NORTH STRATHFIELD RAIL UNDERPASS A ONE PASS SYNTHETIC FIBRE REINFORCED SHOTCRETE LINING FOR A VERY SHALLOW COVER TUNNEL

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ABSTRACT

The single track North Strathfield Rail Underpass (NSRU) consists of two dive structures either end of a 148m long driven tunnel. The underpass is for diesel hauled freight trains up to 1.5km in length. The tunnel excavation dimensions are 7 to 8m in height and 9m in width with has a horseshoe shaped profile. The permanent tunnel structural lining consists of synthetic fibre reinforced shotcrete. The ground cover over the crown of the tunnel varies between 2.5m and 3.5m. The ground profile is predominately Ashfield Shale with graded weathering from the surface to fresh shale. Finite Element analysis, calculated the stresses in the shotcrete lining, was also used to assist in predicting surface settlements. Apart for the initial steel canopy tubes no other steel support is installed in the driven tunnel (a first for civil transport tunnel in Australia). A repeated grid pattern of thirty-five grouted 12m long fibreglass dowels ensured tunnel face stability. The tunnel face was mapped daily. The 12m long canopy tube array installations are staggered relative to the 12m long face dowels by 4.5m. The excavation/shotcrete support cycle advanced in increments and the next cycle cannot commence until the initial 150mm thickness of shotcrete has reached an early strength of 6MPa. Early strength measurements of the shotcrete are a vital part of the construction. At the tunnel face, to support the train live loads, there are three levels of redundancy, the canopy tubes, the shear capacity of the ground slot to the surface and the structural/deflection capacity of the rails. The synthetic fibres in the shotcrete provide shrinkage crack control, residual strength if cracking occurs due to deformation and enhanced durability of the tunnel lining compared any alternative using steel such as steel lattice girders(and with no electrical stray current issues). Both the macro and micro synthetic fibres (the latter in the final 100mm fire protection layer placed over a spray-on waterproofing membrane) will reduce potential fire event related shotcrete spalling. Surface settlement minimisation relies on the construction methodology with the shotcrete over the arch always being very close to the tunnel face not allowing the ground to relax. Real time surface settlement monitoring was carried out using robotic scanning theodolites aimed at reflective prisms. In tunnel monitoring included convergence taping and optical targets. Excavation of the driven tunnel commenced in February 2014 was completed in late August.

1. INTRODUCTION

The NSRU project is designed to grade separate south bound diesel haled freight trains from the electrified suburban rail network north of Sydney at North Strathfield Railway Station. The freight line will pass under three heavily trafficked suburban railway lines and one line currently not in use. The original concept for the underpass included a cut and cover tunnel that would take between three and five years to construct due to the very limited number of track possessions available during any one year. The subsequent reference design stage, developed a driven tunnel option which was further refined during the detailed design phase to have a shotcrete final lining reinforced with synthetic fibres. During the reference design the underpass was also moved northwards 60m to take advantage of more favourable geological conditions. The shallow cover over the driven tunnel necessitated that the tunnel design and construction methodology minimise surface settlements so that there would be no speed restrictions placed on the operation of the existing train services. This was achieved using a combination of canopy tubes and the placement of the shotcrete lining as close as possible to the excavated face of the tunnel. The final driven tunnel length was 148m with the ground cover between 2.5m and 3.5m. The tunnel excavated dimensions were around 8m high by 9m wide. No disruption to train services occurred during the construction of the tunnel. This paper describes some aspects of the design development from the reference design stage through to the detailed design and other minor changes made during the construction phase. An unforeseen 800mm wide dyke was also intersected by the tunnel, though there was some speculation before construction commenced that an anomaly could exist about half way along the tunnel length.

To ensure that tunnelling operations did not impact upon the integrity of existing track infrastructure, a need was identified to design and implement a 24 hour a day, 7 days per week Automated Deformation Monitoring System (ADMS). This was also required to operate within existing client maintenance specifications (Gonzalez, et al 2014).

2. DESCRIPTION OF PROJECT

The NSRU is one of the first projects to be undertaken under the Northern Sydney Freight Corridor Program. This program includes a number of infrastructure projects to improve freight and passenger rail services along the 155 kilometre rail corridor between Sydney and Newcastle, and is a joint federal and state government funded project under the Nation Building Program. The NSRU is being delivered by Transport for NSW(TfNSW) under an Alliance contract with the John Holland Group and Bouygues Constructions Australia. A design JV of Sinclair Knight Merz and Parsons Brinckerhoff were the lead designers with Mott MacDonald Australia as the driven tunnel designer.

A plan of the skewed driven tunnel traversing beneath the railway tracks is shown in Figure 1 and a section schematic in Figure 2.

Figure 1: Tunnel plan alignment.

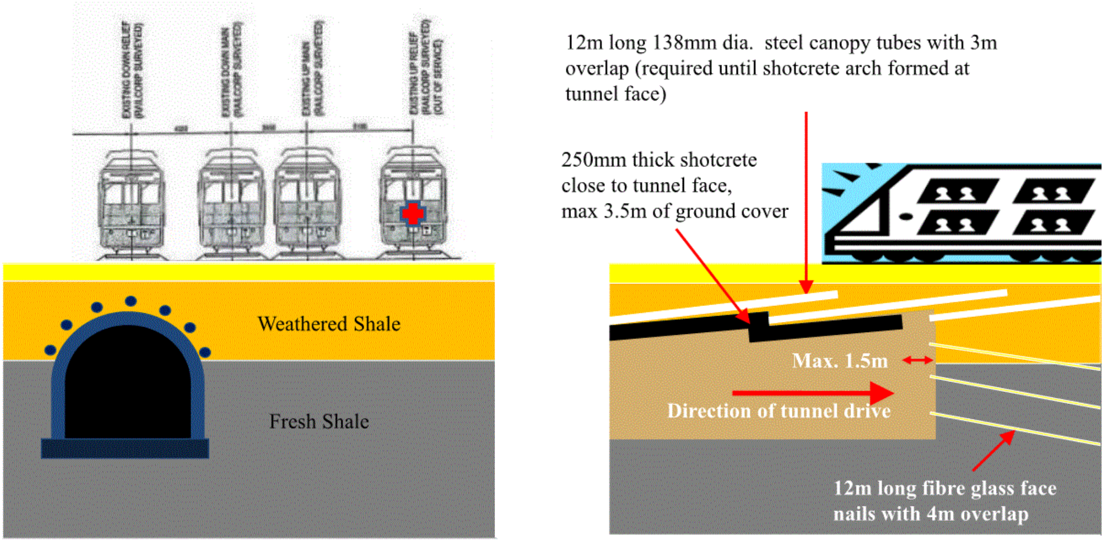
The project involves the construction of a new rail underpass (dives plus driven tunnel) between North Strathfield Station and the Strathfield Junction which will allow the UP Freight movements to the Flemington Goods Loop to be provided via a route beneath the UP Relief, UP Main, DN Main and the DN Relief, thereby eliminating conflicting at-grade movements. The dive structures either end of the driven tunnel are around 350m long each. Trains using the underpass driven tunnel are travelling south down the northern dive across and out through the southern dive. The tunnel alignment (the UP Relief Line) is on a reverse curve of 400m radius with a down gradient on the north dove of 2.8% and on the south dive of 2.2%.

