Geotechnical appraisal report

December 2017

Mt Messenger Alliance

Technical Report 14

NZ TRANSPORT AGENCY
WAKA KOTAHI

Mt Messenger Bypass

New Zealand Government

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Executive Summary

The NZ Transport Agency is to develop a new section of SH3, north of New Plymouth, to bypass the existing steep, narrow and winding section of highway at Mt Messenger. The Project comprises a new section of two lane highway, some 6km in length, located to the east of the existing SH3 alignment.

This report presents a summary of the principal geotechnical considerations affecting the design and construction of the Mt Messenger Bypass. This includes:

- Design of deep road cuttings (mainly in rock) for static and seismic conditions;
- Design of low road embankments on weak soils for static and seismic conditions;
- Design of high earth embankments on elevated terrain for static and seismic conditions;
- Design of high mechanically stabilised earth (MSE) embankments on weak soils for static and seismic conditions;
- Geotechnical input to design of bridge foundations for static and seismic conditions;
- Geotechnical input to design of a relatively deep, short tunnel through soft rock; and
- Suitability of cut to fill materials and need for offline borrow and/or disposal areas.

The design of road cuttings in the soft rocks of the Mount Messenger Formation is primarily focused around the control of shallow instability, particularly slabbing-type failures. Proposed mitigation for this hazard includes adoption of appropriate batter slopes, a rock fall catch-ditch and debris barrier, and rock fall drape for the higher cuttings. Soil nailing is proposed where deep soils are present overlying bedrock.

Design of low road embankments founded on weak soils requires consideration of short-term stability during construction, post-construction total and differential consolidation settlement magnitudes, seismic-induced ground deformations resulting from liquefaction/lateral spreading and cyclic softening. Principal mitigation measures considered include staged construction with basal reinforcement and preloading through fill surcharge in combination with wick drains.

The design of high embankments located on more elevated terrain, where foundation soils are typically shallow and underlain by rock, are focused around maintaining stability during construction.

Design of high MSE embankments on weak soils includes similar considerations to those for low embankments on weak soils, but mitigation measures are likely to include load transfer platforms or undercutting of weak soils as opposed to staged construction and preloading.

The principal geotechnical consideration affecting the design and construction of the bridge foundations relates to the depth of soil cover and strength of the bedrock to support shallow foundations, or the requirement for a piled foundation solution. Access restrictions for construction impact on the geotechnical design approach. Stability of the slopes above and below the foundations, in terms of large-scale instability and shallow slabbing-type failures are also assessed.

Design of the tunnel construction sequencing and support is controlled primarily by strength of the intact rock.

Based on the current alignment and geometry, there is an estimated excess of cut to fill material. A number of potential disposal areas are being assessed.

1 Introduction

1.1 Purpose and scope of this report

This report forms part of a suite of technical reports prepared for the NZ Transport Agency's Mt Messenger Bypass project (the Project). Its purpose is to inform the Assessment of Effects on the Environment Report (AEE) and to support the resource consent applications and Notice of Requirement to alter the existing State Highway designation, which are required to enable the Project to proceed.

This report is focused on assessing the geotechnical aspects associated with the Project alignment as shown on the Project Drawings (AEE Volume 2: Drawing Set).

The purpose of this Geotechnical Appraisal Report (GAR) is to summarise the current understanding of the principal geotechnical constraints and opportunities that affect the route development, and how these geotechnical aspects interact with other design disciplines to help minimise environmental impacts and reduce cost and programme risks to the Project. Particular attention is given to understanding how incorrect assumptions made during the resource consent design stage might constrain subsequent design and construction phases.

1.2 Project description

The 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 SH3 will bypass the existing steep, narrow and winding section of highway at Mt Messenger. The Project comprises a new section of two lane highway, approximately 6km in length, located to the east of the existing SH3 alignment.

The primary objectives of the Project 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.

A full description of the Project including its design, construction and operation is provided in the Assessment of Effects on the Environment Report, contained in Volume 1: AEE, and is shown on the Drawings in Volume 2: Drawing Set.

2 Sources of Information

2.1 Previous Studies

Prior to this Project, two former geotechnical studies have been completed to investigate options for improvements to the SH3 route at Mt Messenger. These include:

1. Mt Messenger Investigation. Preliminary Geotechnical Appraisal Report. Prepared for Transit New Zealand Ltd by Beca Carter Hollings & Ferner Ltd. February 2001.
2. Mt Messenger Options Assessment – Resilience. Prepared by Opus International Consultants Ltd. 15 June 2016. File: 5-C3195.02.

2.2 Published Information

The site is located close to the boundary between two published geological maps prepared by the Institute of Geological & Nuclear Sciences (GNS Science), as detailed below:

- Edbrooke, S.W. (compiler) 2005: Geology of the Waikato area: scale 1:250,000. Lower Hutt: GNS Science. Institute of Geological & Nuclear Sciences. Institute of Geological & Nuclear Sciences 1:250,000 geological map 4. 68 p. + 1 folded map.
- Townsend, D.; Vonk, A.; Kamp, P.J.J. (compilers) 2008: Geology of the Taranaki area: scale 1:250,000. Lower Hutt: GNS Science. Institute of Geological & Nuclear Sciences 1:250,000 geological map 7. 77 p. + 1 folded map.

There are a number of published papers detailing previous experience of constructing road cuttings and embankments in areas of New Zealand dominated by Miocene-age 'soft rocks', colloquially referred to as 'papa' (Section 3.2). This includes papers specific to the Mount Messenger Formation which is the dominant bedrock through which the route passes.

2.3 Recent Geotechnical Investigations

A number of geotechnical investigations have been completed by Opus International Consultants Ltd (Opus) during 2017 as part of the options assessment process. This has included machine boreholes and cone penetration tests (CPTs) in the more readily accessible locations along the route.

The machine boreholes have been advanced to depths of up to 100m below ground level, and included downhole geophysical logging of rock defects and installation of stacked vibrating wire piezometers for measuring groundwater pressures at different elevations above the proposed tunnel location. CPTs have been used primarily to investigate the soils present within the low-lying valleys.

The results of these preliminary geotechnical investigations are presented in the Opus Geotechnical Investigation Factual Report and Addendum (May and August 2017, respectively).

Further geotechnical investigations are currently being advanced along the Project alignment. These include machine boreholes, CPTs, hand augers, dynamic cone penetrometers (DCPs), trial pits and laboratory testing on recovered soil and rock samples.

These are focused on investigating the geotechnical conditions at specific locations to aid design of embankments, cuttings, bridge, tunnel, retaining walls, culverts and potential borrow/disposal areas. These works are expected to continue through October 2017.

No intrusive investigations were undertaken by Beca (2001) or Opus in connection with the previous studies.

3 Regional Setting

3.1 Topography and Geomorphology

North of Urenui, the existing SH3 route cuts through the fluvially-dissected north Taranaki hill country, consisting of narrow ridges with steeply sloping valley sides and a deeply entrenched drainage network that typically have very small or no flood plains.

Heading north towards Mt Messenger, the existing SH3 route follows the valley of the Mimi River at an elevation of around 50 metres above sea level (mRL). It then winds its way up a steep grade for approximately 2km to reach a maximum elevation of around 195mRL immediately east of Mt Messenger (306mRL), avoiding the steeply-incised gullies that drain to the Mimi River.

The route then descends into the valley of a tributary to the Tongaporutu River which discharges into the sea approximately 5km to the north. As it descends from the high point, the road passes through the existing Mt Messenger Tunnel, then traverses along the headscarp and across a large landslide feature, before reaching the valley bottom at an elevation of around 12mRL.

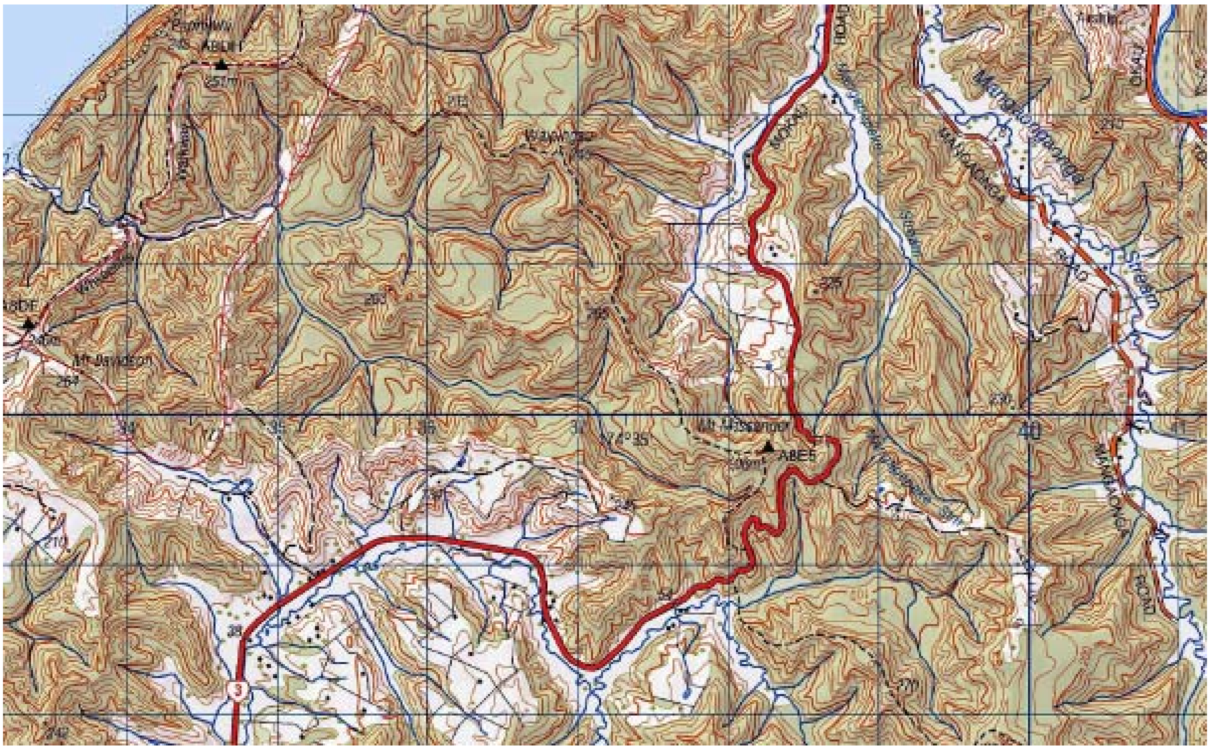

Figure 3.1: Extract of NZ Topo 50 Series map showing current SH3 route at Mt Messenger.

