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NATM for Singapore

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ABSTRACT: Singapore has some experience in the application of NATM for tunnel construction. To date, tunnels of smaller size have been constructed using this technique. The Fort Canning Tunnel is a three-lane vehicular tunnel with a span of approximately 48 feet and is the first wide span tunnel in Singapore constructed by NATM. Its location in soft ground combined with a high groundwater elevation and shallow cover of about 12 feet called for special design considerations in particular in the use of systematic pre-support techniques. A special pre-support umbrella (AGF) consisting of steel pipes and synthetic grouts for ground improvement is utilized. The tunnel design has been developed in close cooperation with the contractor Sato Kogyo in a design-build framework. As a result of the tunnel collapse that occurred on a cut-and-cover construction for Land Transit Authority's subway system the design development was subjected to a rigorous review and checking process.

1 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) is used for the first time for a large span (14.7 m – 48 ft) tunnel in Singapore. The contract has been tendered in a "design and build" frame work 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 three lane highway tunnel, is 180 m (590 ft) long and has a cross section area of 135 m^2 (1440 ft^2). It is constructed in residual soil under an overburden between 3 m and 9 m (10 ft and 30 ft). The tunnel approaches at the north and the south portal of Fort Canning Tunnel are constructed using cut and cover techniques. 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, At one point, the primary tunnel lining even touched the retaining wall.

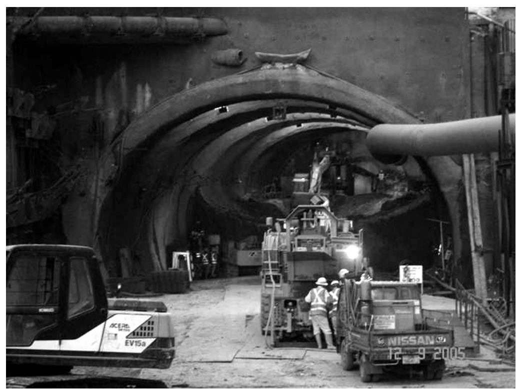

Figure 1. Fort canning tunnel.

2 DESIGN CONCEPT

2.1 *Geology*

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 boulders in a hard Sandy Silt or Sandy Clay with Silt matrix. 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 various thickness (1 m to 5 m) 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 1 m below surface level, i.e. 2 m to 8 m above future tunnel crown level.

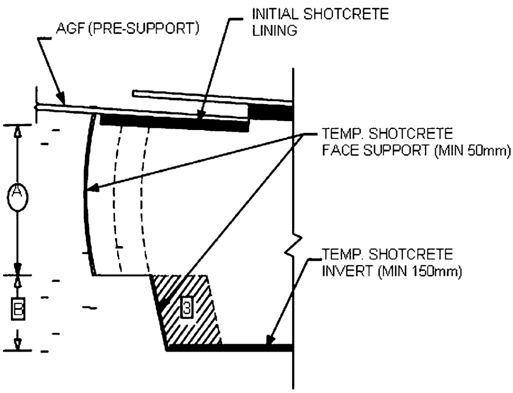

Figure 2. Long section, top heading excavation sequence.

2.2 *Pre-support system*

Fort Canning Tunnel is constructed under very limited overburden. Maximum and minimum overburden is approximately 9 m and 3 m (30 ft and 10 ft) respectively, i.e. 60% and 20% of the tunnel width. Due to this shallow overburden an AGF pipe umbrella is used as a continuous pre-support system over the full length of the tunnel.

The AGF ("All Ground Fastened") pipe umbrella consists of a single layer of steel pipes drilled at 400 mm spacing in the crown of the tunnel top heading. The outer diameter of the steel pipes is 114 mm (4.5 in), pipe wall thickness 6 mm (1/4 in); overall pipe length is 12.5 m (41 ft), installed in four segments (3.5 m – 3 m – 3 m – 3 m). The steel pipes are installed at a 7% outwards gradient (see Figure 2). A 3.5 m overlap of two succeeding pipe umbrellas is provided, resulting in 9 m long segments to be excavated before the installation of the next AGF umbrella following in sequence.

A poly-urethane two component grout is injected through the AGF pipes via grouting ports at 0.25 m (3/4 ft) spacing along the length of the pipe. The grouting process is both volume and pressure limited for each individual pipe.

