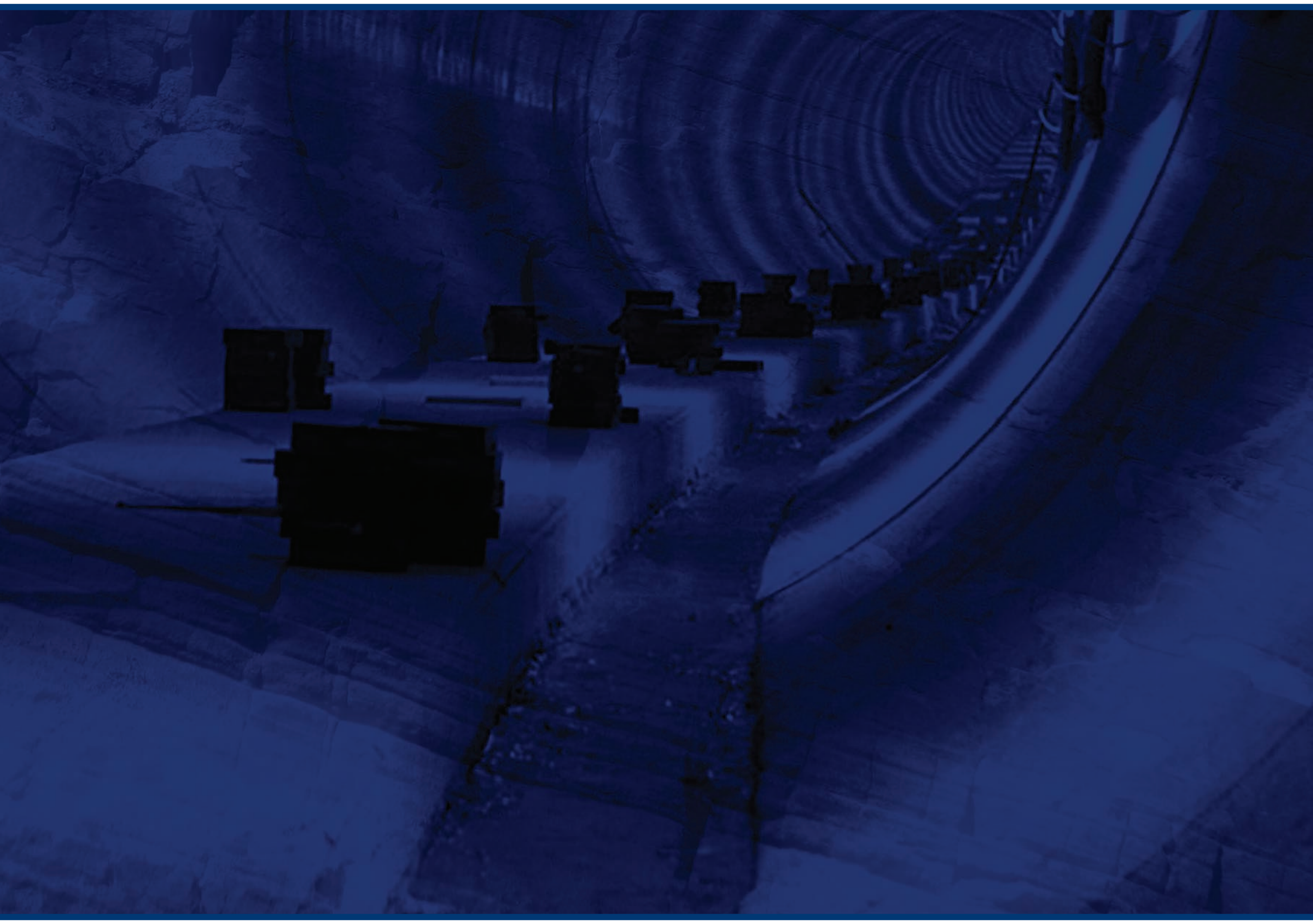
PROCEEDINGS

2018 AUSTRALIAN GEOMECHANICS SOCIETY VICTORIAN SYMPOSIUM

Geotechnics and transport infrastructure

Wednesday, 24 October 2018, 8:00am – 6:00pm
Rydges Hotel, 186 Exhibition Street, Melbourne



AUSTRALIAN GEOMECHANICS SOCIETY
VICTORIA CHAPTER

Global Synthetics

PREFACE

The Victorian chapter of the Australian Geomechanics Society invited academics and practitioners in the field of geotechnical and ground engineering to attend the 2018 Australian Geomechanics Society Victorian Symposium on 'Geotechnics and transport infrastructure' held on 24 October 2018.

