sandy silty clay matrix. The soil matrix contains high clay and silt content and has low permeability values of 10-8m/s or less with high undrained shear strength (at least 150 kPa and often considerably higher). The boulders are quartzite with unconfined compressive strengths in the range 100-200 MPa. Whilst the soil matrix provided the tight and competent ground that is suitable for mining excavation, the boulders presented difficulties to the miners during the tunnelling works. As Hor et al. (2006) explain, these boulders varied in size (up to a maximum of 3m x 3m x 4m), and the larger boulders had to be perforated before breaking them carefully in order to minimize the disturbance to the adjacent unsupported tunnel face. Figure 5 shows the distribution of SPT-N values of the FCBB soils.

Fig. 3 Top heading (a) face dumpling, (b) temporary invert and enlarged footing

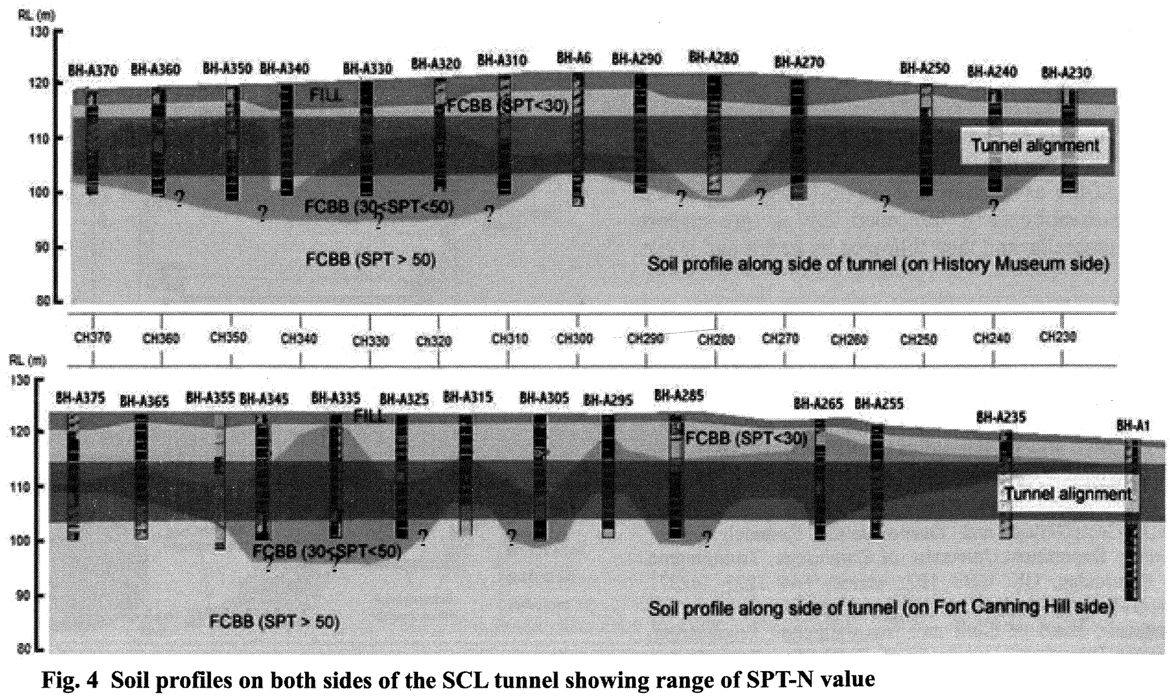


Fig. 4 Soil profiles on both sides of the SCL tunnel showing range of SPT-N value

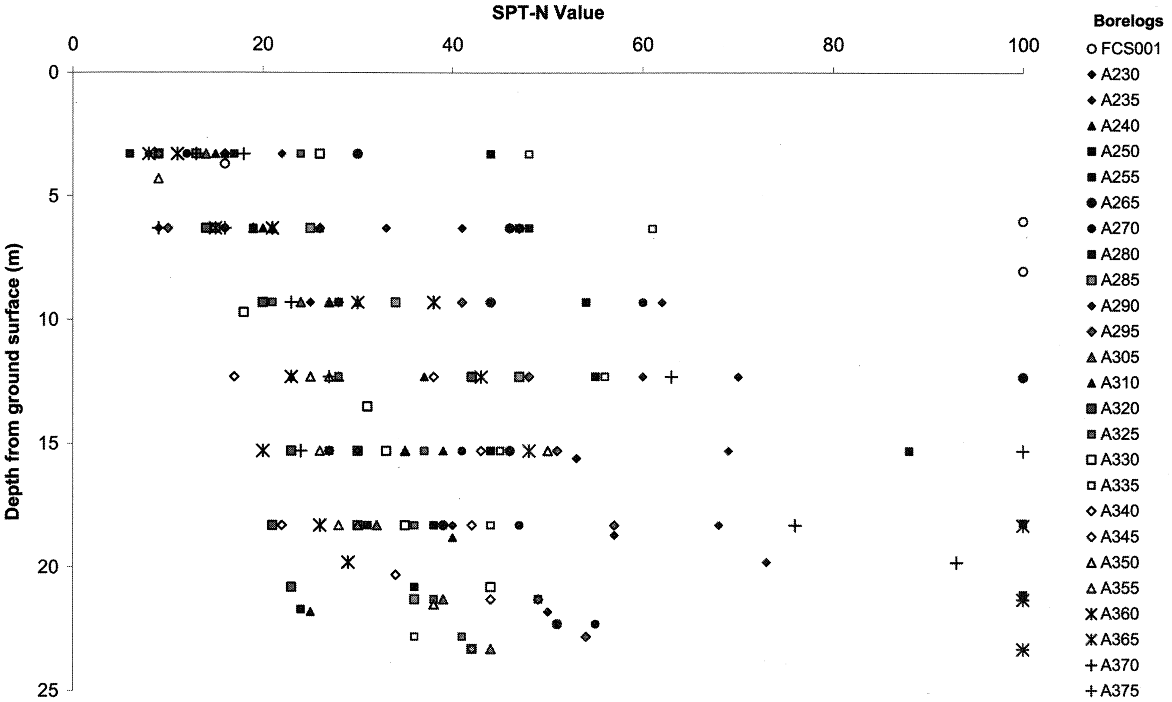
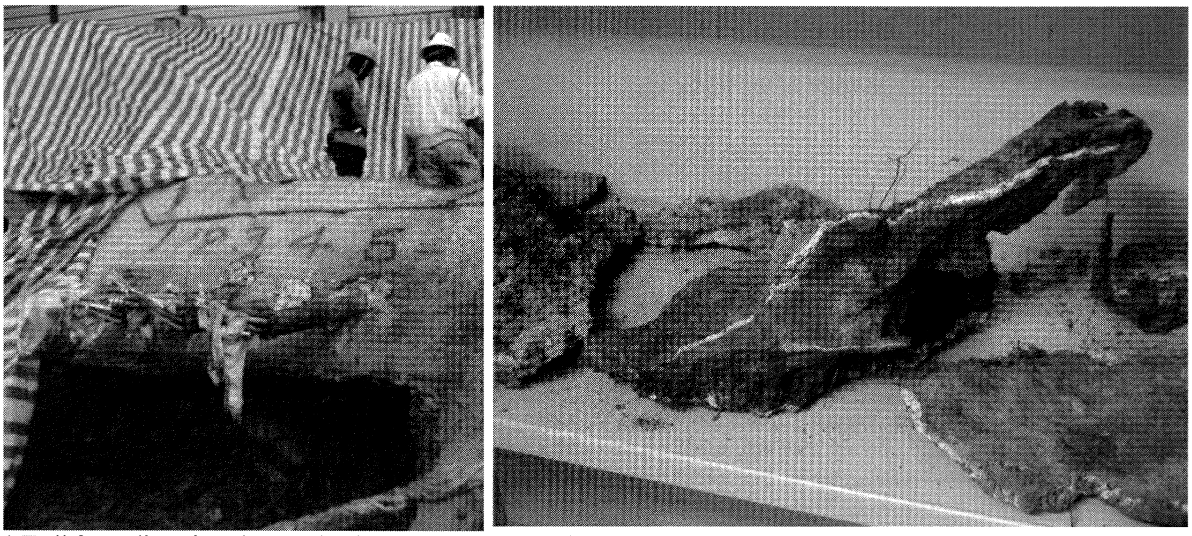