For the tender, the Alliance had developed their own tunnel design and construction methodology which differed quite significantly from the approach taken by the designer for the reference design(Nye, 2013). The reference design had also referred to lattice girders and the builder proposed light steel sets and continuous grout bags to make contact with the rock. The reference design had flagged the possible use of fibre only reinforcement in the shotcrete and not using lattice girders.

The final detailed design of the tunnel lining progressed on the basis of a synthetic fibre reinforced shotcrete without steel sets or lattice girders and this form of tunnel lining has now been successfully constructed.

Also the reference design was developed as a heading and bench excavation and it was anticipated that for such a short tunnel the builder would use or have available a small road header. For efficiency of the construction and to allow large plant to pass each other in the tunnel the Alliance requested that the tunnel be a full height heading excavation and the tunnel at least 1.5m wider. Additional calculations were made by the designer to confirm that this would have minimal additional impact on the predicted surface settlement. Another advantage of a full height heading was the seamless shotcrete lining over the full tunnel profile adding to its durability and lower permeability. This construction change had no impact on the final structural lining shotcrete design thickness which was 250mm.

The reference design driven tunnel was 170m long. This was reduced to 148m by allowing the piling works in the southern dive structure to be constructed closer to the operating railway line.



a) Tunnel Cross Section

b) Tunnel Long Section at Tunnel Face

Figure 2: Schematic Cross section of tunnel and railway tracks, 3.5m maximum ground cover. Tunnel traverses the tracks on a skew.

A proposed 25mm passive fire protection layer in the driven tunnel reference design was removed and replaced with a final 100mm thickness shotcrete layer (increased from 75mm thickness) and containing micro fibres to reduce spalling of the shotcrete. This final layer of shotcrete also will protect the spray-on waterproofing membrane. The tunnel lining has been designed to resist without collapse, a 4 hour duration hydrocarbon fire even allowing for a significant loss of the overall lining thickness. The design fire size had been increased significantly from that used in the reference design.

3. GENERAL BACKGROUND DISCUSSION

Before launching into the details of this project paper it is important to understand some of the drivers to its design and the construction method adopted.

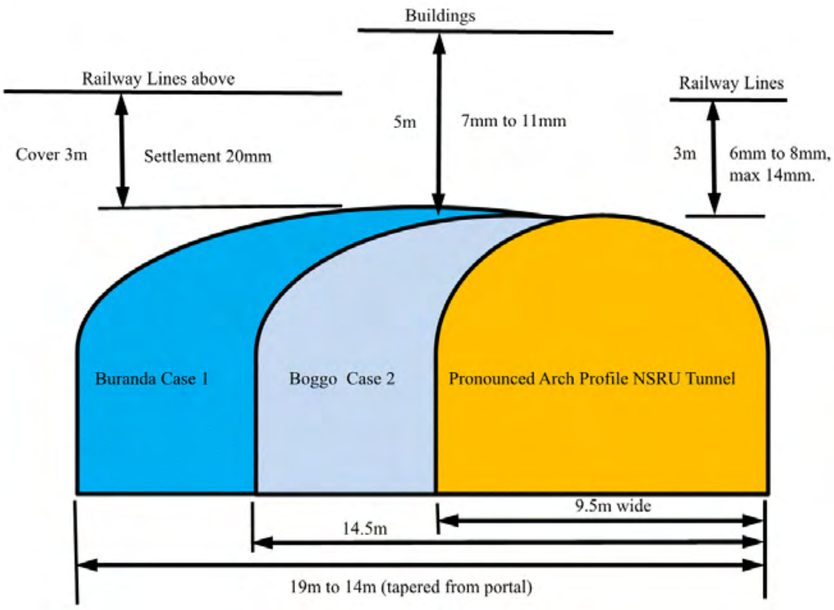


Figure 3: Comparing previous shallow cover tunnels with the NSRU tunnel.

Firstly, as shown in Figure 3 the NSRU tunnel is very shallow but compared to the two Brisbane tunnels (Buranda and Boggo), also excavated in weak rock, the tunnel is narrower and more importantly has a pronounced curved profile over the tunnel crown. The Boggo Road Busway Tunnel had lattice girders embedded in shotcrete as the temporary support, and this was then followed by an in-situ concrete lining for the permanent support. Shotcrete spray trials on lattice girder sections were conducted at the start of the project to assess whether the shotcrete and lattice girders could be used as permanent support. Figure 4 are a series of photographs taken from these trials at Boggo Road. It can be readily seen that voids have formed around the steel lattice girder elements within the shotcrete, which in the long term would be expected to result in steel corrosion followed by shotcrete spalling. The formation of these voids is accentuated by the use of a curing accelerator in the shotcrete which is necessary for its overhead sprayed application in a tunnel.

Figure 4: Evidence of shadowing of shotcrete (when used with accelerator) around steel lattice girders.

The curved profile of the NSRU tunnel is important to ensure that the arch is basically acting in compression and that there are no bending moments and hence no significant tensile stresses in the shotcrete. The synthetic fibres used in the NSRU tunnel provide a reserve of flexural capacity but they were not expected to be mobilised and this was the case in practice with absolutely no longitudinal cracking evident in the crown of the tunnel. It would also appear that fibres negated any visual evidence of shrinkage cracking in the shotcrete.

The other issue at the NSRU tunnel (say compared to Boggo Road) was that the cover was so low that the idea of having an initial ground support (which would also have to negate all of the possible ground settlement) then followed by an in-situ concrete lining would in effect reduced the ground cover. There was simply not enough room to fit a thicker tunnel lining because of the constraints associated with the track vertical alignment and the track gradients.

4. SITE INVESTIGATIONS

Some of the site investigation boreholes and test pit excavations that were located within the rail corridor track zone had to be carried out during the long week-end track possessions in June 2011 and September 2012. Golder Associates carried out the site investigations on behalf of TfNSW with inputs from both the designers and their geotechnical consultant Douglas Partners. Additional boreholes were also drilled outside track possessions where they were not located in the rail danger zone.

The boreholes drilled on the track during the weekend possessions were drilled to a maximum depth of 11m using a small tracked drilling rig.

Visual inspection of the corridor at reference design during a track possession revealed surface rock outcrops on the east embankments extending from the southern end of North Strathfield Railway Station up to the Pomeroy Street Road Bridge. This was the prime factor leading to moving the tunnel alignment northwards from the original concept design cut and cover tunnel as it was anticipated that the geology improved together with a slight improvement in ground cover for a driven tunnel.

The site investigation failed to detect an 800mm wide dyke within a 2m wide shear zone that was intersected at an oblique angle during the tunnel excavation, Figure 5. Though not of great consequence to the project in terms of tunnelling progress, given that the tunnel construction methodology included canopy tubes that could accommodate ground changes quite readily, Figure 6. The interpretation of the groundwater levels discussed later may have identified this dyke, however, the presence of a dyke seemed a highly unlikely event even though the possibility was discussed

Test pits were excavated between sleepers in critical locations along the track to determine the depths of the ballast and the ballast sub-base during track possessions. After the award of the contract to the Alliance excavated additional test pits at the south end and west side of the tunnel as this was likely to be the worst near surface material.

