MONITORING SUCCESSFUL NATM IN SINGAPORE

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ABSTRACT

The Fort Canning Tunnel, Singapore, is an approximately 50 feet (15m) wide vehicular tunnel under shallow ground cover that has been constructed according to the principles of the NATM. Its shallow location in soft ground combined with a high groundwater elevation in close vicinity of historically important features called for special design considerations combined with a rigorous monitoring scheme. Surface, subsurface and in-tunnel instrumentation was installed to monitor the performance of the ground and tunnel support during construction. Comparison of actual reading data with the results from the computer modeling carried out during the design phase is provided in this paper.

INTRODUCTION

The Fort Canning Tunnel forms a part of the project "Contract PE101A—Design and Construction of Fort Canning Tunnel and Realignment of Stamford Road," sponsored by the Land Transport Authority (LTA) of Singapore. At Fort Canning Tunnel (FCT) the New Austrian Tunneling Method (NATM) was used for the first time for a large span (14.7m–48ft) tunnel in Singapore. The contract has been tendered in a Design–Build framework and the team with the members Sato Kogyo (S) Ltd. (Contractor), TY Lin Ltd. (Engineer) and Gall Zeidler Consultants (NATM designer) has been awarded the contract. Fort Canning Tunnel is a 180m (590ft) long, three-lane highway tunnel, and has a cross section area of 135m^2 (1440ft^2). It was constructed in residual soils under an overburden between 3m and 9m (10ft and 30ft). The tunnel approaches at the north and the south portal of Fort Canning Tunnel are constructed using cut-and-cover techniques.

LOCAL SETTINGS

Surrounding Structures

Whereas at the northern end of the tunnel no buildings are located in close vicinity to the tunnel structure, a retaining wall for the newly constructed Singapore History Museum is positioned in immediate proximity of the tunnel next to the south portal.

The alignment leads underneath the historic Fort Canning Park with its preserved trees and a historic cemetery. Fort Canning Rise, a public road crosses the tunnel alignment at a vertical clearance of approximately 5 m between the tunnel roof and the road surface.

In vicinity of the south portal, a settlement sensitive Church building is located.

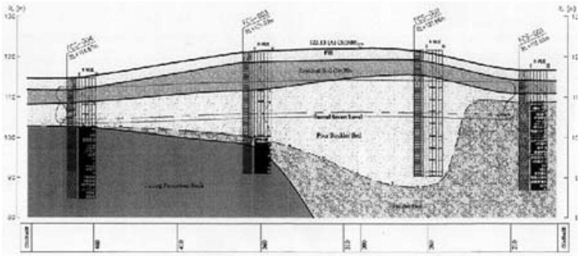

Figure 1. Original schematic geological profile (from initial site investigation report by Kiso-Jiban Consultants Co. Ltd., 2003)

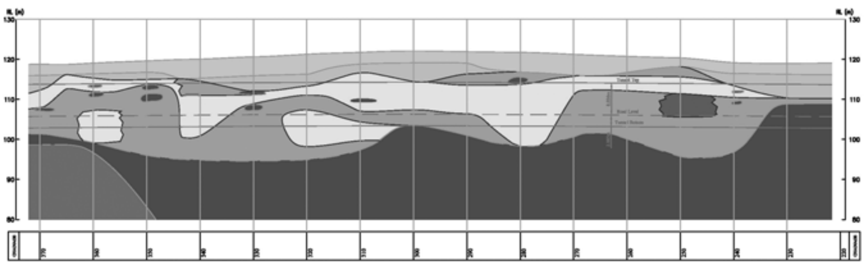

Figure 2. Updated schematic geological profile (Kiso-Jiban Consultants Co. Ltd., 2004)

Geology

The Fort Canning Tunnel is constructed in the residual soils of the Fort Canning Boulder Beds. The Fort Canning Boulder Bed is a colluvial deposit of Pleistocene age that underlies parts of the central business and commercial district of Singapore. It consists of sandstone boulders in a matrix of hard sandy silt or sandy clay with silt. The matrix is of deep-red, red and white or mottled red, yellow and white color. For classification purposes the residual soil layer was subdivided based on the SPT N-value; RS I (N<15), RS II (15<N<30), RS III (30<N<50) and RS IV (N>50). The residual soils of Fort Canning Boulder Bed are overlain by a man-made fill layer of varying thickness (1m to 5m) and are underlain by the bedrock of the Jurong Formation, a sedimentary rock (Sandstone).

