Shotcrete: Elements of a System – Bernard (ed)

Shotcrete application on the Boggo Road Busway driven tunnel

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ABSTRACT: The Boggo Road Busway Project includes 430 m of driven tunnel. The tunnel, with an excavated width of 15 m, was constructed beneath the heritage listed Boggo Road Jail with low ground cover in very variable geological conditions. Shotcrete was the main initial arch support type relied upon to minimize surface ground settlement through this shallow section of tunnel. To achieve a high arch stiffness the shotcrete was applied to its full thickness of 350 mm as near as practical to the tunnel face. Other support types used along this section of tunnel were an array of canopy tubes, steel lattice girders and fibre glass face nails. Beyond the jail initial tunnel arch support types consisted of shotcrete and rock bolts or shotcrete and lattice girders. Fibre glass face nails were used along the full tunnel length for face stability.

1 INTRODUCTION

The driven tunnel examined in this paper forms part of the Boggo Road Busway project in the Queensland capital, Brisbane. This paper describes a number of aspects of the design and construction of the driven tunnel with an emphasis on the application of shotcrete for tunnel support. The driven tunnel is 430 m long with an excavated width of 15 m and a full tunnel excavation height of 8 m. The first 120 m long section of the driven tunnel was excavated under the heritage listed Boggo Road Jail. The main jail buildings and perimeter walls were built in 1908 and are of brick construction.

Ground cover over the tunnel beneath the jail site varied from 5.5 m to 8 m and as a consequence the predicted settlement values and their potential to cause damage to the buildings were critical to determining and obtaining approval for the selected tunnel alignment. The geology along the driven tunnel alignment was extremely variable, providing mixed face tunneling conditions and ranging from surface residual soils to high strength rock at depth. The maximum ground cover above the driven tunnel is 20 m. The initial ground support included shotcrete, and shotcrete was also replied upon as the permanent support at the four 19 m long jet fan niches and at both driven tunnel portals (in combination with permanent rock bolts). The remainder of the tunnel permanent support consists of steel reinforced 300 mm thick concrete over the tunnel arch (the tunnel has a horseshoe shaped profile, Figure 1). Where shotcrete was used as permanent support the water-proofing membrane was a spray on product, in contrast to the sheet product used where there was an in-situ concrete lining.

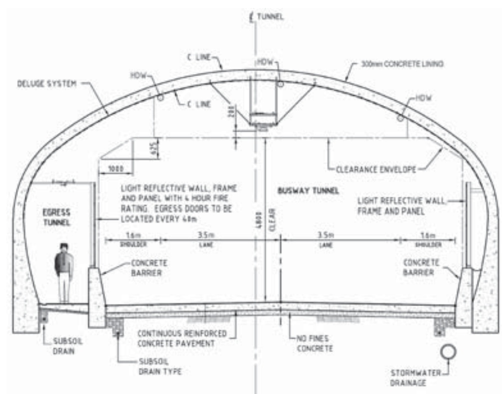

Figure 1. Standard tunnel section through completed tunnel.

2 DESCRIPTION OF THE PROJECT

The Queensland Government funded A$326 million Boggo Road Busway is a dedicated public transport link to the South East and future Eastern Busway at Buranda and provides the connection from the Eastern and Southern Busway corridors to the University of Queensland with a total travel length of 2 km.

The Boggo Road Busway commences at the new Princess Alexandra (PA) Hospital busway station and continues through a cut and cover underpass

Figure 2. Plan of Boggo Road Busway and Eastern Busway alignment, driven tunnel on left below bus turning bay.

beneath the Queensland Rail lines, then travels parallel to the railway lines to reach the new Park Road busway station and existing Park Road rail station (Figure 2). From this point the busway goes underground through a 630 m long tunnel that passes beneath the Boggo Road Jail, Gair Park and Annerley Road before emerging at the new 330 m long cable-stayed Eleanor Schonell Bridge in Dutton Park which then connects across the river to the University of Queensland. At its eastern end the Boggo Road Busway connects to the South East Busway via the first section of the Eastern Busway which starts at the PA Hospital and travels over Ipswich Road and under the Pacific Motorway.

The Boggo Road Busway was opened in July 2009 and will be used by 600 buses daily (this equates to around 13,000 passengers). The Boggo Road Busway project and the first stage of the South Eastern Busway have been designed and constructed using the Alliance method of project delivery. The Alliance team members are Queensland Transport (the client), Thiess Contractors (the builder) and Sinclair Knight Merz (the designer). The route alignment is shown in Figure 2 above with the tunnel alignment on the left side of the figure (Nye et al, 2009).

3 DESCRIPTION OF THE DRIVEN TUNNEL

The busway tunnel's full length is 630 m and consists of 430 m of driven tunnel with cut and cover tunnels at both ends of the driven tunnel. At the north end, the cut and cover tunnel is 130 m long and includes a 34 m diameter bus turnaround area at the driven tunnel portal. At the south end of the driven tunnel the cut and cover section is 70 m long and daylights into Dutton Park.

