The North Strathfield Rail Underpass – Driven Tunnel Design and Construction

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ABSTRACT

The single track North Strathfield Rail Underpass (NSRU) consists of a dive structure either end of a 148 m long driven tunnel skewed under three live railway tracks. The tunnel is 8 m high, 9 m wide, with a horseshoe profile, excavated by full face using a road header. Ground cover varied between 2.5 m to 3.5 m. Settlements were 5 mm to 15 mm (the latter due to a dyke). A one pass synthetic fibre reinforced shotcrete lining was installed (no lattice girders or steel sets were used). A system of automated robotic total stations was used to provide real time monitoring of the rail track and associated ground formation. No disruption to train services occurred.

INTRODUCTION

The North Strathfield Rail Underpass (NSRU) project is designed to grade separate south bound freight trains from the electrified suburban rail network north of Sydney. 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 constructed 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 developed during the detailed design phase to be a shotcrete only final lining reinforced with synthetic fibres. The underpass was also moved northwards 60 m relative to the concept design location 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 settlement 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 148 m with the ground cover between 2.5 m and 3.5 m. The tunnel excavated dimensions were around 9 m wide × 8 m high. No disruption to train services occurred during the construction of the tunnel. This paper describes some aspects of the design development from the reference stage through detailed design and other minor changes made during the construction phase. An unforeseen 800 mm wide dyke was also intersected by the tunnel.

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 km 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 joint venture 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.

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, Down Main and the Down Relief, thereby eliminating conflicting at-grade movements. The dive structures either end of the driven tunnel are around 250 m 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 400 m radius with a down gradient on the north dove of 2.8 per cent and on the south dive of 2.2 per cent.

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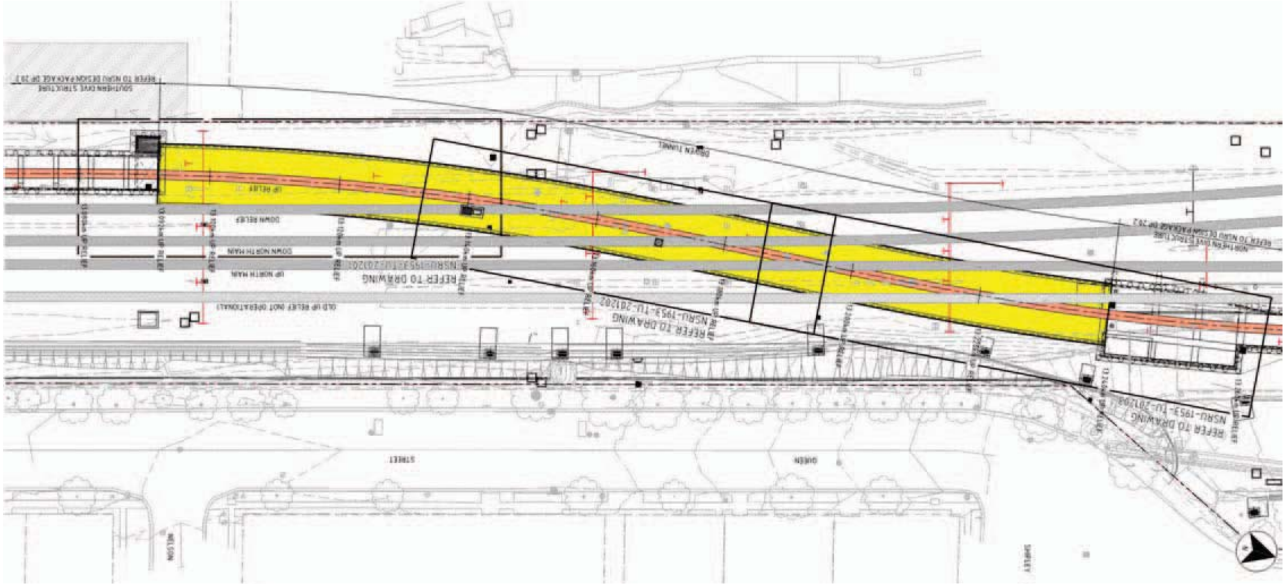

FIG 1 – Tunnel plan alignment.

