Pacific Highway Upgrade from Tintenbar to Ewingsdale (T2E), St Helena Tunnel

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ABSTRACT

St Helena tunnel is the centrepiece of the Pacific Highway upgrade from Tintenbar to Ewingsdale, abbreviated and commonly known as T2E. At the deepest point the twin tunnels run 45 m below the ridge line of St Helena Hill, located about 5 km west of Byron Bay. The tunnel breakthrough was achieved early in 2014 and the works for permanent lining will be completed in early 2015.

Once the project will be handed over to the public in 2015, T2E will deliver a safer highway and uninterrupted traffic flow between New South Wales and Queensland. The design speed in the project area has been set to 110 km/h by the client Roads and Maritime Services (RMS), which will reduce the travel times between Brisbane and Ballina to just over two hours.

The two tubes of the tunnel have been designed to accommodate three lanes in each direction with shoulders and walkways on either side of the carriageway. These requirements result in a huge cross-section of about 20 m width and an excavated area of about 210 m^2.

With the tunnel alignment being situated in basaltic rock, St Helena tunnel is conventionally driven. The selected excavation method was predominantly drill-and-blast with temporary support being installed immediately after the excavation. Despite the wide span of the tunnel, the geology allowed the tunnel perimeter to be supported by only rock bolts and a layer of shotcrete over the majority of its length.

Due to environmental reasons one of the approval criteria of the project was that the tunnel must not have a detrimental long-term impact on the groundwater system within St Helena Hill. Consequently the tunnel has been designed as a fully tanked structure with a waterproofing sheet membrane installed on the temporary shotcrete.

The permanent structure of the tunnel has to meet a 100-year design life and it is designed as a cast *in situ* concrete lining. Considering the water pressure on the lining the concrete is reinforced with a significant amount of bar reinforcement.

The presented paper provides a more detailed explanation of the geology and the consequential temporary and permanent support measures together with the waterproofing system.

INTRODUCTION

In October 2011 RMS awarded the contract for the Pacific Highway upgrade from Tintenbar to Ewingsdale (T2E) to Baulderstone, now Lend Lease. The project area (Figure 1) is located in northern NSW, with the northern end being about 5 km west of Byron Bay.

The T2E section of the highway is the only remaining part between Brisbane and Ballina that currently operates with two-lane traffic providing only single lanes in each direction. The new highway will provide two lanes in each direction with the option to upgrade to three lanes once they are required by increasing traffic volumes.

After providing a project overview, the subsequent sections of this paper will outline the tunnelling works focusing on the following areas:

- the geology in the project area
- the selected excavation methods
- the temporary support measures
- the permanent support
- the waterproofing system.

PROJECT OVERVIEW

The 18.6 km of newly constructed highway has an overall value of $862 M and it is the last remaining section between Brisbane and Ballina to be upgraded to a minimum of two lanes in each direction. In order to allow a smoother journey and safe travelling at the design speed of 110 km/h, the new alignment has been straightened and the elevation has been levelled. This required a number of civil structures to be constructed and a significant volume of earth to be moved.

In summary the overall project comprises:

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FIG 1 – Project area.

- 16.3 km of dual carriageway
- 10 km of access roads
- nine bridge structures, three underpasses, two culverts
- 3 200 000 m^3 bulk earthworks
- 434 m long tunnel through St Helena Hill.

St Helena tunnel is considered the centrepiece of T2E, with its northern portal located about 1 km south of the exit to Byron Bay, next to a sharp right turn of the existing highway that takes it up to St Helena Hill. The new highway will go straight through St Helena Hill, pass about 45 m under residential properties along St Helena Road and surface again 434 m further south.

After lowering the cut to the level of the tunnel, the excavation works for St Helena tunnel started on 25 March 2013 with the first blast at the southern portal. Drill-and-blast was also the method chosen for the majority of the tunnel with temporary support installed following immediately after the excavation. The first break through in the northbound tube was achieved on 24 January 2014.