Figure 5: 800mm wide dyke in tunnel face.

Figure 6: Canopy tube installation with shotcrete lining in foreground.

5. GEOLOGICAL MODEL

From the site investigation data it was possible by interpolation to develop a geological long section with cross sections taken through the tunnel at 10m intervals. The typical mixed face conditions consisted of extremely low), to low strength (Unit 1) Ashfield Shale in the crown, with low to medium strength shale (Unit 2) down to the tunnel spring line and high strength shale in the lower half of the tunnel (Unit 4). Above the shale bedrock there was an undulating layer of residual clay of variable thickness underlying fills and railway ballast. The residual soil thickness was very thin above the northern half of the tunnel as the railway is in cutting, and increases in thickness in the southern half as the railway transitioned into an embankment (refer to Table 1 for the Geological profile and parameters).

The geological long section indicated a fractured rock band (Unit 3) would be encountered near to the invert over much of the tunnel. The daily geotechnical mapping determined that the fractured rock band was in fact linked to thrust faulting associated with the dyke and shear zone, which was aligned nearly parallel with the tunnel. The series of thrust faults had lots of bedding shears due to the fault movements giving an increased fracturing in the face and floor. Apart from the dyke and shear zone that intersected the tunnel 60m from the north portal and suddenly terminated within the 2m wide shear zone at the 90m point, the geological interpretation has proven to be very accurate.

Table 1: Geological profile and parameters

#	Strata	Thickness	Description	UCS	Range E (MPa)
Material above the tunnel crown of the driven tunnel					
1	Ballast	500mm	Gravel and cobbles	-	100 - 200
2	Capping L-1	250mm	Clayey gravel/gravelly clay	-	50 -100
3	Capping L-2	250mm	Silty gravel/ sandy gravel	-	50 -100
4	Filling	0 to 1000mm	Mostly sandy gravel	-	60 -100
5	Residual Clay	0 to 1000mm	Mostly stiff to very stiff residual clays	-	12 - 20
Material intersected at the tunnel face of the driven tunnel				**Min. UCS (MPa)**	**Range E (MPa)**
6	Rock Unit 1	1m to 3m in the tunnel face	Extremely low to very low strength, fractured to highly fractured	0.5	100 - 200
7	Rock Unit 2	1m to 4m in the tunnel face	Low to medium strength, fractured	2	300 - 500
8	Rock Unit 3	0.5m to 2.5m in the tunnel face	Medium to high strength, fractured to highly fractured	7	700 - 1000
9	Rock Unit 4	3m to 6.5m in the tunnel face	Medium to high strength, unfractured, RQD > 70%	15 - 25	2000

The 800 mm thick dyke within a 2.0m wide shear zone was intersected approximately 58m from the north portal on the eastside tunnel wall. The vertical dyke was aligned obliquely to the tunnel at approximately 15 degrees to the alignment centre line. The dyke when first encountered was medium to low strength however it deteriorated rapidly when conjugate faults crossed the dyke (at 80m point) perpendicular to the tunnel alignment. The dyke consisted of stiff clay with a contrasting golden colour compared to the surrounding dark grey shale rock (Figure 5). The dyke sheared material was more susceptible to weathering and had weathered to a soft to firm clay in the tunnel crown and firm to stiff clay below the spring line.

An additional 23 fibre glass dowels where installed in the upper tunnel face at around the 79m point in additional to the standard 35 face dowels pattern that had just been installed at 76m point. A second round of 19 additional face dowels were installed in the upper tunnel face at the 89m point prior to the dyke finishing within the shear zone around 4m short of the west tunnel wall. Interestingly the dyke had little or no impact on the surrounding Unit 4 shale rock below the tunnel springline while a deeper more weathered zone developed in the Unit 2 shale above the spring line. The increased weathering on the uphill side of the dyke may be the result of wetting and drying effects on the shale rock as the groundwater finds its way around the weathered dyke which acts like a dam to the groundwater.

From a lining design perspective it was important that the Unit 4 shale be at or above tunnel spring line so that the walls of the tunnel would be constrained laterally. This point is discussed further in Section 6.

During the detailed design various sources of information were used in the kinematic assessment including a stereo net projection of the data from the Optical Tele-viewer and RaaX imagining data was produced of both the Common Joint & Fault Sets in Ashfield Shale Table 1 from Deep Excavtion in Shale: (Andrews & Braybrooke, 2001) and the Metro West Borehole 3103_130. The typical Joint Sets / Faults / Shear Zones / Dyke encountered during tunnelling have been summarized on Table 2 below and in Figure 7 DIPs 6.0 stereonet.

Douglas and Partners our geotechnical sub-consultant provided three potential rock planar and wedge failure cases based on their experience with Ashfield Shales. The three cases included:

- a small planar failure of a 45 degree wedge (**Minor Joint Set J6- 28/180**) occurring on the face, which was assessed using Rocplane to determine the bolt loads and Unwedge for the Tunnel shotcrete sidewalls;
- a large wedge occurring on the face and Tunnel shotcrete side walls from the intersection of two persistent major joint sets at 60 degrees (**Major Joint Set J2 - 60/020**) and 70 degrees (**Major Joint Set J3 - 70/290**) intersecting the face were assessed using Unwedge.;
- planar failure of a large persistent 70 degree wedge (**Major Joint Set J1- 85/185**) on the face, which was assessed using Rocplane along the length of Tunnel at 10m chainage intervals;

Table 2: Joint Sets / Faults / Shear Zones / Dyke encountered during tunnelling

Tunnel Bearing	*Defect Type*	*Dip*	*Dip Direction*	*Quantity Mapped*	*Persistence (m)*	*Peak Defect Property*		*Comments*
						Friction (degrees)	*Cohesion (kPa)*	
163˚ to 154˚	JM1	**89**	**020**	33	10	28	0	Joint Set with a single small Fault
	JM2	**85**	**343**	47	10	28	0	Potential for Large Planar Failure x 2
	JM3	**84**	**275**	90	10	28	0	Shear Zone with Dyke (800mm)
	JM4	**82**	**073**	38	10	28	0	Joint set with small shear zone
	With Minor sets							
	Jm5*	**76**	**310**	83	3	25	0	Small Fault where wedge broke out
	Jm6*	**37**	**018**	13	3	25	0	Shear Zone crossed Dyke
	Jm7*	**49**	**158**	16	3	25	0	Shear Zone / Thrust Faults
	Jm8*	**72**	**119**	26	3	25	0	Fault 4 east side wall beyond dyke
	Major Features							
	FT9	**41**	**269**	46	>100m	20	0	Thrust Faults 1&2 Aligned parallel to tunnel prior to Dyke
	B10	**10**	**091**	15	>10	27	0	Bedding
	Joint Swarm	**85**	**085**	8	>10	25	0	Joint Swarm at Southern Portal

