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Design of the Te Ara O Te Ata – Mt Messenger Bypass project tunnel

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ABSTRACT: This paper discusses the design of a 235m long, single bore highway tunnel that forms a part of the upgrade of State Highway 3 in rural Taranaki, New Zealand. The tunnel is excavated within the Mt Messenger Formation, which comprises marine turbidite origin sandstones and mudstones, which are relatively soft horizontally bedded rocks. The tunnel will be excavated using the sequential excavation method with a roadheader to achieve an asymmetric horseshoe shaped tunnel with permanent shotcrete installed in each excavation advance using robotic spraying equipment. This paper describes the functional requirements and the tunnel lining design with a focus on the single pass shotcrete permanent lining. It presents lessons learned in designing this solution, being innovative for New Zealand road tunnels.

1 INTRODUCTION

The Mt Messenger Bypass project involves the construction and ongoing operation of a new section of State Highway 3 (SH3), generally between Uruti and Ahititi to the north of New Plymouth. This new section of two-lane highway will bypass the existing steep, narrow and winding section of SH3 at Mt Messenger. The primary objectives for the new section of state highway are to enhance the safety, resilience and journey time reliability of travel on SH3, and contribute to enhanced local and regional economic growth and productivity for people and freight. North of Uruti, the existing SH3 route cuts through the north Taranaki hill country, consisting of narrow ridges with steeply sloping valley sides and deeply entrenched drainage networks that typically have very small or no flood plains.

The Mt Messenger Bypass is designed and constructed by the Mt Messenger Alliance (MMA), which comprises NZ Transport Agency Waka Kotahi (NZTA), Downer Enterprises, HEB Construction, Tonkin & Taylor and WSP.

1.1 *Background*

The bypass is approximately 5.3km long, with a 235m long tunnel east of the Mt Messenger peak to cross a local ridge line and avoid an excessively deep cutting. The tunnel is bi-directional, has a slight curve with radius 1100m, 3% super elevation and a crest of radius 4200m. The tunnel is located at the highest elevation along the proposed bypass route. The finished road level at the northern portal will be at an elevation of approximately 111.3mRL. The tunnel is approximately 12m wide and 9.0m high to allow passage of over-size vehicles; equating to a cross-sectional area of approximately 100m2. It is asymmetric in cross section due to inclusion of a longitudinal pedestrian egress passage on one side. The approaches to each portal are in cut with associated steep rock slopes.

1.2 *Functional requirements*

The overall functional requirements for the tunnel were specified in the Minimum Requirements and are summarised below at a high level:

- Enable safe and efficient movement of traffic;
- Comply with the Building Code;
- The tunnel Fire Life Safety provisions shall be acceptable to Fire and Emergency New Zealand (FENZ) based on a risk-based assessment;
- Be designed as an over-dimensioned route;
- In this rural location, accommodate cyclists and pedestrians on the shoulders;
- Permit the passage of dangerous goods.

The passage of large oversize loads with a vehicle envelope of 10m x 6m drove the selection of a single bi-directional bore for the tunnel which has the disadvantage of not having a second bore available to egress to in an emergency and increases the risk of vehicle collision.

The fire-related performance requirements proved a particular challenge. Whilst a relatively short tunnel, sufficient provision needed to be designed to maintain safety for users and asset protection in the event of a fire. Considerations include the tunnel's remote location, about 1 hour away from the nearest emergency services and isolated from water utilities, it's bidirectional traffic flow and dangerous goods are permitted.

Based on a project risk assessment and NZTA guidance codes the final solution relies on self-evacuation from portals via a dedicated longitudinal egress passage in the event of a fire. Lighting, fire hydrants, remote detection monitoring, flame traps and communications are included but no deluge system or jet fans are provided.

The tunnel portal canopies are required to protect road users from rock fall and soil slips. The tunnel is considered a significant gateway to the New Plymouth region with the constructed portals being visually dominant so that appropriate consideration was given to aesthetics and cultural representation, the final form including enhancements (textures, rebates and patterns) of the visible faces of both the portal and egress passage on their pre-cast unit panels. The tunnel electrical plant and control systems will be located in a free-standing Tunnel Control Building located on the new highway embankment approximately 150m to the south of the tunnel.