In recent years Victoria has seen significant investment in transport infrastructure as part of a plan to manage the demands of a growing population and expanding urban fringe. The construction of Melbourne Metro, a second crossing of the Yarra River, rail and freeway upgrades as well as numerous level crossing removal projects are just some of the major transport projects currently underway in Melbourne and regional Victoria. Many of these projects carry numerous complex geotechnical challenges.

The 2018 Australian Geomechanics Society Victorian Symposium covers a variety of geotechnical challenges associated with transport geotechnics and present overviews of current infrastructure challenges, state of the-art practices, innovation, new research results and case studies demonstrating applications of advanced techniques and cost effective solutions in the construction and design of local transport infrastructure. The Symposium brought together professional engineers, researchers, specialist contractors, regulators, educators and students to share and discuss their experiences on the topic of transport infrastructure and associated geotechnical challenges and applications.

ORGANISING COMMITTEE*

Vladimir Lopez Suarez (Chair)

Daniel King

Richa Shukla Potdar

Erin Lee

David Glover

*a sub-committee of the AGS Victoria committee

SPONSORS

Platinum

Global Synthetics Pty Ltd

Gold

Chadwicks Geotechnics

Acciona Geotech

Silver

Geotesta

Foundation Specialists Group

TECHNICAL REVIEWERS

All technical papers in these proceedings, excluding the keynote addresses, were peer reviewed. The reviewers are acknowledged and listed below:

Daniel King (Editor)
Chris Coulson
Joel Gniel
Jay Lee
Jie Li
Nimal Nilaweera
Bhavikh Riyat
Ben Shannon
Sri Srithar
Manh Tran

AUSTRALIAN GEOMECHANICS SOCIETY

A technical society of ENGINEERS AUSTRALIA

Waterview Connection Southern Tunnel portal design and construction

N. Korte[1], M. G. Wansbone[2] and S. J. Cartwright[3]

[1] Principal Geotechnical Engineer, [2] Senior Geotechnical Engineer, [3] Senior Engineering Geologist, Tonkin & Taylor Ltd, 105 Carlton Gore Road, Newmarket, Auckland, New Zealand, emails: nkorte@tonkintaylor.co.nz, mwansbone@tonkintaylor.co.nz and scartwright@tonkintaylor.co.nz

ABSTRACT

The Waterview Connection project comprises 5 km of new motorway in urban Auckland, New Zealand (NZ). The new motorway connects the existing south-western and north-western motorways to complete the Western Ring Route and provide a direct connection between the central city and Auckland International Airport. The project includes twin 2.4km long, three lane tunnels up to 45m deep and retained portal approaches up to 29m deep. This paper focusses on the Southern Approach Trench (SAT) geotechnical design and performance during construction. The SAT was constructed in challenging geological and hydrogeological conditions, with these conditions dictating the design solution. A significant amount of temporary works requirements were built into the permanent structures, such as cement stabilised blocks behind the headwall to facilitate a safe TBM launch and retrieval and allowance for TBM loading on the headwall. Detailed geotechnical investigations, in-situ testing and construction observations and analysis of monitoring data during TBM launch and breakthrough at the Northern Portal facilitated improvements and optimisation of the permanent and temporary works design and will also allow design optimisation of future designs of this nature.

Keywords: tunnel, portal, headwall, retaining, TBM

1 PROJECT INTRODUCTION

The NZ Transport Agency's (NZTA) Waterview Connection is the largest and most ambitious roading project ever undertaken in New Zealand. The project that provides an alternative major transportation route by completing the "western ring route" that circumvents the congested Auckland southern motorway. The project includes 5km of multi-lane motorway, 2.4km of which comprises twin, 3 lane, bored tunnels. The motorway joins the existing SH20 motorway to the south and the SH16 motorway in the north, requiring four major multi-level motorway interchange viaducts. The project is located in the centre of suburban Auckland, with the tunnels passing beneath suburban rail, road, commercial and residential buildings. The 14.5m diameter tunnels were constructed using the world's 10th largest (at the time) tunnel boring machine (TBM) designed in Europe and manufactured in China. There are 16 mined cross-passages between the main TBM tunnels and retained portal approaches up to 29m deep.