2.3 *Excavation and support*

The Fort Canning Tunnel is excavated under the continuous AGF pipe umbrella pre-support using an excavation and support sequence comprising of top heading excavation (with temporary invert) and combined bench/invert excavation. Top heading height is 6.0 m that increases to 6.5 m (20 ft to 21.5 ft) at the AGF installation location. Advance length in the top heading is 1 m (3.3 ft). During the trial period, the temporary ring closure of the top heading (temporary invert) was installed in 2 m increments at maximum 4 m 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 increase the stability of the tunneling face a face stabilization wedge of 2 m depth at its base is left in place.

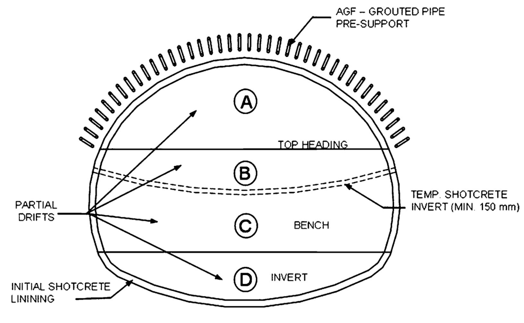

Figure 3. Excavation and support sequence.

The combined bench and invert excavation with 2 m increments in bench followed by the 2 m increments in the invert follows top heading excavation with a minimum distance of 20 m between top heading and bench/invert excavation faces.

The primary lining consists of minimum 300 mm 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) steel reinforcement (welded steel mesh) of 10 mm diameter bars at 200 mm spacing (circumferentially and longitudinally). The temporary invert has a design shotcrete thickness of minimum 150 mm and is reinforced by one layer of steel reinforcement (welded steel mesh) of 10 mm diameter bars at 200 mm spacing (circumferentially and longitudinally).

2.4 *Waterproofing system*

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. Therefore, a fully tanked waterproofing system consisting of a geo-membrane/geotextile layer and a PVC waterproofing membrane is utilized to achieve this requirement. The waterproofing system is segmented by a series of circumferential and longitudinal waterbarriers.

2.5 *Final lining*

The final lining at Fort Canning Tunnel is designed to withstand full overburden pressure, full hydrostatic pressure and an additional surface surcharge load as per the LTA criteria. The final lining is a cast in-situ concrete lining formed by a collapsible steel form. The structural thickness of the final lining is minimum 300 mm in the tunnel arch and minimum 350 mm in the tunnel invert. The compressive strength of the concrete final lining is required to be 40 N/mm^2 and the lining will be reinforced with 9 mm diameter bars at 100 mm spacing in the tunnel arch and with 10 mm diameter bars at 100 mm spacing in the tunnel invert.

The final lining is designed to withstand additional movements that may be caused by future construction activities adjacent to the tunnel such as future MRT subway tunnels crossing underneath the alignment of the Fort Canning Tunnel.

3 GEOTECHNICAL AND STRUCTURAL ANALYSES

In order to assess the stability of the excavation sequence and the installed support system a series of finite element analyses and other geotechnical analyses were performed.

Two-dimensional finite element analyses have been chosen as the primary analytical method in order to assess the integrity of the structure during intermediate construction stages and final conditions after completion. Three calculation cross section locations along the 180 m long tunnel were selected to represent the most onerous structural and geotechnical conditions along the tunnel alignment.

Face stability checks were carried out. These checks assessed the safety of the expected maximum-size face wedge against sliding out from the face. Locations at 10 m intervals along the tunnel alignment were analyzed for face stability to provide for the anticipated variations in the ground conditions.

The final lining structure has been checked at five stations, including one where future MRT subway tunnels will pass below the finished Fort Canning Tunnel.

3.1 *Finite element analyses primary lining*

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 for the linings. The soil was modeled as an elastic-plastic material using the Mohr-Coulomb failure criterion; the linings are 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 assess the individual construction stages. Top heading excavation was modeled by softening the soil within the excavation limits of the heading. This approach simulates the excavation and

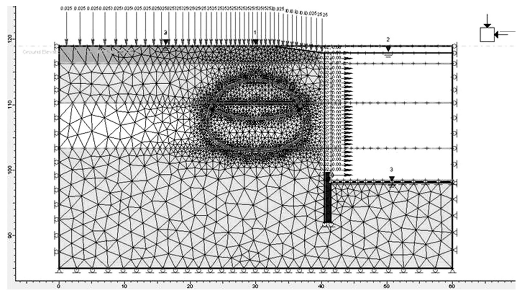

Figure 4. Finite element model (southern end).

