TRANSMISSION GULLY PROJECT

ASSESSMENT OF ENVIRONMENTAL EFFECTS

TECHNICAL REPORT No. 3

GEOTECHNICAL ENGINEERING REPORT

New Zealand Transport Agency and Porirua City Council

Transmission Gully Project
Assessment of Environmental Effects

Technical Report No.3
Geotechnical Engineering Report
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Executive Summary

This Geotechnical Engineering Report presents geotechnical issues of importance to the Assessment of Environmental Effects for the Transmission Gully project. It is based on the content from the Geotechnical Assessment Interpretative report and factual report prepared as part of the Preliminary Geotechnical Assessment, and subsequent geotechnical inputs to Stage 2 of the project.

The engineering geology of the route is described along with the significant hazards which include earthquake ground shaking, liquefaction, the active fault traces, and a number of landslides. Precedent observations in the Wellington region are an essential part of the engineering assessment.

Aspects of route security are discussed and a geotechnical design philosophy is presented. An engineering assessment of geotechnical elements for the route is presented for:

- Cut slopes in rock, including an assessment of rock mass properties, and rock fall
- Cut slopes in soils
- Embankment fills constructed with soils and/or rock, including reinforced soil fills
- Retaining walls, bridges (their abutments and foundations), and tunnels
- Earthworks and construction materials and their properties
- Disposal sites

Route specific issues are identified in relation to the engineering elements and ground improvement treatments are discussed.

Enhancements to the project from a geotechnical perspective, are presented, and provide justification for the changes since the earlier scheme.

Recommendations are presented for each of the various engineering elements.

1 Introduction

1.1 Overview

The Transmission Gully Project (*the Project*) consists of three components:

- The Transmission Gully Main Alignment (the Main Alignment) involves the construction and operation of a State highway formed to expressway standard from Linden to MacKays Crossing. The NZ Transport Agency (NZTA) is responsible for the Main Alignment.
- The Kenepuru Link Road involves the construction and operation of a road connecting the Main Alignment to existing western Porirua road network. The NZTA is responsible for the Kenepuru Link Road.
- The Porirua Link Roads involves the construction and operation of two local roads connecting the Main Alignment to the existing eastern Porirua road network. Porirua City Council (PCC) is responsible for the Porirua Link Roads.

1.2 Transmission Gully Main Alignment

The Main Alignment will provide an inland State highway between Wellington (Linden) and the Kapiti Coast (MacKays Crossing). Once completed, the Main Alignment will become part of State Highway 1 (SH1). The existing section of SH1 between Linden and MacKays Crossing will likely become a local road.

The Main Alignment is part of the Wellington Northern Corridor (Wellington to Levin) road of national significance (RoNS). The Wellington Northern Corridor is one of the seven RoNS that were announced as part of the Government Policy Statement on Land Transport Funding (GPS) in May 2009. The focus of the RoNS is on improved route security, freight movement and tourism routes.

The Main Alignment will be approximately 27 kilometres in length and will involve land in four districts: Wellington City, Porirua City, Upper Hutt City, and Kapiti Coast District.

The key design features of the Main Alignment are:

- Four lanes (two lanes in each direction with continuous median barrier separation);
- Rigid access control;
- Grade separated interchanges;
- Minimum horizontal and vertical design speeds of 100 km/h and 110km/hr respectively; and
- Maximum gradient of 8%;

- Crawler lanes in some steep gradient sections to account for the significant speed differences between heavy and light vehicles.

1.3 Kenepuru Link Road

The Kenepuru Link Road will connect the Main Alignment to western Porirua. The Kenepuru Link Road will provide access from Kenepuru Drive to the Kenepuru Interchange. This road will be a State highway designed to following standards:

- Two lanes (one in each direction);
- Design speeds of 50 km/h;
- Maximum gradient of 10%; and
- Limited side access.

1.4 Porirua Link Roads

The Porirua Link Roads will connect the Main Alignment to the eastern Porirua suburbs of Whitby (Whitby Link Road) and Waitangirua (Waitangirua Link Road). The Porirua Link Roads will be local roads designed to the following standards:

- Two lanes (one in each direction);
- Design speeds of 50 km/h;
- Maximum gradient of 10%; and
- Some side access will be permitted.

1.5 Purpose and Scope of this Report

This report presents the findings of the geotechnical assessment conducted as part of the environmental assessment of the Project.

The purpose of this geotechnical assessment was to:

- Present findings from the Geotechnical Assessment Interpretative report and factual report prepared as part of the Preliminary Geotechnical Assessment;
- Describe the engineering geology along the route and the significant hazards which include earthquake ground shaking, liquefaction, the active fault traces, and a number of landslides;
- Describe precedent observations in the Wellington region that are an essential part of the engineering assessment;
- Discuss aspects of route security, and

- Recommendations made for each of the various engineering elements

This report is part of a suite of documents in support of the notices of requirement for designations and applications for resource consent for the Project.

2 Geotechnical Investigations

The Transmission Gully project is an approximately 27 km long new route north of Wellington, between MacKays Crossing and Linden, east of State Highway 1, see Illustration 3.1. The project is part of the proposed upgrade of SH 1 between Wellington Airport and Levin, one of the Roads of National Significance (RoNS) adopted by the government. New Zealand Transport Agency (NZTA), as the road controlling authority for New Zealand's state highway network, is preparing to modify the designation and obtain resource consents for the project.

Illustration 3.1 - Location of Transmission Gully Route

2.1 Present Role

Opus International Consultants (Opus) has been commissioned by NZTA to provide Road Engineering and Structures advice (including geotechnical engineering) to the development of the scheme and assessment of environmental effects to support the lodging of the designation and resource consent applications.

Opus has worked with NZTA and consultants carrying out other works streams, to further develop the scheme in critical areas, carried out a preliminary geotechnical appraisal for the James Cook, Waitangirua and Kenepuru Link Roads, and prepared technical inputs to the assessment of environmental effects for the project. In addition, the road concepts through the Te Puka Stream valley have been developed to improve route security.

2.2 Previous Geotechnical Investigations

Opus previously carried out a Preliminary Geotechnical Assessment including geotechnical investigations for the route during 2007-2010, under Contract 233PN, and developed a Scheme Assessment (Contract 236PN) and Cost Risk and Value Engineering (Contract 235PN). The preliminary geotechnical assessment (Contract 233PN) was carried out by Opus in association with sub-consultants, GNS Science (GNS) and Golder Associates (Golders), and main sub-contractor Webster Drilling and Exploration Limited.

A preliminary geotechnical appraisal for the main alignment was presented by Opus (2008a)[1]. Site investigations comprising 88 boreholes, 178 trial pits, 14 hand auger holes, 13 Static Cone Penetration Tests, 6 fault trenches, 11 seismic refraction line surveys and laboratory tests were carried out between August 2007 and April 2008, and the results of these site investigations and laboratory tests are presented in the Part 1 - Geotechnical Investigation and Testing Results: Factual Report (Opus, 2008b)[2]. This Geotechnical Assessment Interpretative report (Opus, 2008c) presents a preliminary geotechnical assessment of proposed Transmission Gully project based on the site investigations and laboratory tests completed.

2.3 Stage 2 Geotechnical Investigations

Supplementary aerial photography interpretation and field engineering geology mapping carried out for the link roads is reported in a separate geotechnical paper (Opus, 2010).

Opus' geotechnical team provided advice to NZTA, and to other work streams during Stage 2. The advice was documented in Geotechnical Papers (GA 13 to GA 15).

This report presents a summary of the geotechnical engineering issues, and the measures proposed to mitigate the effects on the environment. It is based on the content from the Geotechnical Assessment Interpretative report and factual report prepared as part of the Preliminary Geotechnical Assessment, and subsequent geotechnical inputs to Stage 2 of the project. Road form and alignment enhancements to the project from a geotechnical perspective, since the 2004 Costed Viaduct Option, are presented, together with the reasoning for the changes.

3 Characterisation of the Route

The Transmission Gully route has been divided into 9 geotechnical sectors, each with distinct terrain and geomorphology characteristics, which present different geotechnical engineering issues for the Transmission Gully project (Opus, 2007)[1].

The extent and characteristics of these 9 sectors are summarised in Table 3.1.

Table 3.1 - Geotechnical Sectors

Sector	Location	Stations	Sector Characteristics
1	MacKay's Crossing – Te Puka Terrace (SH 1)	▪ Stn 300 m to 2,400 m	▪ Flat terrain with low sand dunes and inter-dunal soft ground, with outwash alluvial fan deposits at the northern entrance to the Transmission gully.
2	Te Puka Terrace	▪ Stn 2,400 m to 2,850 m	▪ Flat terrace elevated well above the Te Puka stream, with pine forested hillside to the east.
3	Te Puka Terrace to Wainui Saddle	▪ Stn 2,850 m to 5,800 m	▪ Steeply incised valley with steep hillsides to the east and west, which straddles the Ohariu Fault along this sector.
4	Wainui Saddle to Kenning Stockyards	▪ Stn 5,800 m to 8,700 m	▪ Narrow valley with steep hillsides with Horokiri Stream at the valley floor.
5	Kenning Stockyards to south of Battle Hill Farm where Horokiri Valley swings to the west.	▪ Stn 8,700 m to 12,500 m	▪ Broader valley with steep hillsides to the east, with terrace remnants of older gravel deposits.
6	Horokiri Valley south to terrace immediately north of SH 58.	▪ Stn 12,500 m to 17,500 m	▪ Rolling subdued terrain with a number of older terrace gravel deposits overlying bedrock.
7	SH 58 Crossing at Pauatahanui estuarine plains.	▪ Stn 17,500 m to 17,900 m	▪ Estuarine flats associated with the Pautahanui Inlet, with swampy ground.
8	SH 58 Crossing at Pauatahanui to Duck Creek Crossing.	▪ Stn 17,900 m to 23,300 m	▪ Moderately steep weathered greywacke terrain dominated by Duck Creek, which straddles the Moonshine Fault.
9	Duck Creek Crossing to SH 1 Interchange at Linden	▪ Stn 23,300 m to 27,300 m	▪ Greywacke hillside and erosion K-surface, with steeply incised Cannon's Creek gully.

4 Engineering Geology

4.1 Overview

A detailed description of the regional geology and the engineering geology of the route was presented in the Preliminary Geotechnical Appraisal report (Opus, 2007)[1].

This section presents an overview of the geology and terrain (geomorphic) characteristics of the route (designation and immediately adjacent area), and the geological characteristics of the dominant rock and soils encountered along the route.

4.2 Terrain

The route terrain is characterised (from north to south) as:

- low lying flat plains at MacKays Crossing (Sector 1)
- an elevated alluvial terrace with adjacent incised stream (Te Puka Stream) within bedrock to the west and ancient landslide to the east (Sector 2)
- the relatively linear northward flowing Te Puka Stream valley (north of Wainui Saddle), with steep greywacke side slopes (forested on the eastern flank) and alluvial fan deposits at the mouth of main tributary streams (Sector 3)
- The relatively linear southward flowing Horokiri Stream valley (south of Wainui Saddle), with steep bedrock side slopes and alluvial fan deposits at the mouth of main tributary streams (Sector 4)
- A wide, gently sloping alluvial basin section of the Horokiri Stream in the Battle Hill area, with relatively steep side slopes planted in pine forest on the eastern flank (Sector 5)
- Variably undulating terraces, gullies and subdued hilltops between Horokiri Stream and SH58 (Sector 6)
- A 300 m long section of low lying plain at Pauatahanui Stream (Sector 7)
- The flanks of a significant tributary stream rising to a saddle with Duck Creek, thence traversing along the (recently deforested) east flank of the Duck Creek valley, crossing a number of steep incised tributary streams (Sector 8)
- an undulating plateau between Duck Creek and Cannons Creek, south to a crossing of the deeply incised Cannons Creek gully, thence moderately steep northeast and then northwest facing flanks of a broad ridge top crossing a number of deeply incised gullies (forested at the southern end) culminating at the gentle slopes of Porirua Stream valley at SH1 Linden (Sector 9).

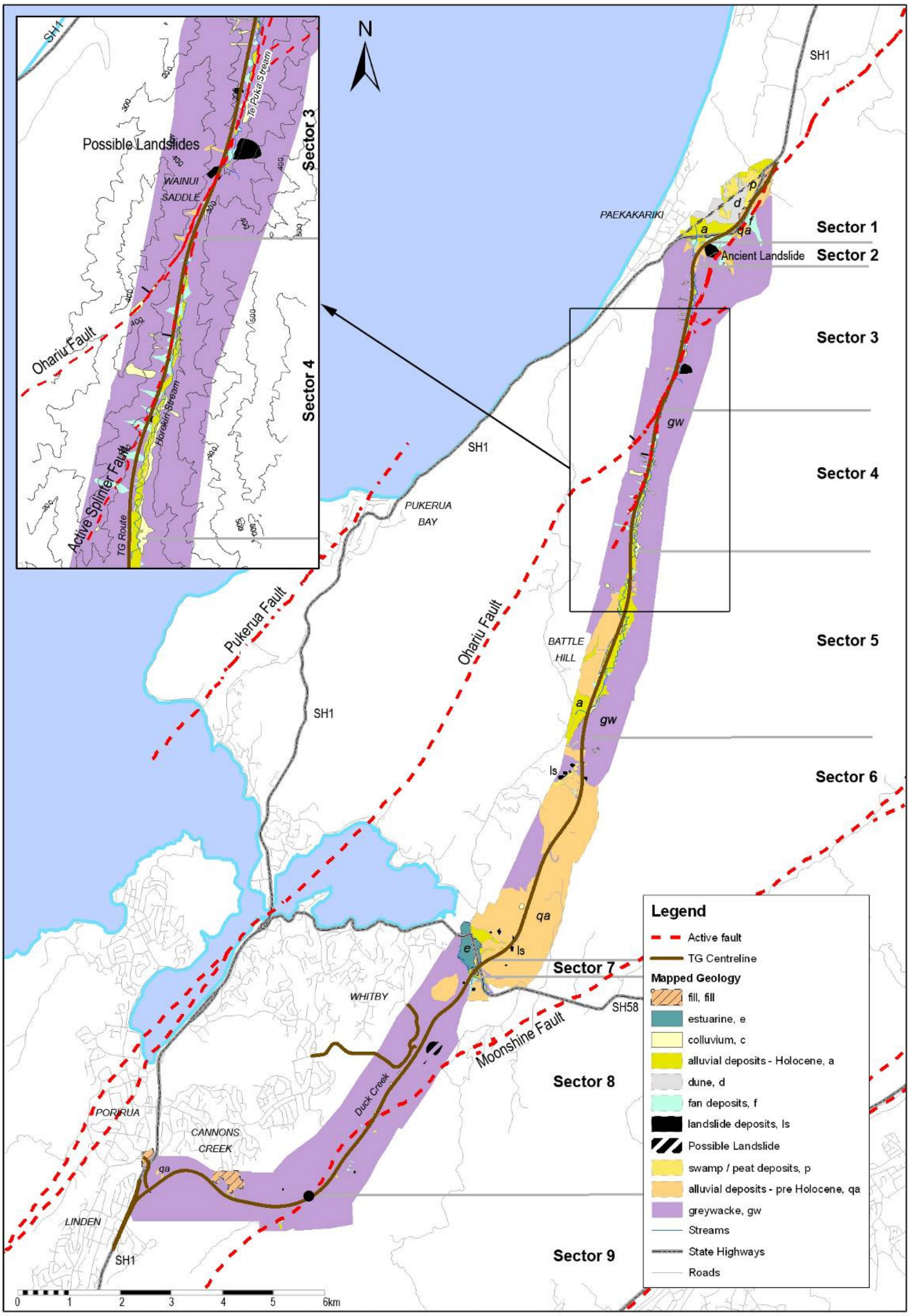

Illustration 3.2 - Summary Engineering Geology along the Route

4.3 Geology

4.3.1 Overview

Greywacke bedrock underlies the entire length of the TG route. Varying thicknesses of old and recent alluvium, estuarine, dune, loess, fan and colluvium deposits overlie the bedrock, see Illustration 3.2 - Summary Engineering Geology along the Route

Where alluvium or estuarine deposits are absent, the thickness of other superficial deposits is inferred to be generally less than 2 m to 5 m thick. An exception is the thicker colluvium and landslide deposits at the eastern flanks of the Te Puka and Horokiri Stream Valleys.