Ground water level at the Fort Canning Hill is established at approximately 1m below surface level, i.e., 2m to 8m above tunnel crown level.

The deposits were found in non-continuous layers and lenses with a varying amount of boulders leading to a inhomogeneous excavation medium with a wide variety of ground properties. Observations in the tunnel face revealed that the ratio between matrix vs. boulders varied between 70% boulders vs. 30% matrix and 30% boulders vs. 70% matrix (Figures 1 and 2).

DESIGN AND CONSTRUCTION

General

The project was tendered based on a Design–Build contract format. The NATM mined construction approach was the Contractor's proposed alternative. The design

Table 1. Soil parameters

Layer		Soil Parameters					
		γ (kN/m^3)	E (MPa)	ϕ' (deg)	C' (MPa)	Cu (MPa)	n'
Fill	Original 1st Variation 2nd Variation	19/ 19/ 19	10/ 10/ 10	30/ 30/ 30	0.005 /0.001/ 0.001	0.040/ N/A/ N/A	0.35/ 0.35/ 0.2
Residual Soil (RS I)	Original 1st Variation 2nd Variation	—/ 20/ 20	—/ 15/ 15	—/ 30/ 30	—/ 0.010/ 0.010	—/ 0.045/ 0.045	—/ 0.35/ 0.2
Residual Soil (RS II)	Original 1st Variation 2nd Variation	20/ 20/ 20	60/ 30/ 30	30/ 30/ 30	0.015/ 0.010/ 0.010	0.180/ 0.090/ 0.090	0.35/ 0.35/ 0.2
Residual Soil (RS III)	Original 1st Variation 2nd Variation	—/ 20/ 20	—/ 60/ 60	—/ 30/ 30	—/ 0.020/ 0.020	—/ 0.180/ 0.180	—/ 0.35/ 0.2
Residual Soil (RS IV)	Original 1st Variation 2nd Variation	20/ 20/ 20	100/ 100/ 100	30/ 30/ 30	0.035/ 0.035/ 0.035	0.300/ 0.300/ 0.300	0.35/ 0.35/ 0.2
Boulder Bed	Original 1st Variation 2nd Variation	22/ —/ —	300/ —/ —	30/ —/ —	0.040/ —/ —	0.500/ —/ —	0.35/ —/ —
Jurong Rock Formation	Original 1st Variation 2nd Variation	24/ —/ —	600/ —/ —	35/ —/ —	0.040/ —/ —	1.000 /—/ —	0.3/ —/ —

for both the tunnel initial support (temporary works) and the permanent support (permanent works) had to be provided by the Contractor for approval by the LTA and the Building & Construction Authority (BCA).

The tragic incident at the Nicoll Highway in early 2004 triggered a series of additional ground investigations and the request for additional numerical analyses for the design of the Fort Canning Tunnel. These analyses included comprehensive ground parameter studies and sensitivity analyses to ensure that a robust design was developed leading to a save and successful tunnel construction.

Design Parameters

Investigation Program. The soil investigation program included approximately 40 borings along the alignment to retrieve cores for laboratory testing and to carry out borehole tests such as Standard Penetration Tests and groundwater observations.

Variation of Design Parameters

During the design verification phase the geological model along the alignment was modified (Figure 2) and a series of parameter studies was carried out. Furthermore, a weak residual soil layer (RS I) was assumed to be present above the crown of the tunnel and a “worst credible” geological profile was introduced. The corresponding FE model was generated. Table 1 summarizes three sets of parameters used during that phase. These were used for the design analyses at different design stages.