Fig. 5 Distribution of SPT-N values for FCBB soils along Fort Canning Tunnel



(a) Trail forepoling pipes (excavation beneath and above after trial (b) Thin sheets of grout (several mm in thickness)

Fig. 6 Forepoling trial showing thin planar bodies of grout associated with hydrofracturing

A pre-tunnelling forepoling trial on the site was initiated to assess the performance of the forepoling installation and to study the behaviour of the in-situ soil during grout injection through the pipes into the ground. In this trial, five forepoling pipes were installed in the ground, and injected with the resin grout (urethane) at the proposed injection pressures (up to maximum of 3bars). Ground was then excavated to expose the trial pipes both from the ground surface and from under the pipes.

It was found that the resin grout was unable to penetrate the low permeability soils other than by hydro-fractures caused by the high grout pressures. Figure 6 shows the forepoling trial and the thin sheets of resin grout formed around the pipes. As noted by Nakagawa et al. (2003), this behaviour is characteristic of low permeability soils where the urethane grout forms thin planar bodies through hydrofracture. Other than verifying the grout behaviour in the actual ground, this trial also provided evidence of the competence of the FCBB soils and hence greater confidence in the ground stand-up time.

INSTRUMENTATION MONITORING AND OBSERVATIONAL APPROACH

CIRIA Report 185 defined the observational method in ground engineering as "a continuous, managed, integrated, process of design, construction control, monitoring and review that enables previously defined modifications to be incorporated during or after construction as appropriate" and that "all these aspects have to be demonstrably robust". Due to variability of the ground conditions as well as variations in construction sequence, the observational method is an important aspect in any tunnelling job. This section discusses the key roles of instrumentation monitoring and the observational approach adopted during construction of the Fort Canning Tunnel.

Instrumentation and Monitoring Control Levels

During tunnel construction, a comprehensive instrument monitoring plan was implemented. This included surface settlement marker arrays every 10m, in-tunnel deformation prisms every 9m, and face movement monitoring during each stoppage of the top heading excavation. Figure 7 shows the instrumentation layout plan whilst Fig. 8 shows the arrangement of the surface settlement array, the in-tunnel deformation array, and the face monitoring points.

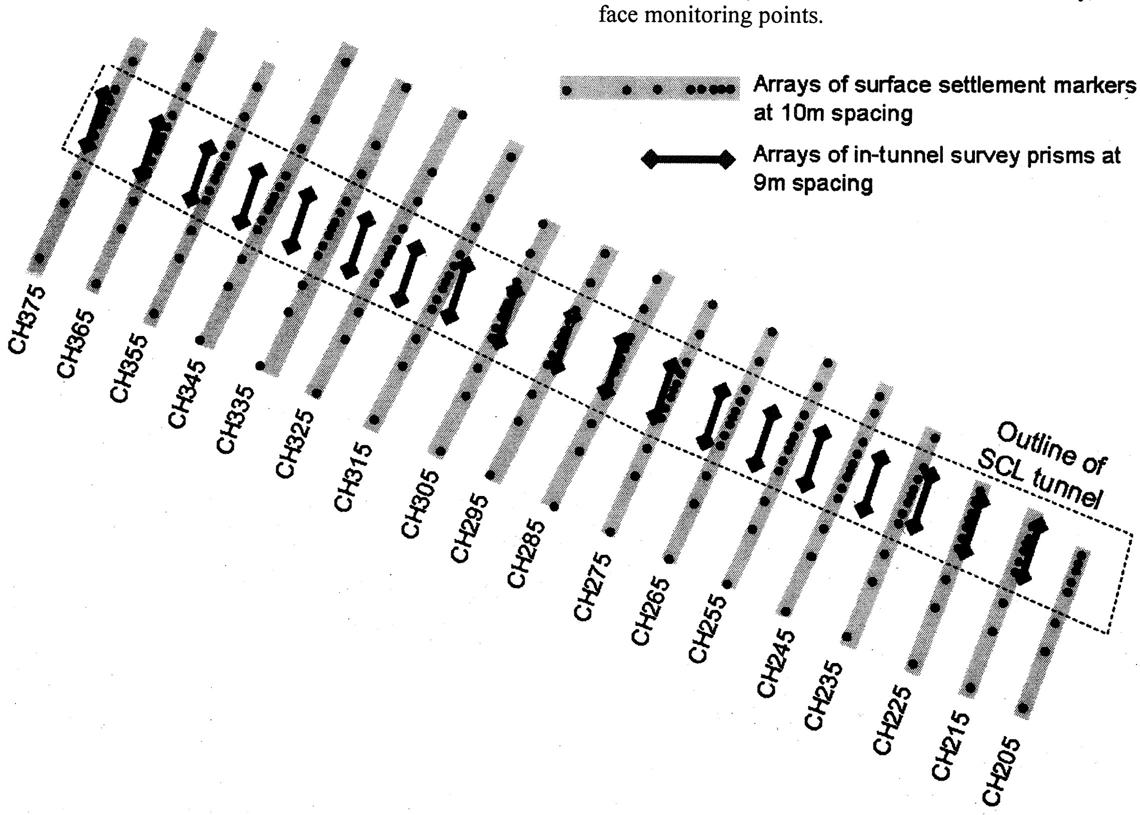


Fig. 7 Schematic layout of surface settlement arrays and in-tunnel survey prisms

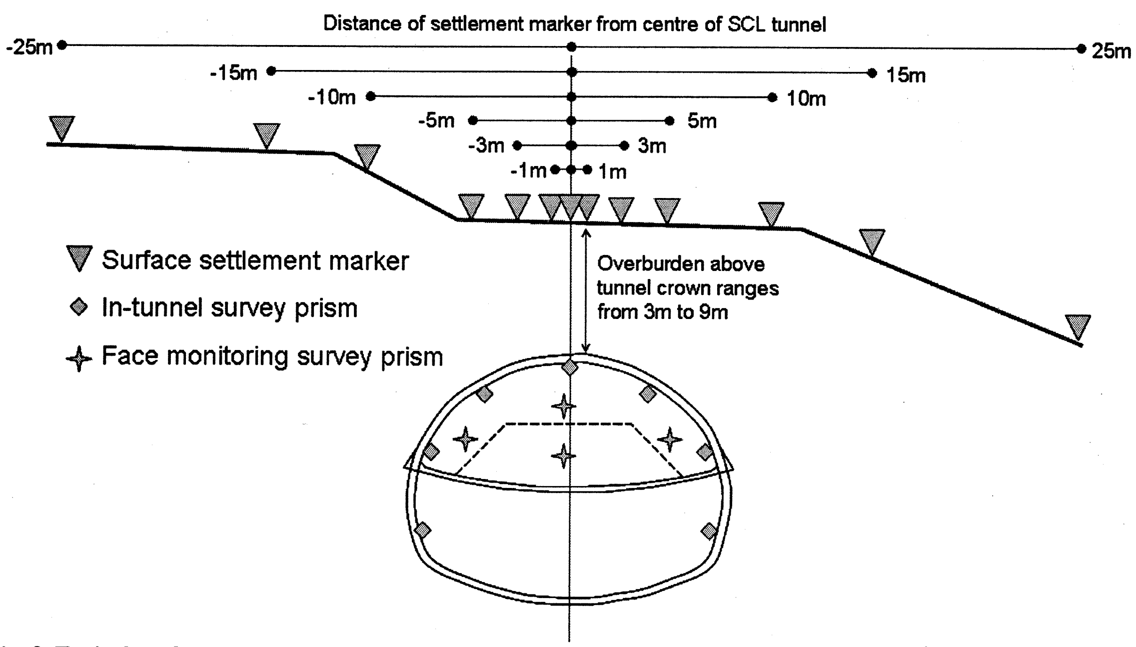


Fig. 8 Typical surface settlement array and arrangement of in-tunnel deformation monitoring