4.3.2 Soil Overburden

Range of Overburden Deposits

Thin colluvium deposits were found on the hillsides throughout the route. Thicker soil deposits were noted along the route, as follows:

- Alluvial fan deposits, dunes and peat swamp at the north end from MacKays Crossing to SH 1 (Sector 1).
- Alluvial terrace, elevated immediately south of SH1 (Sector 2).
- Terrace remnants of older Pre-Holocene deposits along the flanks of the southern part of the Horokiri Valley, between Battle Hill Farm and SH 58 and hilltops in the SH 58 to Flightys Road section (Locally within Sectors 5 and 6). These deposits are particularly extensive along the 3 km section south of Battle Hill to immediately north of SH 58.
- Valley infill and estuarine deposits at SH 58 (Pauatahanui Stream).
- Expected valley infill deposits at SH 1 at Linden.

Pre-Holocene Deposits

The route through Sectors 5 and 6 passes through pre-Holocene (Quaternary) age sediments, over an ancient erosion surface in greywacke bedrock, which may be of late Pliocene age. Drilling investigations in these pre-Holocene deposits have indicated they comprise coarse-grained, rapidly-deposited materials interbedded with finer-grained paleosols, lake or swamp deposits.

The uppermost layers in the Pre-Holocene sequence are more intensely weathered than the lower layers. At the most extreme, firm to stiff mottled orange and yellow-brown sandy silt/clays were sandy gravels when they were deposited, but subsequent oxidation has decomposed the gravel clasts to particles made up of fine sandy silt (ie the gravel clasts are completely weathered).

Lower layers are less-weathered, and the deeper fine-grained layers are stiffer than the uppermost ones. The range of stiffness encountered in the finer-grained sediments (including the completely weathered in situ former coarse grained sediments) is firm to very stiff, and they range from non-plastic to medium plasticity. The coarser-grained layers are generally densely packed, and in places would be better termed conglomerates, because a degree of cementation imparted by minor silt and iron oxide has produced a material that stands vertically up to a height of 6 metres.

The area north of SH 58 has landslides, many of which appear to have active creep. It appears that the fine-grained materials (paleosols) between the gravel layers form failure planes at depths of 5 m to 10 m in the flanks of steeper slopes. Bedding in the pre-Holocene sediments seem to be very close to horizontal.

Colluvial and Fan Deposits

Many of the slopes along the eastern sides of Te Puka and Horokiri Streams (Sectors 2 to 4) are fault facets associated with the Ohariu Fault, and also partly with the active splinter of the Ohariu Fault south of the Wainui Saddle. Investigations have shown that many of these facets have up to 13 m of colluvium, some of which may be partly debris from small rock slides. There are no signs of recent activity, but much of the area is heavily vegetated, partly by pine plantations.

On the west side of Te Puka Stream and the upper part of Horokiri Stream, fan deposits are more common from small tributary streams and gullies, many of which have periglacial dells at their heads. These dells are filled with angular rock fragments and silt (solifluction deposits that were active in the last glaciation, but are now essentially immobile). Although the fans coming down onto the floor of the Te Puka Stream valley have been mapped as Holocene, investigations have indicated the presence of mainly older, weathered in-situ material within them.

These fans are relatively inactive today, having been largely emplaced during the last glaciation, but erosion from the periglacial dells in recent storm events adds small layers of new material to the upper surfaces of the fan deposits in many cases (as observed in the 2003 debris flow event).

Similar fans and periglacial dell features may be present in the east side of the valley, but they are not visible through the trees, but indications are that they are less common, if they are present at all, on the east side.

4.3.3 Wellington Greywacke Bedrock

Composition

The term “greywacke” is applied locally for highly indurated rocks of Palaeozoic to Mesozoic age which comprises alternating beds of muddy sandstone and “argillite” (fissile mudstone), and commonly there are other gradational beds ranging from fine sandstone to siltstone.

The greywacke bedrock (sandstone, siltstone/argillite) was observed to be variably weathered, ranging from slightly weathered to completely weathered, with rock strengths ranging from very strong to extremely weak. The greywacke is typically highly fractured (closely to very closely spaced joints) with fault zones, shear zones and shattered rock zones which are widespread. The rock weathering profile is variable along the route, with the more weathered rock being observed in the upper Duck Creek area. Typically weathering is more intense high on the slopes, and less weathered near the bottom of slopes. In areas along rivers and streams, unweathered rock is often found.

Rock Defects

The several phases of deformation (including the currently active phase) has produced a rock mass with generally three main sets of joints, two of which are more planar, relatively smooth and persistent for 0.5 m to 5 m, and the third more irregular, stepped, very wavy or rough, and only persistent for 0.1 m to 0.5 m. Besides these three sets, one or two random orientations are often present, and incipient joints, which are mainly healed over and partially cemented, form additional planes of weakness that lower the strength of the rock material.

Bedding planes are often sheared and clay lined and persist for tens of metres (to kilometres), so bedding orientation with respect to a slope is important. A significant proportion of minor faults and shear zones with a persistence of tens of metres are related to the bedding. Other faults, both minor and major, are very common in greywacke.

Brecciated Rock

Brecciated rock associated with major faults occurs in zones of the order of 50 m wide. The most recent plane of movement generally has clay gouge perhaps 2 m wide with intensely sheared rock immediately adjacent to it. Beyond this, there is usually a zone of brecciated rock partially cemented by zeolites and other secondary minerals. The crushing and the degree of shearing it has undergone, however, produce sand to fine gravel particles in which the original rock fabric is destroyed. Mineralisation has coated each particle, probably as a result of heat generated by fault rupture events, and forms a weak cement. Typically this material is pale grey with very close, reticulate thin (1 mm or less) whitish veins. It is homogeneous and acts as a soft rock with an unconfined compressive strength in the order of 250 kPa to 1 MPa.

Faults

Faults, both minor and major, are very common in greywacke. Within hundreds of metres of major faults, some minor faults of some form may be found at about every 5 m to 10 m in an exposure. Unlike the bedding, the orientation of these cannot be easily predicted.

4.4 Active Faults

Two active faults run parallel to (and cross) the route at a number of points: the Ohariu Fault (within the Te Puka Stream Valley) and the Moonshine Fault (in the Duck Creek valley).

The two active faults along the TG route were investigated by specialists from GNS and included additional fault mapping, and the excavation of fault trenches in the Te Puka Stream valley to investigate inferred fault expressions on the ground. The Ohariu Fault location was refined based on these investigations

An active splinter of the Ohariu Fault was discovered during this geotechnical investigation, on the western flank of the upper Horokiri Stream valley. The active Ohariu Fault and the splinter fault as well as the now inactive Horokiri Fault are associated with wide crush zones in the bedrock.

5 Ground and Groundwater Conditions

5.1 Introduction

An overview of the ground and groundwater conditions along the route as they affect the Transmission Gully highway is presented in this section. Detailed engineering geological characteristics on a sector by sector basis have been updated from the preliminary geotechnical appraisal report using the results of the site investigations.

Engineering geology sections have been developed using the results from the site investigations and the engineering geology mapping. The selected sections presented illustrate the detailed ground and groundwater conditions along the route. The locations of the sections were chosen to illustrate the key features of the ground conditions and also how they relate to key aspects of the proposed engineering concepts developed for the two alignments, the *"In-Designation Alignment (1A)"* and the *"Preferred Alignment (1B)"*, which are shown on the sections.

5.2 Overview of Ground Conditions

The majority of the route corridor is underlain by Wellington Greywacke bedrock, overlain by a thin (1 m to 3 m) mantle of colluvium on the hill slopes and a variable thickness of alluvium (mainly gravel) on the valley floors.

The Greywacke is typically moderately to highly weathered, weak to moderately strong and closely to very closely jointed. Discrete sheared or crushed zones are common within the greywacke bedrock, typically dipping at steep or very steep angles. These sheared or crushed zones are typically less than 0.5 m thick, but may be many tens of metres locally at the location of major faults (eg the Ohariu Fault, Horokiri Fault and the active splinter fault south of Wainui Saddle in Sectors 3 and 4). In Sector 4, a broad zone of crushed greywacke is present on the western flank of the Horokiri Stream valley.

Locally the rock is more weathered (highly or completely weathered) in the Duck Creek area south to just north of Cannons Creek (Sector 9), where completely weathered rock is present to 25 m depth. Less weathered, stronger rock is present on either side of the Cannons Creek "canyon".

Thicker soils are present at the northern end of the route (Sectors 1 and 2) and Battle Hill to SH 58 (Sectors 5, 6 and 7).

Sector 1 is underlain predominantly by dune sands and thin silt layers (section at Station 1,277 m), with dense alluvial/fan gravels underling the southern part of Sector 1 and the elevated terrace east of Te Puka Stream in Sector 2. Deposits of a large ancient landslide are present at the eastern margin of the route corridor in Sector 2, comprising dense gravel (mixed fine angular weathered greywacke with some large blocks).

Within Sectors 3 and 4, while most slopes have typically less than 3 m of colluvium, some slopes have been found to be underlain by deposits in the order of 10 m to 18 m thick.

These thicker deposits are typically located on the faces of faceted spurs. There is a large landslide on the eastern flank of Te Puka Stream at about Station 4,800 m.

In Sector 5, the valley floor sediments are typically dense gravels from within 0.5 m of ground surface. Greywacke bedrock comprises the eastern margin of the valley. At Battle Hill the western flank of the valley is mantled by erodible loess deposits up to 5 m thick, overlying quaternary sediments - ranging from stiff silts to very dense to slightly cemented gravels.

Similar thick Quaternary Pre-Holocene Alluvium comprising silts, sands and gravels are present in Sector 6, and extends approximately 1 km to the south of SH 58 in Sector 8. Active small to medium size landslides are evident within the Pre-Holocene Alluvium in a number of locations in Sector 6.

In Sector 7 adjacent to SH 58 and the Pauatahanui Stream, dense gravels are present within 3 m of the ground surface with bedrock as deep as 25 m below ground surface. The base of gullies in Sectors 8 and 9 contain relatively thin alluvium (typically less than 1 m to 2 m in thickness).

5.3 Groundwater Conditions

5.3.1 Piezometer Observations

Typically two piezometers were installed in each of the boreholes, to enable characterisation of groundwater conditions at depth, and target zones of possible different groundwater conditions that were identified during drilling. The two piezometers are typically separated by a vertical separation of at least 10 m within the borehole.

Water levels were monitored in piezometers and open holes (where installations were deferred to allow Acoustic Televiewer surveys) following drilling.

Typically water levels have been reasonably consistent over the monitoring period to date, with drops in water levels indicated in some boreholes over the dry 2007/08 summer period. Some rises in water levels were evident in a few (mainly shallow) piezometers, probably in response to heavy rainfall events. The piezometer monitoring results have been updated to include the results to April 2010 (Opus International Consultants, 2010).

5.3.2 Artesian Groundwater Pressures

Artesian pressures were noted in Sector 7 adjacent to Pauatahanui Stream, with a head of at least 2 m above ground level encountered in borehole BH 1 at a depth of 22 m below ground level. Artesian pressures were also noted in BH 79 in Sector 1 at a depth of 7 m, but the height of artesian head could not be measured due to the collapse of the borehole.

5.3.3 Permeable Ground Conditions

During drilling of borehole BH 68 at Station 12,000 adjacent to Horokiri Stream at the southern boundary of Sector 5, large groundwater inflows were recorded in the borehole between 11 m and 16 m depth, indicating a higher permeability gravel aquifer. Very rapid water losses of 1000 litres/ 0.5 m drilling run from 20 m depth within bedrock was also noted in that borehole, possibly indicating open fractured bedrock that is not closely hydraulically connected to the overlying stream.

5.3.4 Groundwater in Hillside Slopes

Within the slopes above the main valleys, groundwater levels are typically about 10 m to 20 m below ground level, but depths of 35 m or greater were evident in holes located higher on some (but not all) of the slopes of Sector 3 and Sector 4.

5.3.5 Springs along Faults in Horikiri Stream Western Flanks

Springs are evident along the Ohariu Fault and the active splinter of the Ohariu Fault discovered by the investigations on the western flank of Horokiri Stream, south of the Wainui Saddle in Sector 4. These springs and the wet ground associated with this fault were readily apparent during the dry 2008 summer as a contrast with the dry surrounding vegetation, see Illustration 3.3. These springs were still flowing in the dry summer months, and this is not unexpected given that these are inferred to be fed up through the shattered rocks associated with the fault zone from deeper groundwater flow.

Illustration 3.3 - Seepages from Active Splinter Fault on Western Flank South of Wainui Saddle

6 Material Properties

6.1 Rock Mass Properties

6.1.1 Geological Strength Index (GSI)

The Geological Strength Index (GSI) is commonly used to classify the engineering characteristics of fractured rock masses, and is an input parameter to derive the Hoek-Brown rock mass strength, as discussed in Section 6.1.2.