The project was designed and constructed by the Well-Connected Alliance consisting of NZTA, Fletcher Construction, McConnell Dowell, Obayashi Corporation, Beca, Tonkin + Taylor and Parsons Brinkerhoff.

2 GEOLOGICAL & HYDROGEOLOGICAL SETTING

The underlying geology comprises East Coast Bays Formation (ECBF), which is a soft sedimentary rock. The ECBF is a division of the Waitemata Group which was deposited in an intra-arc Waitemata Basin ~21 m/y BP (Hayward and Smale, 1992). The predominant ECBF geology comprises interbedded siltstones and sandstones formed by turbidites and debris flow deposits in a bathyal setting. The ECBF have been uplifted and eroded and in the project area are unconformably overlain by alluvial and estuarine deposits of the Tauranga Group between 1.8 million years to 10,000 years old. An eruption 60,000 years ago from Mt Albert Volcano generated basaltic lava flows that mantle the landscape. The Oakley Creek has incised a gully along the edge of the basalt lava flow. ECBF is exposed in the creek and the tunnel alignment roughly follows the line of the creek gully.

A 3D model was developed of the ECBF rock surface and the base of the basalt. The tunnel vertical alignment was designed so that it was almost entirely in ECBF, below the Tauranga Group soils and basalt rock. Figure 1 shows the alignment of tunnels overlaid on a geological map. The geology at the Southern Approach Trench (SAT) can be described as a "toasted cheese sandwich", whereby fractured basalt rock (UCS of 25-120 MPa) is located at the surface with very weak ECBF rock (UCS of 1-5 MPa) at depth. The fine grained soils of the Tauranga Group form the cheese in the sandwich.

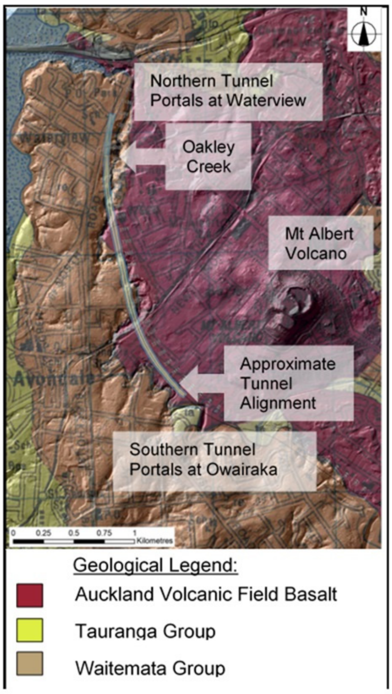


Figure 1 – Location of the Waterview Connection Tunnels in relation to the mapped geology by Kermode (1992) overlain on LIDAR ground surface

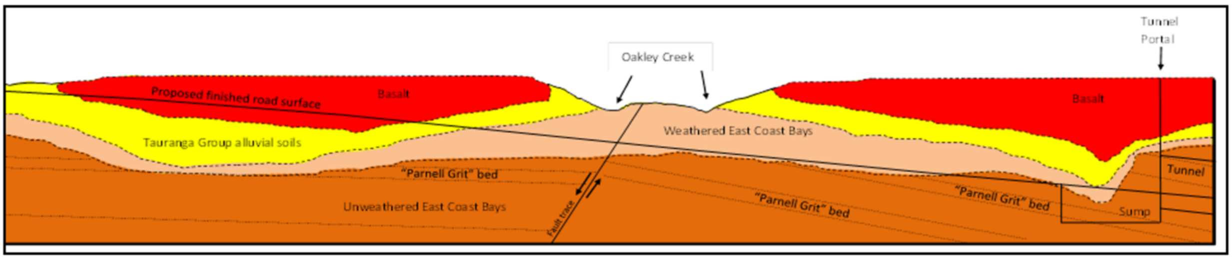


Figure 2: Geological long section through the eastern retaining wall of the Southern Approach Trench

Figure 2 shows the long-section stratigraphy located at the SAT. A soft paleosol exists directly beneath the basalt, which required careful consideration during design development. The RMR of the basalt rock mass varied from 60-80 in the top half of the unit where wide columns were predominant, to 40-60 in the bottom half where platy and horizontally jointed slabs was more common. Isolated highly vesicular and weathered rock forming vertical chimneys and inferred to be infilled gas escape structures (refer Figure 3) made for a highly variable rock mass during excavation. Interconnectivity of the jointing resulted in orders of magnitude higher rock mass permeability relative to the ECBF and Tauranga Group.