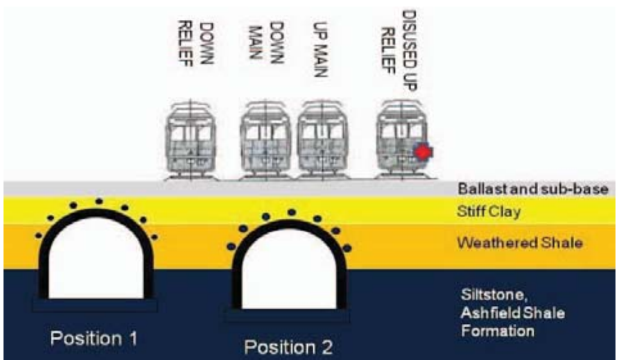

FIG 2 – Schematic cross-section of tunnel and railway tracks, 3.5 m maximum ground cover. Tunnel traverses the tracks on a skew.

SITE INVESTIGATIONS

Site investigation boreholes and test pit excavations were carried out within the rail corridor track zone during long weekend track possession 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 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 11 m using a small tracked drilling rig.

Visual inspection of the corridor during the June 2011 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 cut and cover tunnel concept design location referred to earlier.

The site investigation failed to detect a 800 mm wide dyke, which was intersected by the tunnel excavation. Though not of great consequence given in the future seismic traverses should be considered as part of future site investigations within rail corridors.

GEOLOGICAL MODEL

From the site investigation data it was possible by interpolation to develop a geological long section and cross-sections through the tunnel taken at 10 m intervals. Apart from the dyke that intersected the tunnel 60 m from the north portal and exited the tunnel at 90 m on the west wall, the geological interpretation was proven to be very accurate. The dyke consisted of stiff clay with a contrasting golden colour compared to the surrounding dark grey shale rock (Figure 3). The typical face conditions consisted of extremely low, to low strength Ashfield Shale in the crown, low to medium strength shale down to the tunnel spring line and high strength shale in the lower half of the tunnel. Above the tunnel crown is residual clay, fill and ballast. Test pits were excavated between sleepers in critical locations along the track to determine the depths of the ballast and the ballast sub-base.

VALUE ENGINEERING INITIATIVES

For the tender, the builder developed their own tunnel design and construction methodology which differed quite significantly from the approach taken by the designer for the reference design. The reference design had also referred

FIG 3 – An 800 mm wide dyke in tunnel face.

to lattice girders and the builder proposed light steel sets and continuous grout bags to contact 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.

The reference design was developed as a heading and bench excavation and it was is anticipated that for such a short tunnel the builder would use or have available a small road header, refer Nye (2013). For efficiency of 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 1.5 m wider. Additional calculations were made by the designer to confirm that this would have minimal 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 change had no impact on the final structural lining shotcrete design thickness which was 250 mm.

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

A proposed 25 mm passive fire protection layer in the driven tunnel reference design was removed and replaced with a final 100 mm thickness shotcrete layer (increased from 75 mm 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 four hour duration hydrocarbon fire even allowing for a significant loss of the overall lining thickness. The design fire size had been increased significantly from the design fire in the reference design.

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 6 MPa (aided by the use of an accelerator) before the next excavation cycle could commence. This strength level was achieved after about seven 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 50 per cent 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 versus plastic fibres, fibre contents, shotcrete life, and additives balance. The focus on achieving the 6 MPa 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 6 kg/m^3 – the selection of the synthetic fibres required a reduction in the specified toughness criteria, however this offered lining performance, commercial and durability benefits to the project. The 100 mm fire protection shotcrete layer over the Tam Seal 800 spray-on waterproofing membrane is the same mix design as the structural layer but with Duomix 6 mm long synthetic fibres designed to reduce explosive spalling and with a dosage rate of 2 kg/m^3.

Strength gain of cored or site tested shotcrete (with fibres) are given in Table 1, together with the elastic modulus calculated using the following AS3100 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.

TABLE 1
Summary of shotcrete strength gain with time.

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

SETTLEMENT AND MONITORING

The principle at the core of the tunnel lining design and construction is that the shotcrete should be applied as close to the tunnel face as practical so that in effect, the ground does not have time to 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. Figure 4 is a typical screen shot of surface settlement profiles relative to the advancing tunnel.