The differing elevations of the Mimi River and Tongaporutu River, and proximity to the mouth of the Tongaporutu River, are potentially significant with respect to the geomorphological controls on the alluvial materials present in the valley bottoms over which the Project alignment will be constructed. The depth and texture of the alluvial soils within the valley of the Tongaporutu River, and associated low-lying tributaries, is likely to be directly affected by Holocene-age sea level fluctuations and development of deep buried channels. The more elevated and distant gullies are likely to contain a more limited depth of locally-derived alluvial soils, which will generally present fewer engineering challenges for road construction.

3.2 Geology

3.2.1 Lithology

According to the GNS Science reports referenced, the regional geology of the north Taranaki area comprises Late Tertiary (Miocene-age) sediments of the Wai-iti Group, deposited between approximately 11.2 and 5.3Ma (million years ago). The Wai-iti Group comprises the following formations, from youngest to oldest:

Urenui Formation (Miu) – weakly bedded, bioturbated siltstone or mudstone with incised, coarse channel-fill sequence.

Mount Messenger Formation (Mim) – interbedded fine- to very fine-grained sandstone, mudstone or siltstone; with some channelised conglomerate horizons; typically with massive mudstone and sandstone beds near the base.

Mohakatino Formation (Mih) – well-bedded, graded volcaniclastic sandstone and mudstone, with scattered massive sandstone beds.

Tirua Formation (Mit) – bioclastic sandstone and sandy limestone with volcaniclastic sandstone layers.

Managanui Formation (Mig) – massive mudstone with common concretion horizons and some thin fine- and medium-grained sandstone beds.

The Wai-iti Group are predominantly terrigenous-clastic sediments deposited rapidly in subsiding basins during the Miocene as the convergent Australian-Pacific plate boundary propagated through northern New Zealand; and include mass transport deposits (large scale intra-formational slumps).

In the area of the Project alignment, the surface geology is dominated by the Mount Messenger Formation (Mim), which comprise marine turbidite sands and muds deposited in outer shelf to basin floor settings in the Taranaki Basin during the Late Miocene. These typically soft rocks include a continuum of silty, fine-grained sandstones to silty mudstones. These are sub-horizontally bedded dipping gently (typically 2 to 4°) towards the west; locally varying between WNW and WSW. Bedding is typically thick to massive and indistinct with syn-depositional structures. They are exposed in numerous rock cuttings and natural outcrops along the existing SH3 route in the Mt Messenger area.

3.2.2 Tectonic Setting

The present Australian-Pacific convergent plate margin began to develop in northern New Zealand during the latest Oligocene and was the major influence on Miocene geology in north Taranaki. The region experienced the effects of arc-related volcanism, variable compressional tectonism and a change from carbonate- to terrigenous-dominated sedimentation. Middle to Late Miocene normal faults are common in Miocene and older rocks, the most common fault directions being either NNW and NNE or north-east.

Post-Miocene tectonism has been limited in the north Taranaki region as the active convergent plate margin moved east towards the Taupo Volcanic Zone (TVZ). However, active extension and strike-slip movement is believed to be ongoing to the west (offshore), associated with the Cape Egmont Fault Zone (CEFZ) which led to shallow earthquakes (5-20km).

This means there has been limited active faulting and folding that would lead to the development of significant rock structural defects and a regional stress field, such that the principal stress within the rock mass should be effectively vertical within the Wai-iti Group sediments, including the Mount Messenger Formation rocks.

3.2.3 Existing Large-Scale Landslides

A number of large landslips are included on the QMAP for Waikato and Taranaki within the dissected hill country underlain by soft Miocene-age deposits, including an extensive landslide over which the current SH3 route passes north of Mt Messenger Tunnel. The southern section of the landslide, which appears to be bedding-parallel, is actively moving from mid-slope to the toe judging by a recently formed graben structure and tension cracks. The landslide is judged to have a similar age to the Pukearuhe landslide to the west, which was dated by GNS Science at 20,000 years.

3.2.4 Hydrology and Hydrogeology

As detailed above, the deeply dissected terrain dictates the catchments and sub-catchments of the north Taranaki hills are of limited size and respond rapidly to rainfall, with minimal surface infiltration due to the steep slopes and low permeability rocks and soil cover. Storm hydrographs are typified by rapid peak flows with a quick drop-off to background levels. Stormwater control will need to allow for these high, but short-term surface runoff events.

The deep rock cuttings required for the Project alignment cover quite large areas and therefore direct precipitation onto the cutting slopes and road reserve will be high, in addition to any surface water runoff from the natural slopes above (which should ideally be diverted to adjacent gullies where possible).

No data is currently available on groundwater depths within the hilly terrain. A continuous hydrostatic water table is expected to be present at considerable depth, but perched groundwater is likely to be present within the more porous sandstone layers. Groundwater movement is expected to follow individual sub-horizontal beds with higher hydraulic conductivities, following the gentle dip of the beds towards the west, exiting the slopes at distinct spring lines.

Groundwater levels are expected to be very shallow (<1 to 2m below ground level) within the valley and gully bottoms, responding seasonally to variation in precipitation and runoff rates.

3.3 Seismicity

3.3.1 Historic Records

North Taranaki is an area of low to moderate seismicity compared to other regions of New Zealand.

Local damage has resulted from occasional moderate magnitude earthquakes at very shallow depths within or close to north Taranaki and larger magnitude earthquakes further away.

Since written records have been kept in New Zealand (from about 1840), eight shallow, M5.0 and greater earthquakes have originated from epicentres within or close to north Taranaki. This includes the estimated M6.5 New Plymouth earthquake in January 1853 (the largest recorded), and the M5.4 earthquake located close to Awakino in January 1962.

Similar levels of seismicity as have occurred since 1840 may be expected to continue in the future.

A site specific seismic hazard study is currently being prepared by GNS Science. A preliminary draft was received in early-September 2017[1]. The indicated seismic ground motions are similar to, but generally less, than those indicated by the New Zealand Transport Agency Bridge manual[2] and New Zealand Standard NZS 1170.5 - Earthquake actions (2004). Confirmation of the site specific seismic hazard study is expected in mid-September 2017.

3.3.2 Current Design Standards

Ground motion parameters for preliminary design calculations have been estimated in accordance with the Bridge manual. These will be updated upon confirmation of the GNS site specific seismic hazard study.

Each structure and embankment / cutting will need to be assessed individually against the relevant criteria in the Bridge manual for a Primary Lifeline Route (PLR) and the assessed site subsoil class (as given in New Zealand Standard NZS 1170.5) specific to that location. Examples of the calculated seismic design parameters for different structures are given below:

- Bridge - 1/2500 year return period on site subsoil class B, will require an effective earthquake magnitude of 6.0 and a peak horizontal ground acceleration (•max) of 0.39g.
- Cuttings and low embankments (<6m high) are designed to a return period factor of 1/500, which for site subsoil class B equates to 0.21g.
- High embankments (>6m) are designed to a return period factor of 1/1000, which for a site subsoil class B equates to 0.28g.

[1] GNS Science Consultancy Report 2017/[XXX]. Summary Document for Mount Messenger Expressway. 1 September 2017.

[2] New Zealand Transport Agency. Bridge manual (SP/M/022). Third edition, Amendment 2. May 2016.

- For low embankments on soft alluvial soils (site subsoil class C or D), as present within the valley bottoms along much of the route, the peak ground accelerations increase to 0.28g.

Design of the earthworks will also give consideration to the preliminary findings of a current NZTA research program into topographic amplification of high cut slopes[3].

[3] P. Brabhaharan; D. Mason; E. Gkeli. Research into Seismic Design and Performance of High Cut Slopes in New Zealand. 6th International Conference on Earthquake Engineering, Christchurch, New Zealand. November 2015.