3 SITE CONSTRAINTS & DESIGN OUTLINE

The SAT is approximately 400m in length and at the tunnel portals the temporary excavation reached 29m depth at the base of the portal sump. Key features of the construction works are shown on Figures 4 and 5.
Key site constraints included:

- Complex geological and hydrogeological conditions, including an overlying basalt aquifer.
- Oakley Creek which cut across the portal trench and required diversion to the west of the portal as well as tunnelling directly beneath the stream on the Southbound tunnel, as shown in Figure 4 below.
- Residential housing to the immediate east and potential for damage by blasting vibration and settlement
- Sport fields to the immediate north which were required to remain operational during tunnelling operations which clashed with detailed surface survey monitoring required for the TBM launch.

Figure 3: Highly weathered and extremely weak vesicular water carrying basalt adjacent to strong, unweathered low vesicularity basalt

4 DESIGN CONSIDERATIONS

The design of the SAT and portal headwall had several significant design constraints, challenges and risks as discussed below.

4.1 Portal Position

As discussed in the section above, the geology at the SAT varied from basalt to soft Alluvium and soft rock of the East Coast Bays formation. The TBM was generally designed to tunnel through soft rock and any encounter with basalt would damage the TBM. It was therefore critical that the tunnel entrance needed to be located below the basalt. The proposed portal was located at a basalt infilled valley where the depth of basalt varied both longitudinally and transversely at the portal location.

Structural design of the tunnel lining the portal required the tunnel to be started with overburden cover of approximately 8m. This together with the tunnel dimensions and the need for a large stormwater sump at the portal entrance determined the overall portal depth. Optimisation of the portal position was undertaken to minimise the tunnel length, ensure that the TBM missed the basalt and at the same time kept the portal retained heights within reasonable heights.

4.2 Drained vs Undrained Portal

The project consent conditions placed a limit on the volume of groundwater that could be extracted from the SAT. The basalt was essentially an aquifer with the underlying alluvium acting as an aquitard and there was concern that excavation through the basalt would result

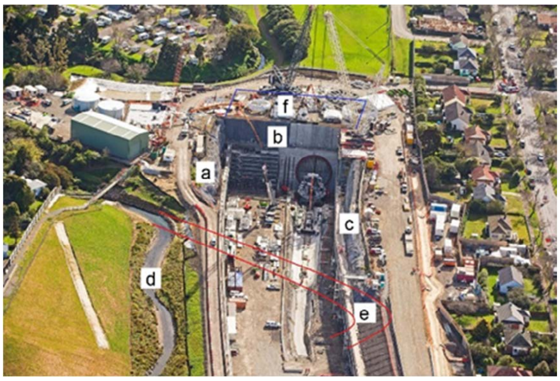


Figure 4: North aerial view of SAT on 7 August 2013. a) Western retaining wall RW909, b) Portal headwall RW910, c) Eastern retaining wall RW911, d) Diverted Oakley Creek, e) Original position of Oakley Creek f) Extent of stabilised block and MSE wall and crane loading platform

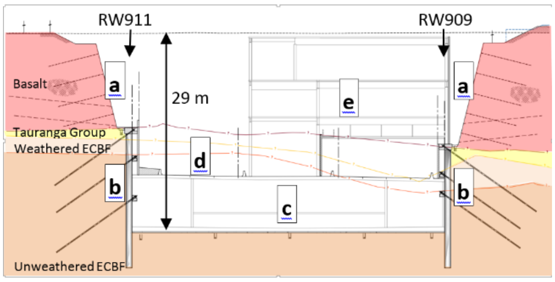


Figure 5: General arrangement of SAT, 20 m south of tunnel portal. a) Basalt cut, b) Anchored bored pile walls, c) Portal sump, d) Road deck, e) Southern Vent Building

in significant volumes of groundwater inflow and also associated settlement. For this reason the original consent design called for a partially "tanked" design solution with secanted piled walls and a grout curtain extending approximately 600m around the portal within the basalt. The grout curtain was identified as a relatively high risk construction activity as there was uncertainty over the ability to create an effective cut-off, the number of grout holes and the grout take required.