For further discussion on this concept please refer to Nye (2012). Shotcrete is considered the most appropriate tunnel support because it can be spayed directly onto the rock surface thus providing immediate support. This method is even more advantageous for full face excavation compared to the alternative of using steel sets or lattice girders in such a large tunnel just for the construction simplicity alone. Figure 5 shows a tunnel section with canopy tubes.

It was anticipated from FE analysis, case histories and previous experience that the magnitude of surface settlement would be around 5 mm, this was based on the assumption that each excavation cycle would be 1 m in length. During the construction phase tunnel excavation commenced with 1.3 m excavation cycles and the surface settlements observed before passing under the first live railway track were the order of 5–6 mm (seven excavation cycles between canopy tube array installations which are 9 m apart, 12 m long canopy tubes with 3 m overlap). Through the permit-to-tunnel (PTT) process the excavation cycle length was later increased to 1.5 m. Excavation cycle lengths were subsequently reduced to 1.3 m, then back down to 1 m when the 800 mm wide dyke was intersected. Then backup to 1.3 m and then as ground conditions improved and after passing the last live track, back to 1.5 m.

The maximum recorded surface settlement was 15 mm at around the half-way point along the tunnel excavation, at

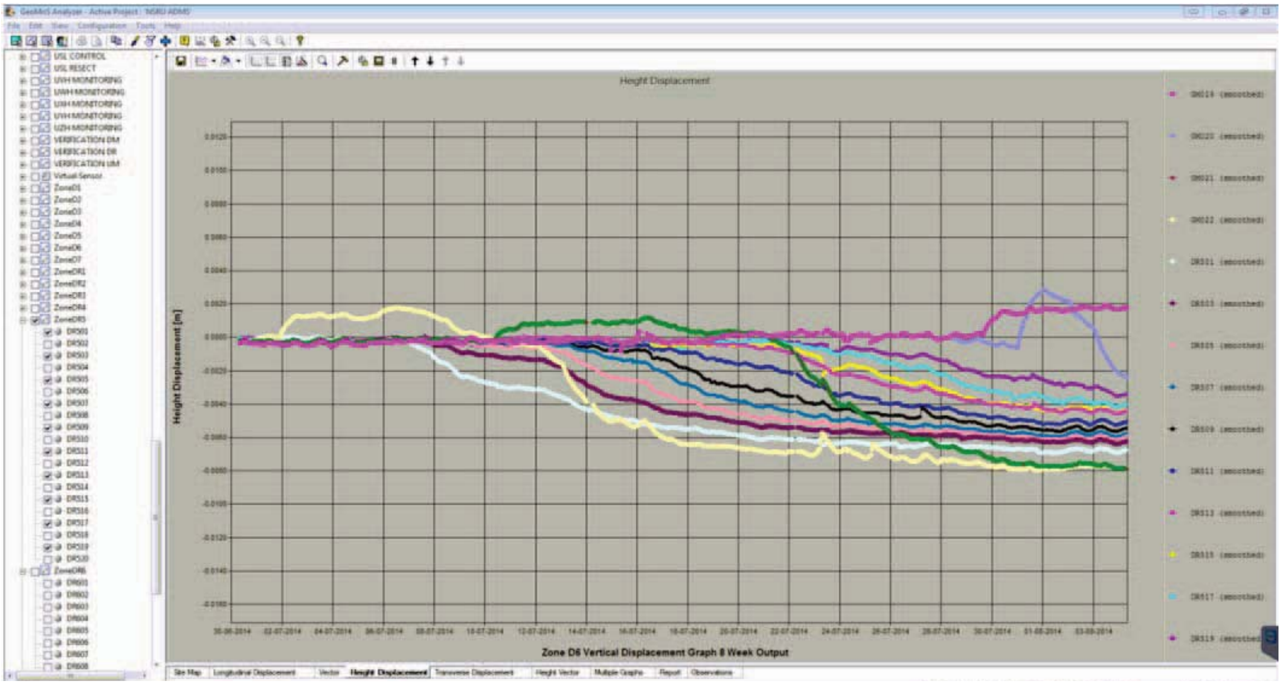

FIG 4 – Typical settlement plot track monitoring (screen capture).