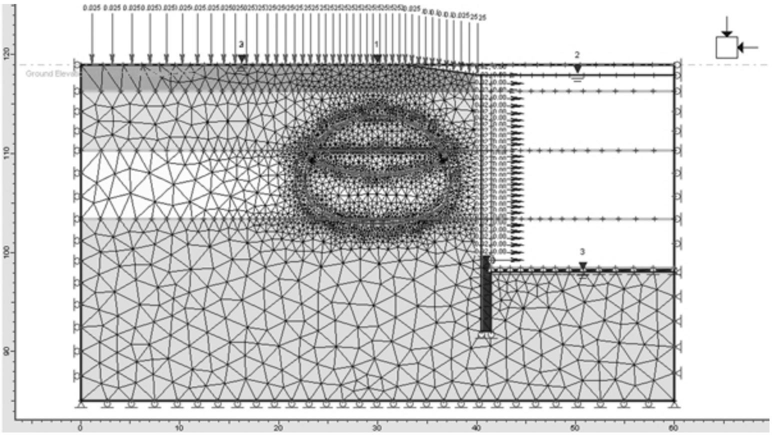

Figure 3. FEM analysis model (CH 240) at the location near the new support wall for the Singapore Natural History Museum

Design Analyses Results

The finite element program Phase2, V5.04 by Rocscience, Inc. was employed for the analyses of loading conditions, ground response to the tunnel construction and to assess the lining forces. Triangular solid material elements were used to model the ground, and beam elements were used for the representation of the linings. The soil was modeled as an elastic-plastic material using the Mohr-Coulomb failure criterion; the linings were simulated as ideally elastic-plastic materials.

All finite element models for the primary lining design utilize a multi-staged modeling approach. An initial stage describing the in-situ stress state of the soil prior to tunneling formed the start point. The modeling stages were established to asses the individual construction stages. Top heading excavation was modeled by softening the soil within the excavation limits of the heading followed by the excavation of the soil elements within the excavation area. This approach simulates the excavation and resulting relaxation of the soil surrounding the tunnel prior to installation of the shotcrete support.

The increase of compressive strength and stiffness of the shotcrete lining after installation was modeled by gradually increasing the stiffness of the lining elements during the following construction stages. By this method, the interaction between ground deformations of the surrounding soil and the initial shotcrete lining is simulated as the excavation progresses.

The modeling sequence of the excavation and support sequence is concluded with bench and invert excavation and installation of the shotcrete lining in the invert.

The shotcrete lining was checked for its integrity during all the intermediate construction stages. The structural capacity of the shotcrete lining was determined in accordance British Standard BS 8110 and Singapore Standard SS CP65.

In addition to the two-dimensional finite element analyses, a three-dimensional finite element analysis was performed to assess the effects of the AGF pre-support umbrella and to confirm the performance of the two dimensional finite element models. All structural design, however, was based on the results of the two-dimensional finite element analyses. Figures 3 and 4 show typical finite element models.

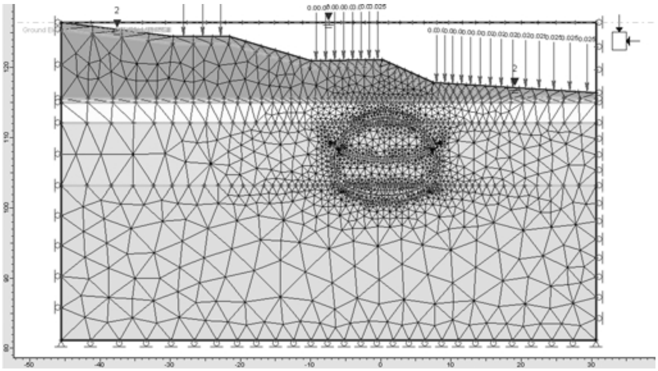

Figure 4. FEM analysis model (CH 340) underneath the Fort Canning Rise

Table 2. Summary of FE results

	Surface Settlements	Tunnel Lining Deformation	
CH 240	max. 104 mm	max. 104 mm (roof settlement)	max. 18 mm (settlement @ springline)
CH 340	max. 39 mm	max. 57 mm (roof settlement)	max. 21 mm (settlement @ springline)
Worst credible	max. 114 mm	max. 138 mm (roof settlement)	max. 48 mm (settlement @ springline)

Utilizing the last iteration of geotechnical parameters (2nd Variation in Table 1) in a series of FE analyses, the deformations at ground level and within the tunnel structure are determined in Table 2.