A detailed assessment of the hydrogeology of the area was undertaken, which indicated that the predominate regional groundwater flow in the basalt was in a north north-westerly direction (away from the SAT) and that there was a much less distinct gradient of groundwater from the centre of the basalt flow to the lateral extents of the flow, where the SAT was located. This lack of connection to the main water bearing flow was confirmed by short term pumping tests. Also by studying flow gauges along the Oakley Creek together with the geological model it was able to be established that there was no direct connection between the basalt and creek, which confirmed the theory of the flow within the basalt occurring along the centre axis of the basalt flow and away from the SAT (France et al, 2015).

2D and 3D groundwater flow modelling indicated that peak groundwater inflows into the SAT were likely less than $1000m^3$/day, which was within the consented water take limits and also that groundwater drawdown and hence settlements would not result in building damage. Modelling actually showed that the differential settlement for the proposed option without a grout curtain was improved as it removed the step in groundwater drawdown and smoothed out the settlement profile.

The SAT design proceeded on the basis of a fully drained portal, which resulted in a significant reduction in the number of piles required (secanted piles were changed to separated bored piles), significantly reduced loadings on the piles as full groundwater pressures where not required to be designed for as well as the removal of the grout curtain. Overall this was estimated to save approximately $5M.

4.3 Portal Headwall Retention Design

Several design, construction and site constraints had to be accommodated in the portal retention design. As shown in Figures 2 & 6, approximately 10-12m of basalt was present above the portal, with alluvial and residual soils extending into the driven portal itself. A primary challenge was the requirement to retain and/or improve the soil in the portal face. Ground improvement techniques from the ground surface, such as jet grouting, were considered but found to be uneconomic due to the presence of basalt and the depth of treatment required. Ground anchors and steel reinforcing could not be used in the circular face of the TBM. Retention with glass fibre reinforced polymer (GFRP) reinforced piles, or GFRP soil nails were considered, but discounted due to the potential for complications/damage to the TBM tunnelling through such materials.

The solution was to over-excavate the basalt behind the portal face, exposing the soils beneath the basalt. From this level at the base of the basalt, a series of partially interlocked weak concrete (5-15 MPa) piles were installed, mainly using a continuous flight auger piling rig to create a block of improved ground. The area of over excavated basalt was then backfilled with an MSE wall, buttressing the basalt cut. The piled block was designed to resist earth pressure loads in two ways, both as a gravity wall and to horizontally distribute load into the adjacent anchored reinforced concrete (RC) piles. This enabled the concrete stabilised block and basalt excavation to be minimised.

In order to act as a gravity wall, the improved ground needed to be able to perform as a coherent block. One potential issue with a piled block with contiguous (non-overlapping) piles only is that vertical planes of weakness are created between adjacent rows, reducing the overall strength of the block. For this reason the piles needed to be interlocked to tie each row together. Shear forces could then be transferred via the cold joints between adjacent interlocking piles. Another reason the piles were interlocked was to provide a stable crown above the TBM during launch to minimise the loads on the TBM shield.

Steel fibre reinforced CFA piles were used in first 3 rows of piles along the portal face to prevent spalling of the otherwise unreinforced concrete during excavation when the face was subjected to tensile stresses.

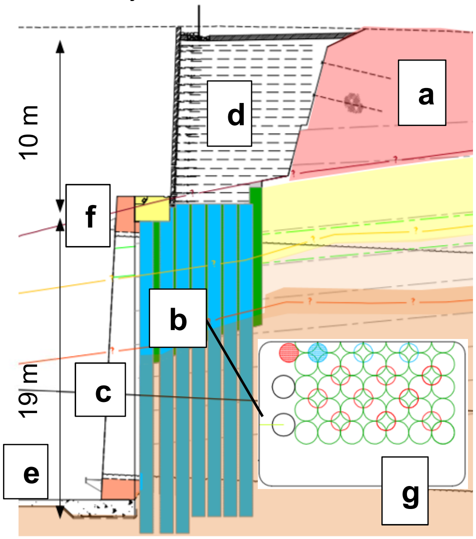


Figure 6: Section through portal headwall showing features in order of construction sequence. a) Basalt excavation and temporary bolting, b) Weak concrete piled block c) Excavation to maximum depth in front of piled block undertaken concurrently with d) MSE Wall, e) sump base slab and, f) RC headwall. g) Plan layout of the interlocking weak concrete piled block.