The busway tunnel profile includes 2×3.5 m bus lanes with 1.6 m shoulders on both sides of the busway together with a 1.5 m wide emergency egress passage along one side of the tunnel. The road pavement to tunnel crown is around 7 m in height. Four jet fans niches are spaced along the tunnel, each required over-excavation of the tunnel crown to accommodate the three jet fans at each fan niche location. The jet fan niches were completed outside the jail buildings but within the driven tunnel. The final lining of the driven tunnel consists of a 300 mm thick in-situ reinforced concrete arch lining for the main running tunnel. At the fan niches, the final lining consists of a pattern of permanent rock bolts and 250 mm of steel fibre reinforced shotcrete.

The waterproofing membrane behind the in-situ concrete lining consists of a two color layer 2 mm thick polyethylene welded sheet over the arch and upper wall of the standard tunnel profile. At the fan niches and tunnel portals a 4 mm thick spray on membrane has been used. The driven tunnel has been designed to be fully drained and has a "no fines" concrete drainage layer under the reinforced concrete road pavement as part of the under pavement drainage system. The tunnel was excavated using an AM105 road-header.

4 GEOLOGICAL MODEL

The geological profile for both the rock type and rock strength varies considerably along the length

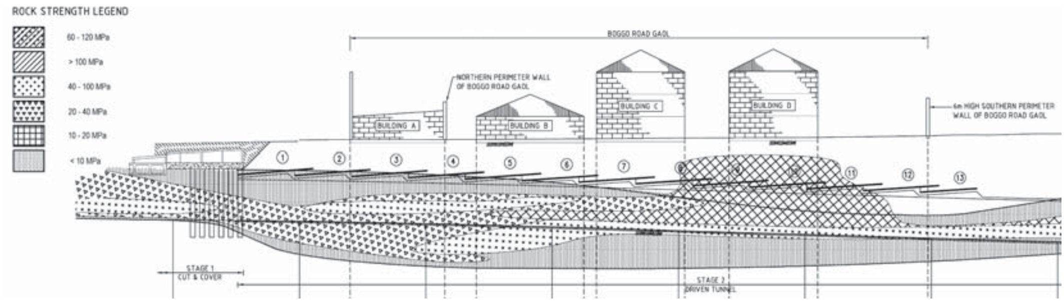

Figure 3. Geological profile under the jail.

of the driven tunnel and across the tunnel section. Developing the geological model profile was quite difficult because of the varying geology. To develop the model required a good understanding of the local geological history as well as the ability to extrapolate the data in three dimensions. It was realized in the design development stage that the geological model would have a significant impact on the selection of tunnel support types along the tunnel and hence the actual cost of the tunnel.

There are industry recognized classification systems which also provide recommended ground support for a given rock mass classification. These include the Q and the Rock Mass Rating (RMR) systems. An initial assessment using both these systems was carried out with the RMR system classifying the ground along the tunnel alignment in the range of 'poor' to 'good' rock conditions. The purpose of the geological model was to provide a prediction of the variation of the rock strata along, across and above the tunnel (Figure 3). Rock strength data was superimposed on the geological model. It was this latter information that was used to develop a very simple site-specific rock support classification system and consequently this was used to determine the extent of each support type along the tunnel. For example, the fully encapsulated resin rock bolts could not be used as initial ground support if the UCS of the rock was less than 10 MPa as this would compromise the desired bond strength along the rock bolt. Elastic Modulus values for rock used in FE analyses for lining design and settlement predictions were in the range of 200 MPa to 5000 MPa depending on the rock type and degree of weathering.

5 INITIAL TUNNEL GROUND SUPPORT

The design of the initial ground support for the tunnel is briefly described below. During construction very few modifications were made to the initial design of the initial tunnel support (Table 1). One significant change during construction was the deletion of steel fibres from the shotcrete for Support Types 1 to 3. This was possible because the ground behavior over the tunnel arch was better than expected. In contrast the face stability was of more concern, as evidence by an increase the number of fibre glass dowels installed at the tunnel face. One localized face collapse also occurred during tunneling, justifying the inclusion of the face nails. Variations in the length of the support types over the tunnel arch from the original design estimates occurred along the tunnel outside the jail section (Table 2). Rock strength data principally consisted of Point Load Test (PLT) and laboratory Unconfined Compression Tests (UCS) results.

Table 1. Initial tunnel support types.

Type	Description of tunnel initial ground support
1	3.6 m long 21 mm diameter, M24 thread, 28 mm hole, 310 kN ultimate capacity fully resin encapsulated rock bolts on a 1.5 m grid, 50 mm thickness shotcrete. To be used in "good rock" where >80% of rock over the arch is high.
2	As above but 1.2 m grid and with 100 mm shotcrete (also modified during construction to 1.2 m (over) by 1.5 m along tunnel grid). To be used in "fair rock" where >50% of rock over arch is high strength.
3	As above but 150 mm shotcrete. To be used in "poor rock" where >80% medium strength rock over tunnel arch and UCS >10 MPa.
4	Triangular profile lattice girder at 1.2 m spacing (170 mm deep section) with 300 mm shotcrete. To be used in "very poor rock" where UCS less than 10 MPa.
5	Alwag 140 mm diameter canopy tubes at 500 mm spacing (27 no.) over tunnel arch. The canopy tubes are 12 m long with 3 m over lap, plus triangular lattice girder 170 mm deep (with 350 mm shotcrete). Low cover tunneling under Boggo Road Jail. The tunnel width was kept constant, i.e. the canopy tubes were only angled up vertically not horizontally.