Pre-Support

The Fort Canning Tunnel is constructed under very limited overburden ranging from 3m (9ft) to 9m (19ft); i.e., 20% and 60% of the tunnel width. Due to this shallow overburden an AGF pipe arch was used as a continuous pre-support system over the full length of the tunnel (Figure 5).

The AGF ("All Ground Fastened") pipe arch consisted of a single row of steel pipes drilled at 400mm spacing in the crown of the tunnel top heading. The outer diameter of the steel pipes was 114mm (4.5"), pipe wall thickness 6mm (¼"); overall pipe length is 12.5 m (41ft), installed in four sections (3.5m–3m–3m–3m). The steel pipes were installed at a 7% outwards angle. A 3.5m overlap between succeeding pipe umbrellas was provided. 9m long tunnel sections were excavated followed by the installation of the next AGF umbrella in sequence.

A polyurethane two-component grout was injected through the AGF pipes via grouting ports at 0.25m (¾ ft) spacing along the length of each pipe. The grouting process was both volume and pressure limited for each individual pipe.

Excavation and Support Sequence

The Fort Canning Tunnel was constructed using an excavation and support sequence comprising of top heading excavation (with temporary invert) and combined

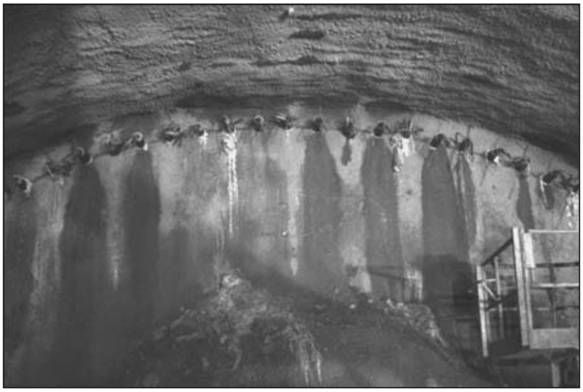

Figure 5. Pre-support umbrella installation

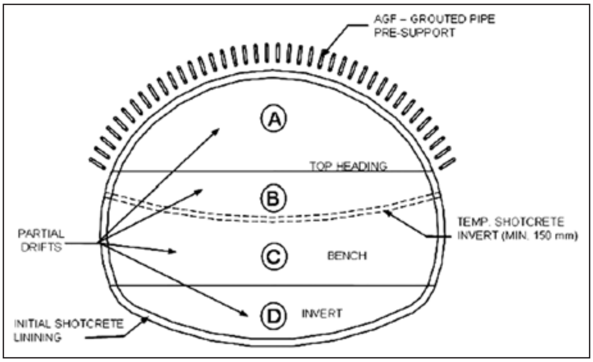

Figure 6. Excavation and support sequence

bench/invert excavation. The top heading extended over the entire width of the tunnel and its height was 6.0m that increased to 6.5m (20ft to 21.5ft) at the AGF installation location. Advance length in the top heading was 1m (3.3 ft). During the trial period, the temporary ring closure of the top heading (temporary invert) was installed in 2m increments at maximum 4m distance behind the excavated tunnel face. The temporary invert installation was increased to 3 m blocks at max. 6 m distance to the face following the trial section. In order to enhance the stability of the tunneling face a face stabilization wedge of 2 m depth at its base is left in place.

The combined bench and invert excavation with two 2m increments in bench followed by the two 2m increments in the invert was carried out in two steps immediately following each other. A minimum distance of 20m between top heading and bench/invert excavation faces was maintained (Figure 6).

Tunnel Support

Initial Support. The primary lining consists of minimum 300mm thick shotcrete with a design compressive strength of 40 N/mm^2. The primary lining is reinforced by one three bar lattice girder installed in every excavation round at 1m spacing and two layers (inside and outside) welded steel mesh of 10mm diameter bars at 200mm

Figure 7. Initial shotcrete tunnel support in top heading

spacing. The temporary invert had a design shotcrete thickness of minimum 150mm and was reinforced by one layer of welded steel mesh of 10mm diameter bars at 200mm spacing (Figure 7).