Figure 7: Continuous Flight Auger (CFA) piling rigs forming the stabilised block behind the tunnel portal headwall.

As the sidewalls were discrete bored piles, the in plane lateral capacity of the sidewalls was limited and the headwall was therefore designed to be structurally independent of the sidewalls. A layout of the piled block is shown in Figure 6 and the CFA rigs in operation in Figure 7.

4.4 Loadings & Analysis Methods

Analysis of the portal headwall retention was required to assess the overall stability of each individual tunnel portal location as well as overall portal excavation, as well as to determine loadings on the structural elements of the headwall for structural design.
Loadings considerations included:

- Earth and groundwater pressures
- Assembly of the TBM, including crane loading and lifting of the TBM cutter head and main drive (up to 390 tonne lift) which was undertaken from a purpose built crane platform directly adjacent to portal headwall.
- Imposed loads on the headwall due to TBM operations during launch and retrieval including both directly by bentonite slurry and grout pressures on the sealing rings and indirectly via shear and normal stresses between the TBM and ground.
- The potential for TBM bentonite or grout pressures to "leak" into the surrounding ground and pressurise the headwall over a wider area.
- Seismic loading for a 1 in 2500 year event earthquake.

Stability analysis of the tunnel portals was initially carried out using two dimensional (2D) limit equilibrium methods. This type of analysis indicated relatively low factors of safety for sections analysed through each portal location in the location of the soft headwall retention. However, this failure mechanism did not take account of 3D effects which were considered likely to have a strong influence on the stability. Accordingly, 3D limit equilibrium analysis, using method of columns, was undertaken, which confirmed significantly higher factors of safety.

Due to the analytical uncertainty associated with the method of columns and due to the need to assess loading onto the structural components of the headwall, a 3D finite difference model using the software FLAC3D was also developed of the portal. This model was utilised to confirm the overall stability of the tunnel portal and estimate design loads for the primary structural retention elements. It also enabled the analysis of potential loading scenarios from TBM operations.

4.5 Basalt Stability Overlying Softened Alluvium

One of the more unusual features of the design of the SAT is the open excavation to the base of the thick basalt deposit where the alluvium is daylighted, temporarily without the support of the piled wall.
The strength of the paleosol directly below the basalt was logged in earlier geotechnical investigations contact to be a soft to firm (Su of 12-50 kPa) black organic soil. Using the 3D model to anticipate the base of basalt, the "soft" layer was targeted during drilling and carefully exposed without penetrating into the underlying soils. A seismic dilatometer (SDMT) was then attached to the end of the drill string and pushed into the contact to measure stiffness and, by empirical correlation undrained shear strength (Su) was obtained. Samples were also collected for organic, Atterberg and triaxial testing. The investigation results concluded the contact to be much stiffer than previously logged and almost devoid of organics. The range of Su measured with more careful investigation techniques showed strengths ranged typically between 40kPa and 70kPa. SDMTs penetrating into the Tauranga Group also suggested some overconsolidation of the soils as a result of the basalt overburden.

The analysis of feasible failure mechanisms in the short and long term was of importance in the overall design philosophy. A conventional bearing capacity calculation, with the basalt considered as a uniform surcharge load on the alluvium indicated low factors of safety depending on assumptions regarding the strength and state of drainage of the alluvium. However due to the nature of the basalt rock mass (specifically the persistence and waviness of cooling joints), significant dilation of the basalt would be required for such a mode of failure, as the basalt would need to displace vertically downwards. A more likely failure mode is that of sliding on the upper surface of the alluvium where a block of basalt would only need to fail in tension (which could then lead to a bearing failure once the basalt is "pulled apart"). Figure 8 shows a diagram of these postulated failure modes. The condition of the uppermost portion of the alluvium, as well as the geometry of the contact was therefore of primary importance.

The effort undertaken both in characterising the properties top of alluvium, and of modelling the paleo-

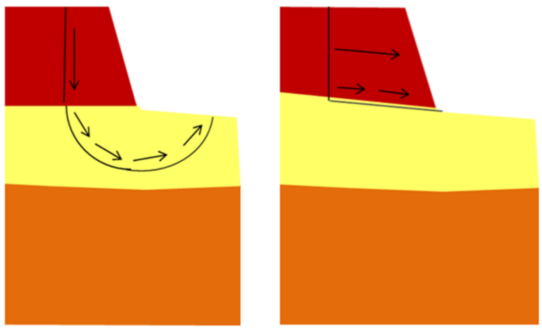


Figure 8: Postulated failure modes in basalt over alluvium. a) 'Bearing' type failure requiring dilation of cooling joints. b) Sliding failure on alluvium with tensile failure in basalt.