The monitoring control levels were established from the design of the tunnelling works. The design of the SCL tunnel was done using the finite element program Phase 2 V5.04 and taking into account the varying loading conditions, to determine the design forces and moments in the tunnel lining (Zeidler and Schwind, 2007); this was a 2D analysis. As there was no empirical experience to draw upon for such a large and shallow tunnel in similar ground conditions, a 3D analysis was also undertaken for both the shallowest section (3m overburden) as well as the deepest section (9m overburden) by Prof Lee Fook Hou from the National University of Singapore using ABAQUS (Yeo et al, 2009). Due to the inherent empiricism of the 2D tunnel analysis, the instrumentation control levels were established primarily from the results of the 3D analysis. These are summarised in Table 1.

The monitoring frequency varied from as intense as twice daily readings to as infrequent as monthly readings depending on the degree of criticality, such as whether the instrument was near to the excavation face or whether the instrument readings had stabilized. Through daily instrumentation meetings, the trends were assessed and interpreted carefully before proceeding with the next phase of the works.

Observational Approach During Construction

The ICE design and practice guide (1996) emphasized the difference in observational approach between SCL construction in soft ground and in rocks. In the case of SCL construction in soft ground and in urban areas, ground movements should be kept at a minimum and there is little or no scope for allowing delayed support. Instrumentation and monitoring therefore should be used for design validation rather than for reducing the primary support system. Nevertheless, the excavation sequence and the exposed face areas may be varied to a limited extent in response to the results of the monitoring.

In the spirit of the above guidelines, the project team adopted an observational approach to ensure a safe but smooth execution of the works. Daily review meetings were carried out on site to discuss the work activities, the instrumentation readings and trends, visual inspections of the shotcrete lining, as well as other health, safety and quality issues related to tunnelling. Construction sequences and advance rates were varied depending on the instrumentation readings, and contingency measures were developed ready for implementation if instrumentation indicated alert and control levels being reached.

Whilst the primary support system and sequence of construction remained unchanged, there were adjustments to the bench length (i.e. distance between top heading face and completed bench/invert lining) as well as to the length of each temporary invert closure (i.e. distance between top heading excavation face and the temporary ring closure). Figure 9 shows the SCL excavation alternating between top heading and bench/invert construction and progressing from Step 1 to Step 8 in the longitudinal direction.

The variations in the bench lengths, and lengths to temporary invert closures, are also indicated. To illustrate the implementation of the observational approach, the monitored tunnel crown settlements were less than 20mm when the top heading excavation had reached CH235. This was considerably lower than the control level of 85mm (see Table 1). To increase the production rate, the project team therefore increased the length to closure of the temporary invert from 2m to 3m. At CH244, it was also decided to proceed with the top heading excavation beyond the maximum design bench length of 27m. As the tunnel settlement started to approach the control level, the length to closure of the temporary invert was reduced from 3m back to the original 2m at CH310. Concurrently, as the surface settlement readings were approaching the control level of 120mm, the bench length was also reduced to less than the original design length of 27m. By reducing the length to closure of the temporary invert and also by reducing the bench length to achieve the more stable final tunnel geometry earlier, the rate of ground and tunnel settlement started to reduce.

The challenge in adopting the observational approach is to find the balance between mitigating impact on the surroundings against maximizing production rate, whilst maintaining safety at all times. When the monitored readings approached the alert and control levels, a key emphasis was on the visual inspection of the tunnel, especially at the shoulder and footing areas corresponding to the maximum shear force and bending moments predicted in the design. However, no sign of distress in the sprayed concrete lining was observed. This gave the confidence to proceed even when some of the monitoring values were approaching the control levels, albeit at reducing rates. Subsequently as settlement readings were stabilizing towards the end of the top heading excavation, the production rate was increased by increasing the length to temporary invert ring closure to 3m as well as allowing a maximum bench length of 30m

Table 1 Design and instrumentation control levels

	Instrumentation Control Levels	
	During top heading excavation	Incremental during bench / invert excavation
Surface settlement	120mm	30mm
Tunnel settlement – Crown	85mm	15mm
Tunnel settlement – Footing	60mm	20mm
Tunnel settlement – Bench/Invert	n.a	20mm
Note: Alert levels were set at 70% of the control levels to trigger a review of the construction activities if these levels were reached.		