Tunnel Final Lining. The Land Transport Authority (LTA) as the Client of Fort Canning Tunnel requested in the project design criteria that no ground water may be drained by the tunnel structure. A waterproofing system that extends around the full tunnel circumference and that consists of a geotextile layer and a welded PVC waterproofing membrane was utilized to achieve this requirement. The waterproofing system is segmented by a series of circumferential and longitudinal water barriers.

OBSERVATIONS DURING CONSTRUCTION

General

Excavation for 180m (590ft) long tunnel started in March 2005 and was completed in March 2006. Excavation was carried out using excavators and breakers.

In general, the ground conditions and ground behavior was found in agreement with the expectations.

Ground Behavior

The deposits were found in non-continuous layers and lenses with a varying amount of boulders leading to inhomogeneous ground conditions with a wide variety of ground properties. Depending on the density of the soil matrix, number and size of boulders in the excavation face and water saturation the ground varied from very hard to very soft (Figure 8).

Most of the sandstone boulders displayed a more-or-less thin weathered mantle. Inside the weathered zone the boulders were very hard generating significant resistance to excavation and breaking. While blasting of the larger boulders was not an option, excavator mounted hoe rams were used to remove the boulders from the excavation face and to break them up. Boulder removal lead sometimes to loosening of the surrounding soil matrix and soil layers due to the vibration energy exerted by the breakers.

While the clay typically yields low permeability, sand and silt admixtures, layers and lenses generated water paths that lead to groundwater discharge through the tunnel face. Despite the limited overburden thickness, the hill foot location in combination

Figure 8. Boulders in the bench face

with frequent tropical rainfall generated sufficient water recharge to cause wet tunneling conditions. Pump sumps had to be provided to avoid softening of the ground in the bench and invert area.

The inherent stand-up time provided by the ground was considered too limited for a safe tunnel support installation. That led to the decision to employ a pre-support system over the entire length of the tunnel early on in the design phase. The lateral extent of the pre-support arch proved sufficient to safely install the shotcrete tunnel support after each excavation round. The grouting material penetrated the various soil deposits including the very fine silt materials introducing cohesion for improved ground strength.

The slightly domed tunnel face of the top heading was sealed after each excavation round. A flash layer of shotcrete was sufficient to stabilize the face and to prevent desiccation of the soil between the excavation operations. Face instabilities have not been observed. This may have been caused—in parts—by the continuous pre-support but also by the face stabilization wedge. During longer excavation interruptions, such as for the AGF umbrella installation, full shotcrete face support was installed (Figure 9).

Surface Settlements

Surface settlements were monitored using surface monitoring points either anchored into paved surfaces or into the soil. Readings were carried out using precise survey instruments. Generally, settlement readings were carried out on a daily basis. If undue readings were recorded, the reading frequency was increased to twice or several times a day (Figures 10 and 11).

Tunnel Deformation

Tunnel deformation monitoring was carried out with total stations. The daily readings yielded 3-D movement data that were evaluated immediately after taking the readings. In tunnel sections were the full final ring closure had been established and the tunnel lining deflections had ceased, the reading frequency was reduced to once per week (Figures 12 and 13).

Figure 9. Top heading face shortly after excavation with exposed pre-support pipes

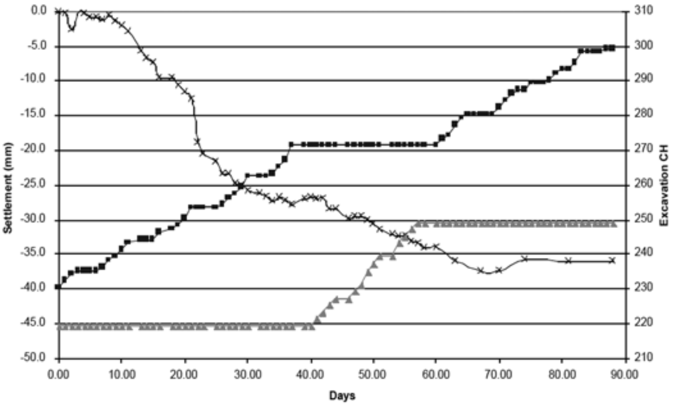

Figure 10. Surface settlement CH 245 (overburden 5.3m)