4 Project Alignment

4.1 Overview

The Project alignment covers a total distance of around 6km. It diverges from the current SH3 (Mokau Road) in the valley of the Tongaporutu River, turning SSE to hug the eastern side of the Mangapepeke Stream valley. It gradually rises from an elevation of around 12mRL through a combination of cuttings and embankments as it crosses the approximately east to west orientated ridges and valleys draining to the Mangapepeke Stream. The alignment then turns towards the south-west, climbing out of the Managapepeke Stream valley via a large earth fill embankment, and is then to be tunnelled through a prominent SE-NW orientated ridgeline at a maximum ground level of around 115mRL.

The alignment then descends into the valley of the Mimi River, through a combination cuttings and high embankments and a bridge crossing a deep gully, where it re-joins the existing SH3 at an elevation of around 50mRL. The maximum road elevation is approximately 80m lower than the current SH3 route.

To achieve the relatively gentle grades and curves proposed throughout the Project route, the following earthworks and structures are included:

- Ten principal rock cuttings typically between 30m and 60m deep, covering a combined distance of around 1.9km (including the tunnel portals);
- Ten earth embankments that are typically less than 3.5m high (but includes two embankments at approximately 16m and 27m high), covering a total distance of approximately 1.8km;
- A single bridge, approximately 120m long, over a tributary of the Mimi River close to the southern end;
- An approximately 230m long tunnel passing up to 80m below existing ground level close to the existing Mt Messenger Tunnel;
- Stormwater culverts at approximately 15 locations; and
- Minor retaining walls, tunnel portal structures and mechanically stabilised earth (MSE) embankments.

The following sections provide a brief description of the main earthworks and structures currently included in the design. The principal geotechnical considerations impacting the further site investigation requirements, design, costing and construction of these works are detailed in Section 5.

4.2 Earthworks

4.2.1 Cuttings

Based on the current understanding of the geological structure and material parameters of the Mount Messenger Formation rock and overlying soils (completely weathered bedrock, colluvial materials and/or volcanic ashes), the road cuttings have been developed on the basis of the following geometry:

i From road verge level, an 8m high cut in rock formed at 12V:1H (approximately 85˚);
ii From the top of the 12V:1H cut, 1V:0.5H (approximately 63˚).

On the upslope of the cutting (where the existing ground level typically continues to rise above the top of the cut), the 1V:0.5H profile will continue to the ground surface, with soil nails used to stabilise the surficial materials, where necessary. In most cases it is not practical to cut the slope at a shallower angle to avoid the need for stabilisation measures, as the natural slope is typically steeper than a design batter for the surficial materials (1V:2H, 26°).

It is estimated soil depths will typically range between <1 and 5m deep (and locally up to 15m deep). The current design assumes the full 5m depth of soil throughout all cuttings (this is a conservative approach). This results in an area of approximately 28,500m^2 that will require stabilisation with soil nails and subsurface drainage or similar. Investigations are currently underway to more accurately identify soil depths along the Project alignment.

On the downslope side of the cutting (where the existing ground surface typically drops away from the top of the cut), the batter slope of the upper 5m of the cut has been reduced to 1V:2H (approximately 26°) to avoid the need for soil nailing. The assessed cut volume for this upper part of the cut is dependent upon the assumed soil depth. This will also be revised upon completion of additional ground investigations.

The base of the cutting currently includes a 3.0m wide surface water drain / catch ditch, which rises 0.9m to the road verge at 1V:2H. This provides a catchment area for potential minor rock falls and slabbing-type local rock face failures. Wider and/or deeper catch ditches may be necessary in some cuttings where the size, volume and frequency of potential rock falls / slabbing failures is greater. The risk of failures will not be fully understood until the cut faces are excavated and so will need to be monitored during construction. Rockfall drapes will be installed to address and manage this risk. The anchoring and installation of rock drapes will likely need to be completed prior to the bulk excavation of affected cuttings.

Increasing the road verge width is currently being investigated to improve travel sight distances within some of the cuttings.

A typical cutting profile is shown as Figure 4.1 (rock drapes excluded for clarity).

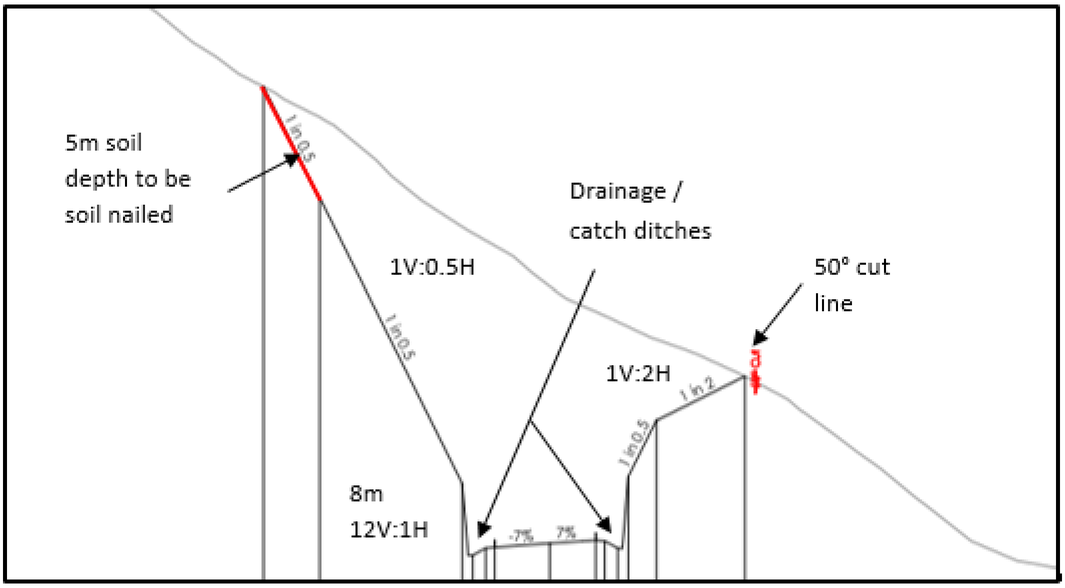

Figure 4.1 – Example cutting profile (Chainage 1160)

A summary of the principal cuttings along the Project alignment is presented in Table 4.1. This includes an indication of the maximum depth at each cutting and the proposed profile.

Table 4.1 – Summary of cuttings for Project alignment.

ID	Chainage (m)		Length (m)	Maximum Depth Section	Maximum Depth (m)	Existing Slope Angle (°)	Proposed Geometry
	From	To					
A	260	490	230	440	48	30	8m @ 12 in 1, 43m @ 1 in 0.5[1]
B	1100	1280	180	1160	52	31	8m @ 12 in 1, 44m @ 1 in 0.5[1]
C	1560	1670	110	1600	26	37	8m @ 12 in 1, 18m @ 1 in 0.5[1]
D	1950	2280	330	2160	38	33	8m @ 12 in 1, 30m @ 1 in 0.5[1]
E	2450	2850	400	2540	57	54	8m @ 12 in 1, 49m @ 1 in 0.5[1]
F[2]	3300	3400	100	3350	32	39	8m @ 12 in 1, 24m @ 1 in 0.5[1]
G[2]	3630	3680	50	3640	29	28	8m @ 12 in 1, 19m @ 1 in 0.5[1]
H	3900	4140	240	4020	49	37	8m @ 12 in 1, 41m @ 1 in 0.5[1]
I	4270	4370	100	4340	31	30	8m @ 12 in 1, 23m @ 1 in 0.5[1]
J	4430	4550	120	4520	30	36	8m @ 12 in 1, 22m @ 1 in 0.5[1]

NOTES:

[1] Currently assumed 5m depth of soil / completely weathered rock which requires soil nailing for stability when cut at 1 in 0.5 on upslope side of cutting. For downslope side, upper 5m of cut is formed at 1V:2H.

[2] Cutting extends to base of the tunnel portal.

Section 5 provides information pertaining to the basis for the proposed cut profiles and soils depths, and the factors that require further investigation and assessment for detailed design of the cuttings. The permissible cutting profiles are of particular significance at this stage of the Project in respect of the corridor width required for consent and designation purposes.

4.2.2 Embankments

Based on the current understanding of the anticipated materials to be excavated from the cuttings, the Project alignment and embankment profiles have been developed on the basis of constructing a ‘core’ of general fill with 1V:1H batters, supported by buttress slopes constructed to 1V:4H. The buttress slopes can be constructed from either general fill or landscaping fill, which is not suitable for use in the core; i.e. derived from excavated soil and completely weathered rock.

It is anticipated that during excavation and subsequent placement / compaction, the rock fill materials will break down largely to their constituent particle sizes; typically fine sands and silts with occasional clayey soils from the mudstone layers and/or heavily weathered sandstone and siltstones. These materials may not be stable in the short-term at the proposed 1V:1H batter slopes. It will therefore be necessary to place the buttress fill as the embankment core is raised.

The earth embankments can generally be divided into two types:

- Typically lower (<5m high) embankments constructed on relatively level (or sidelong) ground along the edge of the valleys at the southern and northern ends of the Project alignment;
- Larger (up to 27m high) embankments crossing erosion gullies on the higher, central portion of the Project alignment.

The ground conditions associated with these two embankment types are very different and this is reflected in the principal geotechnical considerations detailed in Section 6.

Typical embankment profiles for the valley bottoms and more elevated gully crossings, are shown in Figure 4.2 and Figure 4.3, respectively.