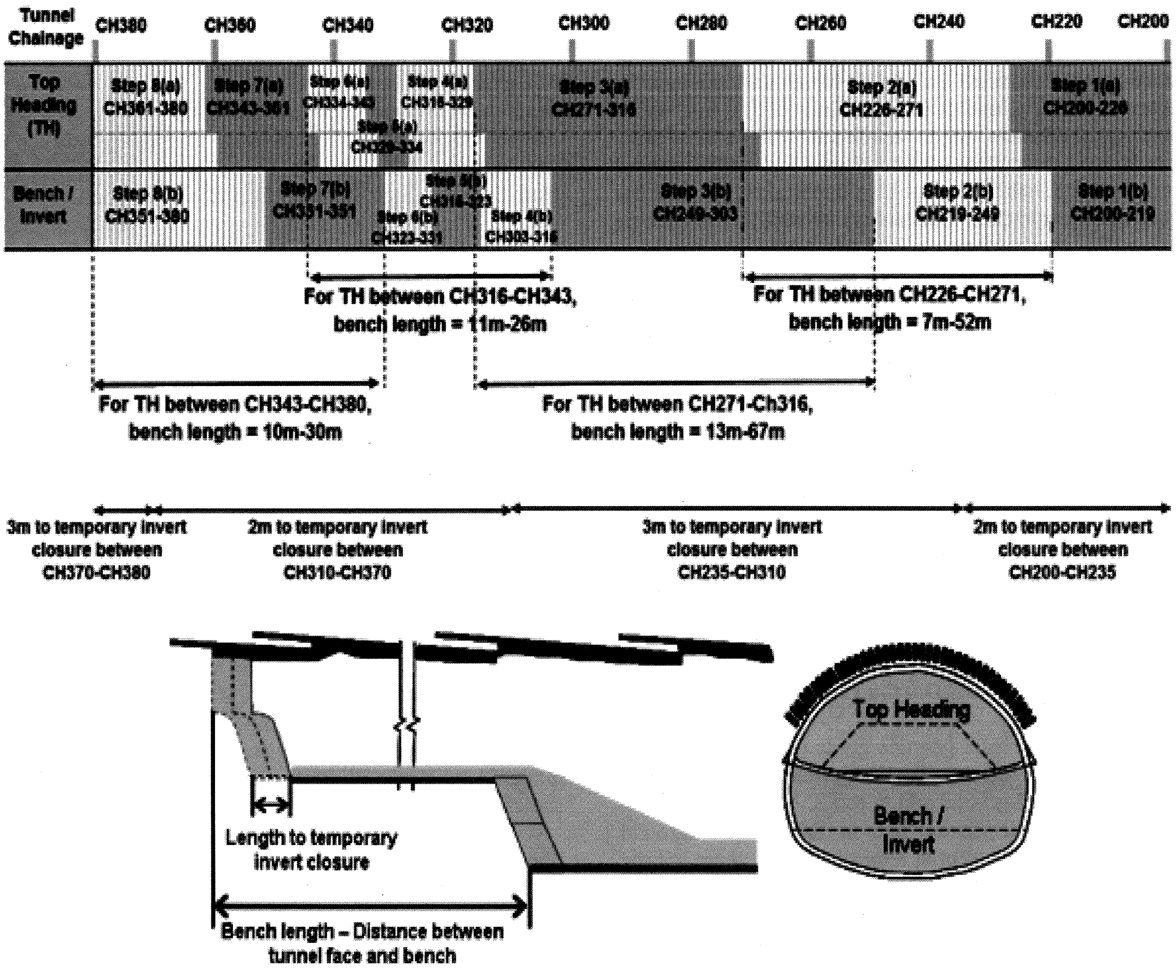


Fig. 9 Progress of SCL excavation from CH200-CH380

GROUND SURFACE SETTLEMENT

Settlement Response of the Ground

The intensive arrays of surface settlement markers were particularly useful in monitoring the performance of the SCL activities taking place beneath the Fort Canning Hill. A typical surface settlement plot against time is shown in Fig. 10, from which the major causes of ground movements can be interpreted in terms of pre-excavation relief, top heading excavation, and bench/invert excavation.

As seen in Fig. 10, the ground was observed to be settling even before the top heading excavation arrived below the settlement markers. For this location at CH295, the pre-excavation settlements started at about 10m ahead of the tunnel face. The casting of the temporary invert followed 2-3m behind the face of the heading excavation. Subsequent top heading excavation caused more surface settlements but at a decreasing rate. When the bench/invert excavation approached the settlement array (typically from around 20m-30m away the array), there was an increase in the rate of ground settlement again. However, the settlement stabilised soon after the bench/invert was cast below the settlement array to achieve the final ring closure of the tunnel.

Figure 11 summarises the maximum surface settlement monitored at various chainages above the tunnel, and at different stages of tunnel construction.

The soil overburden is defined as the distance from the ground surface to the tunnel crown. A higher soil overburden increases the final surface settlement caused by the tunnelling works, and also causes greater pre-excavation settlements in front of the tunnelling face. Once the bench/invert was completed below the settlement array, the additional settlement arising from subsequent tunnelling activities was reduced substantially.

Pre-Excavation Relief Ahead of Tunnel Face

The extent of the pre-excavation stress relief can be investigated by plotting the maximum surface settlement profile in the longitudinal direction ahead and behind the tunnel face. Figure 12 shows the longitudinal surface settlement profile along the tunnel, ahead and behind of the tunnel face. Although the magnitude of pre-excavation settlement increased with soil overburden (Fig. 11), the proportion of settlement due to pre-excavation relief appeared to be almost independent of the soil overburden (Fig. 13).