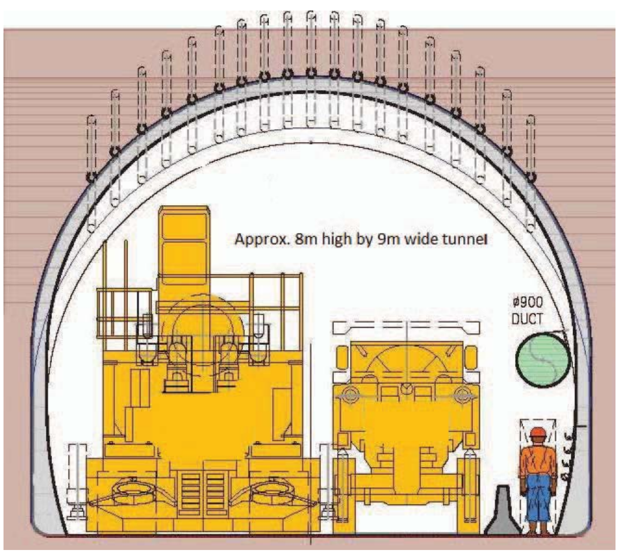

FIG 5 – Tunnel cross-section.

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 1 m, installing additional face dowels in soft material high in the heading and increasing the tunnel lining thickness from 250 mm to 300 mm.

Further analysis and interpretation of all the data is ongoing, however, while there was some surface settlement above predictions this was not reflected in the deformation measurements taken in the tunnel of the shotcrete lining. The maximum convergence reading was around 3.5 mm (where the dyke was intersected) and the maximum vertical settlements at the apex of the crown inside the tunnel were no more than 1 mm. Numerous measurements were taken close to the tunnel face (sometimes at 1 m intervals), the closest reading being 3 m from the excavated face, so there may well be some initial movement not measured, however, given that the maximum surface settlement occurred at least one tunnel diameter and probably more from the face it would appear that the surface settlement was not directly related to ground losses (ie deformations within the tunnel excavation profile and tunnel face). The most likely cause for the higher settlement values at this stage appears to be consolidation of the fill and residual soil over the immediate crown of the tunnel probably as a result of groundwater table changes in combination with the train live loading.

MONITORING PLAN

A comprehensive monitoring plan was developed for the tunnel that also included face mapping and a daily PTT meeting between the designer, geotechnical engineer and the constructor. Often meetings also included the tunnel superintendents and the track certifier.

Deformation monitoring was one mechanism used to verify that the tunnel construction was being adequately controlled in accordance with allowable settlement limits and deformations. Within the tunnel tape convergence and survey readings were taken as close to the tunnel face as practical. Prior to commencing any construction work on-site a baseline survey was taken of the track and surface marks. The condition of the track was monitored by both robotic survey methods with prisms spaced at 2 m intervals on both rails of each track and also by inspection and assessment by track certifiers. In all there were approximately 450 survey prisms used covering the three live tracks and surface ground marks. The robotic survey data including generated settlement plots (Figure 4) could be accessed off-site, saved and printed on any computer or hand held device (eg smart phone or tablet) connected to the internet.

Ground vibration due to passing trains and the potential monitoring of these vibrations was flagged during the design phase. In practice train vibrations in the tunnel were for all practical purposes imperceptible and therefore they had no impact whatsoever on the tunnel construction. A rumble noise from passing freight trains was audible in the tunnel.

TRACK MONITORING

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, seven days per week Automated Deformation Monitoring System (ADMS). This was also required to operate within existing client maintenance specifications (Railcorp standard SPC 207).

The alarming criteria specified in the monitoring plan were originally specified in line with the parameters in SPC 207. The project engaged Sydney Trains in a consultative process to tailor the alarm levels to align with the track maintenance specification TMC 203, on the basis of the frequency and accuracy of the proposed monitoring system as well as the emergency response process developed for the project. The extended base lining in period of the monitoring system was used as further demonstration of this. A wavier was granted by the chief track engineer (Sydney Trains/ASA) to relax the alarm levels on this basis.

The project monitoring system must measure and alarm on the following four key rail variables:

1. long twist, measured over a baseline length of 14 m at 2 m stations
2. short twist, measured over a 4 m baseline length at 2 m stations
3. top, change in elevation measured over a 4 m baseline length at 2 m stations
4. line, deviation of the track in plan over a baseline length of 8 m at 2 m stations.