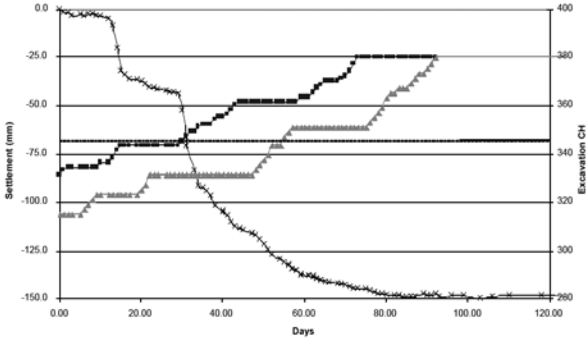

Figure 11. Surface settlement CH 345 (overburden 5.1m)

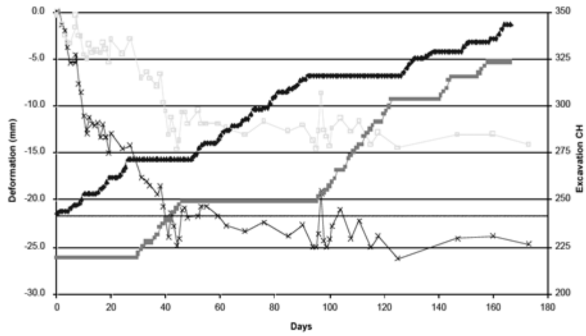

Figure 12. In-tunnel deformations CH 242 (overburden 4.8m)

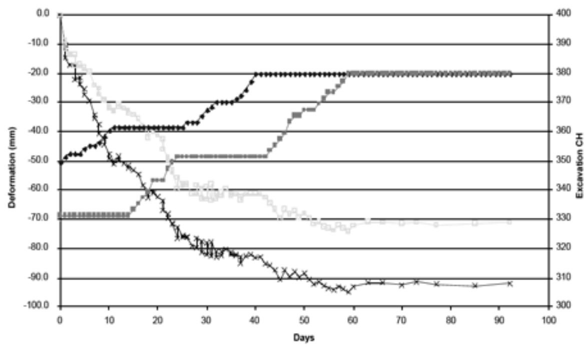

Figure 13. In-tunnel deformations CH 350 (overburden 4.5m)

COMPARISON OF ACTUAL READING DATA TO DESIGN DATA—SURFACE SETTLEMENTS AND TUNNEL DEFORMATIONS

Comparison of the analytical predictions with deformation measurements during construction yields partially inconclusive results. Using the ground properties stated above (2nd Variation), surface settlements for CH 240 are considerably over-predicted (104 mm predicted vs. 36 mm measured). Similarly, tunnel lining deformations were predicted too high (104 mm roof settlement and 21 mm settlement at the spring line) compared to the measured data (25 mm roof settlement and 13 mm settlement at the spring line). Back analyses of the FE model at CH 240 show that the introduction of an improved material layer in the tunnel roof modeling the AGF umbrella and an increase of the stiffness of the soils by 50% and the shear strength by a factor of 10 yield results more comparable to the measured deformations (24 mm roof settlement and 15 mm settlement at the spring line). The effect of the AGF pre-support in the model was ignored during the later design stages, to ensure a more conservative design approach (Figures 14 and 15).

For the back analysis, the strength and stiffness parameters of the improved soil have been assessed by multiplying the strength of each component (soil, steel) by the cross sectional area of each component and then dividing the sum of these products by the total area of improved soil under consideration (see Equation 1).