Table 2. Initial tunnel ground support type—predicted and actual.

Type	Percentage of tunnel length	
	Predicted	Actual
1	20	5
2	16	14
2 modified	0	33
3	5	0
4	32	21
5	27	27

On this project a multiplication factor of between 17 (welded tuff) and 20 was used to estimate the UCS of the rock from the much larger test sample of PLT results.

In the design phase of this project careful assessment of the ground support requirements were conducted over very short lengths of the tunnel due to the wide range of geotechnical parameters. The geological model and the superimposed strength data were continuously reviewed throughout the design and subsequent construction phase. This review resulted in two additional boreholes in Annerley Road (after tunnel construction had commenced), daily geological mapping data from the tunnel, additional field PLTs on rock samples taken from the tunnel, and regular pullout tests on rock bolts in the tunnel walls and crown as tunnel excavation progressed.

The reason for the "rigid" initial Support Type 5 was to minimize ground relaxation due to excavation of the tunnel and hence prevent damage to the heritage listed brick building of the Boggo Road Jail. The predicted settlement was 10 mm and the actual settlement under the buildings ranged between 7 mm and 12 mm with no damage to any of the building or the perimeter 6 m high jail walls. The steel lattice girder provided a visual tunnel profile upon which a check could be made regarding the accuracy of the excavation to achieve the design profile and also provide roof early protection until the shotcrete was applied. The shotcrete was the main structural feature limiting the settlement of the surface once the 3D effect of the canopy tubes at the tunnel face was completely lost, probably after the tunnel had advanced only a few metres. This was the reason why it was necessary to apply the shotcrete at the tunnel face at near its maximum required thickness of 350 mm. Extensive finite element modeling was carried out during design development to determine the required minimum shotcrete thickness. The estimate of support for each type and the actual percentage used is given in Table 2 above.

Type 5 support under the jail was not modified. Type 4 had the shotcrete thickness reduced to 200 mm thickness after monitored settlement readings were less than expected beyond the jail. Steel fibres at 45 kg/m^{3}were deleted from supports Types 1 and 2 saving A$3/kg which became a very significant saving for the project. Type 2 modified support was used for the majority of the length of tunnel beyond the jail.

The fan niches had the same arch profile as the standard tunnel profile, and therefore had the same arch support for given geological conditions. The fan niche length of 19 m also includes the 10% graded transitions from the standard tunnel profile, required for the efficient operation of the jet fans.

6 SHOTCRETE STRENGTH AND TESTING

Under the jail the design intent was to provide initial tunnel ground support that would not allow ground relaxation. This was achieved using a staged construction approach, including forward installation of canopy tubes, steel lattice girders, 12 m long fibre glass face dowels and then the early application of the 350 mm thick shotcrete over the arch to be built up progressively at the tunnel excavation advanced in 1.0 m increments (refer Figures 9, 10 and 11).

Early strength of the shotcrete was critical to the success of this approach as the shotcrete was the stiffest structural element once the 3D constraining effect of the tunnel rock face and near face support (canopy tubes, lattice girders and face nails) was lost. Table 3 above is from the project shotcrete specification and gives the strength requirements of the shotcrete for example from as early as 3 hours after application, together with the

Table 3. Type 5 support, early and minimum strengths specified, plus calculated E-shotcrete.

Age	Minimum strength (MPa)	Test applied	Elastic modulus (MPa)
3 hours	1 MPa	Initial Strength Meyco Needle Penetrometer	1,000
12 hours	6 MPa	Sprayed Beam Compression (ASTM C116)	12,000
24 hours	18 MPa	Core Compressive Strength (AS1012)	20,000
3 days	26 MPa	Core Compressive Strength (AS1012)	24,000
7 days	35 MPa	Core Compressive Strength (AS1012)	28,000
28 days	40 MPa	Core Compressive Strength (AS1012)	30,000

test used to measure the shotcrete strength. The stiffness of the shotcrete, in the form of the Elastic Modulus, has been calculated using the following AS3100 formula at Clause 6.1.2.

$$E_c = 0.043\rho^{1.5}\sqrt{f_c'} \qquad (1)$$

in which ρ is the density of the concrete in kg/m^3 and f_c' is the compressive strength. Referring to Table 4, the initial strength gain of the shotcrete in the first 12 hours was consistently good, generally meeting the required specification target strengths and hence stiffness. Although there were some quality issues with the shotcrete supply with core strength sometime very low there was no apparent negative impact on the surface settlement (further comparisons between early strength testing of shotcrete can be found in Clements, 2004).