Note * indicate minor joint sets

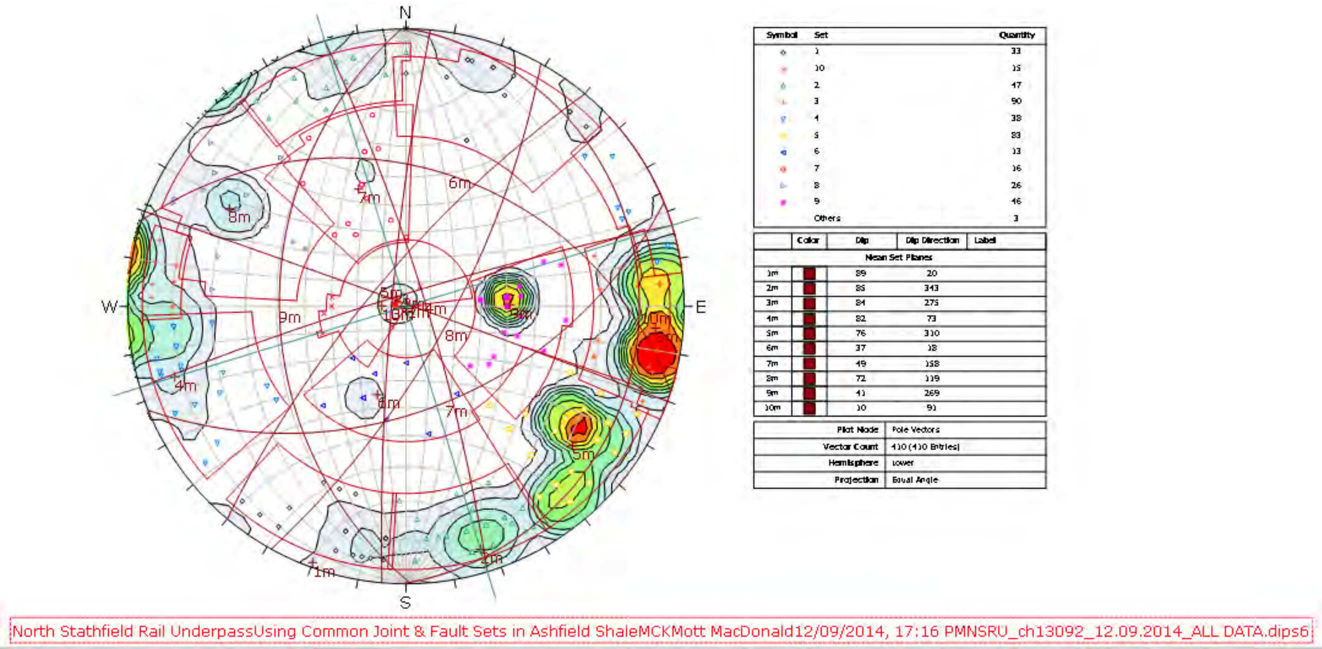


Figure 7: showing DIPs 6.0 stereo-net showing Joint Sets & Fault / Dykes encountered during construction

The daily geotechnical mapping assessed rock failure mechanisms for the tunnel during constructions which are summarized in Table 3. From the start of tunneling at the northern portal a series of thrust faults FT9 (41/269) were intersected that were aligned parallel to the tunnel alignment. The thrust faults caused lots of bedding shears that increased the amount of fracturing in the face and invert. Prior to encountering the 800mm wide dyke, joint set JM2 (85/343) caused the potential for two large potential planar failures at dip angles of around 70 degrees, which had been one of the Douglas and Partners potential rock planar expected failure mechanisms that had resulted in the requirement for 35 No. standard pattern face dowels.

Two rows of additional side wall dowels at 1.5m x 1.5m pattern spacing were installed into the side walls where the dyke and shear zone were entered and left the tunnel. Beyond the 2.0m wide shear zone with the 800mm wide dyke, a fault was encountered in the eastern wall Jm8 (72/119) which was also aligned near parallel to the tunnel alignment. The fault created a hanging wall and it was decided to continue the two rows of additional side wall dowels through the faulted section. The poorest rock conditions where encountered at the southern portal where the rock have been sheared to highly fractured rock by two intersecting shear zones (Joint Swarm (85/085) and JM3 (84/275)) creating a joint swarm.

Table 3: Typical rock failure mechanisms encountered during tunnelling

Geotechnical Domain	***Wall Azimuth (°)***	***Typical rock failure mechanisms***
Tunnel South Wall - Tunnel Face	341	A few large potential planar wedge were encountered **2m (85/343) and** small planar wedge **JM2 (85/343)** intersecting bedding shears**;** Wedge failures were typically **JM2 (85/343), JM3 (84/275)** intersecting **Jm5 (76/310)** and random bedding shears intersecting **Fault 1 & 2 (41/269)**; and Potential toppling failure **JM1 (89/020) and JM2 (85/343)** on the bedding **B10 (10/091**).
Tunnel East - Side Wall	253	Occasional over break on **Jm5 (76/310)** and planar failure **3m (84/275)**; Some small wedge failure on **JM3 (85/275)** intersecting **Jm5 (76/310)**;
Tunnel West - Side Wall	073	Occasional small over break on **JM4 (82/073)** Some small wedge failure on **JM4 (82/073)** intersecting **Jm8 (72/119)**;

Face stability is an important issue when constructing a shallow rock tunnel beneath multiple live railway lines. There are essentially three types of deformation mechanisms associated with the face stability and they are pre-convergence settlement that occurs ahead of the tunnel face (which extends out in front of the tunnel and may indicate poorer ground conditions ahead of the tunnel face); convergence that occurs at the face (settlement that occurs during excavation and prior to the ground support being installed) and extrusion of material in the face (face loss). If face stability is not maintained then it may lead to excessive settlements, spalling, large over-break, and potentially to a face collapse.

The stability of the tunnel face was maintained by installing 12m long fibreglass face dowels on a 9m advance sequenced with a 4m overlap. The 35 No. fibreglass dowels were installed in a standard pattern of 1.0m x 1.0m grid in four rows in the top half of the tunnel face, with 2 rows spaced a 1.0m x 2.0m grid in the lower half of the face, which was combined with a 50mm thick layer of shotcrete in the top half of the face. The 12m long fibreglass dowels effectively create a stiffened core of material ahead of the tunnel. In the more competent rock the fibre glass dowel provide support to key rock wedges.

The highly weathered faulted dyke material that was encountered at the 80m point comprising soft to firm clay in the tunnel crown and firm to stiff clay below the spring line had caused pre-convergence settlement ahead of the tunnel face. It was agreed to install an additional 23 No. fibreglass dowels that reduced the dowel spacing to 0.5m x 0.5m pattern across the face. The large number of dowels in the face effectively created a stiffened core of ground ahead of the tunnel face and this helped to reduce the pre-convergence settlement. Shotcrete was also spayed on the upper half of the face to seal the clays from the moisture and to reduce face loss from squeezing of the clays, and to prevent loosening of the highly fractured rock mass below the residual soils. The timely application of shotcrete onto the face and arch straight after the excavation finished also helped to reduce convergence settlement that was observed in the real time track monitoring prior to the support being installed.