The tunnel is designed as a drained structure and the drainage system will serve to collect any water entering the tunnel from the ground, tunnel wash down water, fire-fighting water or spillage. The carriageway has a consistent 3% fall away from the egress passage. The tunnel drainage system consists of open drains along the southbound barrier face, catch pits to act as flame traps, and a main carrier pipeline. The tunnel pavement structure is designed for a minimum design life of 25 years and the asphalt surfacing has a design life of 10 years. Services are located inside the egress passage at roof level, on cable trays suspended from the tunnel roof or in ducts incorporated into the TL4 vehicle barriers.

2 GEOLOGY

The Mt Messenger Bypass tunnel is excavated within the Mount Messenger Formation (MMF), a Late Miocene sedimentary sequence comprising interbedded marine sandstones and mudstones of turbidite origin. These rocks are characteristically very weak to weak, with slightly weathered to fresh materials typically encountered within 2–3 metres of the surface, deepening to around 10 metres on exposed ridges. The formation is sub-horizontally bedded, with a gentle dip of 2–4° southwest. Bedding is typically thick to massive and often indistinct. The MMF sediments are exposed in numerous rock cuttings, short tunnels and natural outcrops along the existing State Highway 3 in the Mt Messenger area and along the 'White Cliffs' coastline to the west.

The engineering geological model identifies two broad classifications of rock mass conditions relevant to tunnel design: Rock Mass Class 1 and Rock Mass Class 2.

- Rock Mass Class 1 represents the typical ground conditions along the tunnel alignment, characterised by slightly weathered to unweathered, very weak to weak rock with relatively widely spaced defects and intact rock blocks (GSI 70 to 85+).
- Rock Mass Class 2 represents localised zones of more fractured or broken rock, associated with closely spaced joints, occasional shear zones, or fault-related disturbance (GSI 50 to 70). These zones are typically narrow (2–4 metres wide) but appear to be sub-parallel to the tunnel axis and if encountered, are expected to migrate across the tunnel from left to right (when viewed in the direction of the tunnel advance).

Figure 1. MMF rock exposures in the Taranaki area

Laboratory testing of intact rock samples indicates that the Mount Messenger Formation exhibits a typical unconfined compressive strength (UCS) ranging from approximately 2 MPa to 10 MPa, with an average UCS of around 4 MPa used for tunnel design purposes.

Hydrogeologically, the Mount Messenger Formation exhibits generally low permeability; however, perched groundwater conditions can develop along discrete sand-dominant beds and in defect networks. These factors informed the drainage strategy and support design, ensuring stability and durability during both construction and operation.

3 TUNNEL

3.1 *Tunnel cross-section*

The asymmetric tunnel cross section is shown in Figure 2.

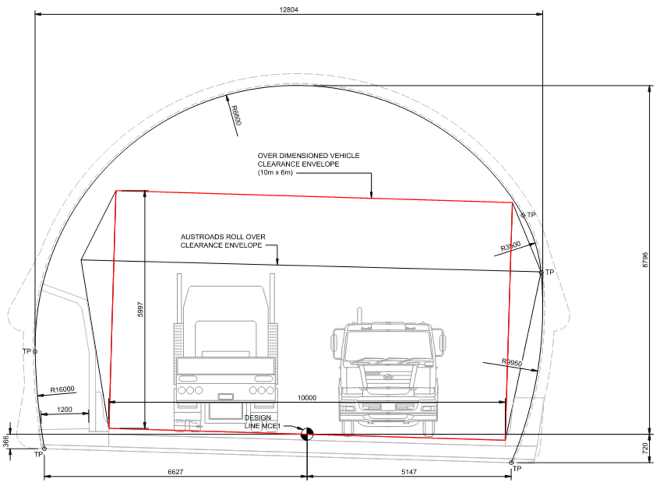

Figure 2. Tunnel cross section

The tunnel shape is space proofed to accommodate the oversize vehicle envelope (6m x 10m) and the tunnel width is then enlarged asymmetrically to provide space for an egress passage on one side of the tunnel cross-section. The passage minimum width is 1200mm. Vehicle clearances and sway allowances are made, and the shape allows room for shoulders and vehicle collision barriers. The large ceiling space that results is more than adequate for lighting and space proofed for jet fans should they be required in the future. Ancillary equipment, such as fire hydrants, fire main, drainage, mechanical, electrical and control equipment have been designed in line with standard approaches and are routed within the longitudinal egress passage.