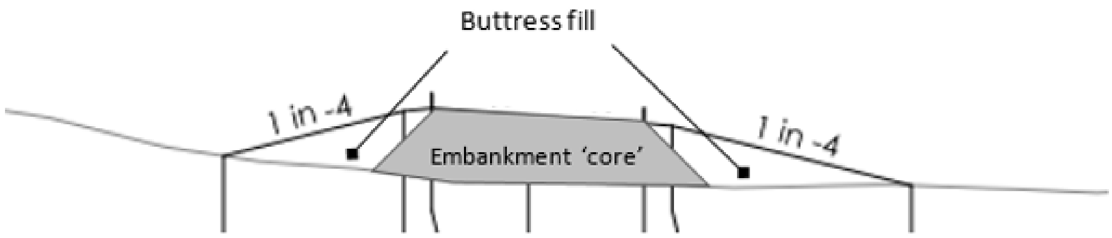

Figure 4.2 – Example embankment profile for valley section (Chainage 920)

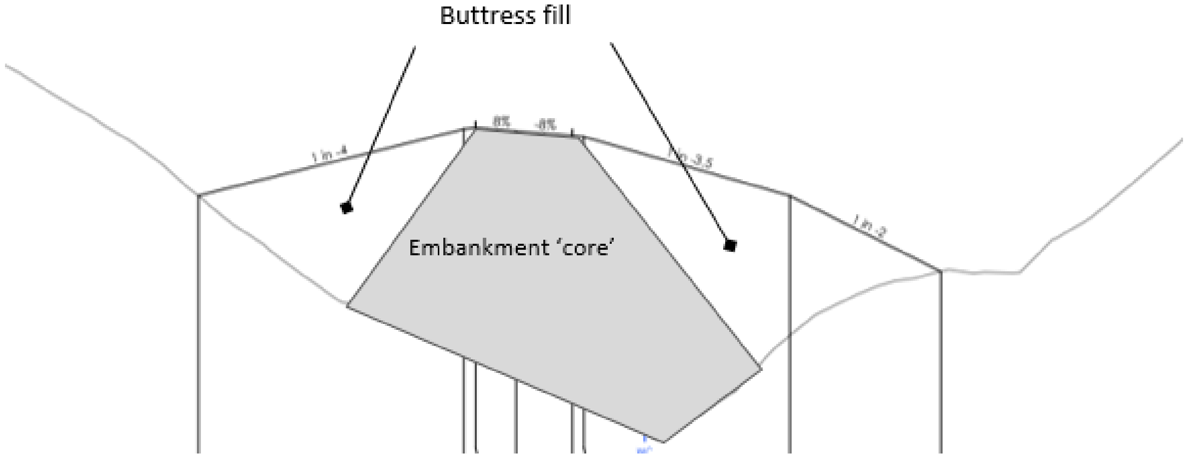

Figure 4.3 – Example embankment profile for gully section (Chainage 2940)

A summary of the principal embankments along the Project alignment are presented in Table 4.2. This includes an indication of the maximum height of each embankment and the anticipated subgrade conditions.

Table 4.2 – Summary of embankments for Project alignment.

ID	Chainage		Length (m)	Maximum Height Section	Maximum Height (m)	Existing Slope Angle (°)	Anticipated Subgrade Materials
	From	To					
A	550	970	420	620	3.0	0	Alluvium + slope deposits on sidelong ground.
B	1300	1370	70	1320	3.5	0	Alluvium + slope deposits on sidelong ground.
C	1510	1560	50	1540	3.5	0	Predominantly alluvium.
D	1700	1950	250	1860	3.5	0	Alluvium + slope deposits on sidelong ground.
E[1]	2300	2430	130	2420	16.0	12 – 45	Slope deposits / Alluvium.
F	2850	3300	450	2900	27.0	30	Across base of gully – slope and alluvial materials.
G	3680	3890	210	3840	16.0	30	Across base of gully – slope and alluvial materials.
H[1]	4370	4420	50	4400	6.0	13	Slope deposits / Alluvium.
I	4560	4660	100	4600	4.0	14	Predominantly alluvium.

ID	Chainage		Length (m)	Maximum Height Section	Maximum Height (m)	Existing Slope Angle (°)	Anticipated Subgrade Materials
	From	To					
J	4740	4790	100	4760	1.5	0	Predominantly alluvium.

Notes:

1 Embankments E and H comprise mechanically stabilised earth structures with one of the side slopes constructed at 1V:1H to minimise the road footprint on areas of high ecological value.

4.3 Structures

4.3.1 Bridge

The Project alignment requires a single bridge with raked piers to carry the road across an approximately 120m wide gully between two steep-sided ridge lines where the new alignment falls towards the Mimi River valley (Chainage 4150 to 4270). A high-value wetland is located a short distance downstream of the base of the gully, making an embankment option or the use of intermediate piers founded within the valley floor undesirable.[4]

The approach to the bridge from either direction is through relatively deep rock cuttings, but localised filling on the steep, sidelong ground is required at both abutments. This will likely require some form of retaining wall, mechanically stabilised earth (MSE) structure or local realignment of the bridge abutments.

Initial site inspections suggest minimal soil cover on the steep ridge slopes such that shallow foundations may be feasible for the bridge abutments, subject to further intrusive investigations and geological mapping.

4.3.2 Tunnel

The highest point along the Project alignment is to be tunnelled through a steep-sided ridgeline close to the summit of the current SH3 route. A road level of around 115mRL is proposed requiring an approximately 230m long tunnel, with the invert up to around 80m below the ridgeline crest. The tunnel is to have an arch shape with a vertical height of around 9m giving a maximum cover depth to the crown of approximately 70m. A single bi-directional tunnel is proposed with the dimensions controlled by the “oversize” load envelope. The base of the tunnel is to be approximately 12m wide, allowing for 3.5m wide carriageways with a 0.6m wide central median and 1.2m wide shoulders and a walk / cycle way (egress passage) adjacent to the northbound carriageway.

Approximately 25 to 30m deep rock cuttings are currently required at the approach to each tunnel portal.

A nearby borehole indicates sub-horizontally bedded very weak to weak, fine-grained sandstones are present at the level of the tunnel and immediately above the crown.

[4] This is based on advice from the Project's expert ecologists.

It is expected that the tunnel will be excavated in stages by road header initially on a top heading with support consisting of fully bonded passive dowels at 1.5m centres in combination with a 100mm thick fibre-reinforced sprayed concrete lining, followed by excavation of the bench.

A typical tunnel cross section is shown in Figure 4.4 below.

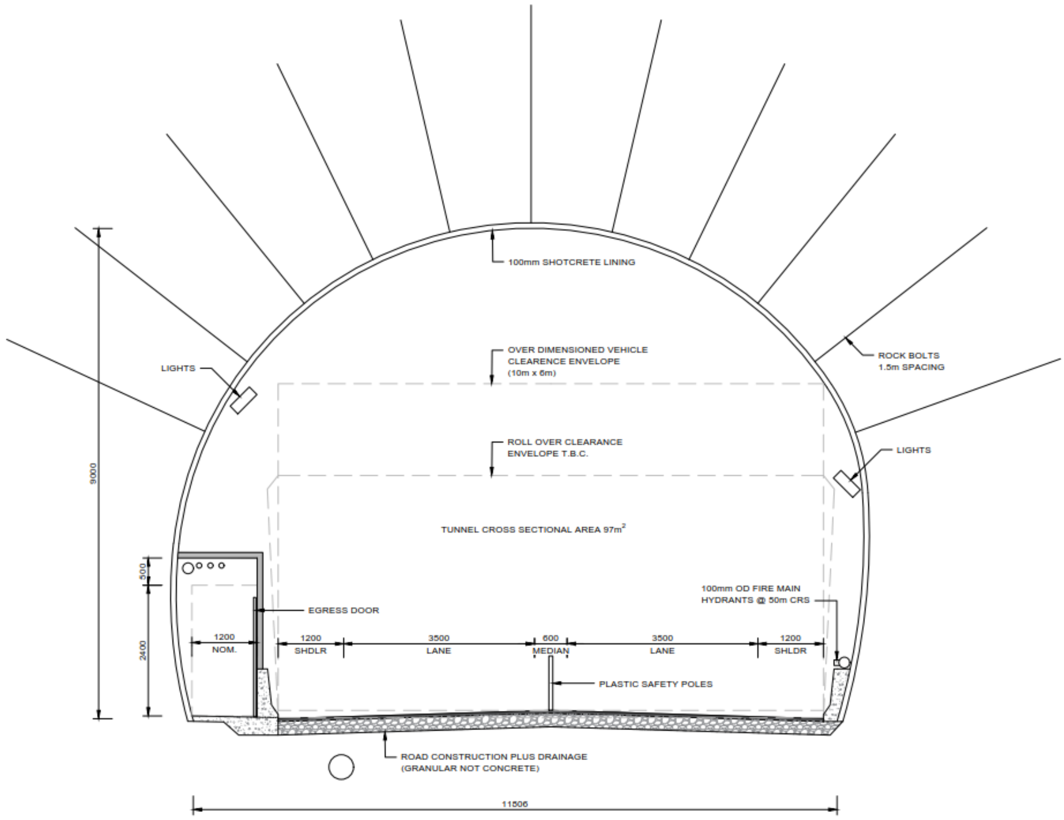

Figure 4.4 – Current typical tunnel cross section.