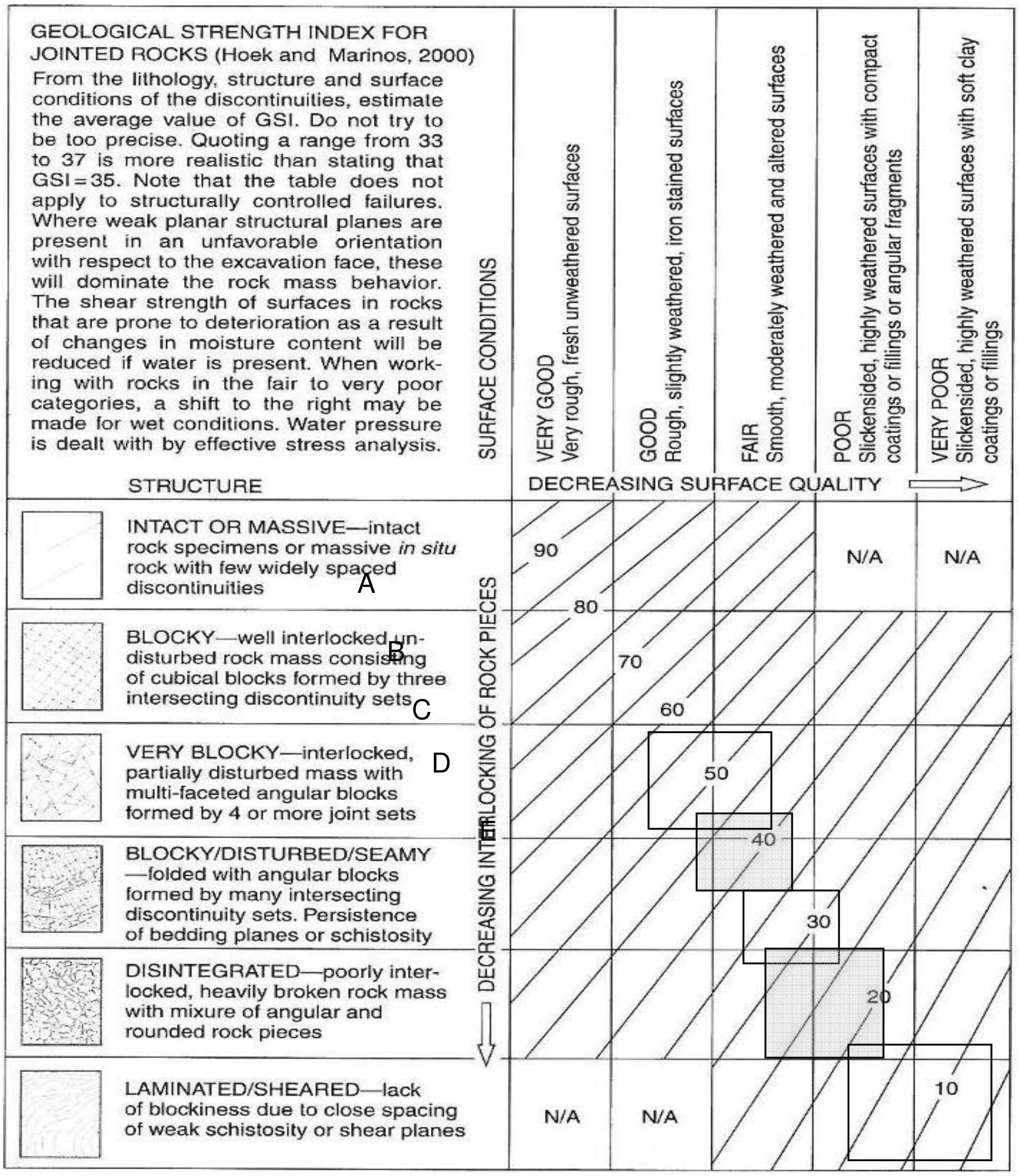

Illustration 3.4 - GSI Characterisation of Greywacke Rock along Transmission Gully

Marinos and Hoek (2000)[3] presented the method of characterising the rock mass using GSI with the aid of a chart (see Illustration 3.4) based- on:

- geological structure (intact to laminated / sheared),
- surface condition of defects (very good to very poor).

The classification gives GSI values for rock masses ranging from 0 to100.

Read et al (2000)[4] considered the application of this chart to classify New Zealand Greywacke rock masses, and presented annotations to the chart to facilitate consideration. They also propose five classes (Class I to V) of rock classified according to the range of GSI values, based on unweathered greywacke rock.

A classification scheme to characterise the greywacke rock mass at the TG site has been developed to enable the derivation of rock mass strengths. This classification is based on the observed properties of the rock core from boreholes along the route.

Table 3.2 - Geological Strength Index Characterisation of Rocks along TG

GSI Class	Rock Mass Structure	Defect Surface Conditions	Typical GSI Range
A	Very Blocky *Slightly Fractured* typically 200-400 mm defect spacing	Rough to smooth, unweathered to slightly weathered surfaces	45 – 55 (50)
B	Blocky / Disturbed *Moderately Fractured* Typically 60 to 200 mm defect spacing, joints typically closed	Rough to smooth, slightly – moderately weathered surfaces	35 – 40 (37.5)
C	Disturbed - Disintegrated *Highly Fractured* Typically intact but locally fragmented, typically 40 – 60 mm defect spacing;	Smooth, moderately weathered surfaces	30 – 35 (32.5)
D	Disintegrated *Shattered* poorly interlocked with angular rock fragments, typically 10 to 40 mm defect spacing	Smooth to polished, moderately to highly weathered surfaces, may have coatings/infillings of silt or angular fragments	20 – 30 (25)
E	Laminated / Sheared *Crushed / Sheared* Typically defects spaced less than 20 mm	Polished or highly weathered surfaces, with silt or clay infillings / coatings eg. crushed / gouge zones	5 – 15 (10)

Notes: Slightly modified from Marinos and Hoek (2000)[6] and Read et al (2000)[7]

Rock encountered along Transmission Gully has been characterised into five “classes”, in terms of representative rock fracturing and defect conditions based on the Geological Strength Index (GSI) procedure (Marinos and Hoek, 2000)[6].

Some adjustment has been made to the GSI for weathered rocks, this is predominantly to reflect the weathering along defects resulting in a poorer defect surface condition.

While, weathered rocks generally have a lower compressive strength, this is not taken into account in GSI. Rock strength is a separate input to the derivation of rock mass strength as discussed in Section 6.1.2.

6.1.2 Rock Mass Strength Properties

The Hoek-Brown failure criterion (Hoek, 1994)[5] provides for the estimation of the rock mass properties on jointed rock masses, with a curved failure envelope, that is, variation of the shear strength with the normal stress level. Rock mass strength properties for the various GSI classes of rock has been derived based on the Hoek-Brown failure criterion (Hoek, 1994)[11]. These strengths can be used to analyse the rock as a homogeneous jointed rock mass, with failure through the jointed rock mass.

6.1.3 Dominant Rock Defects

In addition to the strength of the rock mass, the properties of the rock will also be influenced by the presence of rock defects. The generally closely to very closely jointed nature of the rock mass, and the presence of generally non-persistent joints in Wellington Greywacke, can be characterised by the rock mass strength from the Hoek-Brown approach.

However, persistent defects cannot be characterised by the Hoek-Brown approach. Wellington Greywacke does have widespread persistent rock defects such as faults, shear zones and crush zones, which need to be considered in the characterisation of the Wellington Greywacke rocks.

The shear surfaces generally have an infill or coating of silt or clay, and no tests have been carried out on these materials to date. A preliminary value of effective angle of internal friction of the order of 20° may be appropriate, and testing of these materials in a direct shear apparatus or ring shear test apparatus should be considered in the next stage of geotechnical investigations to verify this.

6.2 Soil Properties

Laboratory tests were carried out on soil samples recovered from boreholes and trial pits, and included soil classification tests, soil strength tests and soil compaction tests. The results of the tests are presented by Opus (2008b) and summarised by Opus (2008c).

The soil samples included colluvium from Sectors 3 and 4, Pre-Holocene alluvium from Sectors 5 and 6, and weathered greywacke, colluvium and alluvium from Sectors 8 and 9.

6.2.1 Soil Classification

The soil classification tests results are plotted on the Casagrande Chart to show the distribution of the soils relative to the A-line, and are shown on Illustration 3.5.

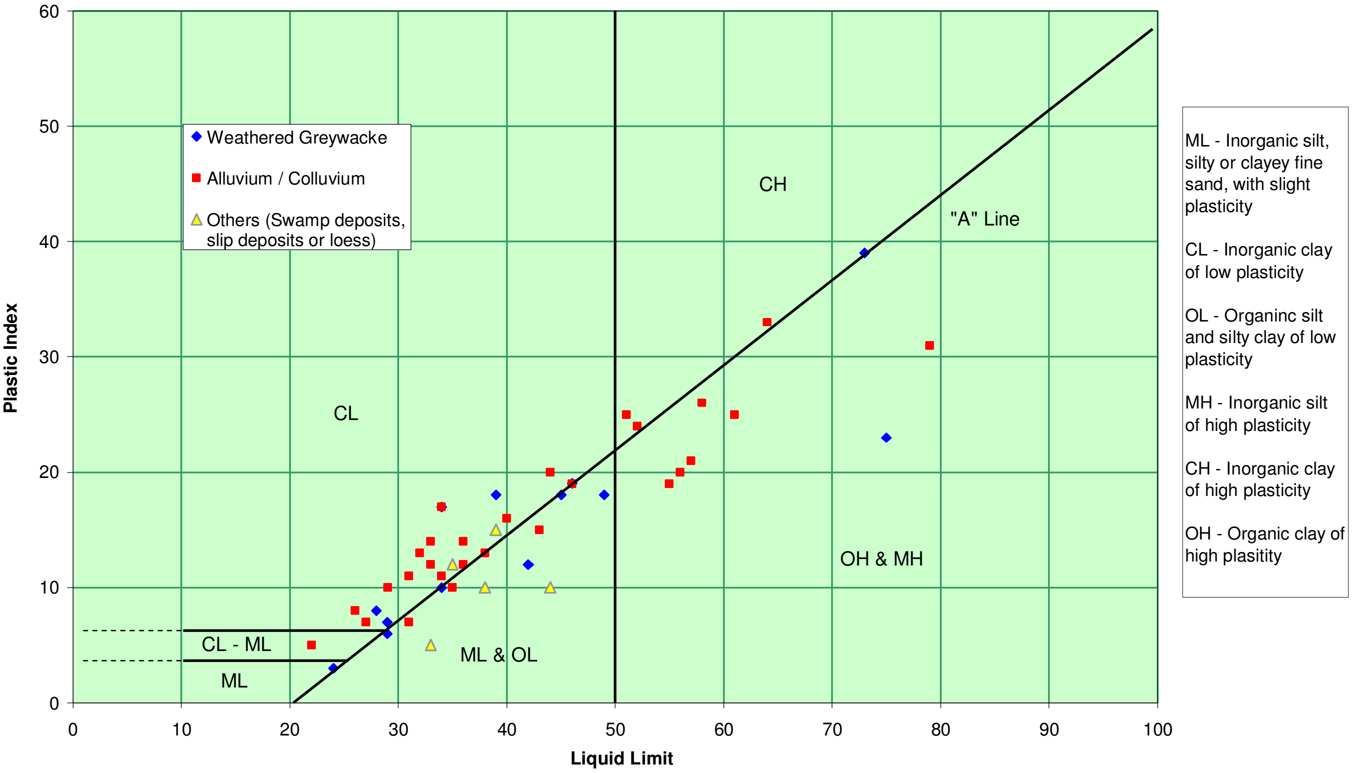

Illustration 3.5 - Classification of Soils from Transmission Gully

The chart shows that the majority of the soils recovered and tested from the Transmission Gully corridor are of low to medium plasticity and fall close to the A-line.

6.2.2 Strength Tests

Consolidated undrained triaxial compression tests were carried out on samples of in situ Pre-Holocene alluvium, alluvium, colluvium and completely weathered rock. Much of the samples were from trial pits and hence only represent the upper few metres of soils, which is appropriate for the colluvium, thin alluvium and completely weathered rock.

Consolidated undrained triaxial tests on samples taken from soils recompacted at optimum moisture content on a variety of soils gave low c' values of 0 kPa to 8 kPa and ø' values of 29° to 38°, with the ø' values greater than 33° being in the sandy gravel materials from either colluvium or weathered greywacke rock.

This indicates that the bulk of the compacted embankments in Sectors 5 and 6 with clayey or sandy silt Pre-Holocene alluvium will have ø' values of 29° to 33°. It should be noted that all these results on recompacted soils were on samples from the upper 3 m of soil encountered in trial pits.

6.2.3 Compaction Tests

Compaction tests were carried out to the New Zealand Standard Compaction and New Zealand Heavy Compaction standards (Standards New Zealand, 1986)[6]. The plot shows the natural moisture content, as well as the optimum moisture content from standard and heavy compaction tests.

The results show that the soils are generally wet of optimum, and are significantly so in Sector 6, where significant thicknesses of Pre-Holocene alluvium are present.

The compaction tests indicate bulk densities of 1.9 t / m^3 to 2.2 t / m^3.

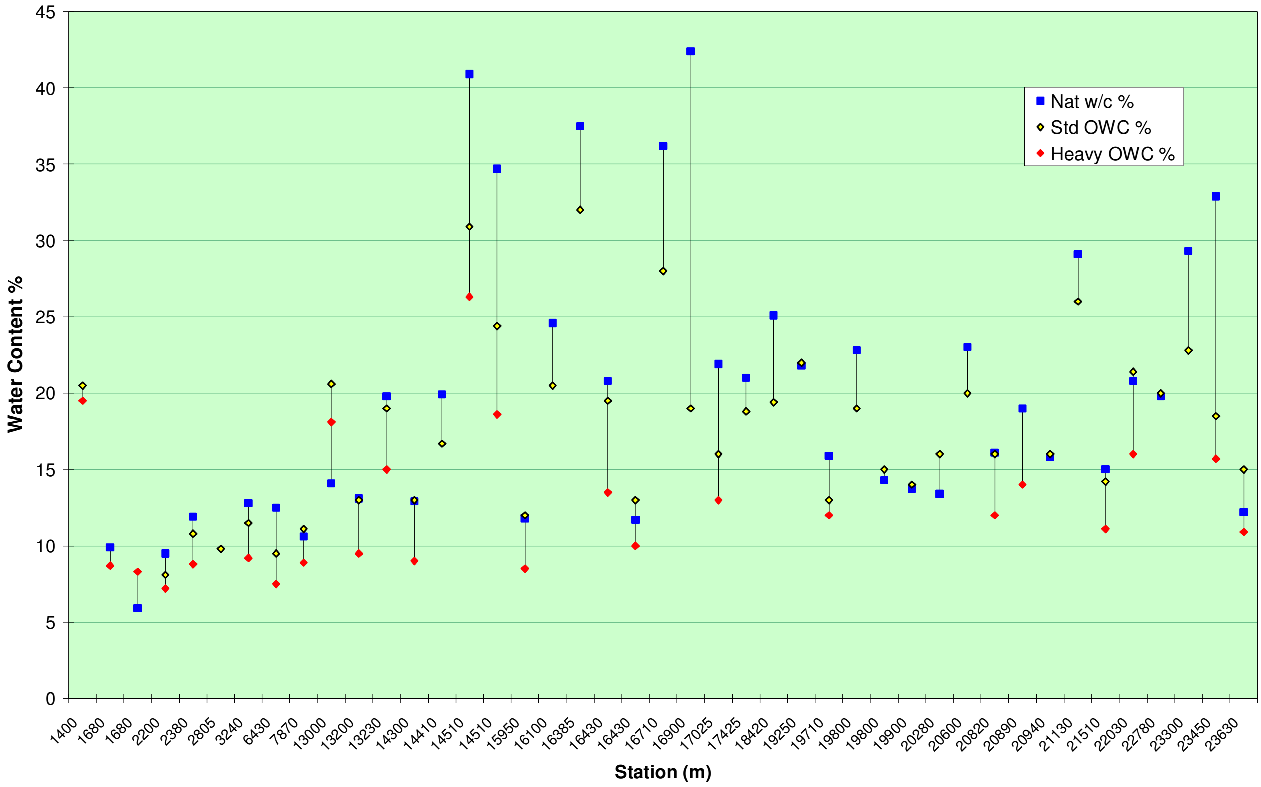

Illustration 3.6 - Soil Compaction Test Results

7 Hazards

7.1 Geological Hazards along the Route

The regional geological hazards and the hazards along the route are presented in the preliminary geotechnical appraisal (Opus, 2007)[1].