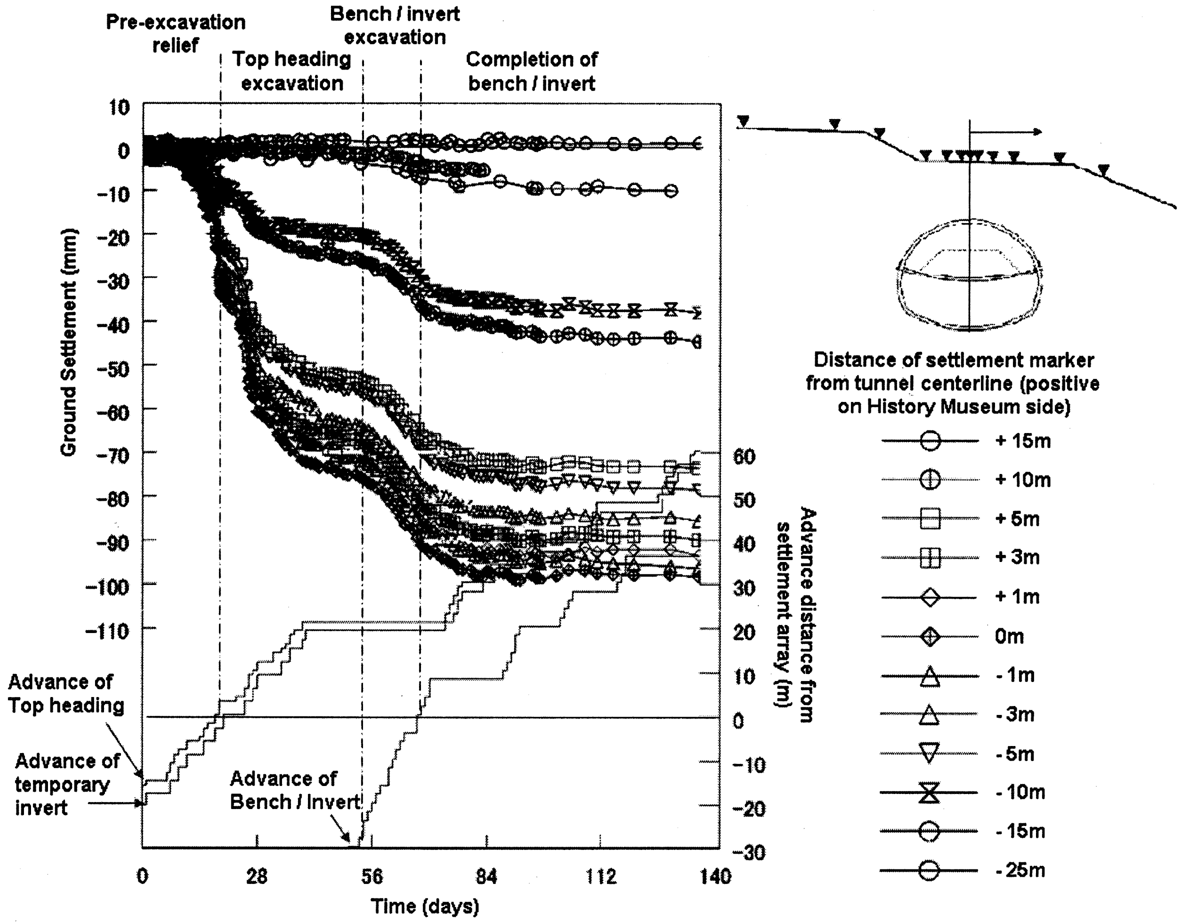


Fig. 10 Surface settlements at CH295 and its components

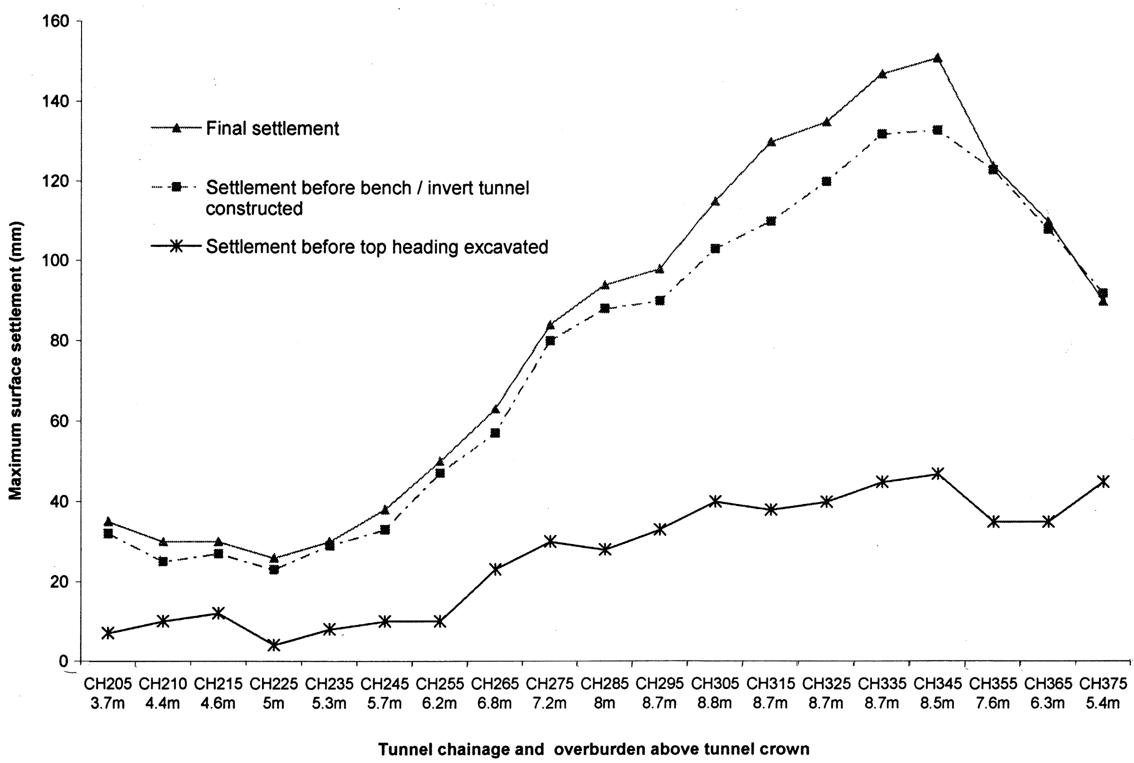


Fig. 11 Influence of soil overburden on maximum surface settlement

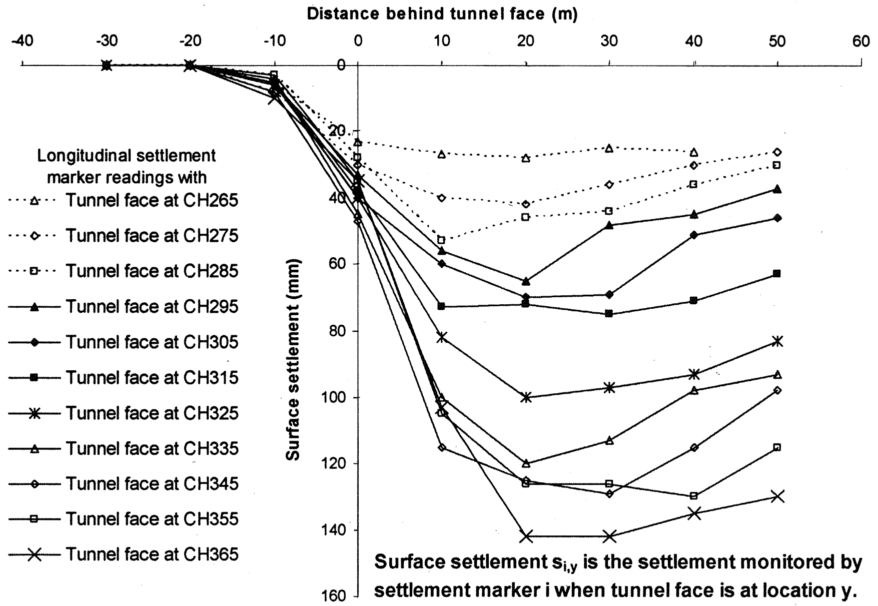


Fig. 12 Longitudinal surface settlement ahead and behind the tunnel face

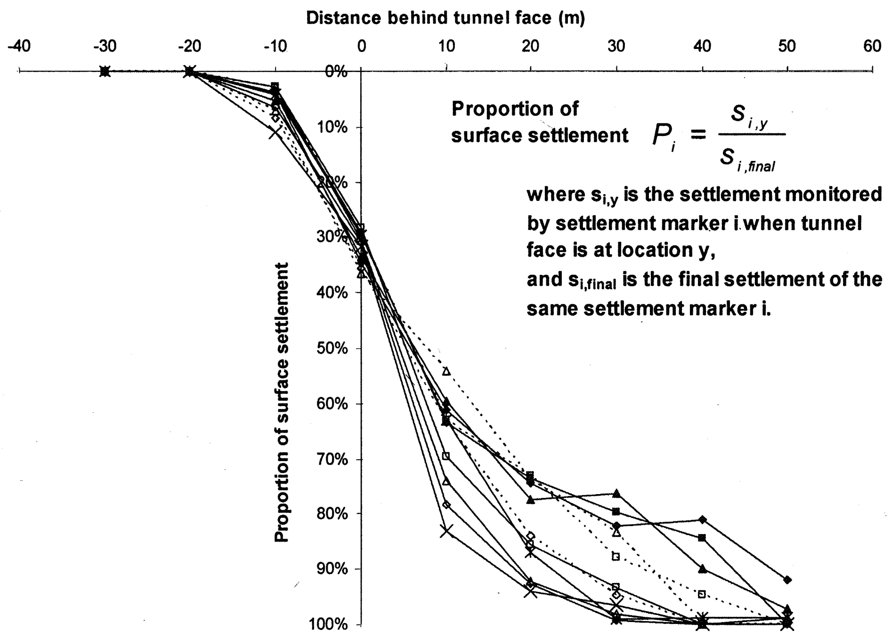


Fig. 13 Proportion of longitudinal surface settlement occurring ahead and behind tunnel face

This proportion varied in a range of 28% to 37% of the final settlement. In spite of the forepoling pipes installed ahead of the excavation face, the amount of pre-excavation settlement remained significant.

However, the proportion of pre-excavation settlement is lower than the 30%-50% range observed in other case histories of open-face tunnelling for firm to stiff clays (Attewell and Woodman, 1982, Mair and Taylor, 1997, Dimmock and Mair, 2007). This may be attributed to the reduction in the pre-excavation face relief associated with the forepoling. Using centrifuge modelling to study the mechanism of tunnel reinforcement in dry sand, Date et al (2008) found that both facebolts and forepoling can contribute to reducing the minimum tunnel support pressure required to maintain stability by up to 29% when compared to the unreinforced tunnel face.

Inference of Volume Loss and Observations

One approach to estimating tunnelling-induced ground settlement is an empirical method based on the concept of volume loss. Peck (1969) proposed that the cross-sectional surface settlement troughs correspond approximately to a normal Gaussian distribution; O'Reilly and New (1982) defined the parameters of that idealized transverse surface settlement profile, so that an empirical value of volume loss can be used to estimate the surface settlement profile. Although this has been derived mainly for circular and shield tunnelling, it is a useful concept for predicting settlements that can be caused by a mined non-circular tunnel.

Based on an excavated cross-sectional area of 136 m^2 and from the surface settlement troughs observed on site, the volume loss can be calculated by expressing the area under settlement trough as a percentage of the excavated area of the tunnel.