4.3.3 Retaining Walls and Mechanically Stabilised Earth (MSE) Embankments

There are currently two mechanically stabilised earth (MSE) embankments proposed along the route (embankments E and H in Table 4.2 above). These are located at:

- Chainage 2300 – 2430 (130m)
- Chainage 4370 to 4420 (50m)

The proposed MSE embankment at Ch. 2300 to 2430 crosses a steep sided gully in the valley of the Mangapepeke Stream. The embankment is up to 16m high and extends over a length of approximately 130m, grading into rock cuts at either end. The downslope (valley) side of the embankment needs to be constructed at 1V:1H; the upslope side is to remain at 1V:4H. An option is currently being considered to increase the batter slope to 1V:0.5H (or replace with a MSE wall) to allow the road alignment to push out further into the valley, thereby reducing the cutting depths at either side.

The proposed MSE embankment at Ch. 4370 to 4420 is located across a short gully which feeds into the Mimi River towards the southern end of the route. The embankment is up to 5.0m high and extends over a length around 50m. The downslope side of the embankment needs to be formed at 1V:1H, with the upslope side typically cutting slightly into the hillside.

The extent of any retaining walls or further reinforced soil embankments required along the route will be finalised during detailed design. These are likely to be required at the bridge abutments and possibly at a number of further locations where existing ecological features are to be retained that require over-steepening of the embankment slopes, or to minimise the length of culverts passing beneath the high embankments.

4.3.4 Stormwater Culverts

Preliminary design has identified 15 locations along the Project alignment where significant stormwater culverts are required, as detailed in Table 4.3. A large proportion of these are located beneath relatively low embankments in the low-lying valley areas where significant depths of very soft to soft, highly compressible soils are anticipated.

These areas are expected to undergo large static settlements and require the risk of short-term instability (i.e. during construction) to be managed. This will likely require some form of ground treatment, preloading and/or staged construction to control stability and long-term differential settlement.

Careful cooperation will be required between the geotechnical, stormwater design, ecologists and construction team to identify a cost-effective and timely procedure for sizing of culverts and monitoring their installation.

Table 4.3 – Summary of stormwater culverts required for Project alignment.

ID	Chainage (m)	Diameter (mm)	Length (m)
1	250	1050	24
2	300	825	26
3	570	1500	67
4	750	600	81
5	870	1350	87
6	1300	1350	27
7	1500	1200	36
8	17100	1200	35
9	1850	4x1350	56

ID	Chainage (m)	Diameter (mm)	Length (m)
10	2220	750	37
11	2300	750	25
12	2400	1200	74
13	2700	600	15
14	2900	900	117
15	2960	2550	210
16	3800	1500	115
17	4400	825	22
18	4750	2100	29
19	4750	2100	43
20	5150	1650	40
21	5650	1350	34

4.4 Route Geology

Based on existing published information, preliminary site inspections / mapping and initial geotechnical investigations, the current understanding of the likely ground conditions along the Project alignment are as follows:

4.4.1 Recent Alluvial Deposits

Recent alluvial deposits, comprising predominantly of reworked bedrock materials (fine sands, silts and clays) of intermediate to high plasticity, are expected within the Mimi River valley and more elevated gullies. The depth of these deposits has not been thoroughly investigated to date, but it is expected these may be relatively limited in the higher gullies (3 to 5m) and deeper in the Mimi River (5 to 10m+).

As indicated in Section 4, the depth and texture of the alluvial soils present within the streams and gullies feeding into the Tongaporutu River at the northern end of the Project alignment, are likely to include buried channel deposits resulting from Quaternary sea-level fluctuations and the associated episodic down-cutting of watercourses and subsequent drowning of the low-lying valleys.

Preliminary investigations at the northern extent of the Project alignment indicate very soft to soft (becoming firm) soils present to depths in excess of 30m at some locations within the Mangapepeke Stream.

4.4.2 Bedrock

Sub-horizontally bedded, very weak to weak, thickly- to massively-bedded, fine- to very fine-grained silty sandstones interbedded with siltstones and silty mudstones of the Mount Messenger Formation, have been mapped in cuttings exposed along the current SH3 route at Mt Messenger (Figure 4.5), along the steep valley sides north and south of the proposed tunnel and at the proposed bridge foundations.

Figure 4.5 - Mount Messenger Formation rock exposures close to the summit of Mt Messenger.

The road cuttings are typically formed at angles of between 60 and 90° and show evidence of shallow slabbing failures, rock falls and slippage of overlying soils (often with trees or other vegetation).

There are relatively few persistent discontinuities other than the sub-horizontal bedding planes, as expected from the minimal tectonic activity since deposition in the Late Miocene (in comparison to the more seismically active regions of New Zealand). However, a number of steeply inclined (40 to 60°) conjugate defects dipping towards the NW-NNW and SE-SSE have been mapped along the existing SH3 cuttings (Figure 4.6) and at the coast (Pukearuhe). These defect sets are most likely of tectonic origin and occur in persistent swarms. Other defects present appear to represent syn-depositional and penecontemporaneous structures rather than post-depositional tectonic-induced movements due to their truncated forms.

The defects are typically tight and up to 10m long, some with observable offsets of up to 1.5m. Good examples of these joint sets are exposed in the side walls and roof of the existing Mt Messenger Tunnel and road cuttings (Figure 4.6). Similarly orientated defects were recorded in the downhole geophysical logging of a vertical borehole located near the summit of SH3.

Figure 4.6 – Steeply dipping defects in the Mount Messenger Formation.

Whilst these do not appear to represent major structural defects that will likely result in large-scale planar and/or wedge type failures of the proposed cuttings[5], where present, these defects will exacerbate the number and size of stress-relief controlled shallow slabbing failures, particularly where cuttings are aligned unfavourably to the defect orientations, as evident in the existing SH3 road cuttings.

Localised stress-relief controlled shallow slabbing failures are ubiquitous along the existing SH3 road cuttings, even in locations where these defects are largely absent. These are more commonly associated with the mudstones and siltstones, often terminating above or below sandstone layers. Rock falls appear to be largely the result of previous slabbing failures, where undermining of harder sandstone layers permits release of individual sandstone blocks.

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[5] Geotechnical investigations, including boreholes with downhole defect mapping, are currently being undertaken at the main cutting locations, to better define this risk. Careful assessment of the plunge of the main defect sets will help identify where large-scale planar and/or wedge failures could occur relative to the cutting orientations. However, close monitoring by engineering geologists will be required during construction of the cuttings to identify such features and provide remedial measures, as required.

Individual slabbing failures rarely exceed areas greater than 1 to 2m high, by 3 to 4m wide and 200 to 400mm thick (typically less than 1m^3 of failed material). Rock fall blocks are typically less than 0.3m^3, often <0.1m nominal diameter and come to rest within 1 to 2m of the base of the cut slope, but occasionally reaching the nearside carriageway; noting there are very few open drainage / catch ditches along the existing road.

The existing SH3 cuttings are typically less than 20m high, which limits the impact of these slabbing failures on the road. The proposed new cuttings are up to 60m high and will require additional rock fall mitigation measures, as discussed in Section 5.1.3.

4.4.3 Colluvium and Tephra

Limited investigations or mapping of the surficial soils (colluvium and tephras) have been completed to date. Based on visual inspection of the top of existing road cuttings, where visible, and limited inspections and testing of the steep-sided valley slopes, soils depths may typically extend for 1 to 3m on the steep-sided slopes, locally up to 5m on flatter sections.

5 Principal Geotechnical Considerations

5.1 Cuttings

5.1.1 Soil Depths

As detailed in Section 4.2.1, it is currently assumed that soil depths associated with the formation of rock cuttings will be in the order of 1 to 5m deep and the existing cut profiles are based on the assumed 5m depth. This dictates that a significant face area will require some form of stabilisation measures (soil nailing) for the upslope cuts and battering at 1V:2H on the downslope side.

Based on the assumed 5m depth of soil, the current alignment would require in the region of 28,500m^2 of soil nailing. On the downslope side, increasing the batter slope from 1V:0.5H to 1V:2H will increase the cut volume and present a face area that would require some form of planting and/or hydroseeding. This also increases the land take necessary over that required for the 1V:0.5H slope, which has been allowed for in the establishing designation boundary.

Hand auger and dynamic cone penetration (DCP) tests are currently being completed at the principal proposed cutting locations to better understand the typical soil depths on the higher slopes. Early indications are that for the high steep slopes, soil depths less than 3m are common, such that fewer areas may require soil nailing.

5.1.2 Rock Types

5.1.2.1 Excavatability

The interbedded fine- to very fine-grained sandstones, siltstones and silty mudstones of the Mount Messenger Formation are typically very weak to weak with unconfined uniaxial compressive strengths (q_u) of <10MPa and are not expected to be highly abrasive. These should be relatively straightforward to excavate in cuttings with a combination of excavators and bulldozers equipped with rippers. Improvements in productivity of excavation may be achieved through the use of some light blasting.

Deep boreholes are currently planned for each of the principal rock cuttings. These will allow better indications of the proportions and layering of the different rock types (and therefore volumes), and provide core samples for strength and abrasiveness testing across the entire project and at individual cutting sites.