The primary hazards along the route are:

- Slope Instability
- Debris Floods and Flows
- Earthquakes and Ground Shaking
- Fault Rupture
- Earthquake Induced Landslides
- Liquefaction
- Tsunami

These hazards are briefly discussed below.

7.2 Slope Instability

The natural slopes along the Transmission Gully corridor are typically 25° to 40°, with only a few instances of major slope instability. The main areas of slope instability are:

(a) Large (50,000 m^2) prehistoric landslide at the northeastern end of the Te Puka Terrace, immediately south of SH 1 at Paekakariki. This is considered to have been triggered by a movement on the Ohariu Fault, and is now inactive. It is unlikely to affect the Transmission Gully route, provided it is not destabilised by construction activities. The *Preferred Alignment (1B)* runs along the toe of the landslide on a fill, and hence buttresses the landslide, and would improve its stability.

(b) A large landslide on the eastern flank of Te Puka Stream valley, at about Station 4,600 m to 5,000 m, south of Wainui Saddle. Some 18 m of landslide debris was encountered in borehole BH 18 near the toe of the landslide. The *In-Designation Alignment (1A)* crosses this landslide on a viaduct.

(c) Suspected rock slides on fault facets on the west side of Te Puka Stream.

(d) Small landslides in Pre-Holocene alluvium are present along Sector 6 and 8, with numerous shallow instabilities in Sector 8.

7.3 Debris Floods and Flows

Debris flows associated with the October 2003 storm event affected State Highway 1 at Paekakariki, as discussed in the preliminary geotechnical appraisal (Opus, 2007)[1]. A number of subsequent events in 2003-2006 also affected SH 1. The October 2003 event also caused debris flows in tributary gullies on the western flank of Te Puka Stream valley.

There is a hazard of debris flows affecting the Transmission Gully route along the Te Puka Stream and Horokiri Stream valleys, and this hazard needs to be taken into consideration in the design of the highway.

7.4 Earthquakes and Ground Shaking

The Wellington Region is in an area of high seismicity in New Zealand. Earthquakes and associated strong ground shaking can therefore be expected in the Region, including the Transmission Gully route. The expected levels of ground shaking is summarised in the preliminary geotechnical appraisal report (Opus, 2007)[1]. The extent of ground shaking has been further considered as part of this study, and is summarised in Section 8.

7.5 Fault Rupture

The Transmission Gully route crosses two active faults, the Ohariu Fault and the Moonshine Fault. An active splinter probably associated with the with the active Ohariu Fault (referred to in this report as the *Active Splinter of the Ohariu Fault*) was discovered during the geotechnical investigations undertaken as part of this study, and runs along the western side of Horokiri Stream, to the south of Wainui Saddle. These faults have been further investigated as part of this study, and the hazards are summarised in Table 3.3.

Table 3.3 - Fault Rupture Hazard

Active Fault	Recurrence Interval	Fault Rupture Hazard
Ohariu Fault	1,500 – 2,200 yrs	▪ TG route is close to the fault between SH 1 at Paekakariki and Wainui Saddle area. The alignments cross the fault with a fault rupture hazard to the highway. Section 9.3.4 provides recommendations for management of the hazard associated with the crossing and route security concerns.
Active Splinter of the Ohariu Fault (south of Wainui Saddle)	2,000 yrs ?	▪ This newly discovered fault also poses a fault rupture hazard where it straddles the TG corridor south of Wainui Saddle. Section 9.3.4 provides recommendations for management of the hazard associated with the crossing and route security concerns. Advice has been provided to adjust the alignment to minimise hazard.
Moonshine Fault	> 11,000 yrs	▪ Investigations indicate that this fault is represented by a wide fault zone in the southern part of the Duck Creek area. Given the recurrence interval of more than 11,000 years, it presents a low risk to the Transmission Gully route.

7.6 Earthquake Induced Landslides

Earthquake induced landslide potential is discussed in detail in the preliminary geotechnical appraisal report (Opus, 2007)[1]. The potential for earthquake induced landslides will be influenced by the height and slope of cuttings formed for the Transmission Gully project. This is further discussed in Sections 9.3.4, 12.6 and 18.

7.7 Liquefaction

Liquefaction as a consequence of earthquakes could lead to subsidence and lateral spreading, which could affect the highway and associated structures.

The preliminary geotechnical investigations indicate a low hazard along the route, with potential hazards mainly at:

- SH 1 crossing at Paekakariki
- Kenepuru Link near Porirua Stream

7.8 Tsunami

The tsunami hazard is very low along the Transmission Gully route, with little potential for damage. Some seiching in the Pauatahanui Inlet could result in localised flooding of SH 58 at Pauatahanui, and given that the Transmission Gully embankment across SH 58 will be higher than the surrounding area, to allow for a grade separated interchange, the seiching is not expected to affect the TG route.

8 Seismicity

8.1 Regional Setting

The Transmission Gully route is located in the Wellington Region, an area of high seismicity in New Zealand. There are a number of major active faults and the subduction zone in the region capable of generating large earthquakes of up to magnitude 8+ on the Richter scale.

8.2 Active Faults

There are a number of major active faults in the region, which are listed by GNS (2008)[18]. This includes the major faults identified in Table 3.4.

Table 3.4 - Active Fault Earthquake Sources

Fault ID	Fault Source	Segment	Magnitude (Richter)	Recurrence Interval (years)	Distance from Transmission Gully (km)
11 - 13	Subduction Zone		7.8 – 8.4	420 – 1,200	22 - 31
1	Wellington Fault	Wellington- Hutt Valley	7.6	700	6 - 16
10		Tararua east	7.3	650	20 - 32
2	Wairarapa		8.1	1,000	21 - 27
5	Ohariu Fault	Central	7.4	1,800	0 – 4.4
6		South	7.4	2,300	1.5 - 19
9	Wairau Fault	Offshore	7.5	1,900	14 - 21
7	Pukerua – Shepherds Gully Fault		7.4	3,450	4.9 - 8
3	Akatarawa Fault	Moonshine - Otaki	7.4	5,150	10 - 17
4	Moonshine Fault		7.1	11,150	0.1 - 11

Note: After GNS (2008)[18].

The location of these faults in relation to the TG route is shown in Illustration 3.7 , and the distances are summarised in Table 3.4. The route straddles and crosses the active Ohariu Fault (Central Segment) between Wainui Saddle and the intersection of the route with SH 1 near Paekakariki, in Sector 3. The route also crosses the Moonshine Fault zone in the Duck Creek area in Sector 8.

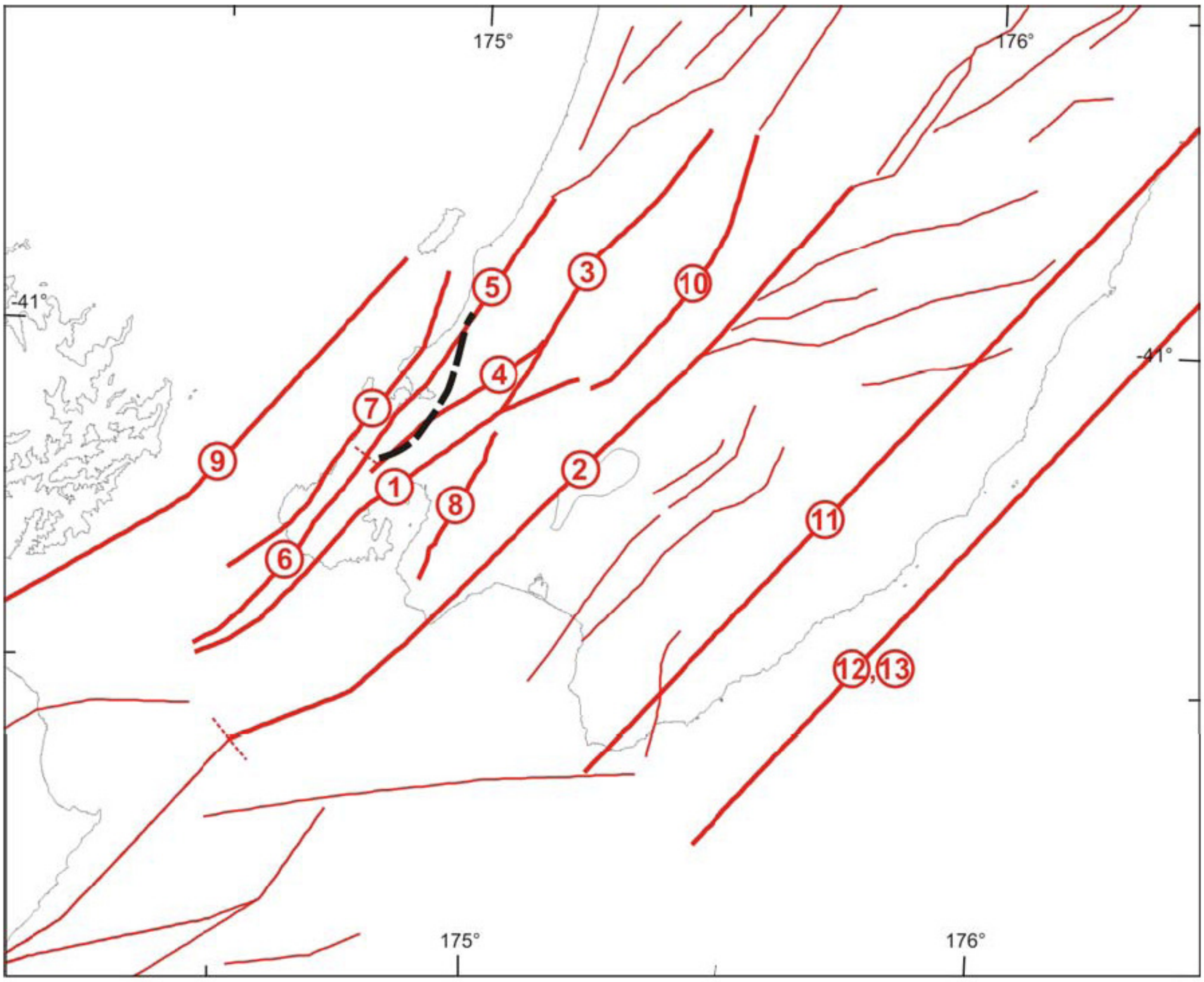

Illustration 3.7 - Active Fault Sources in relation to Transmission Gully Route
After GNS (2008)[18]

8.3 Ground Shaking

The high seismicity and the active faults present in the region can give rise to significant levels of ground shaking along the Transmission Gully route.

The regional ground shaking hazard maps published in the early 1990's indicated ground shaking along the route of Modified Mercalli Intensity (MMI) of MM VIII to IX along the route, with expected peak ground accelerations of 0.3g to 0.6g in a Wellington Fault (Wellington to Hutt Valley) segment event (Wellington Regional Council, 1992)[7].

The earthquake loadings code NZS 1170.5 : 2005 (Standards New Zealand, 2005)[8] provides the peak ground accelerations summarised in Table 3.5 for the return periods for design recommended by the Bridge Manual.

The Bridge Manual (Transit New Zealand, 2003)[9] requires that a site specific seismicity study be undertaken for highway structures of value exceeding $ 1.1 Million on highways of importance category 1, and within 10 km of active faults with a recurrence interval less than 1000 years. The provisional amendment to the Bridge Manual (December 2004) currently in force does not allow the seismic loads to be reduced from that given in NZS 1170.5 based on site specific seismicity studies. It should be noted that further amendments being considered could allow a reduction of up to 30%.

Table 3.5 - Peak Ground Accelerations from NZS 1170.5

Return Period (Years)	Peak Ground Acceleration		Applicable to Design for
	Site Class 'B'	Site Class 'C'	
475	0.36g	0.48g	▪ Lower importance structures not affecting security of route.
1,000	0.47g	0.62g	▪ Embankments and Cuttings
1,500	0.54g	0.72g	▪ Free-standing retaining walls
2,500	0.65g	0.86g	▪ Retaining Walls associated with Bridges

A site specific seismicity study has been undertaken for the Transmission Gully route by GNS Science (2008)[10]. The peak ground accelerations from the site specific seismicity study are summarised in Table 3.6.

Table 3.6 - Peak Ground Accelerations from Site Specific Seismicity Study

Return Period (years)	Peak Ground Accelerations	
	Site Class B	Site Class C
500	0.35g - 0.39g	0.47g - 0.54g
1,000	0.45g - 0.52g	0.47g - 0.71g
2,500	0.59g - 0.69g	0.78g - 0.94g

Note: After GNS (2008)[18].

Spectral accelerations and spectra have also been derived and presented in the seismicity report (GNS, 2008)[20].

The peak ground accelerations from the site specific seismicity study summarised in Table 3.6 show that the peak ground accelerations are of a similar order to that given in NZS 1170.5, with a range of values below and above. The ground shaking at the southern end of the route is higher, given the proximity (6 km to 8 km) to the active Wellington Fault (recurrence interval of about 600 years), which dominates the seismicity of the Wellington Region. Such a variation is not uncommon, as NZS 1170.5 gives typical values for each city and region. The Bridge Manual together with the current provisional amendment requires that the results from the site specific seismicity be used for important and high value structures for the Transmission Gully route, rather than those from the NZS 1170.5, provided that the values are not reduced.

9 Design Philosophy

9.1 Route Security

9.1.1 Context

NZTA has a responsibility to proactively manage the risks to its lifeline state highway networks from natural hazards.

The Civil Defence Emergency Management Act 2002 identifies roads as a key lifeline utility, and requires operators to be able to demonstrate that they have assessed the risks to networks, and taken proactive measures to ensure that the lifelines (roads) are able to function to the fullest extent possible after natural hazard and other events (Ministry of Civil Defence Emergency Management, 2002)[11].

9.2 Existing State Highway Vulnerabilities

It is generally recognised that Wellington City will be cut off from the rest of New Zealand due to disruption of land access after a large earthquake and perhaps large storm event, as both the existing State Highway 1 and State Highway 2 routes and railway routes are vulnerable and are likely to be closed for many weeks (Opus, 2008c)[12]. One of the major concerns is that this transport disruption would seriously impair the ability of Wellington to recover after a major earthquake.

Major Vulnerabilities on State Highway 2 to the East are the Hutt Road and the Rimutaka Hill Road section, which are very difficult to mitigate.

The major areas of vulnerability of State Highway 1, north of Wellington are:

(a) Pukerua Bay to Paekakariki Station section, closed by major landslides in large earthquake event.

(b) Porirua to Paramata Bridge section, liquefaction induced lateral spreading towards Porirua Harbour, and damage to Paramata Bridge in Ohariu Fault rupture event

(c) Ngauranga Gorge, particularly landslides along the Johnsonville bypass section (though there are alternatives via local roads)

Mitigating the risk from large earthquakes is very difficult in the current highway corridors, given the potential for large landslides and fault ruptures that would close the road. These are extremely difficult and impossible to mitigate.

Porirua City Council and the Wellington Lifelines Group made submissions when the Western Corridor project was being considered, highlighting the need for improved route security and for this to be considered as part of development of the upgrade to the northern access into Wellington.