With settlements in the range of 7 mm to 12 mm under the two jail cell block buildings the dominating variable that was the causing this difference was the varying geological profile. These two identical buildings, C and D shown on Figure 3, each are equal to a dead load over the tunnel of 60 kPa or 3 m height of fill. The surface settlement prediction in open ground was between 6 mm and 10 mm. A lot of effort went into investigating the variability of the shotcrete strength during construction but at no stage was the construction progress impacted upon. The investigations and testing consisted of changing the shotcrete mix design, varying the percentage of accelerator and obtaining test data from an alternative supplier. Apart from the tunnel arch, shotcrete also provided a continuous footing at the springline level of the tunnel heading. Where highly weathered claystone was encountered a localized widening of the base of the shotcrete arch was carried along one side of the tunnel. This is further described in Section 7.

Table 4. Minimum and maximum strength range of shotcrete for various age ranges and tests.

			Core strength*		Cyl. strength*	
Age	Days	Spec (MPa)	Min	Max	Min	Max
3 hrs	0.125	1	0.6 (MP)	1.2 (MP)	0.6 (MP)	1.2 (MP)
12 hrs	0.5	6	5.2 (B)	7.3 (B)	5.2 (B)	7.3 (B)
24 hrs	1	18	7.33	31.33	16.5	32
3 days	3	26	12	35	34	48
7 days	7	35	17	48	43	55
15 days	15		18	49	49	65
28 days	28	40	27	65	51	94

* Unless otherwise noted, MP = Meyco Needle Penetrometer, B = Sprayed Beam Compression.

There were three shotcrete specifications issued for the tunnel project. One specification was issued for each of the temporary and initial shotcrete, with and without steel fibres. The third specification was for the permanent shotcrete with steel fibres. The permanent shotcrete was used at the fan niches and at both tunnel portals. The Department of Main Roads, Queensland required that all permanent shotcrete have a minimum of 20% fly ash. The specified minimum characteristic strength of all shotcrete was 40 MPa, with maximum of 10 mm aggregate size and slump of 120 mm. The initial shotcrete layer did not require a smoothing layer prior to the installation of the waterproofing membrane sheet. This was partially due to the skill of the applicators of the shotcrete and also to the adoption of a 10 mm maximum aggregate size. The polyethylene sheet was also specified to be fitted with a geotextile backing layer for additional protection against penetration damage.

One of the options considered for the final lining during design development was to use a combined lattice girder and shotcrete lining and not to have an in-situ concrete lining. There were potential cost savings and productivity gains with this option compared to using an in-situ concrete lining. Shotcrete shadow trials (Figure 4) on test panels with sections of lattice girders did not prove satisfactory because it could not be demonstrated that the reinforcement would be fully embedded in shotcrete (Figure 5). This is potentially a long term durability issue. In the final design the fan niches consisted of steel fibre reinforced shotcrete used in combination with a pattern of permanent rock bolts. The pattern of rock bolts also included the 21 stainless steel rock bolts used to support the three jet fans located in each fan niche.

Figure 4. Lattice girder shotcrete test panel.

Both the north and south portals were also supported by a combination of steel reinforced shotcrete and permanent rock bolts or dowels. At both the four fan niches and at the two tunnel portals the reason for using shotcrete was for construction efficiency which also saved costs compared to the alternative of providing formwork which had no or little potential for reuse. The fan niches required an over-excavation of the tunnel crown to accommodate the three 1200 mm diameter jet fans and did not therefore suit the steel formwork of the standard tunnel profile.

Figure 7 shows the application of a spray-on waterproofing membrane at a fan niche location. The spray-on membrane was adopted only after a number of field trials were carried out. Initial trials were also carried using the polyethylene sheet water-proofing membrane (also shown in Figure 7), and proved unsuccessful because the shotcrete would not bond to the polyethylene sheet, for two reasons. Firstly, there was no adhesion, and secondly the sheeting had air voids underneath, creating an unstable surface area on which to shotcrete.

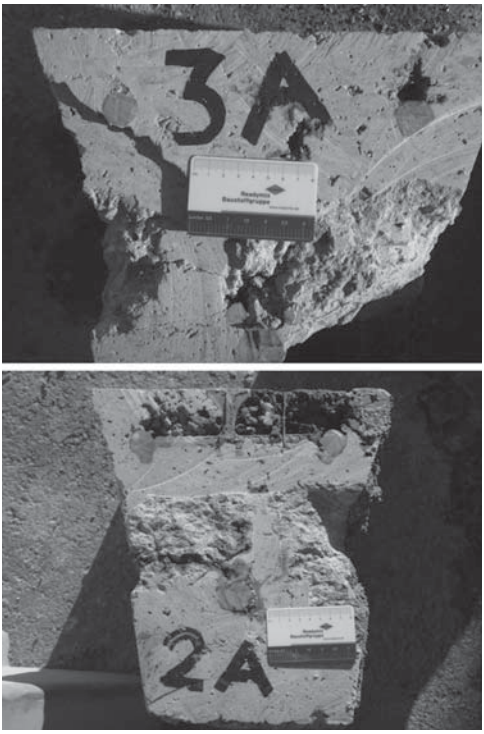

Figure 5. Test panel sections showing voids.