Figure 9: Contact between basalt and Tauranga Group Alluvium exposed in short sections prior to concrete mass block stabilisation. Note the stiff nature of the soils evident by the stability of the vertical cut.

topography as described above was of great use in assessing this potential mode of failure. A photograph of the geological contact is shown in Figure 9. The stiff strength of the alluvial soils is reflected in the good stand-up time of the vertical side walls of the excavated slot.

Based on the result of this assessment the basalt excavation along the side walls of the SAT was able to be constructed without any engineered support elements for global stability which would have been expensive to implement.

4.6 Optimisation of Temporary Retention Design during TBM breakthrough

The "during construction" design of the southern portal headwall was based on assumed TBM operational ranges for face and grout pressures. This included an allowance for the TBM to operate at full "Earth Pressure Balance" face pressures up to the back face of the headwall if required. For this reason, the design of the headwall included a temporary steel reaction frame erected against the headwall face.
The construction sequence for tunnel construction meant that the breakthrough of the TBM at the southern portal was the last part of the mainline tunnelling operation. Over the course of the tunnelling operation, and in particular during the tunnel breakthrough at the northern portal, knowledge was gained as to the appropriate operation of the TBM. As a result the TBM operating range for the southern portal breakthrough was able to be refined such that a reduction in overall maximum face pressures as the TBM approached the southern portal were able to be confirmed.

With this updated information, the portal headwall was able to be reanalysed for lower loads, and the use of a reaction frame omitted for the breakthrough. This change allowed significant programme and cost savings for the TBM breakthrough and disassembly process.

4.7 Support of Crane Platform

In order to minimise the lift radius for the major TBM components during assembly, a temporary crane platform was constructed above the eastern SAT sidewall, together with a smaller auxiliary crane platform to support the cranes superlift counterweight. The crane carried out several major lifts, with the largest being the 390T TBM main drive. After assembly, the crane platform was removed and rebuilt on the western side on the SAT for disassembly of TBM after the completion of tunnelling. The crane platform was supported on one side by columns founded on the sidewall capping beams, and on the other by a footing on the basalt. Figure 10 shows a photo of the configuration. The loadings from the crane platform did not significantly affect the side wall design, although the embedment lengths of the relevant piles were lengthened slightly for vertical capacity.

The basalt cut face under the crane platform footings was supported by a number of multi-strand anchors (pre-loaded to approximately 200 T) to reduce the risk of defect controlled failures in the rock mass due to the crane platform loading. The ground anchors were founded in the basalt itself, beyond the vicinity of the crane platform footing. Generally the anchors bearing plates were able to bear directly onto the basalt face, with some levelling concrete used to infill depressions in the face. For areas of weathered or highly fractured basalt (refer Figure 3), a RC bearing pad was used to distribute anchor loads over a wider area of face.

4.8 Instrumentation

A large number of monitoring instruments and methods were used to confirm the adequacy of the SAT design during construction including:

- Multi-levelled vibrating wire piezometers
- Inclinometers within the retaining wall piles
- Surface movement marker pins around top and base of the excavation to monitor settlements and heave
- Real time deflection monitoring of retaining walls during TBM launch and retrieval
- Ground anchor load cells

4.9 Overall Performance

Monitored settlements were generally less than that predicted by modelling (up to approximately 20mm compared with greater than 50mm predicted).

Figure 10: TBM cutterhead being lowered into position by crawler crane on platform

Essentially no settlement attributable to groundwater drawdown was observed as a result of the excavation.

"Mechanical" lateral deflection, due to deformation of the portal excavation was significantly less that predicted. This is considered to be attributable in part due to the basalt performing better than had been assumed in modelling. Zero tensile strength in the basalt had been conservatively assumed and, as a result, the modelled load on the portal retention due to the basalt overburden was significant. However during construction there was very little displacement of the basalt rock mass – less than 10mm.