Once the tunnel is excavated it is fully tanked with a waterproofing sheet membrane and lined with a cast *in situ* reinforced concrete structure. The first permanent lining concrete was poured on 24 February 2014 and the entire concrete lining is scheduled to be finished in December 2014.

Some key data of the tunnel includes:

- road tunnel – two tubes, three lanes each (future consideration)
- length of tunnels – 2 × 434 m
- cut and cover sections at each end – 2 × 39 m
- spacing of egress cross passages – 120 m
- area of tunnel cross-section – 210 m^2
- excavated volume – 150 000 m^3.

The huge cross-section of the tunnel is a result of the space requirements in the tunnel. With three lanes of 3.5 m in each direction, shoulders of 1 m on both sides as well as traffic barriers and 1.4 m wide walkways also on both sides the tunnel has an internal width of more than 20 m.

Even though it is relatively short, St Helena tunnel has been designed with mechanical and electrical infrastructure equivalent to that for longer tunnels. The installed equipment includes jet fans for ventilation, a deluge system, signage, lighting, video surveillance and an unmanned tunnel control centre on top of St Helena Hill.

GEOLOGY

The project area is located on the Alstonville Plateau, which is characterised by the Lismore basalt sequence and consists of layered sequences of basalt and interbedded sediments. This sequence comprises the southern extent of the Tertiary Period Lamington volcanics and is associated with the Tweed Shield volcano. This volcanic sequence represents a succession of north-westerly dipping felsic/mafic eruptions that took place some 20 million to 23 million years ago. Today the most prominent volcano in the area is Mt Warning.

The Lismore basalt formation irregularly overlies eroded and weathered rock of the Neranleigh-Fernvale group consisting of shales, siltstones, sandstones, conglomerates and greywackes of Devonian-Carboniferous Age.

The Lismore basalt typically consists of sub-aerially extruded lava flows of up to 15 m thickness. In St Helena Hill the height of these flows was observed to be 3–5 m. Within the lava flows, the slower cooling process results in the typical hexagonal columnar joint pattern, with vertical cooling joints and vertical trending basalt columns. On top and the bottom of the lava flows the basalt is more disturbed and air intrusions can be found in the so-called vesicular basalt (Figure 2).

Prolonged periods between volcanic activities allowed for episodes of weathering, resulting in the formation of fossil soils (residual soils) and weathered, fractured basalts together with deposits on top of the weathered surface. Subsequent volcanic activity resulted in the deposition of new lava overlying the weathered and fractured basalts, clays and fossil soils. The high temperature of the lava flowing over the clay and mostly clayey fossil soils 'baked' these and formed a brick-like rock.

The number of lava flows interrupted by periods of volcanic inactivity created successive sequences of fresh basalt columns overlain by vesicular basalt and the brick-like former weathered and fractured basalts, clays and fossil soils.

The tunnel alignment is situated solely in the Lismore basalt. With the height of the tunnel being about 12 m it was expected that the described layering and sequences were also encountered in the tunnel. Figure 3 shows the basalt columns within a lava flow in the top of the heading, followed by a small band of vesicular basalt and the formerly weathered surface soils now backed to brick-like rock. This sequence sits on top of the previous lava flow that will again follow the same sequence underneath the invert. What was surprising was the size of the basalt columns, with about 2.5 m the diameter exceeded the expected 0.6 m to 0.8 m as observed in quarries and other outcrops in the project area.

The current surface of St Helena Hill is similar to the previous periods of volcanic inactivity characterised by weathered and fractured basalts, clays and fossil soils. At the northern end of the tunnel the weathering of the basalt is high to extreme and reaches fairly deep so that over a length of about 75 m the top heading was at least partially situated in the weak ground with almost soil-like characteristics.