Figure 7. Fan niche waterproofing application.

Figure 6. Shotcreted tunnel portals, north portal shown.

The spray-on membrane required a minimum drying time of 24 hours between layers, much longer than the supplier's estimate of 6 hours.

All shotcrete was applied using a Meyco Suprema robotic rig. Shotcrete and concrete was delivered to the tunnel via two drop pipes located near the north end of the tunnel. Two off-road concrete agitators for use in the tunnel were purchased for the project. Concrete and shotcrete could then be transported efficiently in all weathers and without subcontractors being required to enter the tunnel (Bradford, 2008).

Shotcreting conditions in the tunnel were good with very little or no groundwater ingress impacting on the shotcrete during application. If there was any ground water present this was channeled away to the tunnel invert leaving a dry surface for shotcreting. The tunnel portals initially were established with temporary rock bolts, permanent dowels and an initial shotcrete layer. The permanent support consisted of a pattern of permanent rock bolts, followed by the application of the spray-on membrane, followed by an initial 25 mm of shotcrete without steel fibres, followed by the permanent steel fibre reinforced shotcrete. From Figure 6 above it can also be readily seen that it was possible to include a drainage channel (with a half PVC pipe as the form) across the portal and a vertical niche for the latter matching of the vertical emergency egress side panel from the adjoining cut and cover tunnel (refer to Figure 1). The design life specified for all permanent structural support was 100 years.

7 CONSTRUCTION UNDER THE JAIL

Previous sections of the paper have referred to shotcrete and the other support used under the jail. This section provides more details of the construction sequence aided by a number of sketches which together show the application of shotcrete as used in the shallow cover tunnel section. The aerial view in Figure 8 shows the tunnel alignment under the jail.

Developing the construction sequence under the jail required a lot of forethought and analysis. Two previous examples of shallow tunnel excavations in Brisbane had taken different approaches. In particular, the Vulture Street Busway tunnel completed in 2000 used pre-excavated side drift tunnel filled with mass concrete to create the arch footings. The Buranda Busway tunnel competed at the same time recorded settlement values under over-lying railway tracks of around 20 mm and with less ground cover. Smaller but pre-installed footings were used along the arched section. In the end there was a clear difference in the Buranda design compared

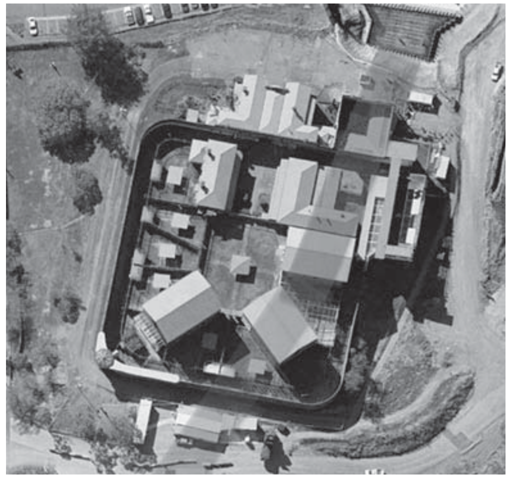

Figure 8. Aerial view of tunnel alignment under the jail.

to that adopted at Boggo. The principle difference was that at Boggo we adopted lighter lattice girders but much thicker shotcrete (350 mm compared to 200 mm thickness over the tunnel arch) and a wall beam, as an arch footing, was only installed as required. To limit the settlement to the predicted values the FE analyses highlighted the sensitivity of the shotcrete stiffness properties on these predictions. This is why during shotcrete trials and construction so much effort was placed on obtaining the early strength of the shotcrete and why a very prescriptive construction sequence was provided on the design drawings for the construction team to follow. In practice the construction team was able to achieve more than specified in Table 5 in terms of applying the full 350 mm of shotcrete near to the tunnel face.

The rate of construction is also an important parameter and the actual excavation rate under the jail was around 9 m per week. This excavation rate was more than adequate to allow the shotcrete to gain sufficient early strength and hence sufficient stiffness. The canopy tubes had the capacity to span alone for 3 m back from the face without shotcrete support (Figures 9, 10 and 11). This was equivalent to at least 2 days excavation production. Referring to Table 3, it can be seen that at 2 days the shotcrete would have attained a stiffness at least above $E = 20{,}000$ MPa (which is well above the average stiffness of the surrounding rock). In any parametric studies, shrinkage and creep values of the shotcrete are almost insignificant given our real knowledge of the actual and obvious wide range of rock mass properties including its stiffness. Behind the tunnel face, after the shotcrete had reached near full strength, the actual stresses in the

Table 5. Tunnel heading excavation and support sequence Type 5 (tunneling under jail).