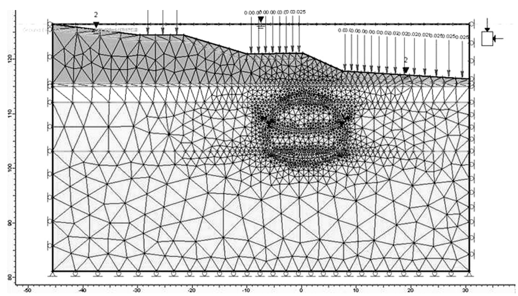

Figure 5. Finite element model (northern end).

resulting relaxation of the soil surrounding the tunnel prior to installation of the shotcrete support.

The commensurate increase of compressive strength and stiffness of the shotcrete lining was modeled by gradually increasing the stiffness of the lining elements. By this method, the interaction between the surrounding soil and the hardening shotcrete lining is simulated.

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

The shotcrete linings were checked for their integrity during all the intermediate construction and final stages. The structural capacity of the shotcrete lining was determined in accordance with British Standard BS8110 and Singapore Standard SS CP65.

In addition, a three dimensional final element analyses was performed in order 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 FE analyses.

3.2 *Face stability*

Face stability checks were performed at 10 m intervals along the alignment of Fort Canning Tunnel. As the residual soils are expected to display different behavior (described by the different Mohr-Coulomb shear

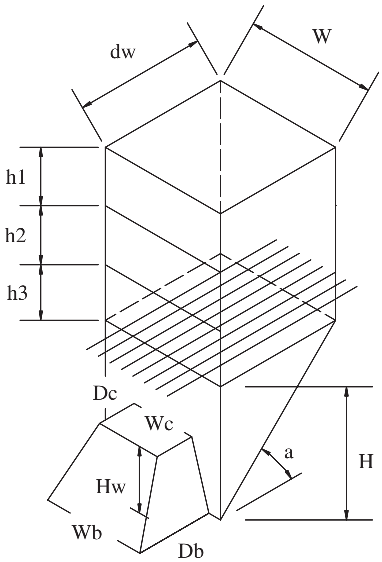

Figure 6. Face stability analysis, analyzed system.

parameters) in the drained and undrained conditions, all face stability checks are performed for both drained and undrained conditions.

The analyses were based on a sliding wedge by comparing "driving forces", against the "resisting forces", (for the analyzed system see Figure 6). The soil mass in question was assumed to be formed by two parts, a wedge in the tunnel face and a prismatic body that reaches to the ground surface above the wedge. Both of the bodies were checked against sliding failure. The upper prismatic body is supposed to be supported by the AGF umbrella, however, for the assessments the support provided by the AGF was ignored. The wedge in the face was assumed to be supported by the face stabilization wedge. Safety factors were computed for both bodies independently. Face stability was determined by the safety factor of the wedge against sliding out of the face.

As the actual pore pressure distribution in the failing body ahead of the tunnel face cannot be described to the required detail in this type of analysis, two extreme cases of pore pressure conditions, i.e. fully drained and fully undrained conditions, were analyzed. Hence, the analyses were based on effective stress principles. It is obvious that the actual pore pressure condition in the surrounding soil during tunnel construction is neither completely undrained nor fully drained, but rather on the path from undrained towards drained conditions. Hence, the actual pore pressure conditions and safety margins lie somewhere in between the two extremes "undrained" (with typically larger safety margins) and "drained" (with typically lower safety margins).

3.3 *Final lining*

Similar to the primary shotcrete lining, the secondary or final lining structure was analyzed using the finite element code Phase2, V5.04 by Rocscience, Inc. For these analyses the following loads on the final lining were taken into account:

- full overburden pressure,
- hydrostatic pressure with ground water table at surface level, and
- surcharge loading as per design criteria.

As per the relevant design criteria, the soil surrounding the tunnel structure is providing only subgrade reaction to the structure, and the soil is not assumed to yield any load carrying or distributing ability. The primary shotcrete lining structure does not carry any load during these analyses. As for the analyses of the load cases during the construction, triangular solid material elements were used to model the elastic response by the ground and beam elements were utilized for the permanent lining simulation.