3.2 *Tunnel excavation and support types*

The tunnel is being excavated using the Sequential Excavation Method (SEM), with a 6.5m high top heading followed by bench excavation. Mechanical excavation is undertaken using a road-header, selected for its suitability to the relatively soft, horizontally bedded rocks of the Mount Messenger Formation. Excavation is proceeding from south to north, with application of ground support following each advance to maintain stability and control deformation.

A single-pass permanent shotcrete lining was selected as the preferred option to enable a fast installation and cost-efficient lining option for this rural tunnel.

Ground support selection was based on the expected variability in ground conditions along the tunnel alignment. Six Support Types (ST1, ST1A, ST1B, ST2, ST3, and ST3A-Portal) were developed to address different rock mass conditions and geometric requirements at the portals. The following summarises the rock mass conditions applicable for each Support Type. Details of the minimum support requirements for each Support Type are presented in Section 4.3.

- Support Type ST1 is the standard design for the majority of the tunnel alignment, where the ground conditions are characterised by slightly weathered to unweathered, very weak to weak Mount Messenger Formation rocks (Rock Mass Class 1), with widely spaced defects and intact rock blocks.
- Support Types ST1A and ST1B were developed to cater for localised zones of more fractured rock (Rock Mass Class 2), typically associated with joint swarms or fault-related disturbance near the tunnel sidewalls. These types allow for enhanced support where isolated zones of reduced rock mass quality are encountered.
- Support Type ST2 was developed for conditions where extensively fractured rock (Rock Mass Class 2) is encountered across the full tunnel face. Although such ground conditions are not expected to be encountered, ST2 provides a contingency support option to ensure safe construction progress through unfavourable zones should they be encountered.
- Support Types ST3 and ST3A-Portal apply specifically to the tunnel sections near the north and south portals. These zones are associated with reduced overburden cover, greater relaxation effects, and a need to accommodate enlarged tunnel profiles for installation of precast portal canopy structures.

Support Type selection during construction is confirmed through an observational approach based on tunnel face mapping, rock bolt installation records, probe drilling results, and tunnel deformation monitoring. This adaptive methodology ensures that ground conditions are appropriately matched with support requirements, maintaining safety while optimising construction efficiency.

4 DESIGN APPROACH AND METHODOLOGY

4.1 *Design approach*

The initial design of the tunnel excavation in the relatively weak rock sought economy by moving away from a traditional primary lining and cast in situ concrete secondary lining by adopting a single pass lining with un-tensioned permanent rock bolts and thin shotcrete lining installed as the tunnel advanced. This relies critically on the strength gain from the grouted rock bolts and shotcrete to facilitate the advance rate of the tunnel and also on the quality of the shotcrete achieving permanent works standards in the challenging environment of an advancing tunnel. Durability and watertightness of a permanent shotcrete lining is also dependent on method of the total thickness build up and workmanship of the shotcrete operators.

In the final design stage, the support system was optimised to accommodate faster advance rates driven by changed project programme demands. The permanent rock bolts were changed to temporary which provided programme and cost savings due to reduced design life requirements. The shotcrete lining build up was thickened and changed from spraying over five advances to spraying over three advances with the last layer applied at a distance from the tunnel face.

The tunnel lining was designed with adequate strength capacity under static and seismic conditions considering:

- Imposed ground and groundwater loads in temporary and permanent cases.
- General loadings (vehicular impact, self-weight and seismic) derived in accordance with the NZ Bridge Manual and NZS 1170 and on the basis of a 100-year design life and Importance Level 4 for the structure;
- Excavation methodology, advance lengths and ground stand up time; and
- Durability for a 100-year design-life

4.2 *Analysis*

4.2.1 *Fem modelling*

The tunnel shotcrete was designed as a single-pass permanent lining, requiring high early strength to allow rapid construction progress and ensure personnel safety. The specified shotcrete mix incorporates steel fibre reinforcement and targets a 28-day compressive strength of 35 MPa.