Confining the face of the tunnel with the dowels and shotcrete helped to reduce the magnitude of surface settlement. However but this will not be significant compared to the overriding influence of the canopy tubes in combination with the stiff shotcrete lining which will closely follow the tunnel face excavation.

6. STRUCTURAL DESIGN OF TUNNEL LINING

The tunnel support design is closely aligned with the construction sequence described in Section 2 and shown in Figure 2b). The canopy tube array over the arch of the tunnel has to partially support the ground and live loading with the fibre reinforced shotcrete lining following very closely behind. The shotcrete is applied in layers starting with an initial thickness of 150mm at the tunnel excavated face and then up to 250mm with 3m of the tunnel face. The canopy tubes have to support ground up to a 2-m span (length wise along the tunnel) allowing for any unforeseen overbreak. The shotcrete taken up to the tunnel face over the arch and walls had to have reached 6MPa before the next excavation cycle of one metre in length can commence to provide both strength and stiffness.

The tunnel lining is what is called a one pass lining. Fibre reinforced shotcrete alone is the tunnel permanent support over the arch and tunnel walls. There is no requirement for lattice girders or steel sets (and as explained in Section 3 these would adversely impact on the lining durability). The tunnel lining behaves as a “confined arch” due to the confinement of the arch by the surrounding rock (ranging from Unit 1 to the better Unit 4) and therefore the compressive thrust in the arch will always be maintained close to the neutral axis of the shotcrete lining.

The two-dimensional plane strain linear elastic finite element analysis was used to model the tunnel lining as a continuation of the continuum of the surrounding rock mass. The compressive stress in the tunnel arch is reasonably uniform with a magnitude over the arch of around 4 MPa (this compares with the Ultimate Strength of the concrete in compression of 40MPa at 28 days). The maximum tension in the lining was 0.25MPa on the inner face in the crown of the tunnel and was easily within the capacity of the fibre reinforced shotcrete tensile capacity if it were to occur. The loading on the model included live loading with trains above and on adjacent tracks and combinations thereof.

In addition to the 2D FE analysis we also carried out 3D FE analysis with train loadings over the tunnel using STRAND7. The results were not included in the design reports for this project submitted to the Alliance or to TfNSW. They proved, however, that the lining stresses predicted by the 2D FE models were conservative.

The canopy tubes were designed as simple continuous members with assumed loading and calculated loadings. There is no single design approach as the mode of loading is very complex. The loads were broken down into individual elements and simple beam calculations used for different loadings or load combinations. For example if the ground above the canopy tubes moves down as a block then the shear capacity of the canopy tubes is the most important criteria and for which the factor of safety is very high. The bending capacity of the canopy tube is limited by the capacity of the threaded joint (if at the point of maximum bending moment) and is around half that of the main casing. In practice additional steel was inserted in the canopy tubes during construction when the excavation span was increased from 1.3m to 1.5m in the form of single reinforced bars. This was to provide a section of uniform strength. The canopy tubes were fully grouted but the grout only allowed the full steel capacity to be used in the analysis (no local buckling assumed once fully grouted). Direct rail wheel loads were reduced at the tunnel crown to allow for the distribution of load under the tracks. Separate calculations determine the stresses in the rails if one or two sleepers were removed (simulating high localised settlement in the tunnel). These calculations demonstrated a high factor of safety overall for the loading assumed to be applied to the tunnel.

The tunnel invert is permanently drained. The tunnel drainage system is described in Section 7. A nominal external ground water pressure with 3m head above the invert of the tunnel has been taken to check the wall for bending and shear capacity, which are satisfactory. The wall could be in compression with a stress of 2MPa. The dive structures either ends of the driven tunnel are 350m long and are also permanently drained.

In principle the tunnel shotcrete walls should bond to the shale rock and this would negate any water pressure applied directly to the wall (and water travelling between the rock and the back of the wall). However, in the field if in some locations this bond did not develop then additional support in the form of cement grouted permanent dowels would be an option. In actual tunnel construction weep holes at 3m centres have been installed along both walls. Approximately 20% of these weep-holes, which were drilled to 2.5m in depth, actually had intermittent, minor water flows. For all practical purposes the tunnel in its current state, even before the application of the membrane and final shotcrete layer is dry.

Further to the discussion about shotcrete bonding to shale (which is applicable to the near vertical walls of the tunnel) there is very little published or actual test information in Sydney. The reference (Hahn and Holmgen) does provide some general test data (but not from Sydney). The range of shotcrete bond strengths against fine grained shales reported varied from 60 KPa (lower bound) to 290KPa (upper bound). For design purposes an Ultimate shotcrete/shale bond strength value of 100 KPa was adopted. The shear capacity of fibre reinforced shotcrete is also an important parameter in addition to bond. Actual test data using fibre reinforced shotcrete (steel or synthetic fibres) has been published. According to these tests there is an excellent correlation between AS3600 $v = 0.34\sqrt{f'_c}$ and at 28 days, shear capacity = 2.15 MPa, however for early strength shotcrete the following formula can be used $v = 0.28\ fc'\ 0.6 - 0.11$. These are ultimate strengths so for working stresses an F of S of 1.5 has been applied. These figures also compare well with the Euro-code values.

The tunnel was constructed without lattice girders or steel sets as they are not required. There are numerous other examples in tunnels where lattice girders (although used) provide no structural support (Boggo Road and the M5 East Exit Ramp). There are also tunnels with shotcrete only (recent Cross Rail station caverns and cross-passages) and other tunnel examples available in numerous other references. The tunnel lining is totally dependent on the shotcrete to gaining sufficient strength to support the tunnel. In an arch profile it only takes around 6 to 7 hours to gain strength (with an accelerator) and within days the shotcrete is actually stiffer than steel lattice girders. There are two reasons for this, firstly the large area of shotcrete relative to the area of steel and secondly the shotcrete arch is acting in compression by a combination of the arch profile and the confinement of the surrounding rock. In the permanent state there is no obvious failure mode of the shotcrete arch with a compressive stress of up to 4 MPa in a shotcrete with a design characteristic strength of 40MPa. High flexural capacity of the shotcrete is not a requirement for the success of the tunnel structural lining design. This assertion is based on the fact that the tunnel walls are rock tunnel preventing the arch from sagging at the crown and thus always acting in compression.

While the probability of a major fire in the tunnel is extremely low as the return interval for such an event is much greater than 10,000 years, nonetheless the tunnel has been designed to at least not collapse after a hydrocarbon fire of 4 hours duration in the tunnel, it is probable that there would be 150mm of shotcrete lining remaining. Our analysis shows that the ground above the tunnel would arch and that the stress in the remaining shotcrete would at the apex of the arch (which provides the surface rock with confinement) would have a uniform compressive stress of around 2MPa (this is under dead load only with no train live loading). If there was a fire larger than even in the most extreme fire case assessed to date we believe that the tunnel could be backfilled from the surface and the surface track reinstated with the lines operational within approximately one week. While the tunnel is being rebuilt the freight trains could be diverted back to the existing surface tracks.