Similar to the shotcrete lining, the structural capacity of the final lining was determined in accordance with the British Standard BS8110 and Singapore Standard SS CP65. Furthermore, serviceability checks for flexural, thermal and shrinkage cracking were preformed. The maximum allowable crack width for the final lining is 0.2 mm.

4 DESIGN REVIEW PROCESS

Throughout the design and construction preparation process the Client (Land Transport Authority of Singapore) has implemented a comprehensive design review process in order to ensure that the design and construction principles and methods meet the project design criteria.

Following the Nicoll Highway collapse in April 2004 a review of the soil conditions for the Fort Canning Tunnel project has been performed. During this rigorous review process the soil properties have been redefined to the effect that shear strength and stiffness values lower than previously anticipated had to be used in the design. The suggested pre-support system has not been changed. It has been requested, however, to alter some details of the intended excavation and support sequence.

The top heading excavation sequence had to be amended such that 1m top heading had to be immediately followed by the construction of 1 m temporary invert. The dimension of the intended face wedge remained unchanged. Furthermore, a more curved shape for the temporary invert has been requested by the Client.

Before the approval of the design, it has been agreed between the Client and the contractor, to build the

Figure 7. AGF installation.

Figure 8. Top heading excavation.

first twenty meter of the Fort Canning Tunnel as a trial section. Throughout the designated trial section, additional monitoring sections (two tunnel convergence monitoring sections within every 9 m long AGF umbrella section) were requested to confirm the predicted structural behavior.

5 TUNNEL CONSTRUCTION

5.1 *Pre-support installation*

The individual steel pipes of the AGF pre-support umbrella have a length of 12.5 m (42 ft). They are delivered and installed in four segments; the first pipe has a length of 3.5 m (11 ft), the others 3 m. Outer diameter of the steel pipes is 114 mm (0.45 in), wall thickness is 6 mm (0.24 in). As the pipe umbrellas overlap over a length of approximately 3.5 m in order to ensure a continuously effective pre-support, twenty AGF umbrellas have to be installed to cover the 180 m long tunnel.

Each individual AGF umbrella consists of 40 pipes drilled around the perimeter of the top heading; pipe spacing is 400 mm. In a first pass the odd numbered AGF pipes are installed at 800 mm spacing and grouted. In a second pass the even numbered AGF pipes are installed and grouted such that finally a spacing between individual grouted pipes of 400 mm is achieved. The grouting volume is limited by 85 kg per pipe at a maximum grouting pressure of 3 MPa. The grouting operation ceases once either of these values is reached. During drilling of the AGF pipes records of soil features such as the presence of hard boulders or particularly soft soil are kept.

Installation time for a 12.5 m of steel pipe is approximately 45 min to 1 h, resulting in an installation time for one AGF umbrella of approximately two working days including preparation time.

Figure 9. Bench excavation.

5.2 *Top heading excavation and support*

The top heading at Fort Canning Tunnel is excavated in two stages A (top heading) and B (temporary invert) (see Figure 3). The advance length for the top heading is 1 m and is typically excavated with an excavator (in soft soil) or a mechanized breaker (see Figure 8 and 9). Due to sub-vertical fissures in the residual soils the harder clay material typically breaks in blocks. The breaker is also used for excavating and trimming of larger boulders. The excavated material is mucked by either a wheel loader or dump trucks.

5.3 *Top heading construction*

Top heading excavation during the first 20 m of the tunnel (trial phase) followed the excavation and support sequence: 1 m top heading excavation immediately followed by 1 m of temporary invert excavation and shotcrete installation. The maximum distance between tunnel excavation face and the installed temporary invert was therefore 2 m to 3 m.

Following the excavation, all exposed soil surfaces around the perimeter of the tunnel and in the tunnel face are sealed with a 50 mm (2 in) layer of shotcrete (flashcrete).

During the next step in the construction sequence, the outer reinforcement and the lattice girder are installed. Following the installation of the first layer of shotcrete lining (approximately 200 mm – 8 in) the inner reinforcement layer is installed and the primary shotcrete lining is completed to required design thickness. All support elements are installed prior to commencement of excavation for the next increment in sequence.

At the stations where top heading excavation is interrupted for AGF pipe umbrella installation shotcrete face support is increased to a minimum 100 mm.