A minimum compressive strength of 1 MPa is required for safe personnel entry following application of the initial 75 mm thick safety shotcrete layer, typically achieved within a few hours of spraying. The minimum strength requirement was checked for potential failure modes including debonding, shear, and flexure using the UnWedge software, considering the anticipated joint set orientations.

The full design thickness of the lining is built up progressively over three layers, with ground–structure interaction and staged excavation analysed using the RS2 Finite Element (FE) software. The Convergence-Confinement Method [1][2] was applied to estimate the three-dimensional nature of an advancing tunnel face by developing Ground Reaction Curves (GRCs) and Longitudinal Displacement Profiles (LDPs) representative of the expected ground behaviour. LDPs were estimated using the simplifying assumption of an axisymmetric model along the axis of the tunnel (with equivalent radius) to predict the increase in ground convergence with distance from the excavation face.

Groundwater loads were incorporated into the analysis to account for the potential build-up of pore water pressures behind the lining over the tunnel's 100-year design life (the tunnel is assumed to be fully drained during construction). The design groundwater pressure distribution assumed a fully drained invert and a linearly varying hydraulic head, with a maximum pressure of 65 kPa at the tunnel crown reducing to zero at the invert level. The potential for unbalanced groundwater pressures acting asymmetrically on the tunnel lining was also considered during sensitivity analyses.

4.2.2 *Advance length*

Optimisation of the top heading advance length was a key factor in balancing construction efficiency with ground stability. For the standard tunnel section (Support Type ST1), an advance length of 1.5 metres per excavation cycle was adopted, based on numerical modelling outcomes and construction practicability. This advance length allows sufficient time for the installation and early curing of the initial shotcrete layer and temporary rock bolts without exceeding tolerable deformation limits.

A reduced top heading advance length of 1.2 metres is adopted for the portal zones (Support Types ST3 and ST3A-Portal) and where more fractured ground (Rock Mass Class 2) is encountered across a significant portion of the tunnel face (Support Type ST2). This reduction provides an additional stability margin in areas where ground conditions are less favourable.

Excavation of the bench is planned only after the full completion of the top heading and assumes that the fibre reinforced shotcrete lining of the top heading has achieved its full 28-day compressive strength. The typical bench advance length is 6 metres for ST1. In portal zones or where more fractured ground is encountered, the bench advance was limited to no more than twice the preceding top heading advance length to maintain structural stability during the staged excavation sequence.

4.2.3 *Structural fire performance*

As no restrictions were placed on the passage of Dangerous Goods Vehicles (DGVs) through the Mt Messenger Bypass Tunnel, a Rijkswaterstaat (RWS) fire curve was applied to the critical areas of the tunnel as the worst credible design fire scenario. This curve assumes that in the worst-case scenario, a 50 m3 fuel, oil, or petrol tanker fire with a fire load of 300 MW can occur, lasting up to 120 minutes. The ambient temperature within the tunnel is assumed to be 20ºC.

Two fire event scenarios, during fire and post fire, were investigated to assess the response of the tunnel structural lining. Structural analysis was carried out using the FE analysis package Strand7. Loading actions applied in the analysis included combinations of loads transmitted from the ground based on the ground conditions and thermal loads induced by the expansion of the lining. Structural analysis was performed considering the type of ground supports installed, ST1 or ST2. A rock mass modulus (Erm) of 1200 MPa was adopted for ST1 in rock mass Class 1 (SC1) and rock mass modulus of 600 MPa was adopted for ST2 in rock mass Class 2 (SC2).

During a fire in the tunnel, heat will cause thermal expansion of the lining. The induced temperature gradient exhibits high intensity temperature at the tunnel face and decreases with distance away from the face. The associated differential thermal expansion through the tunnel lining section will induce bending strain on areas exposed to the heat source. These thermal actions in combination with pre-existing loads were assessed to ensure that the structural capacity of the lining could still achieve sufficient factor of safety, even after spalling had occurred. An allowance of 40mm of spalling of the shotcrete was assumed in the design.