EXCAVATION METHODS

As described the majority of the tunnel was driven in fresh and slightly weathered basalt of a strength of more than 70 MPa. This was considered to be beyond the capacity of road headers. Due to the limited length of the tunnel together

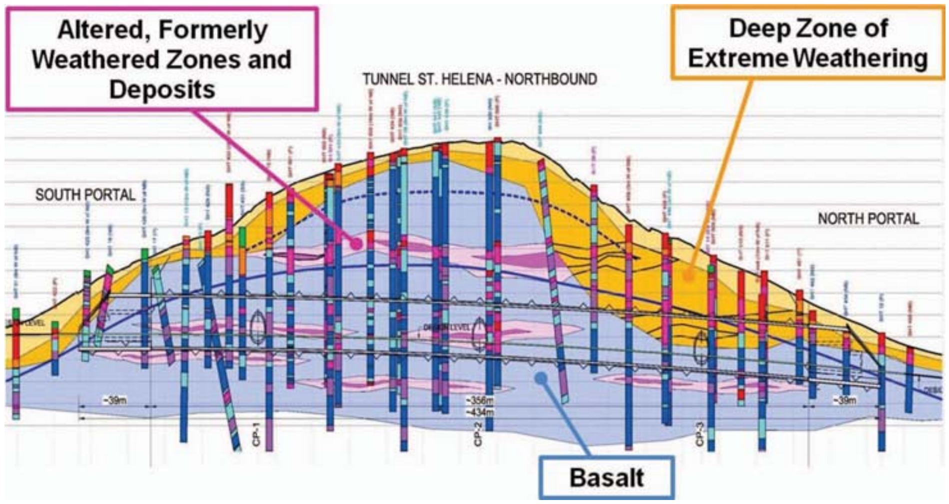

FIG 2 – Expected geology.

FIG 3 – Geology in the tunnel.

with the huge cross-section the use of a tunnel boring machine (TBM) was uneconomical and thus not considered an option.

The tunnel is situated in a rural area with only very few residential properties and no sensitive structures on top of St Helena Hill. Therefore drill-and-blast was the logical excavation method to be chosen together with rock hammers.

About 165 holes with a diameter of 45 mm at a typical spacing of 1 m were drilled at a length matching the required excavation length. These blastholes were filled with emulsion explosives at a rate of 2.9 kg/m^3 of rock. At a spacing of 500 mm to the edge of the tunnel profile so called perimeter holes were drilled at a spacing of 300 mm. The charge for these perimeter holes was significantly less than for the other wholes.

Once the emulsion was placed in the drilled holes, detonators were installed and connected to the trigger instruments. The detonators had delays of 10 ms and not more than four holes were blasted at the same time. These delays together with the perimeter hole led to a low impact blasting with a high accuracy of the blast profile.

The vibration limits were initially set to 5 mm/s at the residential properties on top of the tunnel and due to the low impact were very soon relaxed to 10 mm/s. The vibrations were usually in the order of 7–8 mm/s and therefore well within the limits even for the bigger blasts of up to 5 m length.

After the blast, rock hammers were used to scale down any loose rock and remove tights. The immediate control of the excavated profile helped to reduce overbreak for the blast and resulted in an accurate profile following the profiling with the rock hammers.

At the northern portal the rock in the top heading was predominantly – as described in the previous section – highly weathered with soil-like characteristics. The areas with the weaker material were excavated by an excavator either with a bucket or a cutterhead attached to the boom. In the lower parts of the top heading where the rock got harder a rock hammer had been used on the excavator.

The bench and invert of the tunnel have been entirely excavated by drill-and-blast following the 'quarry method' where the drill holes are being drilled vertically from the temporary invert of the top heading. The profiling was again performed with rock hammers attached to an excavator.

TEMPORARY SUPPORT

Support types

Following every excavation round, ie over the majority of the tunnel excavation following every blast, temporary support measures were installed. This was meant to secure the tunnel perimeter in order to ensure the structural stability and allow a safe re-entry of the workforce into the freshly excavated areas.

Following a comprehensive review of the geology and with the findings of the Geological Interpretative Report (GIR) the designers developed a suite of six different support types. These support types included a combination of the usual support measures like shotcrete, rock bolts, spiling bars, lattice girders and face nails. These combinations and variations allowed an adequate response to the varying geology over the length of the tunnel tubes.