The back-calculated volume losses at various chainages are shown in Table 2; these vary from 0.4% to 2.1%.Mair (1996) reported that open face tunnelling in London Clay had resulted in volume losses generally in the range 0.5-1.5%, and the LTA Civil Design Criteria (2006) suggested that volume loss could vary from 0.5-1.5% based on NATM (SCL) excavation for tunnels up to 6.6m diameter in Singapore's Jurong Formation and FCBB. In comparison, volume losses observed in the much larger Fort Canning Tunnel varied up to 2.1%. From the plots of settlement troughs in Fig. 14, the observed ground settlement was Gaussian in nature and can be described by the trough width parameter $i = K^*Z_o$ as suggested by O'Reilly and New (1982), where Z_o is the depth to tunnel axis and a K value of 0.5 was appropriate for the clayey soils of FCBB

Influence of Soil Overburden on Tunnel Stability

It is observed that the volume loss increases with the amount of soil overburden above the crown of Fort Canning Tunnel. Mair (1989) suggested that volume loss can be related to the load factor LF = N / Nc, where N is the stability ratio of the tunnel and Nc is the critical stability number at failure. Using data from several case histories in overconsolidated clays (including centrifuge model test data from Mair et al (1981), Macklin (1999) fitted a linear regression between volume loss and load factor, as shown in Fig. 15.

From Macklin's regression equation, the load factors at various chainages can be calculated using the observed volume losses as shown in Table 3. The increasing load factors with soil overburden imply that for the shallow Fort Canning Tunnel, tunnel stability reduces with increasing soil load above the tunnel crown.

Table 2 Back-analysis of volume loss

Chainage	CH235	CH245	CH255	CH265	CH275	CH285	CH295	CH305
Overburden (m)	5.3	5.7	6.2	6.8	7.2	8	8.7	8.8
Volume Loss	0.4%	0.5%	0.7%	0.9%	1.1%	1.2%	1.3%	1.6%

Chainage	CH315	CH325	CH335	CH345	CH355	CH365	CH375
Overburden (m)	8.7	8.7	8.7	8.5	7.6	6.3	5.4
Volume Loss	1.8%	1.8%	1.9%	2.1%	1.9%	1.6%	1.2%

Table 3 Back-analysis of load factors

Chainage	Soil overburden (m)	Volume Loss	Load Factor, LF (1)
CH235	5.3	0.40%	0.13
CH245	5.7	0.50%	0.18
CH255	6.2	0.70%	0.25
CH265	6.8	0.90%	0.31
CH275	7.2	1.10%	0.36
CH285	8	1.20%	0.38
CH295	8.7	1.30%	0.39
CH305	8.8	1.60%	0.44
CH315	8.7	1.80%	0.47
CH325	8.7	1.80%	0.47
CH335	8.7	1.90%	0.48
CH345	8.5	2.10%	0.50
CH355	7.6	1.90%	0.48
CH365	6.3	1.60%	0.44
CH375	5.4	1.20%	0.38

Note:
(1) Load Factor is estimated from the linear regression line fitted by Macklin (1999), where volume loss $VL = 0.23\ e^{4.4(LF)}$ for LF > 0.2 and LF values less than 0.2 were ignored.

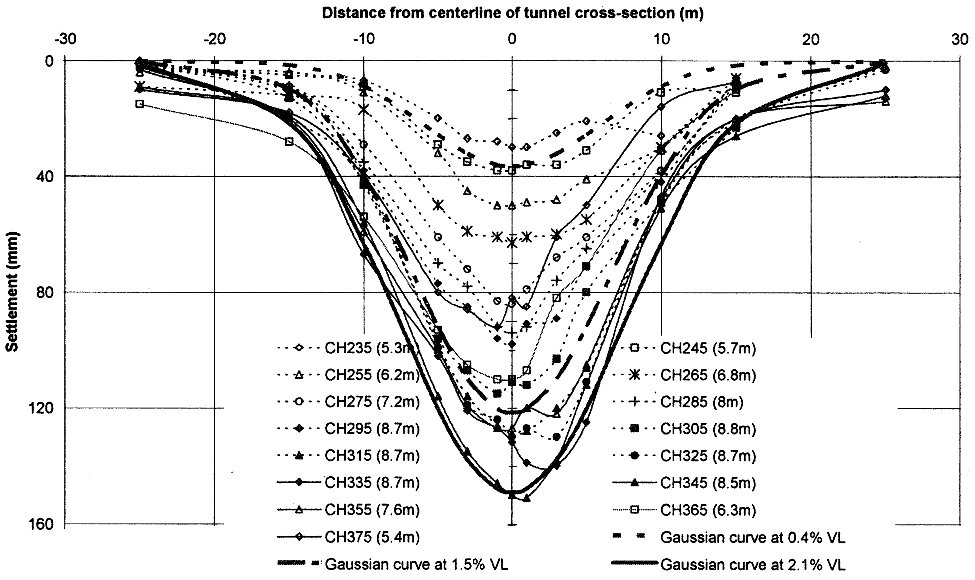


Fig. 14. Monitored settlement troughs versus theoretical Gaussian curves

This might also suggest that the shear strength of soil does not vary significantly with increasing depth for the shallow tunnel, although the soil investigations show a wide spread in the SPT-N values (Fig. 5).

MONITORING OF TUNNEL DEFORMATIONS

Settlement Response of the Tunnel

In-tunnel deformation was monitored using arrays of survey prisms installed on the shotcrete lining around the periphery of the tunnel. Figure 16 shows tunnel settlement behaviour during various stages of excavation. The settlement response of the tunnel lining is similar to that observed at the ground surface, as shown in Fig. 10. Initially, the tunnel crown settled more than the mid-tunnel footing level as the tunnel deformed under increased loads from the top heading excavation. As the top heading construction continued, the initial high rate of tunnel settlements decreased, and this can be attributed to the effect of the temporary invert working together with the stiffening shotcrete lining.

Closing the top heading with a temporary invert and providing an "elephant foot" to increase bearing capacity stabilized the tunnel movements prior to the bench/invert excavation. The rate of tunnel settlement increased slightly as the bench/invert excavation approached the top heading, and then reduced rapidly after the final tunnel ring closure was achieved by completing the tunnel bench/invert construction. After the tunnel bench/invert was completed, the additional top heading settlement was similar to the settlement monitored at the bench/invert level. This implies that the completed tunnel was moving as a whole rather than deflecting at the crown.

Influence of Soil Overburden on Tunnel Response

Fig. 17 summarises the settlement observed at the tunnel crown, at the mid-tunnel footing level, and at the tunnel bench/invert level at different chainages along the tunnel.

The tunnel settlements at the crown, mid-tunnel level and the bench/invert levels increased with higher soil overburden. Moreover, a higher overburden also caused an increased "squatting" of the tunnel crown, as seen from the difference between the mid-tunnel footing settlement curve and its corresponding crown settlement curve in Fig. 17. Hence, it can be inferred that the increased loading for the same primary lining support results in two effects on the tunnel: firstly, there is an increased total movement due to the tunnel settling into the soil, and secondly, there is an increased "squatting" at the tunnel crown. This results in the greater ground movements with higher soil overburden observed in the surface settlement curves, as shown in Fig. 11. Furthermore, once the bench/invert had been constructed, there was no further "squatting" of the crown and the additional settlements occurring in the tunnel were less than 20% of the final crown settlements observed.

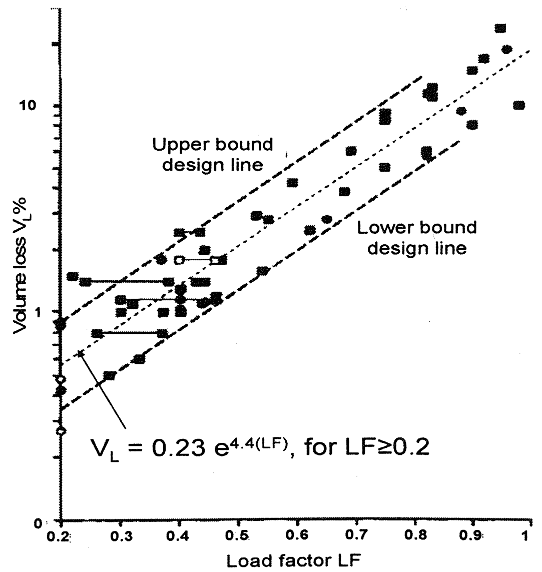


Fig. 15 Field monitoring data from overconsolidated clay sites and centrifuge model test data (after Macklin, 1999)

Tunnel Face Monitoring

During stoppages of top heading excavation, the face was also monitored for any signs of increasing movement. This was done using survey prisms installed on the tunnel face and monitored for out-of-plane movements, i.e. parallel to the tunnel axis. Table 4 summarises the face monitoring data during all the top heading stoppages.