9.3 The Transmission Gully Opportunity

9.3.1 Introduction

Development of a new alternative route such as the Transmission Gully presents a unique opportunity to substantially improve route security for New Zealand's regional and national road network, and this is one of Transit's key objectives for the route. It has been important to consider the route security philosophy in the development of the Transmission Gully route and concepts. It is at the early stages of selection of the route, concepts and road form, that route security considerations can achieve the greatest improvements.

Transmission Gully provides the opportunity to bypass the major hazards (a) and (b) above, and (c) is offset by the presence of local roads. Thus Transmission Gully has the opportunity to substantially improve the route security from and to Wellington via State Highway 1 from the North.

9.3.2 Natural Hazards affecting the TG Corridor

The key natural hazards affecting the TG corridor are:

(a) Earthquakes

- fault rupture (Ohariu Fault, Moonshine Fault and the newly discovered Active Spilnter of the Ohariu Fault)
- ground shaking
- slope failures
- liquefaction

(b) Storms

- slope failures
- debris flows
- flooding

(c) Non-natural hazards involving accidents, perhaps involving hazardous materials could also pose short duration hazards.

The effects on the highway can be classified in terms of the performance, using resilience states that define the three dimensions of resilience (after Brabhaharan, 2006)[13]:

(a) Damage state

(b) Availability state (degree of access) – eg single lane access / full closure

(c) Outage (duration of impairment of access) – eg up to 12 hours / up to 3 days / weeks

9.3.3 Route Security Philosophy

The following route security philosophy has been adopted for the Transmission Gully route:

(a) Highway is open for full access with minimum structural damage in small hazard events with a short return period

(b) Highway suffers limited repairable damage in moderate hazard events, with continued limited access, or highway reopens after a short period of closure, say 12 hours to 3 days[*]

(c) Highway suffers major damage, but does not collapse, in large, long return period events, and limited access can be restored within a reasonable period (say 3 days to 2 weeks[†]).

Such an approach would provide a relatively high degree of security to the route as discussed in Section 9.3.1.

9.3.4 Management of the Major Natural Hazards

The impact of major natural hazards on the Transmission Gully highway can be managed to achieve the defined level of route security through the selection of an appropriate route alignment, road form concepts and design parameters.

While avoiding active faults would provide a higher level of route security, it is generally recognised that these are features that cannot be avoided by lifeline facilities which run over many kilometres and are part of a network (as opposed to buildings that can be sited to avoid active faults).

Earthquake Fault Rupture

The TG route crosses two active faults:

(a) Ohariu Fault, recurrence Interval 2,200 years, movement 3 m to 5 m horizontal, and smaller vertical, and the Active Splinter of the Ohariu Fault, south of Wainui Saddle

(b) Moonshine Fault, Recurrence Interval > 11,000 years

The Moonshine Fault has a very long recurrence interval, and hence a very low probability of rupture. The location, width of fault zone, characteristics and form of the fault is also

[*] The emergency management sector operates on the premise that people should be self reliant for 3 days after major events and after that help would be available.
[†] After 2 weeks, recovery phase will be severely impacted upon by lack of access after the immediate response phase after a major event.

poorly defined. It is therefore of lesser importance to the selection of the TG route and conceptual designs.

The active Ohariu Fault has a recurrence interval of 2,200 years and therefore is a large, moderate return period event.

The following approach has been adopted:

(a) Minimise number of crossings of the active Ohariu Fault and associated splinter faults.

(b) Cross the fault in order of preference on:

- Embankments, where the 3 m to 5 m movement can be readily (within days) modified to reinstate access after a rupture event (provide greater embankment width if necessary); or
- Cuttings, where it is not possible to cross on embankments, with the understanding that fault rupture would lead to large landslides at the crossing and would take longer to restore access (days to few weeks) and more costly to reinstate.

(c) Avoid crossing the fault on a bridge or viaduct structure, if possible, where 3 m to 5 m displacements would cause extensive damage, would be costly to reinstate, and take many weeks to provide temporary access (Bailey bridge) and many months to years to restore.

(d) Avoid crossing the Ohariu Fault in a tunnel, if possible, as fault rupture is likely to cause collapse of part of the tunnel and mis-alignment with a much higher potential for many deaths depending on the time of the earthquake rupture. The tunnel would be very costly and take many months to years to reinstate.

In the case of the Transmission Gully route, it is feasible to cross the active Ohariu Fault on an embankment, or in a cutting, which would enable an acceptable level of route security to be achieved.

Earthquake induced Landslides

Moderate to large earthquakes lead to slope failures in steep to very steep slopes, including cuttings which generally are steeper than the natural hillside slopes.

An appropriate strategy should be to:

(a) Avoid high steep cut slopes that would generate large amount of slope failure materials that would take many weeks to clear. Keep slope angles moderate, for example to approximately 45° or flatter in Wellington Greywacke bedrock for high slopes greater than 10 m to 15 m high.

(b) Avoid high moderately steep (45° to 60°) cuttings through fault disturbed rock.

(c) Provide intermediate benches and berms at the foot of cuttings to trap rock fall and small slips in small to moderate events.

(d) Avoid locating vulnerable highway structures such as bridges and viaducts alongside or at the base of steep slopes and cuttings, where landslides generated can destroy or severely damage the structures.

Earthquake Liquefaction

The site investigations to date indicate the risk of liquefaction to be generally low, except in areas north of SH 1 at Paekakariki (Sector 1) and at the Kenepuru Link (Sector 9), where ground improvement would be considered to protect structures from liquefaction induced ground damage and embankment or foundation failure.

Earthquake Shaking

Appropriate earthquake design should be used to mitigate effects of ground shaking, by:

(a) Structures would be designed to the NZTA Bridge Manual requirements.

(b) Retaining Walls (free-standing) would be designed to allow limited displacement in large earthquakes, using a displacement based design approach.

(c) Embankments would be designed to allow controlled displacement in large earthquakes, using a displacement based design approach, but avoid failure.

(d) Cut slopes in soil would be designed to allow displacement in large earthquakes.

The acceptance of limited displacement would allow emergency access after major events and would not compromise the security of the route, although there may be some damage requiring levelling and reinstatement of the road surface.

Storm Induced Landslides

Manage the effect of storm induced slope failures, through:

(a) flatter cut slopes in overburden or completely weathered rock;

(b) drainage

(c) benches and debris collection berms.

(a) accept small failures.

Debris Flow in Storm Events

Debris control systems should be provided in catchments and debris channels and adequate large concrete box culvert structures would be provided to pass debris under the highway, where there is potential for debris flow.

There are potential debris flow hazards in the Te Puka and Horokiri catchments (Sectors 3 and 4) in particular. It should be noted that debris flow closed State Highway 1 at Paekakariki in October 2003 and twice in subsequent years (Opus, 2007)[1].

9.4 Geotechnical Design Philosophy

9.4.1 Design Standards

Geotechnical design would be carried out to accepted standards of geotechnical practice and the requirements of the Bridge Manual (Transit New Zealand, 2003)[17].

9.4.2 Seismic Loads

Earthquake design loads would be based on the requirements of the Bridge Manual and the results of the site specific seismicity study presented in Section 8.

9.4.3 Cut and Fill Slopes

The cut and fill slopes would be designed for:

(a) normal static conditions

(b) storm and construction conditions

(c) earthquake conditions

9.4.4 Soil Slopes

Soil slopes would be designed for expected ground and groundwater conditions and assessed design strengths of the soils. Soil slopes would be designed to the factors of safety and performance criteria appropriate for design as set out in Table 3.7.

Table 3.7 - Soil Slope Design Criteria

Design Case	Conditions	Performance or Design Criteria
1	Normal	▪ Factor of safety > 1.5
2	Storm and Construction	▪ Factor of safety > 1.25
3	Earthquake	▪ Factor of safety > 1, or displacement < 300 mm in 1,000 year return period event

This preliminary assessment has considered stability analyses, but given the likely lack of certainty over soil strength parameters, consideration has also been given to local precedent performance of slopes in the Wellington Region.

9.4.5 Rock slopes

Rock cut slopes in Wellington Greywacke rock should be designed to achieve the factors of safety and performance criteria appropriate for design as set out in Table 3.8.

There are a number of different mechanisms of failure and hazard issues that need to be taken into consideration in the design of rock cuttings.

The design of cut slopes in rock would consider:

(a) kinematically feasible failures along rock defects

(b) rock mass slope failures

(c) rock fall hazards.

Table 3.8 - Rock Slope Design Criteria

Design Case	Conditions	Performance or Design Criteria
1	Normal	▪ Factor of safety > 1.5 or precedent cut slope angle.
2	Storm and Construction	▪ Accept some small wedge / block failures in modest storms, and larger slope failures in 100 year storm, consistent with route security philosophy.
3	Earthquake	▪ Limited failures acceptable in moderate to large earthquakes consistent with route security philosophy.

9.4.6 Structures

Bridge foundations and retaining walls would be designed to the Bridge Manual (Transit New Zealand, 2003)[17].

The minimum factors of safety required for retaining walls are presented in Table 3.9.

Table 3.9 - Retaining Wall Design Requirements

No	Mechanism	Static Factor of Safety	Seismic Factor of Safety	Comments
1	Sliding	▪ 1.5	▪ 1.2*	▪ Free-standing walls may be designed to undergo a displacement of 150 mm in a design seismic event, assuming flexible wall, without structural damage
2	Overturning	▪ 2.0	▪ 1.5	▪ Overturning is not an acceptable mode of failure in a seismic event.
3	Overall Stability	▪ 1.5	▪ 1.25	▪ Gross stability condition.

10 Road Form and Engineering Concepts

10.1 Road Alignment

Geotechnical advice has been provided to assist with the development of road alignments that take into consideration the route geotechnical conditions, hazards, route security and constructability.

Table 3.10 - General Guidance on Selection of Alignment

Sector	Location	Ground	Guidance
2	Te Puka Terrace ▪ Stn 2,400 m to 2,850 m	▪ Gravel alluvial terrace	▪ Follow terrace on flat terrain with minimum cut into eastern hills ▪ avoid Ohariu Fault zone, avoid landslide
3	▪ Stn 2,850 m to 2,950 m	▪ Te Puka Stream	▪ Cross Te Puka Stream away from Ohariu Fault ▪ Good foundation conditions for bridge
3	Western Flank ▪ Stn 2,950 m to 5,050 m	▪ Greywacke ▪ low overburden	▪ Western flank alignment of flatter, lower overburden and better rock (away from Ohariu Fault zone) ▪ avoid Ohariu Fault crossing ▪ No viaduct or bridges or major stream crossings ▪ Avoid landslide (Stn 4,600 m and 5,000 m) (east side) ▪ Avoid viaducts or bridges along steep slopes /cuttings. ▪ Road on reinforced soil embankment where it is close to the Ohariu Fault.
3	Cross Ohariu Fault ▪ Stn 5,050m to 5,200 m	▪ Te Puka Stream headwaters	▪ Cross Ohariu Fault on embankment (most easy and quick to reinstate after fault rupture) and maximise route security ▪ Te Puka stream crossing on embankment upstream of sensitive area and confluence of eastern side streams.
3	Cross saddle on eastern flanks ▪ Stn 5,200 m to 6,350 m	▪ Greywacke	▪ Cross saddle on eastern flank away from Ohariu Fault ▪ Maximise chances of better rock away from active fault. ▪ Take advantage of flatter terrain on eastern flanks
4	Western Flank ▪ Stn 6,400 m to 7,650 m	▪ Greywacke with fan deposits	▪ Alignment on the flatter, low overburden western side ▪ Use reinforced soil embankments to avoid large cuttings in fault disturbed rock on the western flanks. ▪ Away from Horokiri stream on eastern valley floor
5	Valley Floor Stn 8,800 m to Battle Hill South	▪ Dense alluvium	▪ Alignment on valley floor, crossing the Horokiri Stream and Stn 9,800 m and Stn 11,800 m

10.2 Road Form

There are a variety of road forms that have been considered for the highway. Table 3.11 presents road forms considered to:

- minimise geotechnical and natural (storm & earthquake) hazards
- maximise route security
- reduce costs, and
- take into consideration environmentally sensitive issues.

Table 3.11 - Appropriate Road Forms

Type	Description	Application	Commentary
1	▪ High Cuttings	▪ Cuttings through spurs	▪ Accept localised high cuttings in short section spurs in preference to extensive structures over other sections to reduce cut heights.
2	▪ Reinforced Soil Embankments	▪ Sidling Fill	▪ Use 45° reinforced soil embankments to steepen fills and reduce intrusion into streams. ▪ This would use surplus fill from cuttings. ▪ Can be designed to perform well with some displacement in large earthquakes. ▪ Minimises need for rigid anchored walls or half bridges.
3	▪ Reinforced Soil walls	▪ Sidling walls	▪ Use vertical reinforced soil walls in preference to concrete walls. ▪ This would use surplus fill from cuttings. ▪ Can be designed to perform well with some displacement in large earthquakes. ▪ Minimises need for rigid anchored walls or half bridges.
4	▪ Spilt grade carriageways for northbound and southbound to reduce width of road formation	▪ Avoid large cuttings	▪ Large cuttings following the hillside results from the need for large road platform and alignment geometry. ▪ Locate highway on terraces cut into hillsides or terraces formed on sidling fill / reinforced soil embankments or reinforced soil walls. ▪ This has the potential to reduce the height of cuttings, which can be steeper and reduces earthworks quantities; reduce earthworks footprint, and cost. ▪ Provides a solution that fits better with the terrain.
5	▪ Open box cuts with shallow slopes	▪ Saddle	▪ Form open box cut through the eastern side of saddle to cater for the very poor fault deformed bedrock, which has the potential to: • Reduce depth of cut. • Avoid high costs of steeper supported cuts or cut and cover tunnel
6	Rock Fall Protection	▪ Cut slopes	▪ Use 3 m wide benches in cut slopes at 10 m height intervals to slow rock fall and provide collection areas. ▪ Use narrow rock fall collection ditch (1 m to 2 m wide) with concrete (removable) barrier to allow cleaning. ▪ Use cut slopes of 45° for rock rolling rather than bounce. ▪ Consider intermediate rock fall barrier fences where slopes higher than 30 m

10.3 Geotechnical Enhancements since 2004 Designation

10.3.1 Overview

The road form and alignments have been enhanced considering the geotechnical engineering and route security issues outlined in Sections 10.1 and 10.2, and the design philosophy presented in Section 9. These enhancements are outlined in this section.

The enhancements have significantly improved route security, reduced project risks and have led to significantly reduced projected construction costs.

10.3.2 Alignment on Te Puka Terrace : Station 2,000 m to 2,800 m

The alignment has been moved onto the Te Puka Terrace, which is a flat alluvial gravel terrace, and this has:

(a) Reduced the earthworks necessary, which in the previous 2004 scheme had involved large cuttings in the fault zone on the eastern flank of Te Puka Valley.

(b) Avoided the large pre-historic landslide at the entrance to Te Puka Valley, and the proposed embankment buttresses the pre-historic landslide.

(c) Avoids affecting a long section of the Te Puka Stream. A 300 m section of stream was proposed to be culverted in the SAR (Opus, 2008d), and this has been avoided in the proposed scheme. Only a local crossing of the Te Puka Stream is now required at Station 2,800 m.