The support types also defined the excavation length and the areas of the face that were considered safe to be excavated before support measures had to be installed. The tunnel face was divided into a minimum of three excavation steps namely heading, bench and invert. In the extremely weathered and therefore weaker ground encountered at the northern end of the tunnel the heading has been split up further into two drifts separated by a temporary sidewall. The round length varied from 1.5 m in the soil-like areas to a maximum of 5 m in the fresh basalt.

Support measures

The support measures that formed the support types varied depending on the ground conditions. These variations included a different shotcrete thicknesses and alternative types and lengths of rock bolts to be installed. The shotcrete was fibre reinforced and the thickness ranged from 100 mm in the fresh basalt to 350 mm at the portals and the extremely weathered zone at the northern end. One first for tunnels in Australia was that synthetic fibres were used to reinforce the temporary shotcrete. With numerous applications in mines it was not a big surprise that these fibres worked well and the shotcrete met all specified structural requirements.

The rock bolts installed were either grouted steel bars in less competent rock and split sets in the hard and unweathered basalt. With safety always being the top priority, the capacity of the split sets was tested on sacrificial bolts immediately after installation to ensure the rock provided sufficient friction. The installation of the friction bolts helped avoiding grouting as an additional step in the construction sequence, which on the one hand saved time but also avoided personnel to be required at the tunnel face. Another advantage of the split sets was that they provide their bearing capacity immediately after being installed without having to wait for a grout to develop strength.

In areas of weaker ground and at the portals spiling bars have been installed together with lattice girders to provide immediate support before the shotcrete has gained sufficient strength. The areas of weaker ground were the fractured basalts and weathered areas particularly at the northern end of the tunnel. At the northern portal three rows and at southern portal two rows of spiling bars have been installed on top of each other. This helped to avoid any canopy tubes that were initially considered.

Support type selection

The selection of the support types followed a transparent process with the selection criteria defined during the design phase already. Based on the GIR, the design classified the rock and described the potential failure mechanisms together with the required support.

The designer's site representative, whose official title on T2E is Tunnel Geotechnical Representative (TGR), compared the *in situ* ground conditions with the design assumptions and he then assessed the monitoring results by relating them to the results of the structural analysis. Once the TGR has completed his interpretation of the ground conditions he selected the adequate support type. As a formal direction to the construction team a Ground Support Instruction Sheet (GSIS) was handed over.

The described procedure was part of a Geotechnical Permit to Excavated (GPTE) Process that had to be followed before every excavation round of every individual face of heading, bench or invert. The TGR met at least with the tunnel construction manager to discuss the geology, the monitoring results, the as-built documentation, test results and any observations before the GSIS together with the GPTE could be issued; however, representatives from the Client and the Project Verifier were present in almost every single one of these meetings.

This process may sound quite onerous but after only a few days it became a routine and everyone knew what information had to be provided and reviewed. The huge benefit of these joint meetings was a transparent process, which eliminated the risk of a breakdown in communication. It also reduced the risk of misunderstandings and overall made the excavation and support works of the tunnel a very safe process.

PERMANENT SUPPORT

General

The 100 years design life of the permanent support has to be achieved without considering any of the temporary support described in the previous sections. Consequently a cast *in situ* concrete lining with a minimum strength of 40 MPa has been designed to permanently support the tunnel perimeter. The minimum design thickness of the lining is 500 mm in the mined areas and 800 mm in the cut-and-cover sections.

At an early design stage it was investigated if the concrete lining could be unreinforced or reinforced with structural fibres only. Being a fully tanked tunnel however, the lining has to withstand a water pressure equivalent to a water table of 25 m above tunnel crown. The first results of the analyses indicated that the forces caused by the water pressure could only be borne by a bar-and-mesh-reinforced concrete structure. In addition the invert has been arched similarly to the tunnel roof in order to support the loads from the full water pressure.

Construction of the permanent lining

The concrete of the permanent lining will be cast in longitudinal sections of a typical length of 10 m. These so called blocks are again poured in two independent steps. The invert up to about 0.8 m below the level of the finished carriageway will be cast first with the tunnel vault following in an additional step. The joint between the invert and the vault is an actual construction joint with reinforcement bridging the joint. The joints between the blocks (block joints) are left untreated and no reinforcement or other connection between the blocks has been designed. This ensures that the individual blocks work as a complete ring but do not transfer stresses caused by differential movements from one block to the next.