Another issue that was raised during the detailed design was the possible impact of vibration caused by trains passing overhead during the initial application of the shotcrete. Some investigation of this issue was carried out with the conclusion that there would be no impact. Calculations estimated a possible maximum displacement of the canopy tubes of 0.1mm. During construction vibrations due to passing trains were for all practical purposes non-existent.

7. GROUND WATER AND DRAINAGE

Prior to the start of tunnelling the groundwater table level was approximately 2.6m below ground level (typically 11.0m AHD) between the northern portal and the 90m point and then drops down to approximately 4.5m below ground level (8.0m AHD) between 90m to the southern portal. A possible reason for the groundwater change was initially believed in the detailed design phase was that the groundwater was damming behind a geological barrier like a dyke across the alignment of the tunnel.

Both the driven tunnel and the dive structure at either end of the tunnel are permanently drained. The combined predicted flow rates from these two structures was predicted to be the order of 0.1 to 0.5 litres/second with potential for higher short-term inflows. The actual ground water inflows to date have been very much less than predicted with only localised seepage points along the tunnel either through purposefully drilled weep holes in the lower wall (drilled at 3m intervals along the tunnel) or in circumferential defects in the shotcrete lining which have occurred at only about 6 locations within the tunnel itself (caused by a probable cold joint in the shotcrete at the staging point for the installation of the face nails spaced at 9m intervals along the tunnel).

Initially, circumferential 450mm wide core drains will be applied over the structural shotcrete lining on areas where tunnel lining seepage has occurred, which is at a very limited number of locations along the tunnel. Additional core drains were installed at 4.5m intervals along the tunnel and to take the groundwater seepages from the base of the side wall weep holes down to the sub-track slab drains. A cement based smoothing layer is next applied prior to the membrane application.

Over the smoothing layer a spray-on membrane will be applied over the arch of the tunnel to spring-line level on both walls over the initial tunnel shotcrete structural lining. The membrane will be applied over the smoothing layer which is around 10mm to 20mm in thickness. This layer will be applied by firstly spraying it onto the shotcrete surface and then rolled or brushed to obtain the desired smooth finished surface.

For the spray-on membrane BASF MasterSeal 345 (or equivalent) was proposed in the tunnel waterproofing specification, but Tam800 has now been applied to the tunnel. Both are water based products that are safe to use without specialised applicators. The Tam800 product is applied in two different contrasting colours and this may result in a final membrane that is at least easier to monitor its application thickness. The membrane will have a minimum thickness of around 5mm. The bond strength of the membrane to both shotcrete surfaces (dry and newly applied over the membrane) is typically around 1MPa for these products.

The invert of the tunnel is fully drained to prevent uplift forces acting under the track slab with perforated pipes running in gravel filled trenches excavated in the shale rock and running down both sides of the tunnel to the drainage sump at the low point of the tunnel. The low point sump in the tunnel also acts as a flame proof trap. The low point sump then drains by gravity to the north portal groundwater pump well. This drain was installed by directional drilling and pipe jacking a one pass reinforced concrete pipe for the 100m long drain. The south dive structure groundwater is transferred directly to the tunnel low point sump and then to the north portal sump. The groundwater from the north dive structure does not enter the tunnel but is directed to the northern pump well located outside the tunnel portal. Surface rain water flows from both the dive structures are diverted prior to entering the tunnel into the pump sumps located at each portal. In addition to waterproofing measures in the tunnel spoon drains run along the base of the tunnel walls in the track slab and any seepages from the lower wall if they occurred are directed to the sub-track slab drains.

The waterproofing membrane over the tunnel arch is designed to prevent long term drips developing in the tunnel crown and becoming a maintenance issue for the overhead wiring, the track rails and fixtures and the tunnel services on the tunnel walls. For durability the tunnel is classified as B2 and this is based on a sample of ground water taken within the rail corridor but not close to the tunnel.

8. SHOTCRETE

The tunnel profile with circular arch profile has been specifically developed to ensure that there is no flexure in the lining and that the loads applied result in a tunnel lining purely in compression. The maximum compressive stresses in the shotcrete lining determined from Finite Element (FE) analysis are a little under 4 MPa (dead plus live load). For this reason the shotcrete needed to gain an early strength of 6MPa (aided by the use of an accelerator) before the next excavation cycle could commence. This strength level was achieved after about 7 hours. The full dead load would also not be applied at the face, but the live load due to the trains passing above from calculation and assuming no distribution along the rails would be around fifty percent of the total load. The structural shotcrete had to have a design life of 100 years and as such, the shotcrete mix includes fly ash and silica fume, both of which enhance the durability of the mix. The shotcrete mix was developed through a number of iterations during the construction phase despite having shotcrete trials and testing carried out many months before the commencement of construction. The key parameters refined during the shotcrete trials included accelerator percentage, cementitous content, steel fibre vs plastic fibres, fibre contents, shotcrete life, and additives balance. The focus on achieving the 6MPA strength gain as quickly as possible without compromising the 28 day strength was a key focus. The macro synthetic fibres used in the structural shotcrete were Barchip60 with a dosage rate of 6kg/m^3 – the selection of the synthetic fibres required a reduction in the specified thoughness criteria, however this offered lining performance, commercial and durability benefits to the project. The 100mm fire protection shotcrete layer over the TamSeal 800 spray-on waterproofing membrane is the same mix design as the structural layer but with Duomix 6mm long synthetic fibres designed to reduce explosive spalling and with a dosage rate of 2kg/m^3.

The bond between the spray-on membrane and shotcrete will be checked on-site, however, typically for these products the bond between the both the underlying shotcrete and the sprayed shotcrete onto the membrane, it is expected that this bond strength will not be less than 1 MPa.

Measured strength gain of cored or site tested shotcrete (with fibres) are given in Table 5 below, together with the elastic modulus calculated using the following AS3600 formula at Clause 6.1.2, $Ec = 0.043\rho^{1.5}\sqrt{fc}$. In which ρ is the density of the shotcrete in kg/m^3 and fc is the compressive strength. The initial values up to the 3 days strengths are significantly less than taken at the time of design. However, even the one day old stiffness of the shotcrete is significantly more than the surrounding rock and certainly much greater than the overlying extremely weather shale, fill and track ballast.

Table 4: Shotcrete Mix Design

Material	Weights (kg/m^3)	Comments
Cement (kg/m^3)	370	The shotcrete mix design was modified twice during construction to improve the early strength gain without the addition of more accelerator (which is applied at the shotcrete nozzele). Modifications included reducing the flyash content from 25% to 20% and replacing the loss with cement. The percent of 10mm aggregate and sand was also reduced in the final modified shotcrete mix design.
Flyash	100	
Silica Fume	30	
10mm	520	
Man. Sand	570	
Fine Sand	530	
Admixture	4800	Polymer-based Superplasticiser
Admixture	650	Retarder
Additive	6	BARCHIP (Polyfibre)
Water	215 litre/m^3	
Slump	130 mm	

Table 5: Summary of shotcrete strength and stiffness gain with time

Time	Strength (MPa)	Elastic Modulus (MPa)
7 hrs	6	12 000
1 day	10	15 000
3 days	21	22 000
7 days	33	27 000
28 days	40	32 000

9. SETTLEMENT AND MONITORING

Surface settlement and in tunnel monitoring formed a critical part of the works both prior to and when tunnel excavation progressed under the live railway lines.