A maximum face movement of 63mm was observed at CH343 during the top heading face stoppage for forepoling works and bench/invert construction. Figure 18 shows the development of face movements at this location.

The time dependent behaviour of the tunnel face depends on the soil overburden driving the soil wedge behind the face, the soil shear resistance along the failure wedge, and how long the unsupported face remains unexcavated.

Many of the stoppages to the top heading excavation were due only to the installation of the pre-excavation support forepoling system. During these excavation stoppages, face movement occurred due to the time dependent behaviour of an unsupported soil face as well as due to the disturbance caused during installation of the steel pipes and pressurized grout injection works. After the forepoling system was installed above the tunnel crown, the rate of face movements increasing with time reduced. Figure 19 compares the face movements at the end of the forepoling works with the total face movements. The movements arising from forepoling installation alone were less than 20mm in 16 out of the 18 locations where forepoling was installed.

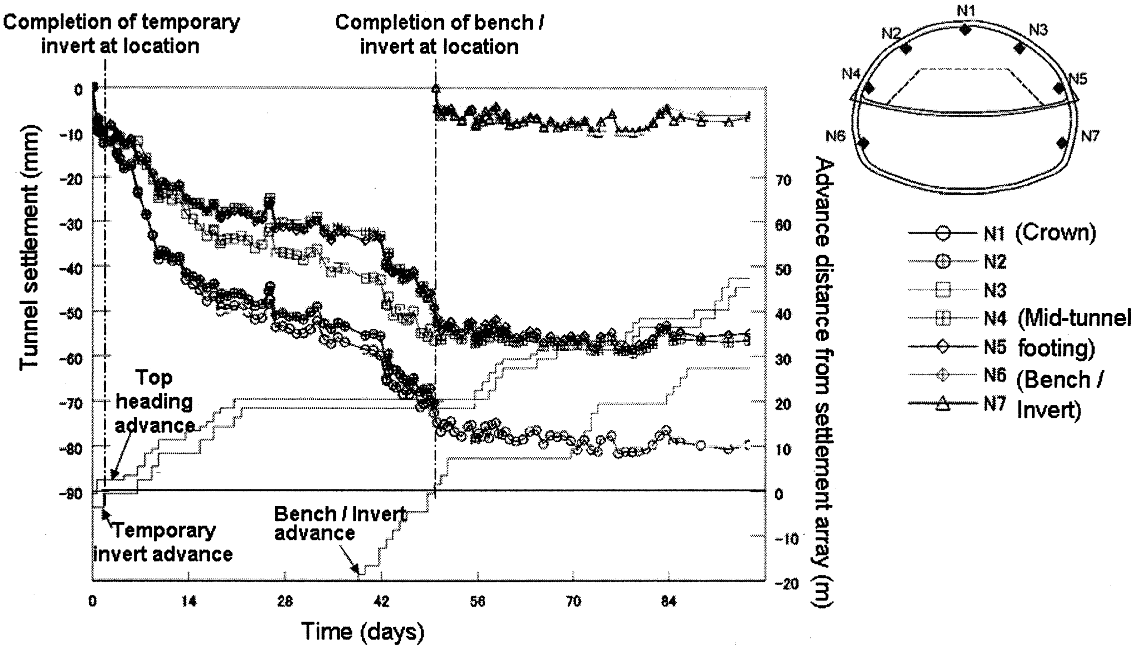


Fig. 16 Tunnel settlement monitoring using survey prisms at CH296

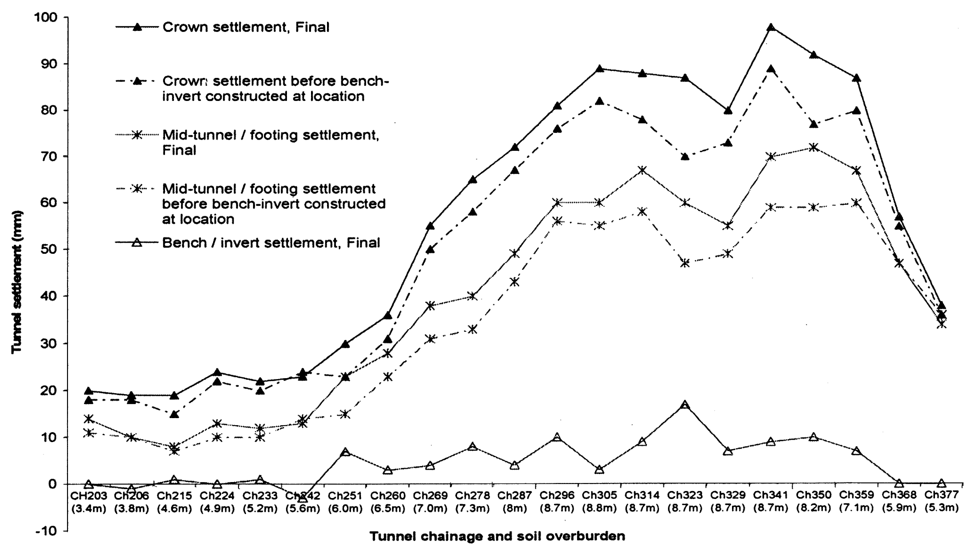


Fig. 17 Tunnel settlement at various chainages

Table 4 Summary of face monitoring data

Chainage	CH208	CH217	CH226	CH235	CH244	CH253	CH262	CH271	CH280
Overburden	4.0m	5.0m	5.0m	6.0m	6.0m	6.0m	7.0m	7.0m	7.4m
Reason for stoppage at face	Fore poling + bench / invert	Fore poling works	Fore poling + bench / invert	Fore poling works	Fore poling works	Fore poling works	Fore poling works	Fore poling + bench / invert	Fore poling works
Max face movement	25m m	3mm	9mm	5mm	0	13m m	9mm	20m m	11m m

Chainage	CH289	CH298	CH307	CH316	CH325	CH334	CH343	CH352	CH361
Overburden	8.4m	8.8m	8.7m	8.7m	8.7m	8.7m	8.6m	8m	6.9m
Reason for stoppage at face	Fore poling works	Fore poling works	Fore poling works	Fore poling + bench / invert	Fore poling works	Fore poling + bench / invert	Fore poling + bench / invert	Fore poling works	Fore poling + bench / invert
Max face movement	9mm	17m m	10m m	29m m	11m m	28m m	63m m	19m m	47m m