Deformations were generally greater in areas without basalt overburden, and in particular in areas where ground anchors were omitted from the capping beam and were only installed at a lower level on the wall. Lateral displacements of up to approximately 30mm for the portal sidewalls were observed (in locations without basalt), compared with predicted displacements up to approximately 100mm.

There was no significant increase in load or deformation on the headwalls during launch or retrieval. Reasons for this included relatively low face pressures, particularly immediately prior to breakthrough, and relatively low grouting pressures, as a result of learnings made with operation of the TBM. In addition, the friction loads on the shield were significantly lower than had been designed for. One reason for the low friction loads is likely to be the ability of the weak concrete interlocked block to span above the TBM shield, preventing "squeezing" of the TBM. This was demonstrated during the launch of the TBM by data from an extensometer that was installed into the weak concrete block immediately above the excavation line of the TBM. This extensometer showed very low levels of vertical displacement as the cutterhead and TBM passed beneath (<1mm).

5 CONCLUSIONS & LESSONS LEARNT

1. Undertaking additional detailed geotechnical investigations and in situ testing at the SAT allowed greater understanding of the geological and hydrological conditions. This allowed risk based decisions to be made to optimise the design and allowed cost savings to be made by changing the design philosophy from a partially "tanked" undrained portal to a fully "untanked" drained portal. This allowed for significant cost savings to be made in the portal design in changing from secanted pile solution to a separated bored piles, reducing the loading on the piles and also allowed the removal of the grout curtain.

2. Detailed monitoring of the TBM operation and loadings applied by the TBM on the Northern Portal were used to revisit the design of the temporary thrust frame at the SAT. This resulted in significant cost and programme savings by allowing the complete removal of the thrust frame on the return breakthrough at the SAT.

3. The overall performance of the SAT excavation was significantly better than modelled during design. There are two areas in particular that are considered to merit consideration in terms of optimisation of future similar designs:

- There is potential for a similar headwall to be designed with lower overall structural demands providing the TBM operational loads are able to be defined more accurately at the design stage.
- Where significant strong rock layers such as basalt overly weaker soils that require retention, allowing for the strength of the rock layer to redistribute loadings into the retained soils is likely to be of benefit in reducing demands on the retention system. Consideration should be given to what reliable levels of tensile strength can be assumed as part of the design.

4. The steel fibre reinforced weak concrete piles were able to be excavated by the TBM without difficulty, however the use of steel fibres did cause some issues with the efficient operation of the machine initially. In particular the steel fibres got caught on the conveyor and made cleaning out of the machine more difficult.

5. The Alliance delivery model was a key component to the successful delivery of the project in challenging geotechnical, environmental and social environment. The Alliance structure allowed true collaboration between the construction and design teams, best for project decisions to be made. In particular it allowed for risk based decisions to be made during design to take advantage of opportunities and for monitoring information obtained during construction to be used to modify the design resulting in cost and programme savings.

6 ACKNOWLEDGEMENTS

The writers would like to thank the New Zealand Transport Agency for permission to publish this paper. The writers would also like to acknowledge the contribution of the other Well-Connected Alliance participant organisations for the roles they played in delivering this successful project.

REFERENCES

France, S.J., Williams, A.L. (2015). "Waterview Connection: Environmental impacts of a deep drained trench"

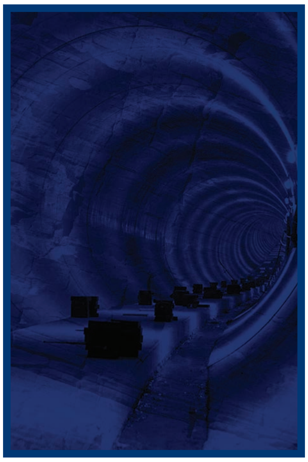
Hayward, B.W, Smale, D. (1992). Heavy minerals and the provenance history of Waitemata Basin sediments (early Miocene, Northland, New Zealand). Journal of Geology and Geophysics, 35:2, 2123-242

Kermode, L.O. (1992). "Geology of the Auckland Urban area, Scale 1:50,000". Institute of Geological and Nuclear Sciences Geological Map 2.

Well-Connected Alliance (WCA) 2014 "Southern Portal Headwall Stage 2 Design Report", Rev D..

2018 AUSTRALIAN GEOMECHANICS SOCIETY
VICTORIAN SYMPOSIUM

Geotechnics and transport infrastructure



AUSTRALIAN GEOMECHANICS SOCIETY
VICTORIA CHAPTER