Sequence No.	Description of excavation and installation support
1	The tunnel is to be excavated by heading and bench.
2	For the heading excavation install an array of 12 m long cement grouted steel canopy tubes from the tunnel over the tunnel arch.
3	In very low to low strength rock install a 1 m grid of 12 m long cement grouted fibreglass dowels.
4	Advance the tunnel heading in 1 m increments. The stability of the face will be continuously assessed by tunnel/geotechnical engineer with each excavation cycle.
5	Stand the lattice girder and holding in position for shotcreting using steel or similar brackets attached to the lining behind the face or welded to any exposed canopy tube, and or by timber blocking.
6	The excavation cannot advance until the lattice girder has been fully embedded in shotcrete as described in Items 7 and 8 below.
7	Up to and including the lattice girder expansion joint in each side of the tunnel fully embed the lattice girder footing and lattice girder arch in shotcrete. A minimum thickness of 200 mm of shotcrete is to be applied over this section of the arch profile between lattice girders.
8	Over the remaining crown arch profile apply 80 mm of shotcrete over any exposed ground between lattice girders and fully embed the last erected lattice girder in shotcrete with 25 mm minimum cover on the inside reinforcement bar(chord B1).
9	As the tunnel advances apply up to 90 mm layer between lattice girders until the minimum uniform 350 mm thickness of shotcrete is achieved behind the tunnel face. i.e. the thickness of shotcrete within 3 m of the face shall not be less than 350 mm before commencing the next excavation cycle.
10	Ensure the shotcrete surfaces including and circumferential joints formed are clean of any debris before applying shotcrete in the next excavation cycle.
11	When the tunnel heading excavation has advanced 9 m repeat items 2 to 10.

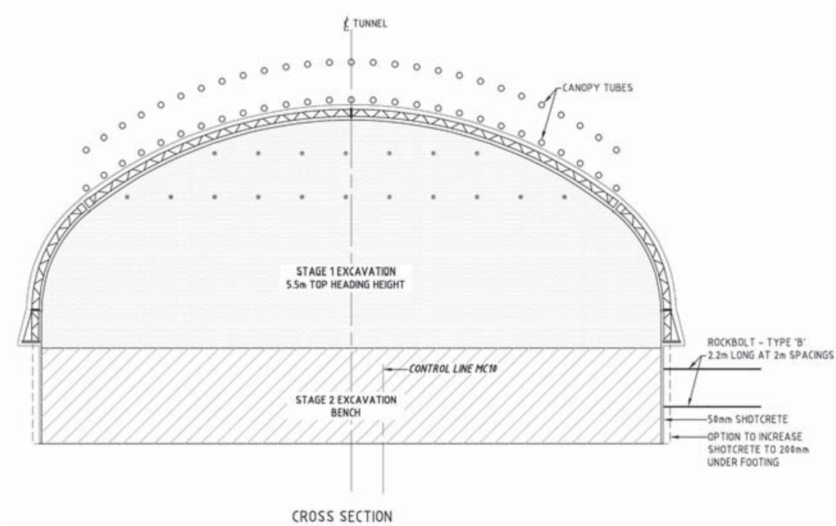

Figure 9. Cross section through the tunnel showing 27 canopy tubes over the tunnel arch.

shotcrete were very low, even with a full overburden loading of around 5 m to 8 m of ground cover, thereby negating creep effects. The shotcrete arch under the jail was also reinforced with steel lattice girders, spaced at 1 m centres. Even considering the rock property uncertainties, the steel lattices girders alone would have negated any potential shrinkage effects of the shotcrete on surface settlement, even if this phenomena were significant.

The widened footing details shown in Figures 12 and 13 were used along the LHS side of the tunnel for a length of 40 m starting from Cell Block Building C (Figure 2), when the tunnel jail intersected highly weathered claystone at the level of the arch springline. The UCS of the claystone rock would have been less than 5 MPa. The sequence of reinforcement and shotcrete installation shown in Figure 13 worked extremely well. The intent of the

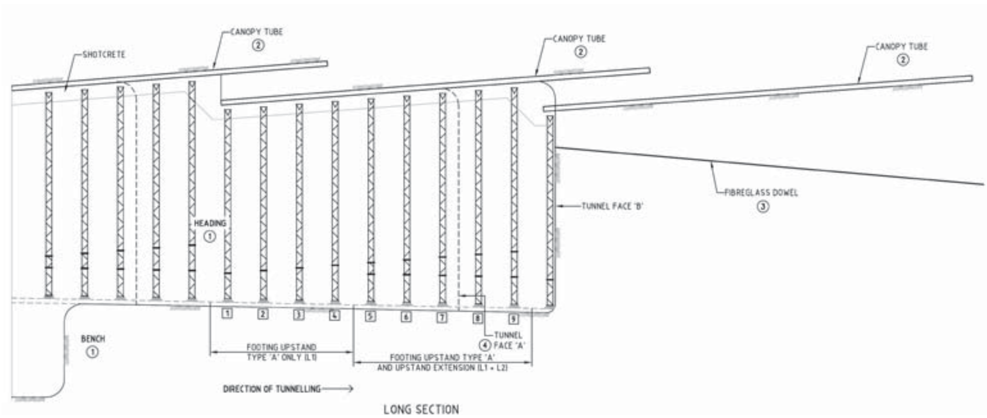

Figure 10. Long section of the tunnel with lattice girders and canopy tubes shown.