The principle at the core of the tunnel lining design and construction was that the shotcrete should be applied as close to the tunnel face as practical so that in effect, the ground does not relax. The canopy tube array installed ahead of the excavation ensures that even during the excavation process the ground has had little opportunity for movement prior to the placement of the shotcrete. The face nails have similar though less influence on surface ground movement. Figure 8 is a typical screen shot of surface settlement profiles relative to the advancing tunnel.

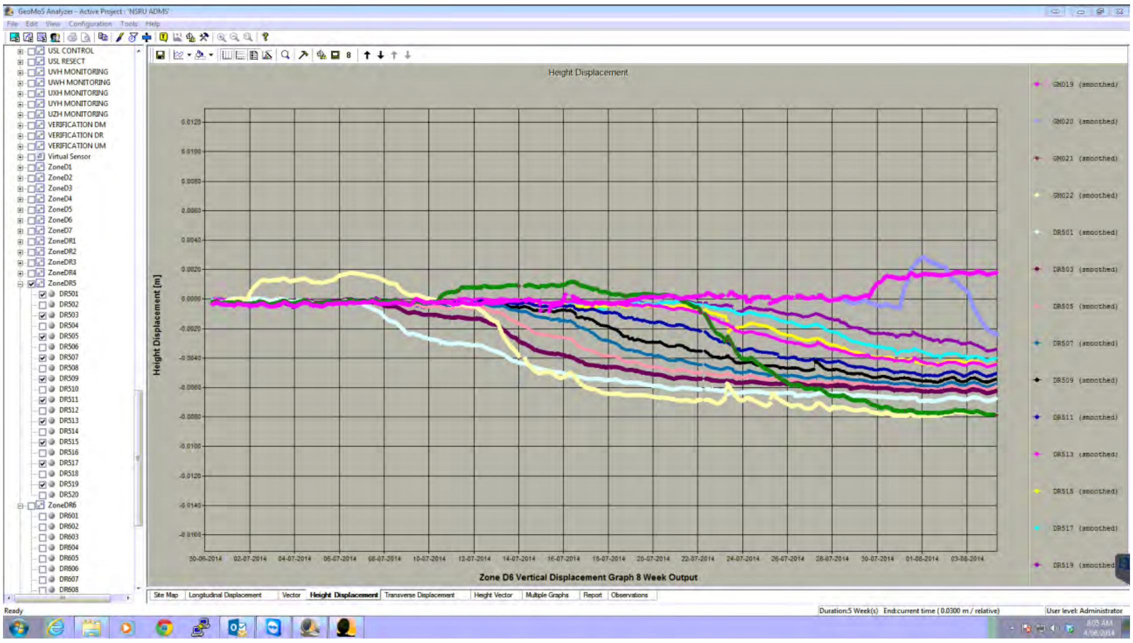


Figure 8: Typical settlement plot track monitoring (screen capture).

During the construction phase tunnel excavation commenced with 1.3m length excavation cycles and the surface settlements observed before passing under the first live railway track were the order of 5-6mm (7 excavation cycles between canopy tube array installations which are 9m apart, 12m long canopy tubes with 3m overlap). Through the daily Permit-To-Tunnel (PTT) process used on site the excavation cycle length was later increased to 1.5m. Excavation cycle lengths were subsequently reduced to 1.3m, then back down to 1.0m when the 800mm wide dyke was intersected. Later reverting back up to 1.3m as the ground conditions then improved and back up to 1.5m again after passing the last live track.

The maximum recorded surface settlement was 15mm at around the half-way point along the tunnel excavation, at which point the dyke coincided with the tunnel crown. Due to the increase in observed surface settlement other mitigation measures were applied including reducing the excavation cycle back to 1m, installing additional face dowels in soft material high in the heading and increasing the tunnel lining thickness from 250mm to 300mm. The maximum settlement started to develop just prior to the dyke and further investigation is required because in addition to the dyke excavation progress in the tunnel also increased prior to the mitigation measures listed above were adopted.

At the design stage the Design Settlement Report listed the following:

"The following summarises our conclusions:

1) The geometry of the tunnel arch (a pronounced curved arch profile) in combination with relatively strong rock strata (at least to tunnel spring-line level, with more weather rock above) is very favourable to achieving low surface settlement values.

2) The construction method proposed has a reliable history of achieving both predicted and low settlement values in similar weak rock tunnels.

3) The 2D finite element model predicts surface settlement in the order of one millimetre. Compared to say Boggo Road which used the same FE analysis approach that did not allow initial relaxation of the ground (pseudo 3D effect in a plain strain FE model, due to the forward installation of canopy tubes) this is a much lower prediction which may be the result of the circular arch profile compared to the flat arch. Either way the approach gave a very

accurate estimate of the surface settlement (calculated 7mm plus 3mm (judgement) and gave 10mm which matched the actual).

4) The magnitude of surface settlement is greatly dependent on the construction sequencing being followed and the quality of construction workmanship. These two factors are considered the controlling drivers on the magnitude of settlement as the theoretical predictions are so low.

5) Our interpretation of the geological profile is that during excavation of the driven tunnel there will be a lowering of the water table, however, changes in the water table will not have significant impact on the predicted surface settlements both in the short and long term.

6) The theoretical settlement values predicted from the FE analysis due to tunnelling construction works are less than a few millimetres, however, these are guide only and for this reason our judgemental estimate of the surface settlement due to the tunnel works is around 5mm."

It is always interesting to review predictions made during the design and while the settlements were higher there were reasons for this, although the actual settlements had no impact whatsoever on the operation of the trains. Rail twist, which is more important that uniform settlement values of the track were always within railway limits with no alarms triggered.

10. CONSTRUCTION

Tunnel excavation commenced on the 7th February 2014, initial progress was slow, with the first 9m of tunnel excavation taking 16 days to complete. Nineteen 12m long 139mm diameter steel canopy tubes were installed over the arch every 9m length of tunnel starting from the tunnel portal. There is a 3m overlap between canopy tube arrays. The excavation cycle was 1.3m with an initial 150mm of shotcrete sprayed over the arch and walls of the tunnel with additional shotcrete layers added immediately behind the face section building up the thickness to a final 250mm. A pattern of 35 face nails 12m long were installed at 4.5m into the excavation from the start of each canopy tube array. By staggering the canopy tubes and face dowels by 4.5m, a delay in construction was achieved which allowed the following shotcrete lining to gain more strength before the next series of excavation cycles.

Apart from observed standard tunnel design support, additional measures were taken based on the mapped geology and deformation readings. For example, when the dyke was intersected on the east wall of the tunnel two rows of permanent fibre glass dowels up to 10m long were installed on this section of the wall. Additional convergence of the side wall was also noted, around 3.5mm, in the vicinity of the dyke. Further along from where the dyke intersected the tunnel, ground conditions deteriorated in the centre of the face of the tunnel with some associated additional surface settlement observed. While it would be difficult to quantify all the components of observed settlement, with certainty, face relaxation of the tunnel was occurring. To mitigate this movement additional fibreglass face dowels were inserted in the top third of the tunnel face.