5.1.2.2 Subgrade

Pavement design for the rock cuttings will be affected by the rock type exposed at subgrade level. Where sandstones are present at subgrade level, these will provide a high stiffness and be reasonably stable requiring a relatively thin pavement with limited undercut. However, where mudstones or clayey siltstones are present at or close to subgrade level, these are likely to break down to a clay or clayey/sandy silt during excavation and subsequent wetting and drying. For longer and/or steeper cuttings, the subgrade materials may vary along the length of the cuttings as differing beds are exposed. At the transitions from cut to fill areas, there will be quite extensive transition areas where soils are exposed at subgrade level which will require undercutting drainage and replacement with a capping layer.

Where mudstones or clayey siltstones with intermediate to high plasticity are present close to subgrade level, these may need to be undercut and replaced with a compacted coarse fill and/or stabilised with lime/cement and pavement drainage included to minimise wetting / drying of the subgrade materials.

Whichever materials are exposed at subgrade level, it will be important to maintain subgrade drainage and a weather protection layer until immediately before pavement construction.

The number of boreholes proposed along the route and at individual cuttings may not be sufficient to reliably determine the rock type likely to be encountered at subgrade level. The pavement design will need to allow for this uncertainty, for instance, by inclusion of a capping layer which may or may not be required. This can only be reliably confirmed once the cuttings are reduced to subgrade level with a range of pavement solutions specified for the range of subgrade conditions likely.

Laboratory testing of the different rock types (reworked) will be completed to provide an estimate of the California Bearing Ratio (CBR) for pavement designs.

5.1.2.3 Fill Suitability

The materials excavated from the rock cuttings are expected to break down to their constituent materials during the excavation, transport, placement and compaction process, resulting in fine sand, silt and silty clay fills. These will provide a suitable source of general fill materials for the embankment cores and buttress fills, but will require varying levels of conditioning during placement to get close to optimum moisture content.

The rocks are generally expected to be excavated close to optimum moisture content, but may require minor wetting during placement. However, it will be important to protect the finer-grained materials from precipitation by avoiding stockpiling or double-handling wherever possible, as drying these fills will take considerable time and impact production rates.

Separating out the coarse-grained (sands) from the fine-grained (silts and clays) may be undertaken at some locations in the cuttings, where thickly bedded sandstones can be identified and excavated / stockpiled separately from siltstone and mudstone beds. This could allow the sandier fill to be used for the high embankments (>15m) where excess pore pressures could occur within the fill during construction and the associated risk of instability if predominantly low permeability soils are used. The coarse-grained soils will also be preferred for use immediately beneath the road pavements in the embankments or as a capping layer in cuttings where fine-grained beds are exposed at subgrade level.

Blending of finer and coarser soils may be possible and advantageous in some situations but will require close monitoring and construction control where intended for use as general fill in the embankment cores.

Optimum moisture content / dry density relationship testing will be completed as part of the current site investigations and during detailed design, along with strength and CBR testing of the different rock (and soil) types (and blended materials).

5.1.3 Stability and Mitigation Measures

The proposed cutting profiles detailed in Section 4.2.1, comprising primarily of a 12V:1H slope over the bottom 8m, with 1V:0.5H for higher sections, is considered the maximum allowable slope profile for overall stability of the cuttings and to minimise the amount and impact of slabbing failures to a level that may be deemed acceptable from a maintenance perspective (i.e. regular clearance of minor slabbing failures collecting within the catch-ditch).

The combination of a steep bottom section and the catch-ditch will help control the impact of slabbing failures, which will generally slide along the generally smooth formed cut face (as opposed to rolling and bouncing off uneven projections on the rock face), to drop into the catch-ditch and pose minimal threat to impacting on the road carriageway. However, on impact, particularly for larger slabs and/or those released from higher sections of the cutting, these are expected to break up on impact with the risk of some minor debris making it on to the road carriageway. This can be minimised in a number of ways, including:

- Increasing the currently proposed 3m wide rockfall buffer zone and 0.9m deep catch-ditch;
- Ensuring the surface materials in the catch-ditch are soft to better absorb the energy of the falling rocks, such as thick topsoil or loose sand as opposed to terminating the catch-ditch on the excavated rock or concrete-lined channels; and
- Inclusion of some form of roadside catch-fence or barrier adjacent to the higher cut faces.

For high cuts where the rock bedding could lead to rock falls, for instance, where hard sandstone layers are present overlying thick mudstone and siltstone beds that may be prone to slabbing failures, leading to undercutting of the harder sandstone bands, inclusion of a drape on the upper 1V:0.5H slope may be required as these failures could roll and bounce down the cut face and be propelled further from the cut face onto the road carriageway.

This profile is unlikely to be safe without the draped mesh. The change in profile to 12V:1H is to both optimise cut volumes and change the vector direction of the rock fall slabs to near vertical, thus minimising the debris run out distance.

For unfavourable bedding combinations, any cuts higher than 20m will need to include a steel drape from the top of the 1V:0.5H section down to 4m below the top of the 12V:1H section.

Careful inspection and mapping of the cut faces by an engineering geologist as the works proceed will help identify potentially problematic sections that can be dealt with whilst access to the slopes is straightforward (i.e. as the cuts are advanced, rather than having to go back after the full cut depths have been excavated).

5.1.4 Groundwater and Drainage

There is currently little or no information regarding groundwater levels and/or perched water tables present within the slopes. This information will be available once the deep boreholes have been completed and piezometers installed.

It is anticipated that a number of perched water tables will be present within the coarser sandstone layers, with limited vertical hydraulic connectivity through the finer-grained mudstone and clayey siltstone beds.

Based on limited information and preliminary modelling, groundwater seepage from the road cuttings is estimated to be in the order of 10 to 30m^3/day.

Groundwater seepages are therefore likely to be largely sub-horizontal, parallel to the bedding, with limited secondary flow expected through the typically tight joints, although this may occur to a limited extent towards the tunnel as these joints open in response to stress relief around the tunnel opening.

General lowering of groundwater levels within the bedrock as a result of the proposed tunnel and rock cuttings is not expected to have a major impact on the moisture availability for vegetation on the slopes beyond the construction works. This is expected to be controlled primarily by pore water held in the near-surface soils from replenished by direct precipitation, rather than root systems reaching down to deeper groundwater sources. However, groundwater lowering and a reduction in moisture availability will occur locally within the soils along the top of cuttings, which may impact on vegetation in the areas affected.

5.1.4.1 Cut Faces

If the groundwater level or perched water tables are present within the cutting depths, such that spring lines occur and/or the cut faces are subject to wetting / drying, sub-horizontal drainage holes may be necessary to control seepage pressures causing local instability of the face and increase the likelihood of larger slabbing failures. However, this is more likely to be necessary at the soil-rock interface. Site observations to date indicate seeps from within the rock mass are unlikely to require treatment.

Allowance has been made for installation of counterfort and/or slot drains near the soil-rock interface where signs of significant groundwater are encountered during construction or are anticipated for the permanent condition.

These drains will normally be installed as the cutting is excavated to ensure the works can be completed from ‘ground level’ rather than via rope access or other methods that require working at height.

Horizontal or sub-horizontal drainage holes need to be carefully designed and installed to maximise their design life to avoid regular maintenance or replacement, which would also require working at height.

5.1.4.2 Pavement Drainage

If groundwater levels are expected to be at or above the base of the cuttings, pavement and highway drainage will be required. Gravel drains on one or both sides of the carriageway (when in deep box cuts) may be required to maintain groundwater levels to a suitable depth below the pavement subgrade. These would ideally be installed from the base of the drainage / catch-ditch.

Highway drains could potentially be excavated using a digger to create the trenches, but it is likely to be more efficient to use a specialist drainage installation technique utilising a rock cutter which installs the pipe and gravel backfill in a single pass.

5.1.5 Surface Water Runoff

Dispersed surface water runoff from the natural slopes above the cuttings can generally be left to flow (trickle) down the face of the cuttings and into the stormwater ditches at road level. However, where surface water flows are concentrated by the landform resulting in high volumes of runoff at specific locations during heavy and/or prolonged rainfall events, could result in erosion and excessive wetting / drying of the face materials leading to local instability. At these locations, concentrated flows will either be diverted away from the cut face via a lined surface water channel along the top of the cutting discharging into the adjacent gullies; or can be intercepted at the top of the cuttings and discharged to the base via lined channels running down the face of the cuttings.

5.1.6 Geotechnical consideration for designation

The current cutting profiles are based on the current understanding of the rock strength and structure visible in road cuttings along the existing SH3 route around Mt Messenger, with limited intrusive investigations completed along the Project alignment to date. Most of the cuttings observed are at higher elevations in bedding layers that will not be encountered in the Project alignment at lower levels. However, based on the limited site observations in the lower route the lower beds may include a higher proportion of sandstones and sandy siltstones. These sandy rocks will generally be more stable.

For less favourable ground conditions, such as weaker, more heavily weathered materials, a more structured rock mass and/or deeper than anticipated soil depths, unsupported cutting slopes may need to be eased in some areas requiring a larger construction footprint or otherwise mechanically stabilised through the use of rock bolts or similar.

In order to provide for works within any area that may be subject to such changes in the required cutting profiles (without the need to include significant support such as rock bolting reinforcement), the possible working areas have been defined by extending the full height of the cuttings at an angle of 50° from the base of the cut slope. This falls within the adopted profile used to define the extent of the designation adjacent areas of cut, being a 45° cut +10m.