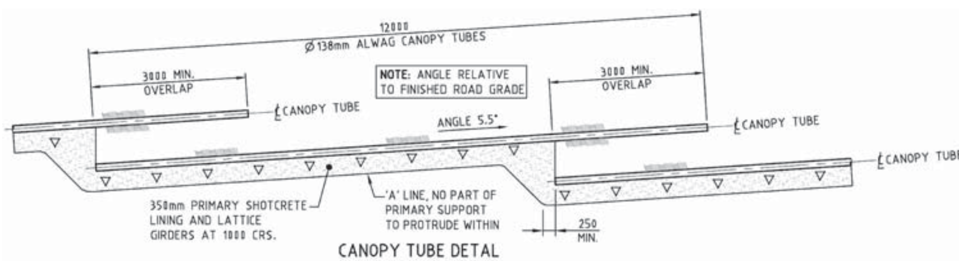

Figure 11. Details of the canopy tube and shotcrete in a section through the tunnel crown.

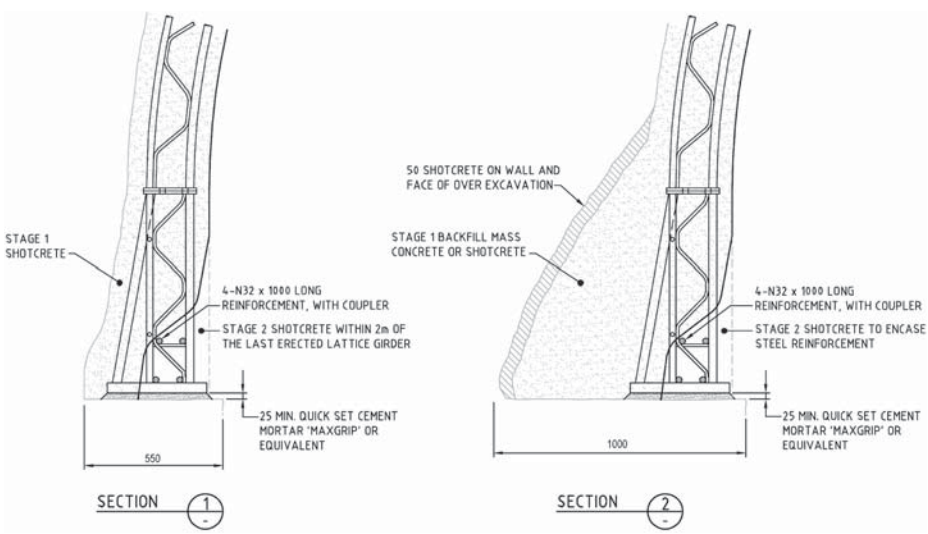

Figure 12. The standard shotcrete and alternative wider footing/wall beam for highly weathered claystone.

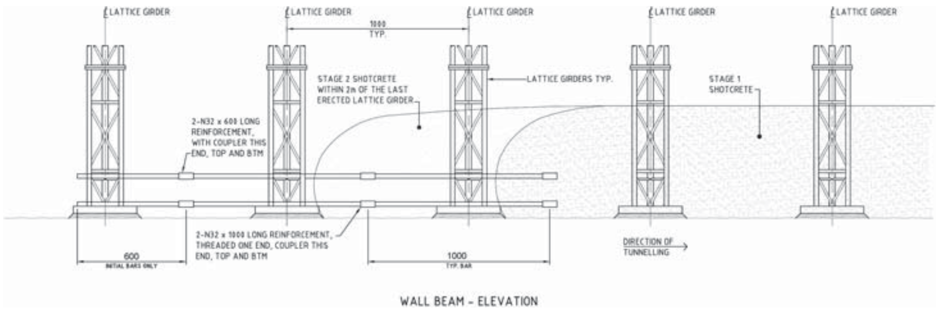

Figure 13. Longitudinal section along wall beam showing the shotcreting sequence.

steel reinforcement was to create a temporary beam prior to subsequent bench excavation to allow the installation of an additional 200 mm shotcrete beneath the beam to act as a continuous deep footing transferring the arch load down to the invert of the tunnel. This last sequence, which is not shown here diagrammatically, was not required.

Together with the tunnel monitoring results during construction the other important tool was the 'Permit to Excavate' process. On a daily basis a team consisting of the geotechnical engineer, the designer and a representative of the construction team would meet to discuss the previous day's construction and prepare for signing a permit for the next 24 hours of tunneling. Items on the agenda included safety, face mapping, monitoring data and the reporting or otherwise of any damage to the surface buildings. Pre-construction surveys of the buildings had been carried out and various instruments were fitted to the buildings to monitoring existing cracks. Visual observations were also regularly carried out, often daily, depending on the criticality of the tunneling work at the time.