The reinforcement is delivered in bent bar and is installed from a custom-made gantry. Before the concrete is poured the reinforcement in the arch has to be self-supporting. Consequently the lattice girders are erected first and the bar reinforcement is assembled using the lattice girder as support and a spacer between the inner and outer reinforcing layers. In addition the reinforcement is supported by so-called BA anchors. The BA anchors consist of a threaded feral together with a ring of sheet membrane at the open end of it. To install the feral a hole must be drilled through the waterproofing sheet membrane and the feral can be installed by grouting it into the hole. The collar of sheet membrane is welded around the hole, which is drilled through the waterproofing membrane and thus the hole could be sealed again. Once the feral is installed a threaded bar is inserted into it, which will then provide temporary support for the reinforcement.

Both parts of the lining are formed with a custom-made steel formwork shutter (Figure 4). The invert shutter is open in the centre section and only has to form the haunches on the sides where the curvature of the lining would prevent the fresh concrete from staying in the required shape.

The shutter for the vault is a massive steel structure that supports the weight of the fresh concrete and has also sufficient built-in vibrators to compact the concrete. This shutter runs on rails and allows the tunnel to be cast in blocks of the standard length of 10 m. Before each pour, the shutter will be moved in place and the skin of the formwork fixed in by built-in hydraulics. After the correct position has been confirmed by a survey check the concrete can be placed through hatches

FIG 4 – Formwork shutter.

in the steel skin from the bottom-up in defined steps. Once the concrete has reached the level of the hatch it will be closed and the concreting pipe will be moved to the next hatch. During the pouring process the concrete is compacted by vibrators that are installed on the formwork shutter.

The formwork can be stripped as soon as the concrete has reached a compressive strength of 6 MPa. This allows a continuous production, which after going through the learning curve is programmed to be up to one pour per day.

Urban landscape

As contractually required in the Deed, the tunnels are designed to have a low visual impact on the environment. This is why the portals have a very basic collar that from an architectural point of view is 'very low key' and the portals follow the natural slope of St Helena Hill.

The concrete of the permanent lining of St Helena tunnel will be exposed to the view for the drivers (Figure 5). This is different to most of the tunnels in Australia where the tunnel lining is 'hidden' behind reflective panels. The concrete lining in St Helena Hill will be coated with a reflective layer of paint with different shades of grey in a tessellated pattern. This coating allows the lighting to be reflected evenly and the tessellation will also provide the motorists with a special experience during their short journey through the tunnel.

The emergency egress cross passages have been designed with prominent shapes and colours. These very pronounced features will make them easy to identify in case of an emergency.

WATERPROOFING

Requirements

The avoidance of an impact on the groundwater system and therefore the reinstatement of the groundwater level after the construction was one of the approval conditions for the project and a major concern for the client. A temporary lowering of the groundwater table during construction was permitted; however, for the permanent state the tunnel has to be tanked with waterproofing sheet membrane.

Tanking of the lining

For St Helena tunnel a 2 mm PVC membrane (Figure 6) with a thin signal layer has been chosen to ensure the water tightness of the lining. As usual these sheets are laid out around the perimeter of the tunnel and heat-welded together with double seams. The void between the seams is tested with a pressure to make sure the welding was 100 per cent tight.

FIG 6 – Waterproofing membrane and reinforcement in the invert.

Experience from other tunnels has shown that it is nearly impossible to have perfectly installed sheet membrane and without even minor leakages. In order to be able to remedy even small water ingress through potential damages within the sheet membrane water stops are installed and welded to the waterproofing sheet membrane in every block joint and the construction joint between the invert and the vault.

In case of any potential leaks the water stops prevent on the one hand water from seeping through the joints and on the other hand form compartments along the tunnel perimeter between the concrete and the waterproofing membrane and thus limit any water inflows to each individual block. By installing hoses that go through the concrete lining into the potential void between the concrete lining and the waterproofing, membrane post grouting is also possible to seal any damages in the waterproofing membrane.