After the fire, these induced structural actions would cease as the lining temperature returned to normal levels. A post fire assessment was undertaken to confirm whether the reduced thickness of the lining due to the fire damage and spalling would remain stable under the pre-existing load combinations to ensure that the subsequent inspection and repair works could be carried out safely.

Calculated equivalent lining cross section dimensions and thermal inputs, applied in the Strand7 model, are summarised in Table 1.

Table 1. Equivalent section size and thermal input for beam element

Fire Type	Total Thickness (mm)	Equivalent Young's Modulus (MPa)	Equivalent Membrane Thickness (mm)	Equivalent Bending Thickness (mm)	Equivalent Tempera-ture (ºC)	Equivalent Tempera-ture Gradient (ºC/m)
RWS	250	13637	145.8	667.1	141.0	3773
RWS	300	14226	186.2	810.8	84.9	2158

The minimum tunnel lining thickness (fibre reinforced shotcrete) adopted for support types ST1 and ST2 were 250 mm and 300 mm respectively. Hence these minimum lining thicknesses were adopted in the Strand7 assessment. The minimum concrete characteristic compressive strength for the shotcrete was 35 MPa. Creep and shrinkage of the shotcrete were considered in the modelling through reducing the shotcrete modulus of elasticity (long term) from 30 GPa to 15 GPa.

Combinations of ground and fire loads were applied on all iterations of the tunnel lining fire assessment. Ground load consisted of vertical uniformly distributed loads (UDL) applied on the full tunnel lining above the tunnel spring-line and horizontal UDL applied on the full tunnel lining from crown level to the invert level. Ground wedge loads consisted of a vertical UDL applied locally on the tunnel crown centreline, a vertical UDL applied asymmetrically on the tunnel crown and a lateral UDL applied on the tunnel sidewall. Fire load consisted of temperature expansion fire load applied at the nodal elements affected by fire and temperature gradient fire load applied on the beam elements of the lining affected by fire.

Sensitivity analysis was performed by varying the rock mass modulus (Erm) from 1200 MPa to 2400 MPa for ST1 and from 600 MPa to 2400 MPa for ST2 with all other conditions remaining unchanged. The assessment for these sensitivity checks indicated that in the event of a fire (as represented by the design fire curve) the lining had adequate capacity to perform as required both during and after the fire event.

4.3 *Tunnel support design*

The detailed design of the tunnel support types was developed to meet the functional requirements for stability, durability, and constructability under the range of ground conditions anticipated along the tunnel alignment. The requirements for each support type described in Section 3.2 were based on ground–structure interaction analyses, structural performance criteria, and practical construction considerations.

Each support type incorporates specific design features, including variations in shotcrete thickness, reinforcement detailing, rock bolt layout, and drainage provisions, tailored to match the expected ground behaviour. The design approach aimed to ensure consistent tunnel performance across both the typical and more adverse ground conditions, while maintaining flexibility to adapt to actual conditions encountered during construction.

Support Type ST1 was developed for the majority of the tunnel alignment, where slightly weathered to unweathered Mount Messenger Formation rocks (Rock Mass Class 1) are encountered. The minimum support layout for ST1 is shown in Figure 3 and comprises:

- 4 m long fully grouted temporary rock bolts installed at approximately 1.5 m spacings,
- A 250 mm thick steel fibre reinforced shotcrete lining, applied in three layers (75 mm + 100 mm + 75 mm),
- No steel reinforcing mesh under typical conditions.