Shotcrete was rarely used to stabilize the tunnel face. After face mapping the geotechnical engineer would provide a pattern for the next array of 12 m long fibre glass dowels (the dowels had the same 3 m overlap as the canopy tubes). The number of face dowels numbered over 30 on a regular basis, and were seen as a more reliable method of providing face stability than shotcrete. The exposed geology of the tunnel face varied both across the face and along the tunnel length. One face collapse did occur when it appeared that the dowels did not bond sufficiently to the surrounding rock. In some cases the dowel failed by shear and in others the rock just broke away leaving the undamaged dowel end suspended in the air. This incident resulted in a change in the installation of the lattice girders in that they were stood using four split sets installed across the tunnel arch using the robotic drilling. The footings of the lattice girders were then shotcreted so that there was no need for anyone to place blocking at the lattice girder footing by hand. The shotcrete would set while the lattice girders were fully suspended from the tunnel arch via the split sets.

8 FURTHER CONSTRUCTION COMMENTS

The primary concerns of the construction team, in the installation of shotcrete is safety, quality, cost, productivity and effect on other processes. The construction team worked closely with the designers to achieve the desired structural outcomes with the most efficient application methods. This was achieved through initial constructability reviews held prior to commencement of a section and continuous reviews through the 'Permit to Excavate' process. Constructability reviews provided a great conduit for the designers to communicate intended sequence and outcomes and for the two teams to identify issues and innovations. The Permit process allowed for fine tuning using the added information of monitoring data.

The primary safety concerns regarding shotcrete are adhesion, fall out and how the shotcrete provides support. The high early strengths required allowed the construction team to progress to the next round without delay. The cutting, lattice girder and bolting process was achieved with a 5 m exclusion zone from the face, which allowed sufficient time for the shotcrete to reach a safe strength. Monitoring for cracks yielded very few results, which gave confidence in the support. There were no issues with adhesion due to the rough surface of the encountered rock mass. Fall out was encountered in the initial stages of attempting to spray 350 mm in one pass, however this was alleviated by mix and techniques improvements. Fall out only occurred in the area being currently sprayed, not in previous shifts areas, which also indicated sufficient bonding.

Quality targets are for a consistent product within specification and a finish that will not require a secondary treatment for the application of waterproofing membrane. Shotcrete has a number of variables which can adversely affect specification results which we found difficult to get to a final mix and application method until half way through the project. The introduction of high early strength requirements can affect other mix characteristics, as does the accelerator dosage. Intensive, prolonged testing regime had to be undertaken to come to a mix/application method that performed to expectations. The use of a 10 mm non fiber mix did meet the finish expectations for smoothness to allow for membrane installation, however this is in no small part attributed to operator experience. Other projects have shown that a 7 mm product will provide a superior finish with less skill. Lamination was present in cores to a minimal extent. This was found to due to the accelerator injection method and was minimized by the one pass spray method.

Of course, all tunnel builders want to minimize costs. The removal of fiber from the mix reduced material costs by 1/3. All designs should be issued with a fiber/non fiber depth option as often the economics of the material supply drives towards the non fiber option. Cost improvement measures were also seen in equipment selection. The Meyco proved to be a reliable, cost effective machine with little down time and good application rates. The method of spraying up to 350 mm in one pass created productivity improvements, thus reducing costs, as did the omission of the smoothing layer due to the quality of finish. The one pass method also reduced the interaction of the spraying process with the other in tunnel activities. Smoothing layer activities, which was omitted from this project, can delay services installation.

9 CONCLUSIONS

The application of shotcrete on this project was critical to its success. Shotcrete was used to provide initial stiff tunnel support in an arch profiled tunnel to control ground settlement to within predicted and necessary limits to prevent damage to the heritage listed jail structures above.

As initial tunnel support, with 10 mm size aggregate, no special additional measures were required to protect the sheet waterproofing membrane with its felt backing during its installation. Shotcrete together with the spray-on waterproofing membrane has also proved successful at the fan niches and at the two tunnel portals. Further research into spray-on water proofing products to reduce drying times would be useful.

Shotcrete as permanent support at the fan niches and the tunnel portals saved construction time and costs as formwork at these locations was not required. The shotcrete performed well in all of the remaining support types, 1, 2 and 4 used as initial ground support in the tunnel (Type 3 support was not used). There was also no significant, if any, cracking observed in the shotcrete along the full length of the tunnel throughout construction.

Apparently complex construction processes, such as the widened tunnel wall beam, were carried out very successfully using large localized volumes of shotcrete with no construction delays.

The shotcrete lattice girder shadow trials demonstrated that there could be durability issues with this type of construction, this is particularly so where a design life of 100 years has been specified. Obviously, there are also other important factors that could impact on the design life (e.g. groundwater chemistry).

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