10.3.3 Alignment on Western Flanks of Te Puka Valley : Station 2,800 m to 5,000 m

The alignment has been moved onto the western flank of Te Puka Valley, from the eastern flank proposed in the 2004 costed viaduct option. This change has:

(a) Reduced the number of crossings of the active Ohariu Fault, from two crossings in the 2004 Costed Viaduct Option, to one crossing in the current scheme.

(b) Enabled crossing the Ohariu Fault on an embankment rather than on viaducts. In the event of an Ohariu Fault earthquake event, with the potential for a few metres of horizontal and some vertical displacement, an embankment can be quickly reinstated by earthmoving machinery, whereas a viaduct would collapse and would take many months to years to restore. This significantly enhances route security and the robustness of the route.

(c) Avoided the risk of instability affecting the route, which is associated with the crossing of large landslides at about Station 2,200 m (see Section 10.3.2 above) and at Station 4,500 m to 4,800 m (the 2004 Costed Viaduct Option had crossed the Station 4,500 m to 4,800 m landslide on a viaduct, making it's piers vulnerable to landslide movements). This change in alignment has also enhanced the security of the route, improved the robustness of the highway and reduced project risks.

(d) Avoided the higher and steeper slopes associated with the eastern flank of the valley, and the associated high earthquake induced landslide hazards.

(e) Avoided the high cuttings up to some 75 m to 80 m high required for the 2004 Costed Viaduct Option, some of the cuttings in the fault disturbed zone associated with the active Ohariu Fault. The eastern flank also generally has a much thicker overburden of soil overlying bedrock, which would have had to be cut at a flatter slope or supported by soil reinforcement such as soil nailing. Avoiding these cuttings has significantly enhanced route security, reduced project risks and significantly reduced costs.

(f) Eliminated the long sections of viaducts associated with the 2004 Costed Viaduct Option, and hence led to substantial reduction in the projected cost of the project.

10.3.4 Reinforced Soil Embankments - Te Puka Valley: Station 3,500 m to 4,900 m

The 2008 SAR scheme (as well as the 2004 Costed Viaduct Option) had proposed a number of viaducts along steep hillsides in the Te Puka Valley (also the Horokiri Valley in the 2004 Costed Viaduct Option).

The author of this report (P Brabhaharan) was part of a New Zealand Society for Earthquake Engineering (NZSEE) Learning from Earthquakes team which visited the earthquake damaged areas of China in November 2008, following the May 12, 2008 Magnitude 8 Wenchuan Earthquake that devastated areas of the Sichuan Province of China. Observations of landslides and the effect on transportation lifelines are documented by Brabhaharan in the reconnaissance report (Yu et al, 2010). Extensive landslides, in somewhat similar steep terrain to the Te Puka Valley, were observed to have led to closures of many highways in Sichuan, see Illustration 3.8.

Illustration 3.8 - Highway destroyed by landslides in the 2008 Wenchuan Earthquake, China

The author also observed and obtained records of many highway bridges and viaducts that had been destroyed or damaged by landslides where they are located in steep terrain prone to landslides, see photographs on Illustration 3.9.

Observations from the 2008 Wenchuan Earthquake illustrate that structures along steep hillsides are prone to damage and destruction from the large forces associated with earthquake induced landslides, or can be undermined by the failure of steep slopes.

Illustration 3.9 - Viaduct / bridges in steep terrain destroyed by landslides in the Wenchuan Earthquake

Viaducts were proposed in the Te Puka Valley in the 2008 SAR preferred alignment (1B) between Stations 3,800 m and 4,800 m (as well as previously in the 2004 Costed Viaduct Option). These viaducts could be vulnerable to severe damage or destruction by large landslides as they are located on steep slopes. These viaducts will also be located alongside the proposed high cuttings which will be steeper than the natural hillsides, and earthquakes induced slope failures are likely as discussed in Section 12.6. In addition, these viaducts are located very close to the active Ohariu Fault and hence are vulnerable to local shaking and ground displacements in the proximity of a fault rupture zone. These structures would therefore not meet the route security philosophy for the project.

Also very high retaining walls on steep hillsides had been proposed in the 2008 SAR preferred alignment (1B), which are also vulnerable to the effect of landslides during earthquakes in this terrain.

The proposed scheme now proposes a reinforced soil embankment up to 40 m high with 45° slopes to replace the viaducts and vertical retaining walls. This would provide an improved route security consistent with the route security design philosophy (Section 9.1) by eliminating the use of viaducts and high vertical walls in the steep terrain along the Te Puka Stream valley.

Key features of this option are:

(a) High vertical walls on steep slopes and viaducts are replaced with 45° reinforced soil embankments (RSE). RSEs are more resilient to landslides as they are more ductile earth structures, and any earthquake induced landslide debris from above would run over the road and down the 45° slopes, compared to imposing large loads and causing damage to rigid structures (such as viaducts and high vertical walls). Debris on the road platform supported by the reinforced soil slope could be relatively quickly removed to reinstate access along the highway. RSEs are also resilient to fault movement nearby as they are ductile earth structures that can deform.

(b) This option would have a greater impact on the Te Puka Stream at the toe of the RSEs. More sections of the Te Puka stream would need to be realigned or reconstructed to facilitate RSE construction.

(c) The toe of the RSE would also need to be protected with rip-rap against erosion by the stream in flood events.

(d) It would be prudent to incorporate berms at intermediate vertical height intervals, to minimise erosion of the RSE slope from natural surface run-off.

(e) Unlike vertical walls and viaducts, the reinforced soil embankments can be revegetated to achieve a natural landscaped slope in the long term. The 45° slope is somewhat similar to the natural greywacke rock slopes in the region, which are predominantly of 38° to 43° slopes.

10.3.5 Alignment on Western Flank of Horokiri Valley: Station 5,500 m to 8,500 m

The alignment has been moved onto the western flank of Horokiri Stream Valley, from the eastern flank proposed in the 2004 costed viaduct option. This change has:

(a) Eliminated the long sections of viaducts associated with the 2004 Costed Viaduct Option, and hence contributed to the substantial reduction in the projected cost of the project.

(b) Avoided the steeper slopes associated with the eastern flank of the valley, and the associated high earthquake induced landslide hazards.

(c) Avoided the high cuttings required for the 2004 Costed Viaduct Option. Avoiding these cuttings has significantly enhanced route security, reduced project risks and significantly reduced costs.

(d) Led to the alignment being closer to the active splinter of the Ohariu Fault, and to improve route security, the alignment has been placed largely on reinforced soil embankments with 45° side slopes, to minimise the height of cuttings and avoid significant intrusion into the Horokiri Stream.

(e) As noted in 10.3.4 (a) above, RSEs are more resilient to landslides as they are more ductile earth structures, and the landslide debris from above would run over the road and down the 45° slopes, compared to imposing large loads and causing damage to rigid structures (such as viaducts and high vertical walls). Debris on the road platform supported by the reinforced soil slope could be relatively quickly removed to reinstate access along the highway. RSEs are also resilient to fault movement nearby as they are ductile earth structures that can deform.

10.3.6 Alignment on Battle Hill Farm Flats : Station 8,600 m to 12,000 m

The alignment has been moved onto the Battle Hill Farm valley floor, from the eastern flank of the Horokiri Valley proposed in the 2004 costed viaduct option. This change has:

(a) Avoided the steeper slopes associated with the eastern flank of the valley, and the associated high earthquake induced landslide hazards.

(b) Avoided the high cuttings required for the 2004 Costed Viaduct Option. The eastern flank also generally has relatively thicker overburden of soil overlying bedrock, which would have had to be cut at a flatter slope or supported by soil reinforcement such as soil nailing. Avoiding these cuttings has significantly enhanced route security, reduced project risks and significantly reduced costs.

(c) Eliminated the long sections of bridges to cross the deeply incised tributary streams of the Horokiri Stream associated with the 2004 Costed Viaduct Option, and hence led to substantial reduction in the projected cost of the project.

10.3.7 Alignment at Duck Creek and Cannons Creek: Station 22,500 m to 24,000 m

The 2004 Costed Viaduct Option had long viaducts crossing the Duck Creek and Cannons Creek valleys and a deep excavation for an interchange (Warspite) on the elevated terrace (K surface) north of Cannons Creek.

The deep excavation is in an area of deep soil overburden and highly to completely weathered rocks, which would have had to be cut to a relatively flat slope leading to substantial earthworks and being in a deep cutting, reduced route security.

The proposed alignment, with reduced lengths of viaducts and the elimination of the Warspite Interchange at this location, has led to:

(a) Improved route security by avoiding the deep cuttings in an area with deep soil overburden and poor rock conditions.

(b) Reduced construction costs associated with reduced earthworks.

(c) Substantial reduction in construction costs associated with the reduced length of viaducts across Duck Creek and Cannons Creek.

11 Tunnels

11.1 Location

A tunnel of up to 500 m length was considered for the Wainui Saddle section in Sector 3. Two tunnels would be needed each involving about 15 m width and 10 m height of excavation, to provide for the two lanes and a crawler lane in each direction, and for safety operational and construction reasons.

11.2 Tunnel Form

Two forms of tunnel were considered:

(a) Cut and Cover Construction

(b) Tunnelling through Bedrock

Cut and Cover construction would be appropriate where the road alignment is at a relatively shallow depth of up to about 15 m, leading to only a small cover over the tunnel.

Tunnelling is suitable where the depth of the road alignment is over 30 m, giving a cover of over 20 m. This assumes a tunnel width of about 15 m and height of about 10 m. Between 15 m and 30 m, tunnelling can still be considered but would require greater support measures.

A combination of cut and cover construction form at either end near the tunnel portals and tunnelling methods in the middle section where the tunnel is at greater depth were considered to be appropriate.

11.3 Cut and Cover Tunnel

A top down construction is appropriate for cut and cover tunnels, so that the excavation is supported as it proceeds down towards the full depth.

With cut and cover technique, both carriageways may be constructed in one trench to form a single wide tunnel with perhaps a separation wall in the middle rather than as two trenches leading to two tunnels.

Given the poor ground conditions, a diaphragm wall or secant pile wall construction option is recommended.

As the depth of excavation becomes significant, the excavation support costs and risks increase, making a tunnelling option more attractive.

11.4 Underground Excavated Tunnel

11.4.1 Tunnel Alignment and Cover

Underground tunnelling (as opposed to cut and cover construction) involves excavating a large hole in the ground, without removal of the ground above the tunnels. Twin tunnels would be more appropriate rather than a single large span tunnel, where tunnel support would be much more costly because the span is too large for the ground to arch across the tunnel and reduce the tunnel roof loads.

Typically tunnels allow for uniform undisturbed ground outside the tunnel extending for one tunnel diameter, as otherwise more detailed analysis, and special precautions are likely to be required (Megaw & Bartlett, 1982)[14]. For a 15 m tunnel diameter, this translates to about 30 m separation between the tunnels, or 45 m centre to centre distance.

The preferred cover to tunnels is a minimum of 1 diameter in hard rock and 2 diameters in alluvium (Ponnuswamy & Victor, 1996)[15]. For the fault disturbed rock present at the Wainui saddle, a minimum rock cover of say 1.5 diameters (or 22 m) would be preferable. Including the soil cover, the preferred minimum cover is 30 m, which would give a depth of 40 m to the road level.

The tunnel alignment would need to be east of the saddle to avoid the active Ohariu Fault and the Active Splinter of the Ohariu Fault on the east side of the saddle, with a recommended minimum separation distance from the fault of 20 m. Given the width of the fault zone, this may be difficult to achieve, and may lead to additional costs due to a further eastward tunnel being longer.

11.4.2 Tunnel Construction

The Wainui Saddle tunnel as proposed (500 m length) is too short to viably use a Tunnel Boring Machine (TBM). Because the majority of the rock is likely to be weak, the use of a road header would be a suitable option rather than adopt a drill and blast approach. Drill and blast is also not preferred given the likely low cover above the tunnel.

Construction issues need to be considered in the design and construction of a tunnel in the fault disturbed weak sheared and shattered rock at the Wainui Saddle area, with the potential for crossing fault zones associated with the historical Horokiri Fault along the main valley alignment, assuming that the alignment is able to avoid crossing the active Ohariu Fault and splinter.

Also a water proof membrane lining between the primary support and the concrete lining is used as a standard feature in modern tunnels, and should be provided to reduce seepages into the lining and discoloration from seepages.

11.4.3 Tunnel Portals

The locations for the tunnel portals should be carefully chosen so that the tunnel enters and exits the ground where the natural topography suits portal structures. This would minimise the extent of excavation and permanent support for the portal and approach walls, to ensure that these are stable under earthquake conditions and do not impose a risk to road users in the tunnel by blockage of the entrances.

It is now common practice to use sloping portals for the tunnels, and is recommended, rather than steep sub-vertical cuttings, which pose a greater risk particularly under earthquake conditions.

11.5 Feasibility of Tunnel Options

Tunnel options were considered during the scheme assessment for the project in 2008, and were discarded given the high cost compared to the adopted surface road option. Also the presence of the active Ohariu Fault and the active splinter limit the tunnel alignments.

12 Rock Cut Slopes

12.1 Design Philosophy

The preliminary design for the cut slopes in rock aim to provide generally stable cut slopes through selection of appropriate cut slope angles and geometry. Provision is also made for implementing localised stabilisation measures during construction, and accepting some failures in large hazard events – storm or earthquake, as discussed in the Route Security Philosophy in Section 9.1.

Three important issues influence the selection of cut slopes in rock, see Illustration 3.:

- stability under normal conditions
- stability and route security under large hazard events, e.g. earthquake and storm
- rock fall hazard mitigation.

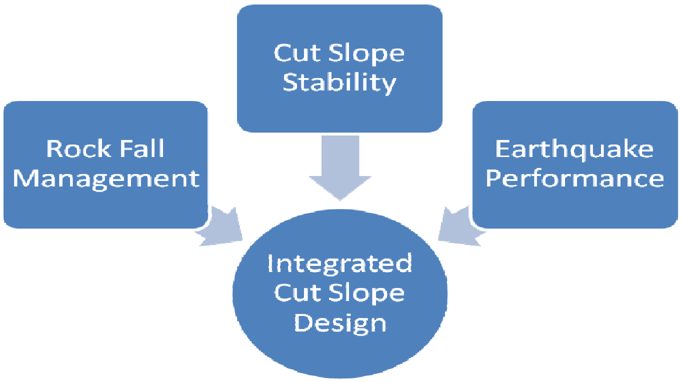

Illustration 3.10 - Important Issues Influencing Design of Cut Slopes

An integrated approach to consider all three issues has been a key consideration in the selection of an appropriate cut slope design. This approach has been fundamental to the achievement of an overall robust and cost-efficient solution that meets performance expectations.

The stability of cut slopes has been considered using four approaches:

(a) Precedent cut slopes

(b) Behaviour of slopes in historical earthquakes

(c) Rock defect analyses

(d) Rock mass stability analyses

Consideration of these approaches for the type and height of cut slopes proposed for the alignment options in Transmission Gully has enabled the development of cut slope and geometry recommendations for scheme assessment level design.

12.2 Cut Slope Configuration

The preliminary design of cut slopes in rock for the Transmission Gully route has been based on an integrated consideration of cut slope stability, earthquake performance, rock fall management and cost effectiveness.