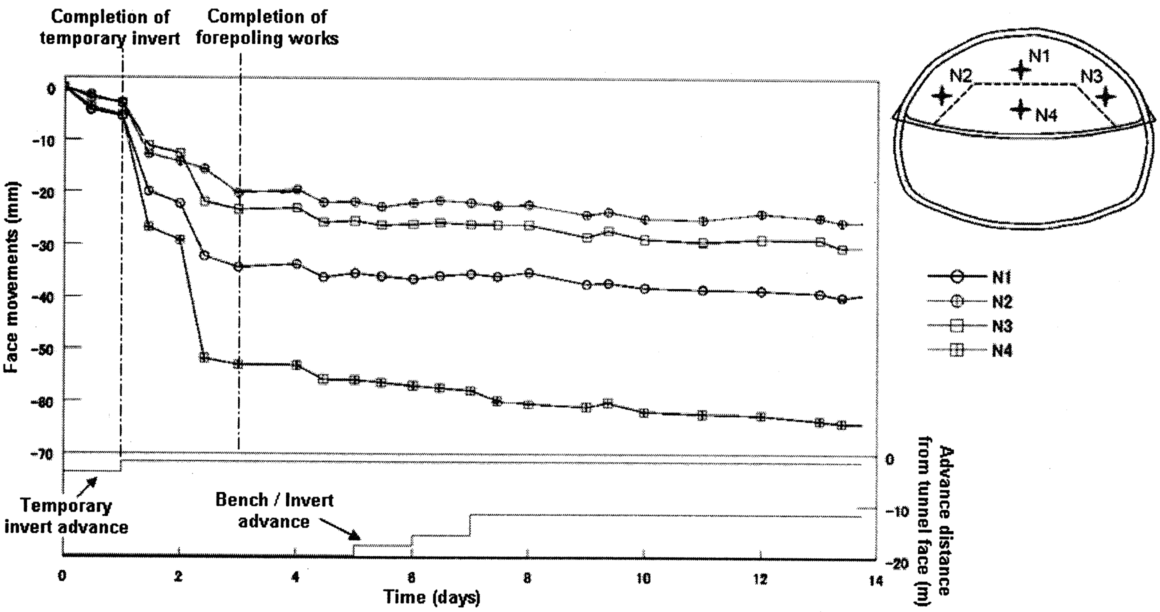


Fig. 18 Face movement monitoring at CH343

At some locations, the top heading face was left open for a much longer time to allow for the excavation of the tunnel bench/invert. During such prolonged exposure of the top heading face, the additional outward movement of the tunnel face was less than 15mm but still significant.

A localized soil slip on the tunnel face at CH206 caused part of the face below the tunnel crown to fall out (Figure 20). This had been attributed to heavy rainfall saturating the shallow overburden (~3.8m) above the tunnel. As the soil is more permeable at shallow depths, the sudden increase in porewater pressure probably led to a reduction in the shear strength of the soil and caused a local slip of the tunnel face. In accordance with the contingency plan, face dowels were installed to stabilize the face immediately

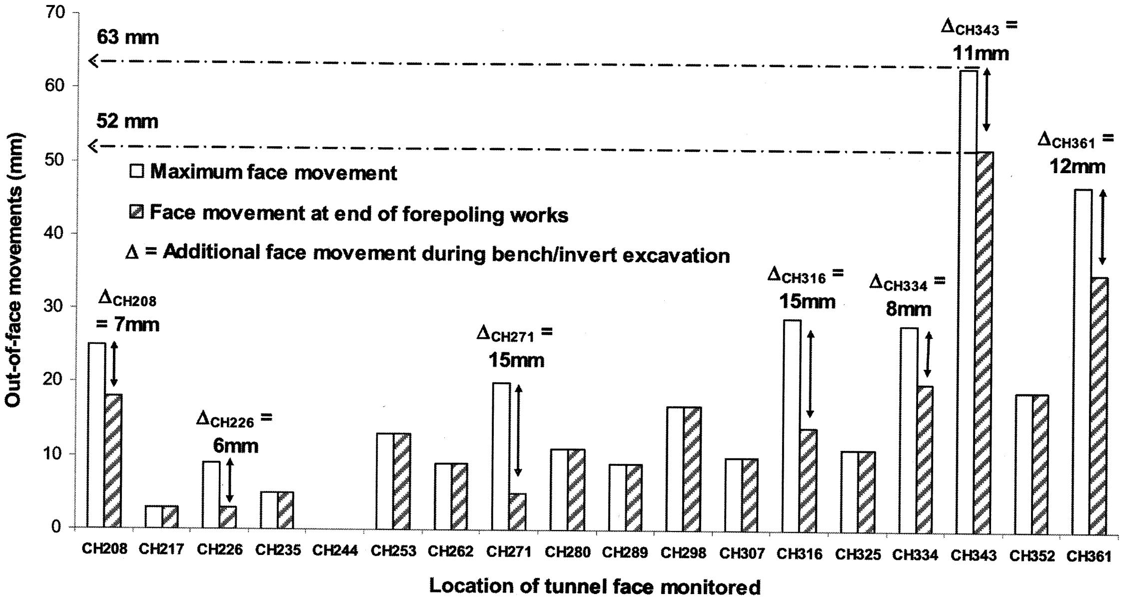


Fig. 19 Face movements along tunnel chainage

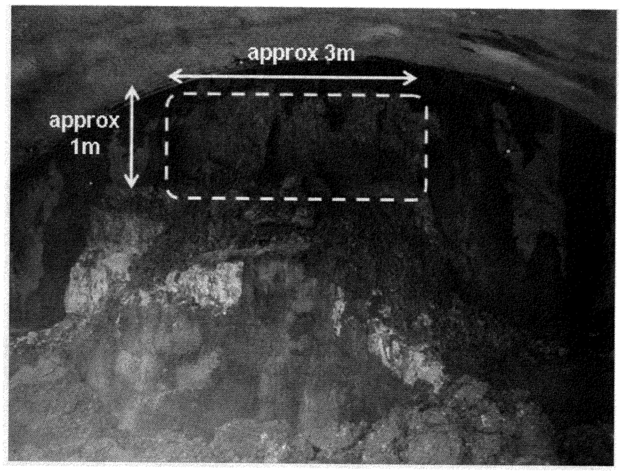


Fig. 20 Slipped face between dumpling and tunnel crown at CH206

CONCLUSIONS

Geotechnical investigations prior to tunnelling were carried out to confirm that the ground conditions were suitable for the proposed method of tunnel excavation at Fort Canning. This included a forepoling trial on site as well as an extensive soil investigation programme. During tunnelling, tight monitoring controls were implemented to allow the works to be carried out safely and progressively. Contingency measures were planned beforehand, and implemented readily when one localised failure occurred. Minor variations in the construction sequencing were made based on a planned observational approach backed up by extensive geotechnical investigations and close construction monitoring.

Analysis of the surface settlement and tunnel settlement readings has revealed interesting information on the SCL tunnel construction at Fort Canning. The volume loss ranged from 0.4% to 2.1%, with the corresponding transverse surface settlement trough following the theoretical Gaussian curve closely.

The volume loss generally increased with soil overburden, and using the load factor approach, it is suggested that the tunnel stability reduced marginally with increased soil overburden (suggesting a relatively constant undrained shear strength with depth). The longitudinal settlement profile showed that the pre-excavation stress relief accounted for up to 37% of the total ground settlement. Whilst forepoling may have helped to reduce the pre-excavation stress relief, substantial settlements occurred ahead of the tunnel face even with forepoling reinforcement. Some increase of tunnel face movement was observed during installation of forepoling.

It is hoped that the experiences gained from the Fort Canning Tunnel project will be useful to engineers planning similar tunnels and will give greater confidence when undertaking large diameter shallow tunnels in soft ground using SCL construction methodology.

REFERENCES

ATTEWELL, P.B. and WOODMAN, J.P. (1982). Predicting the dynamics of ground settlement and its derivatives caused by tunnelling in soil, *Ground Engineering* Vol. 15, No. 8, pp. 13-36.

CIRIA REPORT 185. The Observational Method in ground engineering: Principles and Applications, 1999, Construction Industry Research and Information Association.

DATE, K., MAIR, R.J. and SOGA, K. (2008). Reinforcing effects of forepoling and facebolts in tunnelling, *Geotechnical Aspects of Underground Construction in*