During the trial period it was established that the actual soil conditions in the tunnel heading are less critical than expected. In order to accelerate the tunneling progress, the excavation sequence was modified. First, two top heading construction increments (two times 1m excavation) were followed by one 2 m long segment of temporary invert construction. Later, this was altered to three top heading excavation rounds followed by one 3 m (10 ft) long segment of temporary invert construction. Additionally, the minimum distance between the excavation face and the closed temporary invert was increased from 3 m to 6 m (20 ft). With the described adaptations the production rate of the top heading was improved to an average of 1 m completed top heading per 12 h shift.

5.4 *Bench/invert excavation and support*

Bench and Invert of Fort Canning Tunnel are constructed in two immediately succeeding excavation steps. The advance length for one round of bench and invert excavation is 2 m (7 ft).

In a first step the bench (C, see Figure 3) is excavated and the temporary invert is demolished using a mechanical breaker and backhoe excavators. As for the top heading, a wheel loader or dump trucks are used for mucking of the excavated soil. Similar to the top heading excavation, a 50 mm (2 in) shotcrete layer is installed on the exposed soil surfaces. In the following step the outer reinforcement is connected to the top heading reinforcement by splice bars and the bench sections of the lattice girders are installed. Dependent on the soil conditions, the first layer of shotcrete may be installed in the bench before the excavation of the invert or it may be sprayed later in a continuous operation together with the invert shotcrete lining installation.

Figure 10. Invert excavation.

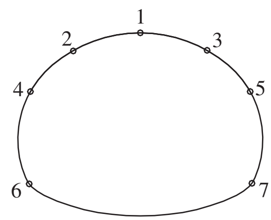

Figure 11. Tunnel lining monitoring arrangement.

Following the bench excavation, the invert portion of the tunnel is excavated (D, see Figure 3) and the primary tunnel lining is built up as previously described. 2 m excavation and shotcrete lining installation are generally performed within one working day.

6 MONITORING AND DESIGN

As an integrated part of the construction, an extensive monitoring scheme has been established at the surface and within the Fort Canning Tunnel. Surface monitoring points are arranged in arrays at 10 m intervals along the tunnel alignment. Surface monitoring points are concentrated in the vicinity of the centerline of the tunnel cross section with the remotest monitoring points outside the expected settlement trough.

Monitoring points within the tunnel to monitor the performance of the shotcrete lining are installed with a frequency of two monitoring cross section in every AGF segment (9 m – 30 ft) in the trial section and one monitoring cross section in every AGF segment (9 m – 30 ft) in the remainder of the tunnel. The arrangement of the monitoring points installed in the tunnel lining is shown in Figure 11. At three stations along the tunnel alignment an invert monitoring point is installed.

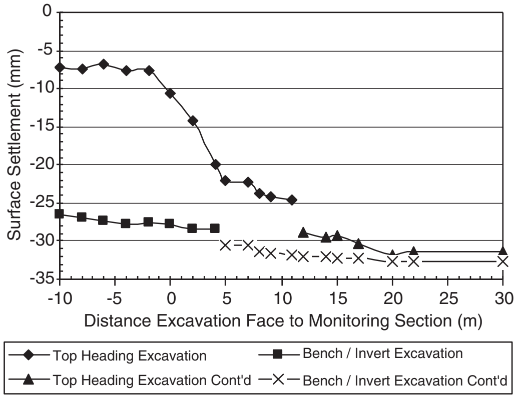

Figure 12. Surface settlements, 15 m along the alignment, 4.6 m overburden.

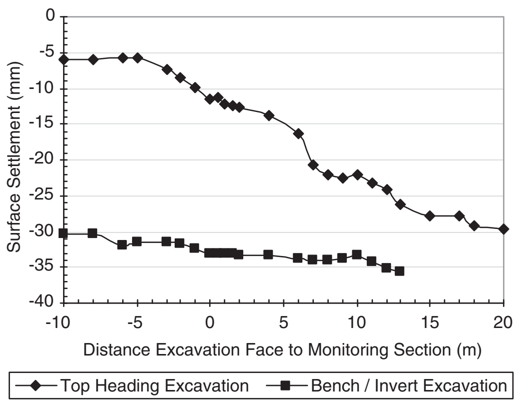

Figure 13. Surface settlements, 35 m along the alignment, 5.3 m overburden.

6.1 *Surface movements*

Surface settlements are monitored at monitoring arrays perpendicular to the tunnel axis at 10 m intervals along the tunnel alignment. Once any excavation face is within a plan distance of 15 m of a monitoring section, monitoring is carried out at a frequency of one reading per day until the surface settlements due to tunneling cease. After that, monitoring sections are read once every month for the remaining construction period. The monitoring accuracy established at the site is approximately ±2 mm (0.008 in).