The principles adopted for cut slope configuration are summarised in Table 3.12.

Table 3.12 - Principles for Cut Slope Configuration

Item	Principle	Influences	Commentary
1	*Overall slope angles of 40° to 45° in rock; 20° to 30° in soils; 35° to 40° in fault disturbed rock (eg Wainui Saddle) and dense gravels (eg Te Puka Terrace).*	▪ Route Security ▪ Rockfall hazards ▪ Long term maintenance ▪ Appearance ▪ Revegetation	▪ Historical earthquake induced landslides indicate large failures in steep slopes, as observed in China after the Wenchuan Earthquake 2008 and previous NZ earthquakes. ▪ Dominant rock defects are predominantly 40° to 75° which makes steeper slopes vulnerable to large defect controlled failures. ▪ More extensive rock wedge failures in steeper slopes and rock fall trajectories exacerbate rock fall hazards. ▪ Steeper slopes mean more slips and maintenance costs as well as safety hazards for road users. ▪ Natural greywacke slopes in the region are characteristically 38° to 43°, and the proposed slopes 40° to 45° better fit into a natural landscape compared to the steeper slopes. ▪ Moderate slopes make revegetation more practical.
2	Slopes incorporate min 3 m wide benches at 10 m height intervals, except first bench above highway at 15 m height.	▪ Rock fall hazards ▪ Route security ▪ Long term maintenance ▪ Appearance ▪ Revegetation	▪ Some rock fall will originate from slopes, and benches will capture and slow rock fall before they gather momentum and energy. ▪ Significantly reduced rock fall safety hazards without extensive rock fall protection interventions, large rock fall collection berm and fence at base of slope. ▪ Benches catch small slumps that soften rock surfaces and reduce rockfall trajectories that pose a safety hazard to road users. ▪ Benches reduce small wedge and planar slope failures from progressing upslope. ▪ Benches reduce velocity of surface water run-off. ▪ Benches encourage establishment of vegetation on the benches and then spread over the slope. ▪ The first bench at 15 m is a compromise to minimise its visibility to drivers using the highway.

Item	Principle	Influences	Commentary
3	Bench Configuration with minimum 10% outward slope, and terminated where height of slope above bench is minimal (< 2 m)	▪ Drainage ▪ Erosion ▪ Appearance	▪ Longitudinally horizontal benches in soils (Sector 6) to minimise potential for erosion by surface water run-off along benches. ▪ All benches have an outward slope min 10% to prevent stagnation of water on benches when small slumps inevitably occur, which would otherwise lead to feeding water into slope causing instability, and concentrated flow causing slope erosion. ▪ Bench termination to minimise benches where they are not essential, and enhance visual appearance.
4	Rounding of slopes at bench edges, top of cut slopes, and horizontally at the ends of cuttings.	▪ Rock fall hazards ▪ Appearance	▪ Sharp edges and corners of slopes particularly in rock, is a source of loose rock, wedge failures and hence rock fall. ▪ The tops of cut slopes are often in overburden weak soils or weathered rock, and will not generally stand at same slope angle as the predominant cut faces. ▪ Rounded slopes and particularly well rounded longitudinal edges of cuttings make them look as more natural hillsides than visually intrusive edges of cuttings.
5	Revegetation of Cut Slopes is encouraged.	▪ Stability ▪ Erosion Protection ▪ Rock fall hazards ▪ Appearance	▪ Revegetation of cut slopes enhances stability, provided this is with shallow shrubs rather than large trees. ▪ Larger trees should be avoided as they may cause tree falls and roots loosening and wedging off rocks. These may naturally take root in the long term, and would need to be managed. ▪ Vegetation minimises erosion particularly in soil slopes or weathered tops of cuttings in soils. ▪ Vegetation softens cut slopes and particularly benches, leading to less bouncing of rocks and hence rock fall trajectories that pose a hazard to road users. ▪ Vegetation catches and slows rock falls. ▪ Vegetated slopes give a better soft appearance and provide a natural habitat.
6	Rock fall protection Interventions will be required, but on an as required basis.	▪ Rock fall hazards ▪ Appearance	▪ The objective is to achieve passive rock fall protection through the cut face configuration, and in the longer term, vegetation. ▪ A rock fall barrier of some form will be required at the base, and is ideally incorporated with the traffic barrier – say a specifically designed NJB. The barrier should provide for ease of clearing rockfall, either space for a front end loader, or a removable NJB. ▪ In some areas with significant rock fall hazards identified during design and construction, a short rock fall fence may be incorporated onto the bench, on an as required basis. This is of value particularly during the early years after construction. ▪ Other rock fall protection measures such as draped netting may

Item	Principle	Influences	Commentary
			be appropriate in specific areas with rock fall hazards, on an as required basis.
7	Drainage measures implemented on a proactive basis.	▪ Route security ▪ Stability ▪ Ongoing maintenance ▪ Vegetation	▪ Sub-horizontal drainage holes will be installed where there is potential for groundwater pressures behind the rock or soil slope face that reduces stability of the cut slopes. ▪ The drainage holes drain onto the slope allowing for natural dispersion of the water on the cut face, and also helping the establishment of vegetation. ▪ Long term maintenance of the drainage holes should be allowed for during the operation of the highway.
8	Slope stabilisation measures implemented on an as required basis, and may comprise mesh, rock bolts, anchors, dental concrete, shotcrete, soil nailing or reprofiling the slope to a flatter slope where practical.	▪ Route security ▪ Stability ▪ Ongoing maintenance ▪ Appearance	▪ Inevitably there are likely to be localised areas that require stabilisation measures to mitigate slope instability risks. ▪ The stabilisation measures should be implemented locally on an as required basis. ▪ The appearance needs to be taken into consideration, to soften its visual impact, using texture, colour, suface treatment etc.

Note: Overburden includes alluvium, loess, colluvium and completely weathered rock.

To progress project development, further geotechnical investigations are recommended to:

- further characterise rock conditions along the route, particularly at large cuttings
- gain more information on dominant shear, crush and fault defects through further boreholes (vertical and inclined) with acoustic televiewer surveys, engineering geological defect mapping, perhaps aided by characterisation of rock mass through trial cuttings in selected areas.
- characterise rock materials including testing of the shear strength of the infill in shear surfaces and crush zones.

12.3 Stability of Cut Slopes in Rock

<u>Based on Precedent Data</u>

Experience in the design, construction and observation of the performance of cut slopes, particularly for state highways in the region, provides background knowledge of stable cut slopes and common stability issues affecting large cuttings in Wellington Greywacke.

This has been supplemented by a survey during this project, of cut slopes in Wellington Greywacke in the region, mainly state highway cuttings (SH1, SH2 and SH58), but also some along local roads and quarries. The cut slope data collated includes 49 data points presented in Illustration 3.11.

The data presented shows the weathering of the greywacke at the cut slopes as well as their stability condition. While there is some scatter in the data, it is clear that most of the data falls below the gold line, indicating the maximum stable angle of cut slopes in greywacke rock in Wellington Region. Many slopes show some local instability features.

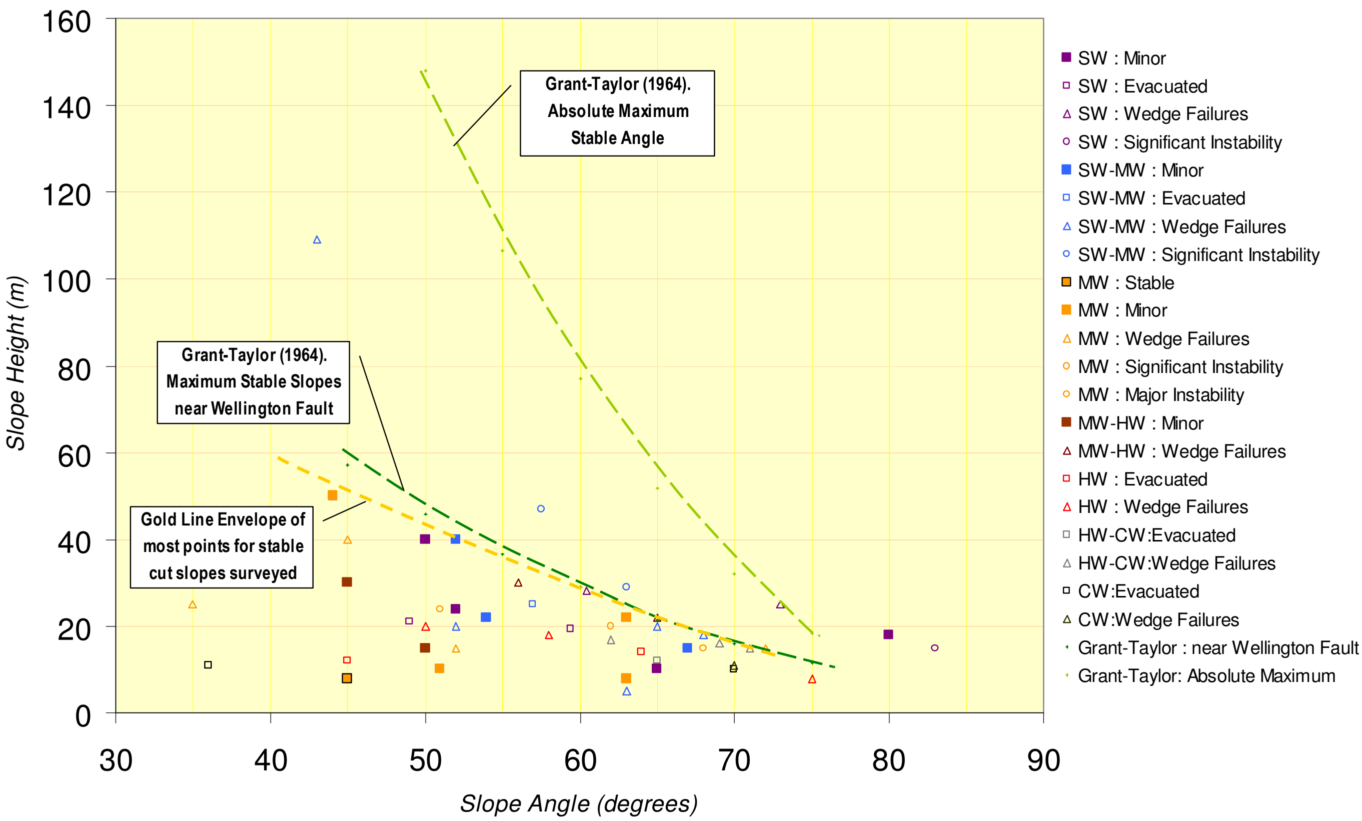

Illustration 3.11 - Superimposition of Grant-Taylor (1964) Curves on the Precedent Cut Slope Data

Grant-Taylor (1964)[16] proposed charts showing two curves representing "*absolute maximum stable angles*", and "*maximum stable angles of slopes near the Wellington Fault*" (within 0.5 mile of the Wellington Fault) based on 38 data points, see Illustration 3.11.

Grant Taylor's curves are superimposed on the precedent cut slope data in Illustration 3.11. In superimposing the two sets of information, it is important to note that:

(a) the Grant-Taylor (1964)[23] curves represents natural hillside and cliff slopes, whereas the precedent data collected by Opus represents cut slopes.

(b) the Grant-Taylor data has a small proportion for natural slopes less than 200 ft (~60 m) high, whereas the majority of the Opus data is for cut slopes less than 60 m high.

(c) the Grant-Taylor chart curves use a significant proportion of the data from natural slopes greater than 200 ft (~60 m) high, whereas the Opus cut slope data has only one point above 60 m height, reflecting the presence of few cut slopes greater than 60 m high other than in quarries.

Interestingly, the gold line which provides an upper bound for the majority of the current survey points (gold line) is very close to the Grant-Taylor curve (dark green) for *"near Wellington Fault"*.

The combined precedent slope plot indicates that the Grant-Taylor *near Wellington Fault* curve provides a useful maximum stable angle for slightly-moderately weathered or better rock, even though the Opus survey points from this investigation lie within as well as beyond the half a mile distance from the Wellington Fault suggested by Grant-Taylor.

Reflecting on Grant-Taylor (1964)[23] observations, he appears to have meant the lower curve to represent, in his words *"intensely sheared rock where planes of weakness at many angles are present",* and in his data set *"such an area is in a zone about half a mile wide from the Wellington Fault westward."* On the other hand Grant-Taylor's upper curve *"will apply only to slopes in material with no planes of weakness at angles less than the batter angle."* With the information we now have, over 40 years later, it is clear that there are extensive areas with rock where planes of weakness at many angles are present, as discussed further in Section 12.4. This reflects the fact that there are numerous faults in the Wellington Region, such as the Ohariu Fault, and together with the overall tectonic deformation of the Wellington Region, has influenced the rock mass characteristics and hence stable slopes in the Region.

Based on the precedent data in Illustration 11, the average 40° to 45° slopes proposed for Transmission Gully project would give slopes that are generally stable up to slope heights of up to about 55 m to 60 m height, with some localised failures, such as wedge failures, in rocks with planes of weakness in many angles being present.

For slopes that are higher (55 m to 75 m) and in intensely sheared, a shallow cut slope would be more appropriate such as proposed for the Wainui Saddle.

A maximum cut slope of somewhere between 40° (based on Grant-Taylor's lower curve) and 60°(based on Grant-Taylor's upper curve) may be appropriate for better quality rock, depending on the nature of the rock mass and rock defects.

In this context it should also be noted that the majority of the very high (few hundred metres) stable natural slopes in Wellington Greywacke in the region are of slopes between about 38° and 43° based on the our observations.

These observations suggest that the stable slope angles are not sensitive to slope height below a slope angle of about 45°, and the stability will be more a function of particular dominant adverse defects and rock mass conditions than just slope height at least up to a few hundred metres. Note that Grant-Taylor's curves do not extend below a slope angle of about 45°, see Illustration 3.11.

<u>Based on Rock Mass Stability</u>

Consideration has been given to the stability of the rock cuttings as a circular failure through a homogeneous jointed rock mass.

The GSI classification of rocks logged in selected boreholes along the route is presented by Opus (2008c). The GSI classification shows that:

(a) Rocks in Sectors 3 and 4 are predominantly of GSI class C and D (GSI values of 20 to 35), with the exceptions of:

- Rock in borehole BH 29 on the eastern flank of Horokiri Valley has GSI Class B and C rock (GSI values of 30 to 40) below about 45 m depth, which means better rock is likely in the lower part of high cuttings along an alignment on the eastern flank of the valley (*In-designation Alignment 1A*).

- Rock in boreholes BH 16, BH 71 and BH 72 at the saddle and on the western flank of Horokiri Stream south of the saddle is weaker GSI Class E and D (GSI values 5 to 30), indicating generally poorer quality rock. The alignment on the western flank (*Preferred Alignment 1B*) has now been modified to minimise the cuttings along this section.

(b) Rocks in Sectors 8 and 9 are predominantly GSI Class C and B (GSI values 30 to 40) below the 5 m to 10 m deep weathering zone, with the exceptions of:

- BH 39 which has Class D rock to 20 m depth

- BH 30, which has GSI Class D rock to about 40 m depth

- BH 52, which has Class E and D rock over its full depth of 25 m.