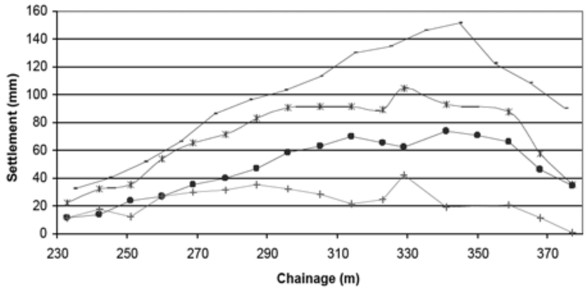

Figure 14. Surface and tunnel settlement along the alignment

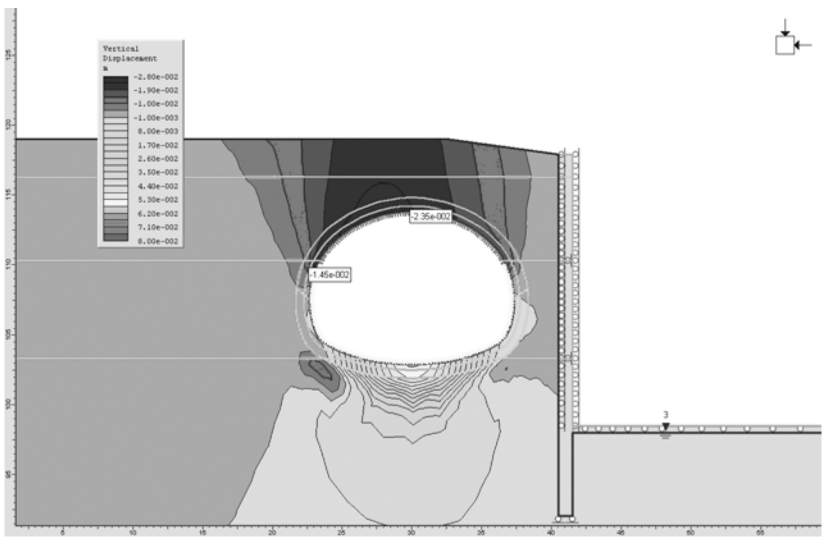

Figure 15. Tunnel deformations CH 240—back analysis with AGF umbrella

$$P_{AGF} = \frac{(P_P \cdot A_P) + (P_S \cdot A_S)}{(A_P + A_S)} \tag{1}$$

When using the same soil properties (2nd Variation from Table 1) at CH 350 and ignoring the beneficial effects of the AGF umbrella (as it was done for all final design calculations), the numerical analyses significantly under-predicted the deformations measured in the field. Whereas, following numerical analyses, surface settlements of 39 mm should have been expected for the prevalent soil conditions, surface settlements of up to 148 mm were observed. Tunnel lining deformations at this location were also considerably underestimated. Tunnel settlements of 57 mm in the roof and 21 mm at spring line level are predicted; 93 mm and 71 mm respectively are measured.

A closer match to the actually measured data at this location forms the model used as the worst credible situation at Fort Canning Tunnel. In the latter analysis, which formed the design case for the tunnel lining, surface settlements of 114 mm and vertical tunnel deformations of 138 mm (crown) and 48 mm (spring line) are determined.

Under normally consolidated soil conditions it may be expected that surface settlements are somewhat smaller, or approximately equal for very shallow tunnels than settlements measured in the tunnel crown. The surface settlement values measured at Fort Canning tunnel are locally significantly higher than the tunnel lining deformations (up to 50% more surface settlement). This may be attributed to insufficiently compacted fill above the tunnel alignment that has not been identified during the soil investigation and a short-term consolidation effect triggered by the tunneling operation below. However, measurement data are not yielding completely conclusive indications for this assumption.

CONCLUSION

Categorizing inhomogeneous residual soils—as prevalent at Fort Canning Hill—according to their SPT-N-values, as well as assessing elasticity and Mohr-Coulomb shear parameters in order to predict the ground response to a tunneling operation constitutes a challenging task. Numerical analyses utilizing a wide range of soil parameters are required to reliably predict upper and lower bound of the expected surface and tunnel lining deformations. In the Finite Element analyses performed for Fort Canning Tunnel, the upper bound settlement values were predicted acceptably well while lower bound values were not closely matched by analytical results using the stated elasto-plastic soil parameters. By using SPT N-values to categorize soil strata, an exact prediction of surface and tunnel deformations at locations where detailed geological stratification data were available (i.e., borehole location) however was not achieved.

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