Surface settlements typically start occurring 3 to 5 m ahead of the progressing top heading excavation face; deformations increase to approximately 8 mm (0.31 in) to 13 mm (0.5 in) once the excavation face reaches the plan location of the monitoring section. Typically, the pre-deformations increase as the overburden to the tunnel crown increases (see Figures 12, 13 and 14).

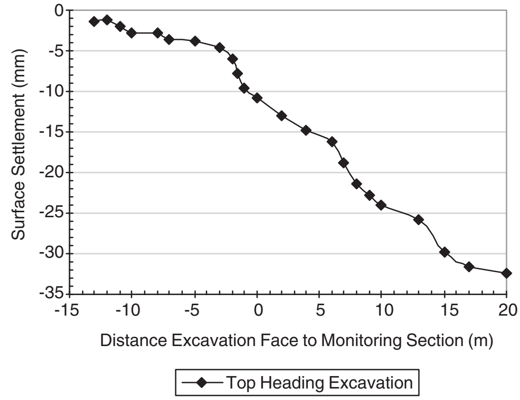

Figure 14. Surface settlements, 55 m along the alignment, 6.2 m overburden.

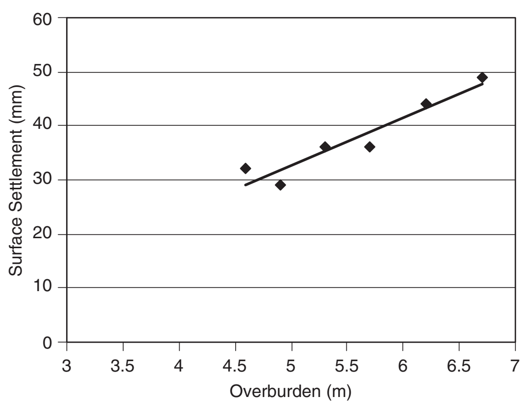

Figure 15. Surface settlement vs. overburden to tunnel crown.

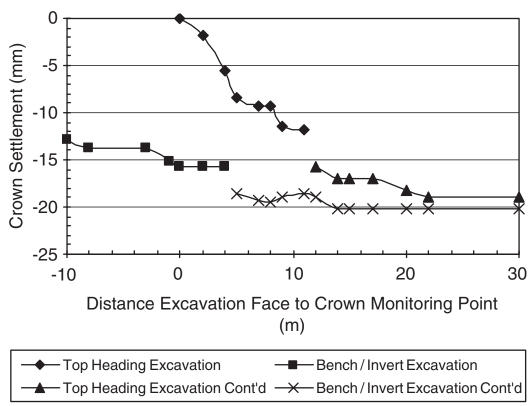

Figure 16. Tunnel crown settlement, 15 m along the alignment.

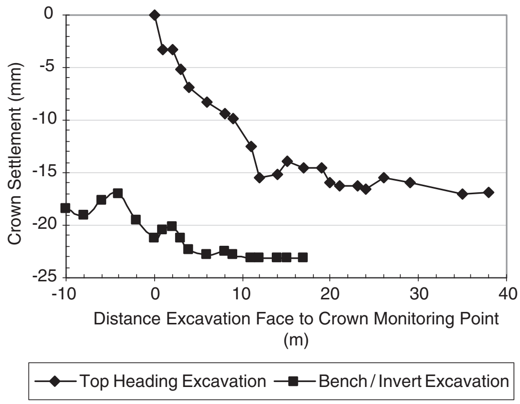

Figure 17. Tunnel crown settlement, 33 m along the alignment.

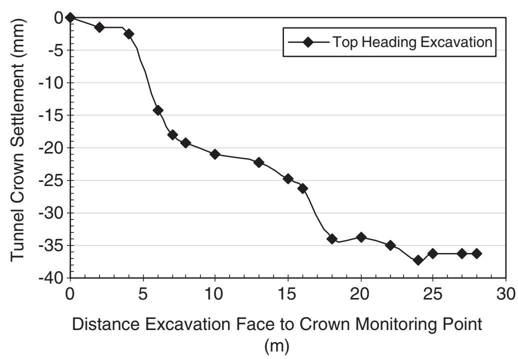

Figure 18. Tunnel crown settlement, 55 m along the alignment.