It should be noted that increasing the catch-ditch width from the current 3m to around 6m would significantly increase the number / volume of rock falls and/or slabbing failures that are prevented from reaching the road carriageway (particularly when used in conjunction with a top-fixed rock drape). This may be preferable to reducing the cut slope angle for the higher cuts, as this will have a much reduced impact on the land take required, and limit the cut heights, volumes and surface area over which rock falls / slabbing failures can occur and require protection.

Consideration is currently being given to reducing the slope gradient on the downslope faces of some cuttings to flatter than 1V:2H to provide additional fill materials in case of a cut to fill shortfall. There are several cuttings along the Mangapepeke Stream where such works are being considered. This possibility has been be provided for within the designation.

A number of local sites are currently being assessed to determine their potential for mass spoil areas, to avoid having to haul large volumes of excess cut material away from the Project area. These are predominantly gullies and are therefore likely to be underlain by weak and compressible soils. Further consideration will be needed of possible requirements for mitigation measures to be included at these sites (for instance, shear keys for high fill embankments on soft ground).

In general, we expect the cutting profile as shown in Figure 4.1 can be adopted for most of the proposed cuttings. However, to account for potential unfavourable conditions being encountered, the designation boundary allows for cuts at 45° +10m. This is intended to deal with localised features, not as being a requirement at all cuttings in general.

5.2 Embankments

The general profile of the earth embankments is described in Section 4.2.2. Depending upon the height and foundations soils, there are a number of specific requirements for different embankments along the Project alignment.

The embankments to be constructed in the Mangapepeke Stream valley are expected to be underlain by deep, very soft to soft and highly compressible soils. Here, additional requirements for embankment construction may include use of a geofabric / geotextile separator layer with a basal drainage blanket, a high strength geotextile basal reinforcement and preload fill. Design of these embankments will largely be driven by maintaining stability of the embankments in the short term, requiring staged construction, limiting long-term total and differential settlements and seismic displacements.

The high embankments constructed across the elevated gullies may require undercutting of weak surface soils and/or a drainage blanket for maintaining stability of the foundation soils. Depending upon the permeability of the general fill materials available and the height of embankments concerned, intermediate sub-horizontal drainage layers may be required to control the build-up of pore pressures within the embankment during construction (e.g. every 10m height of fill). This will be subject to design requirements.

5.2.1 Fill Suitability

The anticipated suitability and handling requirements of materials won from the rock cuttings for re-use in the construction of earth embankments is discussed in Section 5.1.2.3. These are considered likely to be suitable for general fill used to form the embankment core, for use in MSE and buttress fills. Additional potential fill sources will derive from the following:

- soil cover overlying bedrock within the cuttings (colluvium, completely weathered rock and volcanic ash);
- tunnel spoil;
- undercut of shallow weak materials beneath embankment footprints, where required for stability, or at transitions from cut to fill areas; and
- excavations required for bridge abutments and piers.

Some of these materials may also be suitable for use as general fill in the embankment core but typically they will be more difficult to separate the suitable and unsuitable materials and can be used as buttress fill instead (and for surcharge preloading of embankments). Careful earthworks planning will be required, however, as these will be the first materials to be excavated, but will typically be placed last.

It is considered unlikely that any of the cut materials will be suitable for use as the drainage blanket required at the base of embankments to be constructed on soft ground. Suitably coarse-grained materials may therefore need to be imported to site and in many cases these will be at the front-end of the earthworks operations. Suitable borrow areas for such materials are likely to be relatively distant from the work site (not within the Mount Messenger Formation) from an existing authorised facility.

As indicated above, the majority of the fill materials are sensitive to moisture changes and likely to become difficult to handle during heavy and/or prolonged rainfall, which would require significant time and effort to dry if wetted beyond the optimum moisture content. This, along with other factors such as erosion and sediment control and subgrade conditions, may result in relatively short earthworks seasons. However, the intermediate to high plasticity of these soils should limit the amount of dust and erosion of soils by surface water runoff, provided they are kept moist.

5.2.2 Soft Ground

Recent alluvial deposits are present within the valley of the Mangapepeke Stream, Mimi River and the steep-sided narrow gullies along the Project alignment which are to be traversed by earth embankments. These soils are known to be very soft to soft and highly compressible to considerable depth within the Mangapepeke Stream. The embankments constructed in this area will likely require a geofabric separating layer, drainage blanket, basal geofabric reinforcement and preloading to limit post-construction total and differential settlements to acceptable levels. Embankments greater than approximately 3m high may require staged construction to maintain stability. Wick drains are likely to be required in order to speed up dissipation of excess pore pressures (to achieve foundation soil strength gain and allow staged construction to proceed quickly). This will minimise the amount and/or duration of surcharge preloading required.

The depth and strength / compressibility of the alluvial materials in the Mimi River have not yet been investigated but are expected to be less than that of the Mangapepeke Stream, but will nonetheless likely require inclusion of a drainage blanket, basal reinforcement and surcharge preloading. Wick drains are also likely to be required.

Control of post-construction differential settlements at the transition from soft alluvial soils to the rock cuttings will be the critical for the embankments along the Mangapepeke Stream and Mimi River.

The depth and strength of the alluvial soils (valley floor deposits) and colluvial soils (slope wash) present in the base of the higher gullies has also yet to be investigated, but is likely to require some treatment prior to construction of the high embankments required in these areas. If the soils are not too deep, it may be possible to dig these out and found the embankment on stronger materials. While some form of subsurface drainage will be needed below the embankment fill, basal reinforcement, wick drains and preloading are unlikely to be required for these embankments. However, construction control will be required to control excess pore pressures within the fill and foundation soils as the embankment heights exceed 10 to 15m.

The proposed MSE embankment in the Mangapepeke Stream is also likely to be underlain by very soft to soft, highly compressible alluvial soils. The height of the embankment and steep valley-side batter slopes (approximately 1V:1H) dictate that some form of ground improvement or a load transfer platform will be required for the timely construction of this embankment. Options include stone columns, timber piles or deep soil mixing.

For the embankments required in the Mangapepeke Stream, static settlements in the order of 0.5 to 1m may be expected. Considerable volumes of fill will be required as a surcharge which will need to be removed at the end of the preload phase. Planning is required to decide where this preload material will go, either pushed out onto the sides of the embankment as additional buttress fill or taken to a suitable disposal area within the site designation or off-site. In estimating earthworks volumes and cut/fill balance, this 'loss' of material below existing ground level needs to be taken into account.

5.2.3 Liquefaction and Lateral Spreading

Based on preliminary analysis and assessment of limited investigation data, the risk of significant liquefaction or lateral spreading is low. Further intrusive investigations and laboratory testing of recovered samples are required before this can be further analysed. A range of methods for assessing susceptibility of soils to triggering will be completed, in accordance with recent guidance.

If liquefaction is identified as a hazard, then lateral spreading is also likely to be a significant hazard within the valleys of the Mangapepeke Stream and Mimi River. This will require considerable ground improvement works, although preloading with inclusion of basal reinforcement and wick drains may improve the soils sufficiently that liquefaction triggering is no longer a major risk, or that seismic displacements are predictably low.

5.2.4 Cyclic Softening

Very soft to soft soils that are assessed not to be susceptible to liquefaction triggering can be affected by a significant loss of strength and associated large strains during seismic acceleration as a result of cyclic softening of soft clays. The vulnerability of non-liquefiable soils to this type of damage is dependent upon the in-situ undrained shear strength. There is currently insufficient ground investigation data to confirm whether the soils are likely to undergo cyclic softening for the design ground motions, and if so, the magnitude of the ground displacements that would result. However, existing information from the alluvial soils in the Mangapepeke Stream are considered borderline for triggering of cyclic softening and detailed investigation and assessment of this hazard is required.

For those soils to be preloaded, with the aid of wick drains, the strength gain resulting from this ground improvement is expected to be sufficient to prevent the onset of significant cyclic softening or limit the anticipated ground deformations to acceptable levels for an ultimate limit state event.

5.3 Earthworks Volumes

A detailed review of the earthworks cut/fill balance has been undertaken by the Project team. The information presented below is intended to assist in understanding the likely suitability of cut materials for different fill types, possible bulking and compaction factors, how changes to the cutting and embankment profiles and calculated settlements could affect this balance and potential volume and timing of preloading requirements.

This information is presented as preliminary only and may be subject to revision as the geotechnical investigation results and geotechnical analyses are advanced. All materials will be subject to an earthworks specification to be developed as part of detailed design.

- **Topsoil**
  - Typical depths for stripping in cutting areas, assume 300mm.
  - For embankments within the Managapepeke Stream and Mimi River flood plains, it is suggested topsoil materials are generally left in-situ and the first layer of fill (typically a drainage blanket) is placed directly on a geofabric separating layer after vegetation is cleared;
  - Topsoil – typical depths in higher gullies to be filled, assume 500mm;
- Soil (colluvium, tephra and completely weathered rock)
  - Assume a 2m deep undercut within the base and sides of higher gullies to be filled. This material should generally be considered suitable for re-use as buttress fill. Assume a bulking factor of 1.1.
  - Undercut at transition areas from cut to fill zones and bridge abutments / piers, culverts, assume 1m deep. This material should generally be considered suitable for re-use as buttress fill. Assume a bulking factor of 1.1.
  - Tephra (volcanic ash) is expected to have a high allophane content and therefore will require careful handling if to be used as fill.
- Rock (deep cuttings)
  - Assume all materials are suitable for re-use as general fill, structural fill for reinforced earth or buttress fill with a bulking factor of 1.15.