The excavation cycle along the tunnel varied thus the number of cuts within each 9m long array length varied correspondingly. For 1m length excavations, nine cuts between canopy tube array installations and for 1.3m cuts seven and for 1.5m cuts six. The tunnel excavation was being carried out after environmental approvals were granted, on a 24 hours per day seven days per week basis. The Alliance from 108m onwards along the tunnel changed the scheduling of the work such that shotcrete was delivered only in the early morning and reduced the shifts down to two 10 hour shifts with the final 1.5m long excavation cycle up to the south portal. The adjusted shift pattern and advance optimisation allowed for consistent shotcrete timing and mitigated lost time due to variability in curing time and inconsistent shotcrete supply.

In addition to plastic depth markers attached to the exposed ground surface, in order to further prove that the thickness of structural shotcrete is in excess of the design minimum, the Alliance has developed, in conjunction with local based software developer 12d solutions, a system of thickness verification whereby measurements are taken normal to the design profile on both the underlying (rock excavation) and underlying surfaces (shotcrete). A concept to measure the thickness during shotcrete application has been proven and further trials are being considered to further support the construction cycle.

While the waterproofing membrane has not been installed at the time of writing of this paper its installation is expected to go smoothly as for all practical purposes the shotcrete lining is almost dry as the shotcrete lining has little or no defects and there are no voids within the shotcrete matrix.

It is worth noting that there are many disadvantages of using steel sets or lattice girders in the same situation. Apart from the complexity of erecting lattice girders in such a large tunnel (and probably it would have to be a heading and bench operation and not a full face tunnel as used here) there is the issue of obtaining continuous and consistent contact with the excavation profile. Shotcrete on the other hand because it can be spayed directly onto the rock surface provides immediate and consistently continuous support. And because of the sheer volume (hence larger cross-sectional area) of shotcrete the lining is stiffer than alternative steel support that is spaced at intervals of at least 1m along the tunnel. The shotcrete only method is more advantageous for full face excavation because it is simple, continuous, provides high durability and construction safety (robotic application of shotcrete). The need for delivery and site storage of heavy and cumbersome steel components to a small and confined construction site is also eliminated.

11. ACKNOWLEDGEMENT

For permission to publish this paper from the Transport for New South Wales and the excellent work environment created by the Alliance.

12. SUMMARY AND CONCLUSIONS

1) The construction of the NSRU tunnel has proved conclusively that a shotcrete only lining applied close to the tunnel face in combination with the canopy tubes is an extremely effective construction method in this particular geological setting with very shallow ground cover. Without the need to erect lattice girders or steel sets the construction process was significantly simplified and the durability of the tunnel shotcrete lining, which forms the final lining without embedded steel has been considerably enhanced. The alternative to a one pass structural lining would have required a larger excavation further reducing the ground cover and perhaps to a point where the driven tunnel solution may not have been achievable (considering here both technical issues and approvals).

2) Compared to the initial cut and cover tunnel option proposed which required track possessions, the driven tunnel could be constructed continuously without disruption taking at least three years off the original construction program.

3) Shotcrete can be applied directly to the excavated surface while steel sets and lattice girders require some form of blocking which in practice can never be as effective as the intimate contact between the ground and sprayed shotcrete (which can be applied over both the tunnel arch and tunnel face in the same operation, if the latter is required). Using this construction method, settlement predictions are not only reduced but are also more predictable.

4) The tunnel was excavated as a full height heading which would have been less practical had lattice girders been used. Erecting these in an 8m high face would have required additional resources and transportation costs. In contrast the shotcrete was applied robotically and progressively by spraying with mechanical plant without the need for men to be close to the face.

5) The use of synthetic fibres with the shotcrete also proved to be successful. There is also no evidence of shrinkage cracking in the shotcrete lining. For all practical purposes the structural lining is near watertight. There is also a significant cost saving by using synthetic fibres compared to steel fibres. And as with the permanent fibre glass dowels used as a mitigation measure in the tunnel walls in the dyke zone, no possible electrical earthing and bonding issues.

6) The flexibility of the shotcrete only approach to tunnel support was made more apparent when a dyke was intersected. To increase the stiffness of the lining it was just a simple matter of increasing the shotcrete thickness (in this case from 250mm to 300mm , and this was taken as a precautionary measure). Permanent fibreglass dowels were also installed along the tunnel walls on both sides of the tunnel in the vicinity of the dyke.

7) The standard fibre glass dowel pattern of 35 dowels was effective at pre-supporting the tunnel face through various poor ground conditions consisting of mixed face conditions with a typical weathered shale profile that was intersected by faults and sheared zones in which one contained an 800mm wide dyke. An exclusion zone of 3m from the tunnel face was maintained, to protect the worker from potential rockfall and falling wet shotcrete. During the drilling of Canopy Tubes and fibreglass dowels, when the tunnel crew had to work within 3m of the tunnel face, the fibreglass dowels were combined with a minimum 50mm thickness of shotcrete. Additional dowels to the standard pattern were installed when a series of faults intersected the dyke and this helped reduce pre-convergence settlements (that occurred ahead of the tunnel face) by creating a stiff core of ground in front of the tunnel face. There were no incidents like a rock fall.

8) The timely application of shotcrete to the upper face of residual soils sealed the clays from the moist air and also helped reduce convergence settlement that occur during and after excavation until the ground support can be installed.

9) Geotechnical site investigations are more difficult to carry out in an operating rail, nonetheless there was sufficient information to interpret geology along the full length of the tunnel. While a dyke was intersected and not identified by the site investigation it has little real impact on tunnel excavation progress and the construction method was able to accommodate changed geological environment without significant modification.

10) The broad geological model developed for the tunnel proved to be very accurate. The shotcrete arch acted as a compression member without flexure because the high level of Unit 4 rock in the tunnel walls ensured that the lining behaved as per design. No cracking in the shotcrete associated with possible flexure in the tunnel crown has been observed at any point along the tunnel.

11) The PTT process was used very successfully on this project. With a high level of focus on risk of settlement, the process was used to closely monitor settlements and ground conditions and instruct mitigation measures or optimisation as appropriate. Through the PTT process it was possible to enhance tunnel excavation productivity by increasing the excavation cycles in steps between 1m, 1.3m to 1.5m depending on ground conditions and live loading.

12) The maximum surface settlement was around 15mm but this movement was not reflected in the lining deformations which were in the range of 1mm to 2mm in the tunnel crown within the tunnel. Typical surface settlements were between 5-8mm. At springline level tunnel convergence was never greater than 3.5mm across the full width of the tunnel. At no time were either the short or long twist criteria for the railway tracks exceeded.

13) While surface settlement values were higher than predicted at some locations this can be partially offset by the change to the larger tunnel profile taken for the reference design, the geology and particularly the dyke and its localised effect on the immediate surrounding geology, the excavation cycle length changes and also magnitude of settlement and rate of tunnel advance. Groundwater disturbance and possible ground consolidation in the zone above the tunnel arch together with train live loading are other possible contributing factors.

13. REFERENCES

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