The waterproofing sheet membrane prevents inflows into the tunnel which is on one hand important for the operation

FIG 5 – Finished tunnel.

and maintenance of the tunnel but on the other hand is also crucial to ensure that the tunnel will not work as a huge drain pipe, which would prevent the water table from recovering once the tunnel is finished.

Measures outside the lining

The requirement to ensure that the water table will recover meant that the tunnel itself had to be watertight but it also had to be prevented that the area around the tunnel lining would work as longitudinal drain.

Apart from the installation of the membrane the following aspects had to be considered:

- a disturbed zone along the tunnel perimeter that could cause a longitudinal water path along the tunnel lining
- a void forming between the permanent tunnel lining and the temporary lining or rock surrounding the tunnel.

The impact of the disturbed zone around the tunnel was assessed by empirical and numerical methods. Experience from previous tunnel projects compiled in literature and advice from blasting experts suggested that a noticeable blast impact would not go beyond 1 m around the tunnel perimeter and that it would not increase the permeability of the rock by more than one order of magnitude. These parameters have been investigated in a numerical model. The results showed that the impact of the potentially increased permeability was negligible. It was also assessed that only a continuous disturbed zone of higher permeability would act like a longitudinal drain pipe. This however was concluded to be unlikely and therefore no overall detrimental impact was considered.

The mentioned void between the lining and the shotcrete or the rock can have two different causes. Shrinkage of the concrete might cause the lining to separate from the shotcrete or rock by a few millimetres creating a very small and more or less negligible void. The second and more significant effect is a deformation of the tunnel lining caused by the high water pressure. With the relatively wide and flat tunnel profile the analyses shows that the expected uplift particularly in the tunnel invert has a potential to form a void of about 30–50 mm. The analysis of this gap demonstrated that a persistent void of this magnitude gap might have the potential to cause some dewatering of the hill and therefore would prevent a full recovery of the water table.

In order to prevent the persistency of the gap and interrupt a potential water path in longitudinal direction various concepts have been considered. Initially it was intended to close the gaps with bentonite strips at spacings of 50 m. These bentonite strips would swell when in contact with water. Where the swelling capacity was considered sufficient to close the gaps created by shrinkage it could not be demonstrated that the swelling capacity was sufficient to close a gap of 50 mm.

The solution for the larger voids is to install membrane flaps that are designed at locations where different water pressures are assumed in the structural analysis. These flaps consist of strips of sheet membrane that are on the outside of the membrane. One end of the flap is glued to the shotcrete with an epoxy and the other end is welded to the sheet membrane. These flaps have been tested to ensure sufficient capacity to withstand a differential water pressure from one side of the flap to the other.

At the location of the flaps it was also decided to install grout rings in the rock around the perimeter of the tunnel. This measure is meant to prevent a potential short circuit around the flaps created by disturbed rock of increased permeability.

CONCLUSIONS

St Helena tunnel may not be one of the longest tunnels recently built in Australia. It has however one of the largest standard cross-sections of all road tunnels and it will be equipped very similarly to its longer counterparts.

From a geological point of view St Helena Hill presented two challenges. After the initial reviews it was considered that the tunnel would entirely be situated in solid basalt. A thorough study of all data from the comprehensive exploratory program showed that the basalt was intercepted with layers of weaker and more weathered rock that could even have soil-like character. The second challenge was the understanding of the groundwater system and the recovery of the original groundwater table.

Joint efforts that involved the client, the contractor, the verifier and the proof engineer ensured that all challenges could be resolved and the tunnel could be constructed safely and with outstanding quality. Once the project is completed and the road handed over to the public, the journey from Byron Bay will be considerably shorter and safer.

REFERENCES

Hyder Consulting. GH01 – Hydrogeology report G00-GH01-RPT-0056, hydrogeology report for T2E.

Mueller and Hereth. Tunnel design reports and drawings for St Helena tunnel.

Road and Traffic Authority (RTA, now RMS). Pacific Highway upgrade, Tintenbar to Ewingsdale, design and construct project deed.