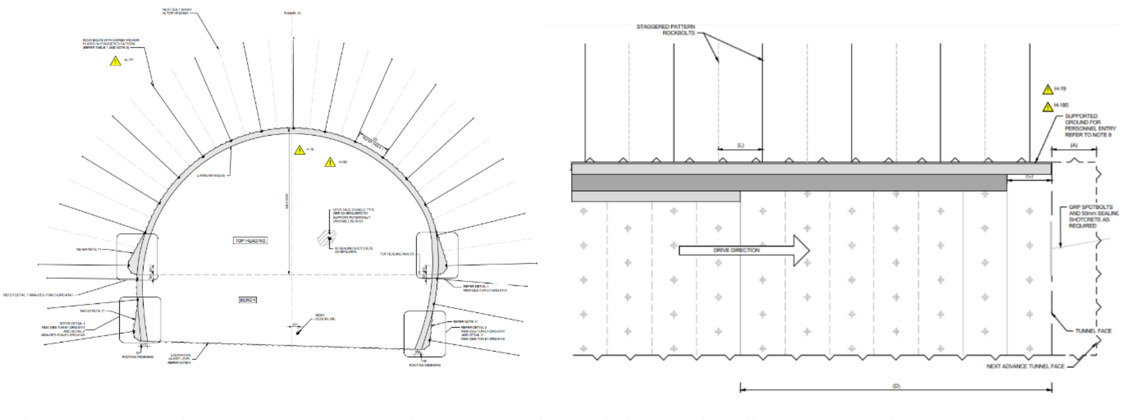

Figure 3. Tunnel support type st1. Left: cross section. Right: top heading construction sequence.

Where Rock Mass Class 2 is encountered on one side of the tunnel, Support Types ST1A and ST1B are adopted. These involve increased bolt density, a widened lining footing at the sidewall, and installation of a single layer of steel reinforcing mesh across the affected top heading sidewall. Shotcrete thickness remains at 250 mm for both support types.

In cases where Rock Mass Class 2 is encountered across the full tunnel face, Support Type ST2 is implemented. This design includes a 300 mm thick steel fibre reinforced shotcrete lining, applied in three layers (65 mm + 130 mm + 105 mm), with high-density rock bolt installation (typically at 1.2 m spacings) around the full perimeter. Two layers of steel reinforcing mesh are provided across the top heading sidewalls and shoulders.

At the portal zones, Support Types ST3 and ST3A address the combined challenges of reduced overburden, increased potential for ground relaxation, and the enlarged tunnel geometry required for the installation of precast portal canopy structures. These designs generally mirror ST1 but include a 300 mm thick shotcrete lining (applied in three layers of 75 mm + 100 mm + 125 mm) and widened lining footings at both the top heading and bench levels. In addition, a canopy of 8m long spile bars provide pre-support to the tunnel.

Minimum Requirements excluded application of waterproofing membrane with the expectation that watertightness is provided by the shotcrete lining alone. Although the Mount Messenger Formation is generally of low permeability, drainage measures were incorporated across all support types to prevent the build-up of pore water pressures behind the lining and divert groundwater to the invert. Groundwater control measures include strip drains at locations where seepage is observed during excavation, regularly spaced perimeter bored drains in the sidewalls and an invert drainage system beneath the tunnel floor. Following installation of the initial safety shotcrete layer, any minor seepage or localised dripping from the excavation surface is managed by installing strip drains prior to the placement of the full lining thickness. In cases where multiple closely spaced water-bearing defects are encountered, the excavation is locally widened to accommodate closely spaced strip drains behind the minimum design lining thickness, ensuring effective long-term drainage and preserving the integrity of the tunnel lining.

5 CONCLUSIONS

The Mt Messenger tunnel is currently under construction. The tunnel design has included a number of innovations for New Zealand encouraged by the need for cost effectiveness and close co-operation between designers and constructors facilitated by an Alliance delivery model. These are application of single pass shotcrete lining, observation-based application of drainage measures in the tunnel and an asymmetric shape around an egress passage to save excavation volume.

It is a bi-directional tunnel to allow very large, oversized loads to pass, principally because it serves New Plymouth oil and gas industries. The risk-based fire life safety provisions effectively balance the need for safety and adequate asset protection for a tunnel that includes two-way traffic and dangerous goods, including petroleum tankers, with capital and operational costs.

Tunnel design tends to focus on the civil works aspects such as ground conditions, excavation and support design noted in this paper. However, this project also demonstrates the importance of the need for appropriate functional requirements, fire life safety, durability, aesthetics and construction access which can drive many key decisions early in the design process.

6 ACKNOWLEDGMENTS

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