Undercut of mudstones / clayey siltstones at subgrade level in base of cuttings for pavement construction. Assume 600mm deep undercut over 50% of cutting areas.

  - o Material can be reused as general fill, structural fill for reinforced earth or buttress fill with a bulking factor of 1.15.
  - o Tunnel spoil and rock excavation for bridge abutments / piers can be reused as general fill, structural fill for reinforced earth or buttress fill with a bulking factor of 1.15.

- 'Loss' of fill material below existing ground level for embankments constructed on deep soft ground in the Mimi River and Mangapepeke Stream flood plains. Assume settlement roughly equal to embankment height (H)/3 to H/6.
- Surcharge fill for preloading embankments on deep soft ground. Assume embankment height (H) x 0.5 for depth of fill required for a period of 6 months (assuming inclusion of wick drains to accelerate dissipation of excess pore water pressures).

5.4 Bridge Foundations

In lieu of any site-specific deep intrusive investigations, both shallow and piled foundation solutions are currently being considered for the bridge abutments and piers. Principal controls on these include:

- Access on the steep slopes and soft valley floor for plant and equipment (reach for drill rigs)
- Soil depth (colluvium, tephra, highly to completely weathered bedrock)
- Rock strength and structure, groundwater pressures / seepage
- Rock slope stability (natural and cut profiles) – risk of slabbing failures above below abutments / piers.
- Bridge span – increasing may allow shallow foundations if can be founded in more competent rock / away from potential slope instability
- Rock support / reinforcement may be required to stabilise slopes above / below abutments and piers.
- Scour protection at base of slope to prevent erosion and future undercutting.

Further details of the geotechnical considerations relating to the bridge design and construction are presented in the structures section of the Design Report.

5.5 Retaining Walls and Mechanically Stabilised Earth (MSE) Embankments

There are currently two sections of embankment which, due to restricted space along the route alignment (to preserve high value ecological assets), cannot be formed at stable slope angles suitable for general fill. The current proposal for these sections is to construct the embankment slopes at 1V:1H (45°) using mechanically stabilised earth (MSE). This comprises earth fill reinforced with layers of geogrid.

These sections are located between approximate chainages 2300 – 2430 (130m long) and chainages 4370 and 4420 (50m long) with maximum heights of around 16m and 5m, respectively. They occur where the Project alignment crosses gullies that discharge to the Mangapepeke Stream and Mimi River, respectively.

No intrusive geotechnical investigations have been completed at these sites so far, but it is expected that there will be a relatively limited depth of potentially weak soils overlying bedrock. If this is shown to be the case during the forthcoming investigations, then it may be more efficient to undercut these weak soils down to a sound foundation layer, rather than a requirement to undertake some form of ground improvement and/or staged construction. If the depth of the weak soils is too deep for undercutting (i.e. greater than approximately 3m) or this process is considered to have a high impact on the adjacent high value ecological area, then ground improvement using stone columns, deep soil mixing or driven timber poles to create a load transfer platform are options to be considered.

Further MSE supported embankment slopes and/or possibly retaining walls are likely to be required, but have not been detailed at this stage.

5.6 Tunnel

As detailed in Section 4.3.2, the proposed tunnel is approximately 235m long with an arch shape, around 9m high and 12m wide giving a total face area of just over 100m^2 (total excavation volume of around 20,000 to 25,000m^3).

The road alignment rises gently about 1m in elevation from the southern end to a maximum elevation of around 115mRL, then falls gently by around 3m to the northern portal.

To date, a single borehole investigation has been completed from a layby adjacent to the existing SH3, approximately 50m north-west of the proposed tunnel alignment. This indicated very weak to weak silty fine sandstones at the tunnel elevation and immediately above the crown (based on engineering description and unconfined compressive strengths (UCS) ranging from 1.5MPa to 4.0MPa. Stacked vibrating wire piezometers were installed to monitor groundwater pressures at various elevations within the borehole.

A second borehole is planned immediately to the east of the tunnel alignment which will include a range of in-situ testing measurements, defect mapping and further vibrating wire piezometers for monitoring groundwater pressures. Boreholes are also to be completed close to each of the tunnel portals.

5.6.1 Excavation and Support

The level of support required is influenced by the weak rock behaviour in a relatively large opening, the depth of cover and excavation sequence. The low permeability and absence of a water table through the ridge line mean that groundwater is not a significant factor in support design.

Preliminary assessments indicate that support will be a combination of rock dowels and shotcrete installed progressively as the tunnel advances. A slightly asymmetric shape of the tunnel is proposed to minimise excavation cross section whilst accommodating the egress passage.

The final shape and support requirements for the tunnel are subject to detailed finite element modelling, but is not expected to change significantly.

The finite element modelling will also be used to determine a preferred excavation sequence (Sequential Excavation Method or SEM). This is expected to comprise mining on a bench and top heading with installation of rock dowels and sprayed concrete lining, followed by excavation of the lower bench. Removal of the bench is likely to occur following completion of the top heading and roof support over the full length of the tunnel. Equipment for the safe operation and maintenance of the tunnel will be required but will not imposed any significant loads on the lining.

The soft rocks can be excavated by road header for the upper bench and standard excavation plant for the bench. (). Instrumentation will be installed during excavation to monitor the performance of the support.

The tunnel excavation will most likely progress from one direction only.

5.6.2 Groundwater Pressures and Seepage

Monitoring of groundwater pressures in boreholes is currently underway, the results of which will be incorporated into the finite element modelling of the tunnel design, including an estimate of groundwater seepages into the tunnel expected during construction and operation to permit design of control measures.

The silty, fine sandstones present along much of the tunnel alignment are expected to be of low primary permeability and with minimal secondary permeability and storativity. The horizontal permeability is expected to be much greater than the vertical permeability, such that seepages are expected to be localised through coarser-grained sandstone layers.

Significant infiltration into the tunnel during construction is not expected and can be dealt with by creating appropriate falls out of the tunnel when working upgrade and by creating falls towards a sump for collection and removal when working downgrade. Approximately 10 m^3/day of groundwater inflow is conservatively estimated for the fully excavated tunnel.

5.7 Stormwater Culverts and Swales

Wherever culverts and swales are required crossing or running along the side of embankments located within areas underlain by soft, compressible soils (as described above), consideration needs to be given to phasing of the construction of the culverts and acceptable levels of post-construction settlements.

For some embankments on soft ground, settlements in the order of 0.5 to 1.0m may be anticipated, for which preloading will be required to reduce the post-construction settlements to acceptable levels. Positioning of culverts in firmer / stiffer ground on the edge of gullies is preferred to control the adverse effects. Alternatively, installation of the culverts in soft ground may need to be delayed until after preloading has been completed at some locations. In most cases, this may be acceptable as embankment heights are relatively limited (<3 to 4m), but will be impractical for greater embankment depths. Temporary stormwater control measures will be required to divert surface water flows around embankments prior to installation of the permanent culverts. Alternatively, the permanent culverts could be installed and then excavated and re-levelled upon completion of the preloading, removing the need for additional temporary structures.

Apart from one short embankment up to 16m high in the central northern section of the Project alignment, the larger embankments occur outside of the areas where very deep, soft soils are expected (within the lower reaches of the Mangapepeke Stream and Mimi River). However, the depth of compressible soils may still be such that unacceptable levels of settlement are predicted for the culverts that need to be installed prior to construction of the embankments. In these cases, it is likely to be necessary to undercut the weaker foundation soils prior to installation of the culverts.

Wherever culverts are required to cross embankments, it is recommended these are designed with greater than the minimum required gradients to allow for construction and post-construction differential settlements and ideally, more flexible pipe materials and/or joints.

MSE supported embankment slopes and/or retaining walls (cantilever timber pole, gabion basket, mass block etc.) may be used to locally steepen the embankment slopes to reduce the overall required length of the culverts. However, these structures may require additional groundworks not required for gentler embankment slopes, for instance, to strength the foundation soils to prevent instability and/or excessive settlement. The time and costs associated with the MSE slopes or retaining walls and any associated groundworks will need to be weighed up against the economics for reducing culvert lengths.

6 Proposed Construction Monitoring and Instrumentation

Careful monitoring and instrumentation by engineering geologists / geotechnical engineers and earthworks supervisors should be carried out during construction to confirm design assumptions and material behaviours. This will be particularly important for confirming the stability of cut slopes with minimal support requirements, maximising separation of suitable and unsuitable materials for different fill types and minimising double handling / moisture conditioning, minimising the volume and duration of preloading and speed of staged embankment construction.

Typical instrumentation requirements will include:

- vibrating wire piezometers for measurement of pore water pressures within embankment foundations, construction fills and around the tunnel excavation;
- Standpipe piezometers installed adjacent to cuttings to monitor groundwater level response to construction works for temporary and permanent conditions;
- settlement markers and profilometers for assessing progress of surcharge preloading;
- inclinometers and survey pins for monitoring potential instability of embankments;
- Inclinometers, survey pins and extensometers for monitoring cutting stability
- Borehole extensometers above the tunnel and precise surveys of the tunnel opening.