In addition to this, any output must display the various readings against its relevant alarming criteria and also provide a graphical display of those results. All of these four key rail variables must be measured across the entire zone of influence of the proposed tunnel measuring some 200 m in length and involving the use of over 450 monitoring prisms. For further details refer to Mares (2014).

In November 2013, the NSRU commissioned the ADMS to measure 3D track deformation at predetermined locations (at surface level) with the primary aim to ensure rail reliability, safe passage of rail traffic and provide inputs into the PTT process throughout the proposed tunnel construction phase. The ADMS consists of three robotic total station instruments used to measure in excess of 450 monitoring targets in near real time on a 24 hour, seven days per week basis. Three monitoring stanchions exist within this ADMS, each with two separate total station mount points, associated communications, mains power and uninterrupted power supply capabilities. This enables the Alliance to instantly double the monitoring frequency should circumstance require it. Monitoring stanchions have been positioned such to minimise the length of the required range measurement, ie from total station to monitoring prism and hence to minimise the parts per million error values associated with all EDM measurements. An added feature of the ADMS is that the total station mount points have been elevated to 3.5 m above nominal ground level providing clear sight to the prisms mounted on the rails and as ground monitoring points. A key feature of the ADMS is the ability to make comparison between measured and baseline values, determine if deformation has exceeded a predetermined range and to generate an alarm accordingly. This alarm is transmitted to project stakeholders. The baseline had been defined prior to tunnelling works with monitoring of the track six weeks before works started.

CONSTRUCTION

Tunnel excavation commenced on 10 February 2014, initial progress was slow, with the first 9 m of tunnel excavation taking 16 days to complete. Nineteen 12 m long 139 mm diameter steel canopy tubes were installed over the arch every 9 m length of tunnel starting from the tunnel portal. Therefore there was a 3 m overlap between canopy tube arrays. The excavation cycle was 1.3 m with an initial 150 mm 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 250 mm. A pattern of 35 face nails 12 m long were installed 4.5 m into each array. By staggering the canopy tubes and face dowels every 4.5 m this created a delay in construction which allowed the following shotcrete lining to gain more strength progressively with the excavation.

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 10 m long were installed on this section of the wall. Additional convergence of the side wall was also noted, around 3.5 mm, 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 cycles within each array length varied correspondingly. For 1 m cycles there are nine cuts, 1.3 m have seven cuts and 1.5 m excavation cycles six cuts. 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 108 m 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 ten hour shifts with the final 1.5 m 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.

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SUMMARY AND CONCLUSIONS

The reference design was further developed in the detailed design phase with close cooperation between the designer and the constructor. The major changes, relative to the reference design were the full height heading excavation and the carrying forward from the reference design of the fibre reinforced shotcrete lining without with steel lattice girders. Without the need to erect lattice girders the construction process was significantly simplified and the durability of the tunnel shotcrete lining without embedded steel has been considerably enhanced.

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. The mitigation measure initiated as a result of intersecting a soft zone in the face associated with the dyke was to install more face nails – as an example. The tunnel design arrangement was also optimised at times through the PTT process as ground conditions and settlements improved, with the number of canopy tubes reduced at times to 17 per array. Predesigned mitigation measures allowed this process to be versatile, responsive, and effective.

Through the PTT process it was possible to enhance tunnel excavation productivity by increasing the excavation cycles in steps between 1 m, 1.3 m to 1.5 m depending on ground conditions and live loading. The process was not encumbered by the need to use steel sets or lattice girders. The advance rate of the tunnel was around 5 m/week, with 24 hour, seven days per week working.

The flexibility of the shotcrete only approach to tunnel support was made more apparent when the dyke was intersected. To increase the stiffness of the lining it was just a simple matter of increasing the shotcrete thickness from the current 250 mm to 300 mm (this was a precautionary measure).

The maximum surface settlement was around 15 mm but this movement was not reflected in the lining deformations which were in the range of 1 mm to 2 mm in the tunnel crown within the tunnel. Typical surface settlements were between 5–8 mm. At no time were either short or long twist criteria for the railway tracks exceeded.

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.

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.

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