Surface settlements continue to increase as the excavation face passes the monitored section and cease once the progressing top heading excavation face is approximately 20 m (66 ft) beyond the observed monitoring location. During the combined bench and invert excavation, surface settlements increase by further 5 mm (0.20 in) to 7 mm (0.28 in). Overall, it can be observed that, due to the generally very shallow soil cover at Fort Canning Tunnel (20% to 60% of tunnel diameter), the surface settlements increase with increasing overburden thickness (see Figure 15).

6.2 *Tunnel lining deformations*

The monitoring points are installed and initialized immediately after the completion of the tunnel lining at the relevant advance. The monitoring accuracy for the tunnel lining monitoring established at the site is approximately ±2 mm (0.08 in).

The typical deformation pattern following top heading excavation and lining installation observed at Fort Canning Tunnel is a relatively uniform. Settlement of all five installed measurement points immediately after installation is uniform. Following the temporary invert installation, settlement at the top heading footings stabilizes at a level of approximately 10 mm whereas the crown of the tunnel lining continues to settle until it stabilizes once the excavation face reaches a distance of approximately 20 m from the monitoring section.

During the combined bench and invert excavation a uniform additional settlement of the top heading structure of roughly 5 mm (0.02 in) was observed. Overall, similarly to the surface deformations, tunnel crown deformations cease once the top heading excavation face has progressed to approximately 20 m beyond the monitoring section and the deformation values increase with increasing overburden. Divergence of approximately 10 mm (0.39 in) between monitoring points 4 and 5 are typically observed when the top heading support structure stabilizes; only insignificant increase in divergence is recorded during the combined bench and invert excavation.

6.3 *Measured vs. predicted deformations*

Ground parameters and other modeling assumptions for numerical simulations are typically chosen at a lower bound of the expected range which is to describe the behavior of the in-situ soil. The results of these simulations are therefore typically conservative estimates. Following the collapse at Nicoll Highway and the re-definition of the soil parameters, these values became even more conservative. However, the overall performance of the utilized numerical simulations is considered satisfactory for determining expected lining forces as well as the prediction of the deformation patterns of the soil mass.

Analyzing the numerical data and assessing these data in comparison with the actually measured in-situ deformations, it can be seen that the predicted pre-deformations ahead of the tunnel face were overestimated, i.e. the effect of the continues AGF pre-support was underestimated (see Table 1).

Similar conclusions can be drawn from the comparison of the predicted deformation and the measured deformations. Whereas the lateral extent of the deformation trough was very accurately predicted by the numerical analyses (with the limited ground cover, the width of the settlement trough is more a function of tunnel width compared to the overburden, and less of the exact match of the soil parameters), the magnitude of the surface and lining deformations is over-estimated (see Table 2).

Table 1. Pre-deformations ahead of the tunnel face.

	Pre-deformations (ahead of tunnel face)
Actual	5 mm to 13 mm (0.2 to 0.5 in)
Analysis (before revision)	15 mm (0.6 in)
Analysis (after revision)	45 mm (1.8 in)

Table 2. Surface deformations (until September 2005, tunnel construction 30% completed).

	Surface settlements
Actual	30 mm to 50 mm (1.2 to 2.0 in)
Analysis (before revision)	45 mm to 75 mm (1.8 to 3 in)
Analysis (after revision)	50 mm to 120 mm (2.0 to 4.7 in)

7 CONCLUSION

At Fort Canning Tunnel it could be demonstrated that by utilizing a continuous pre-support system like the AGF pipe umbrella, it is feasible to use NATM tunneling principles to construct a large tunnel under shallow overburden in soft soil. Furthermore, with the available numerical tools such as two and three dimensional finite element analysis, a realistic and reliable prediction of the structural behavior of the ground and the sequentially installed shotcrete tunnel lining is achievable.

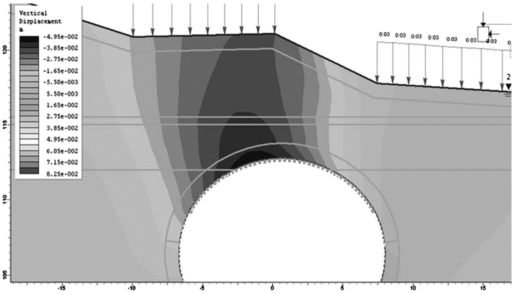

Figure 19. Predicted surface settlements.

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