Stability analyses carried out using Hoek-Brown parameters for the rock mass have indicated that average 45° slopes in Class C and D rocks will have a factor of safety of at least 1.5 under static conditions, assuming average groundwater levels, in the absence of dominant defects which give rise to failures controlled by defects.

In Class C and B rocks, the stability of 45° cut slopes are expected to have a factor of safety much greater than 1.5 under static conditions. In this case a 50° slope will give a factor of safety of at least 1.5, in the absence of defect controlled failures.

In general, the average 40° to 45° cut slopes adopted in rock will provide a acceptable stability against rock mass failure. Further investigation and assessment is required for major cut slopes along the route to better define the rock mass properties and particularly the significant shear zone features and their strength. Systematic investigation and design is recommended.

12.4 Influence of Defects on Cut Slope Stability

The Wellington Greywacke rocks along the Transmission Gully route are generally very closely to closely jointed. However, the joints generally have a very low persistence, and the joint orientations vary considerably. This would suggest that the rock would behave as a "homogeneous" closely jointed rock mass. This is generally true for the case of joints.

However, there are also numerous minor faults, shear surfaces and crush zones present in the rock mass, and these are generally much more persistent (tens of metres) than the joints, and hence have the potential to have a significant influence.

Given that the cores from the boreholes cannot be used to assess the orientations of these dominant defects, acoustic televiewer surveys were undertaken in representative boreholes and also some field mapping of defects was undertaken at outcrops and small cuttings along tracks (Opus, 2008b)[3]. The data from these surveys provide information on the dip and orientation of these dominant defects.

These shear surfaces, faults and crush zones have a dominant dip direction of 110° to 150°, and 290° to 310°. This corresponds to a strike of SSW to NNE as clearly shown in the Rosette plot in Illustration 3.12.

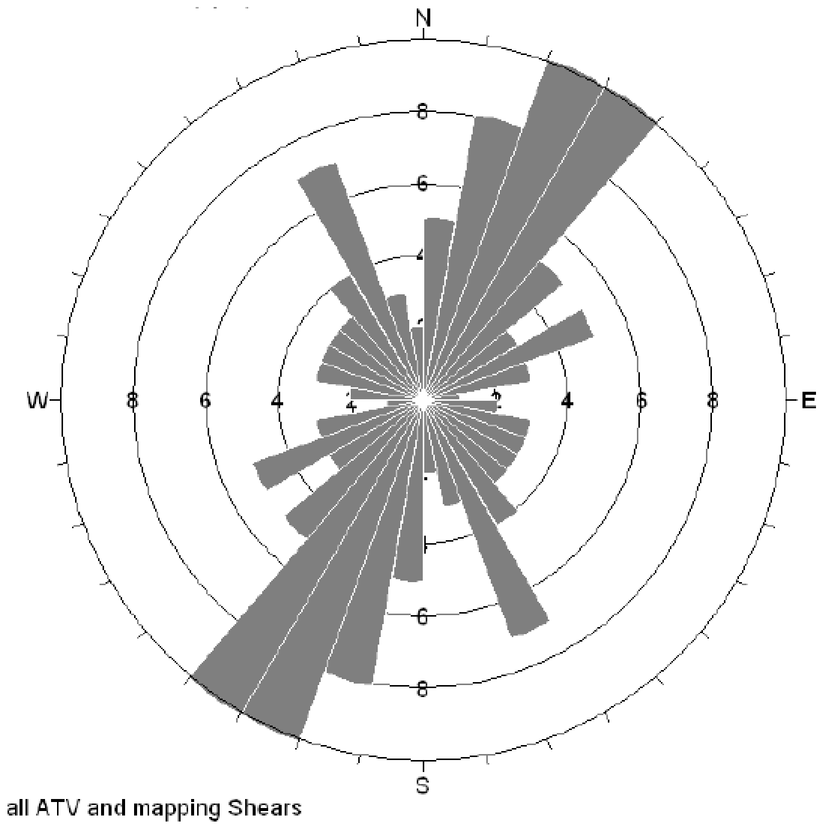

Illustration 3.12 - Rosette Plot showing Strike of Shear Surfaces

The dip and directions of the shear surfaces mapped from the acoustic televiewer and field defect mapping surveys is presented in Illustration 3.13. This three-dimensional plot clearly shows the concentration of dominant defects with dips of 45° to 80° and dip directions of 110° to 140°, and 300° to 330°. Also shows some low angle defects dipping 20° to 30°.

These shear surfaces have a strike that is generally sub-parallel to the strike of the proposed cuttings along the Transmission Gully alignment in Sectors 2 to 8, as shown in Illustration 3.14.

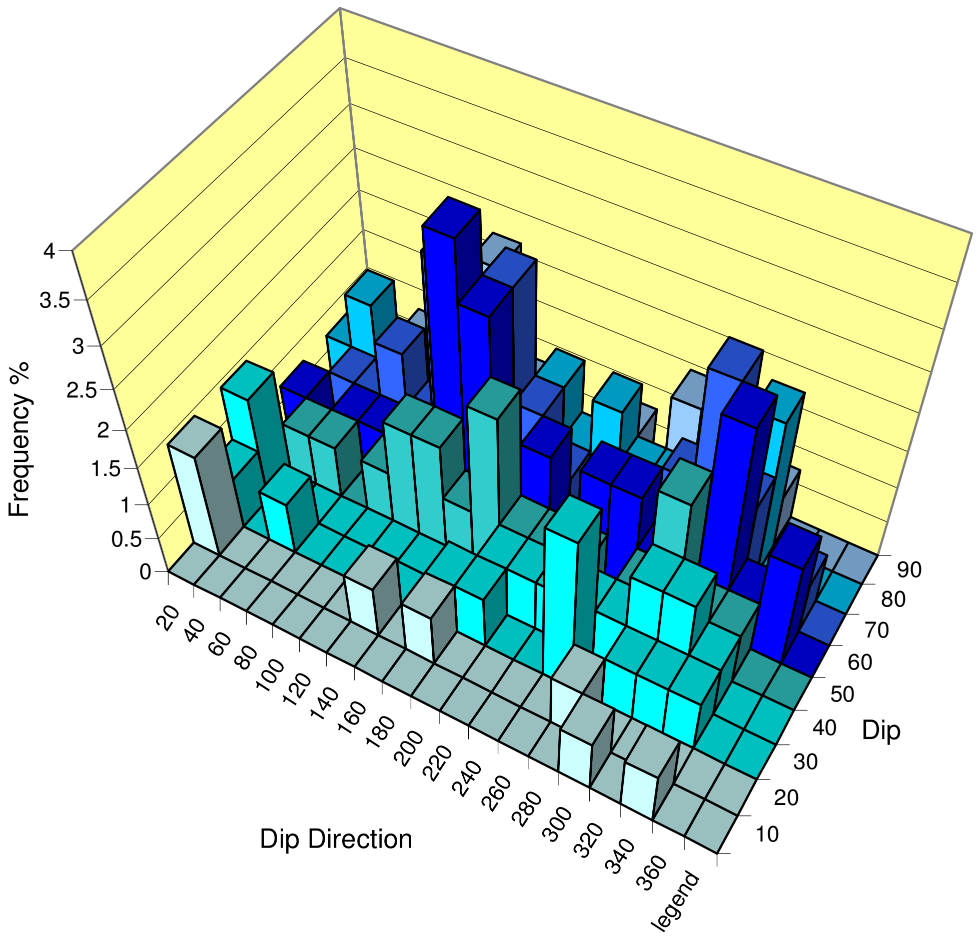

Illustration 3.13 - Frequency Plot showing Dip and Dip Directions of Shear Surfaces in Rock

The parallel nature of the strike of the shear zones to the strike of the proposed cuttings, ie the presence of widespread shear surfaces that dip perpendicular to the plane of the cut slopes, has the potential to cause large planar failures where the cuts are formed steeper than the shear surfaces. Such failures have been observed in cuttings for the State Highway 58 realignment (Hancox & Brabhaharan, 1995)[17], and the Newlands Interchange cutting prior to the late 1990s construction of an interchange (Brabhaharan, 1998)[18].

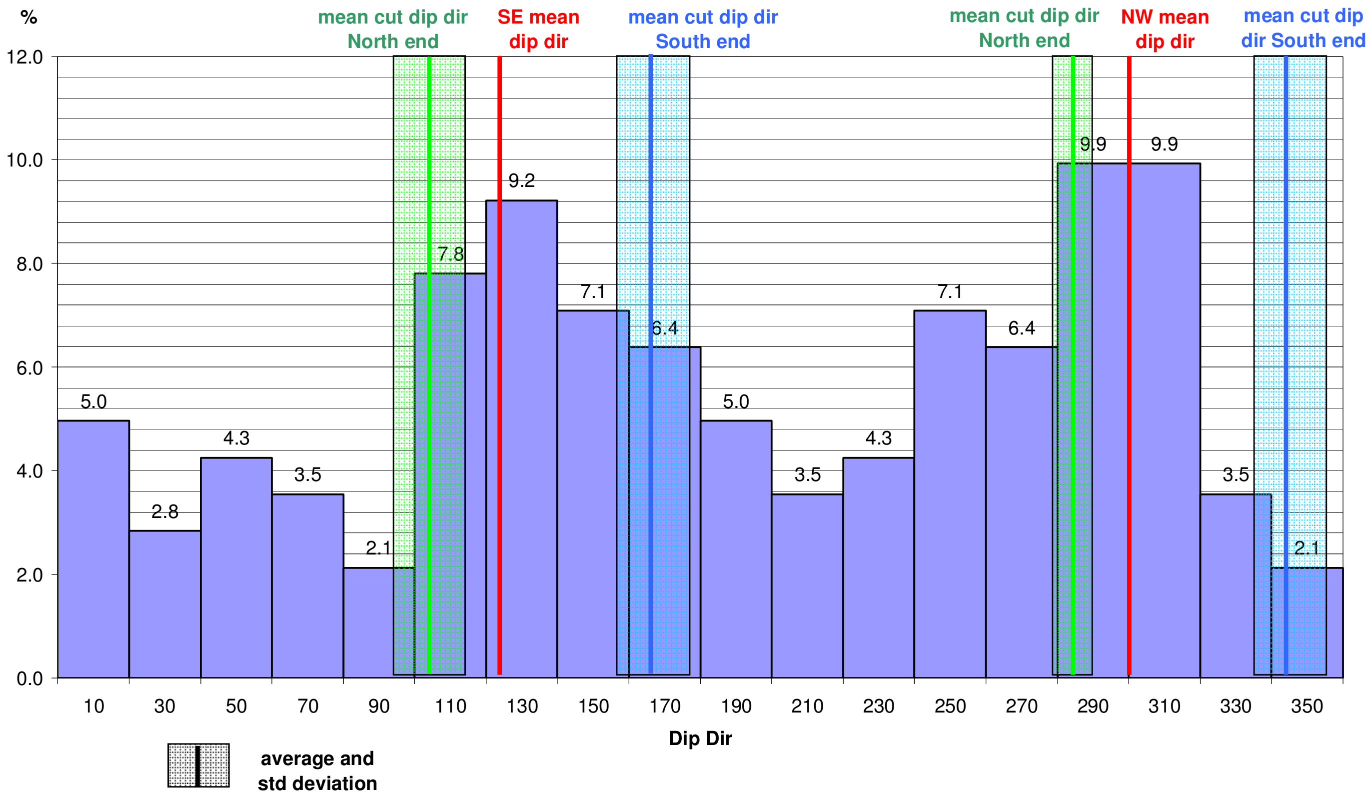

Illustration 3.14 - Comparison of Dip Direction of Cuttings and Shear Surfaces

The direction of the dips in relation to the cuttings on the eastern and western sides of the valleys has been considered, and potential for instability assessed using stereo nets.

The stereonet on Illustration 3.15 shows the E and SE dipping cuts in Sectors 2,3,4 & 8 (red great circles) for the alignment on the western side of the valley in sectors 3 and 4 (*Preferred Alignment 1B*), and S dipping cut slopes in box cuts in Sector 9 (blue great circles) (both *In-designation 1A* and *Preferred 1B Alignments*) in relation to the great circles for the shear zones (grey).

The cuts shown are assumed to be 45°. The friction circles (20° and 30°) in black show that there is some potential for wedge failures, but these are relatively less if the cut is formed at a slope of about 45°. There will also be some potential for toppling failures given the presence of very steep shear surfaces and faults.

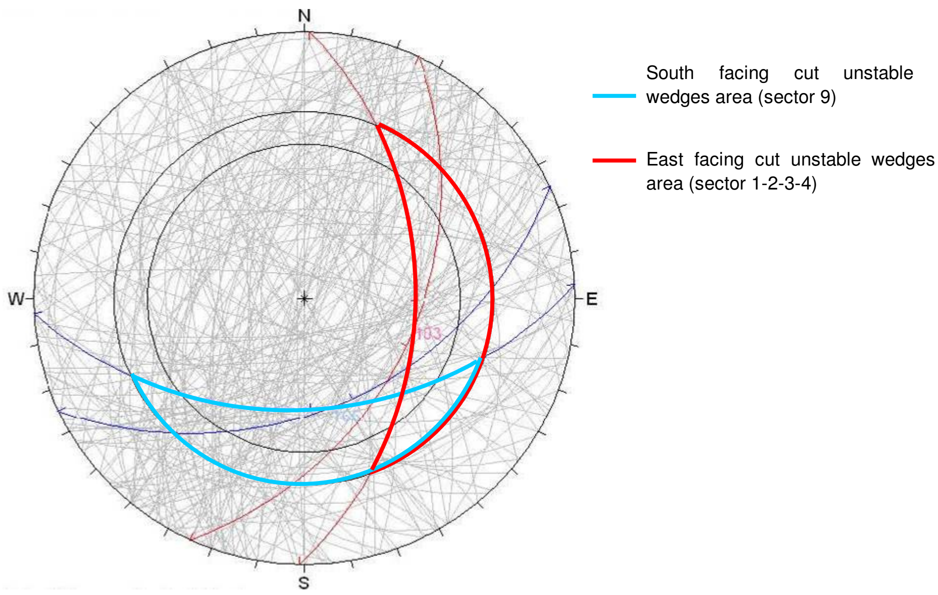

Illustration 3.15 - Stereonet for Wedge and Planar Failures on S & SE Dipping Cuts

The stereonet on Illustration 3.16 shows a similar plot for NW dipping cuts in Sectors 2,3 and 4 (red great circles) for the alignment on the eastern side of the valley (*In-designation Alignment 1A*), and the dominant north facing cuts in Sector 9 (both *In-designation Alignment 1A* and *Preferred Alignment 1B*). The more frequent intersections of the great circles of shear surfaces (grey) between the great circles of cut slopes (red / blue) and the friction circles (black) indicate the greater potential for wedge and some planar failures in this case.

The NW facing cuts will apply only for the *In-designation Alignment 1A* on the eastern flanks of the Te Puka and Horokiri Valleys, but the N facing cuts (blue) in Sector 9 will apply for both alignments. The reliability of the assessment for Sector 9 is low given the very limited data on dip and orientation of shear surfaces currently available for Sector 9 (No acoustic televiewer data available for Sector 9). These will need to be considered further in design, potential wedges stabilised and managed during and after construction.

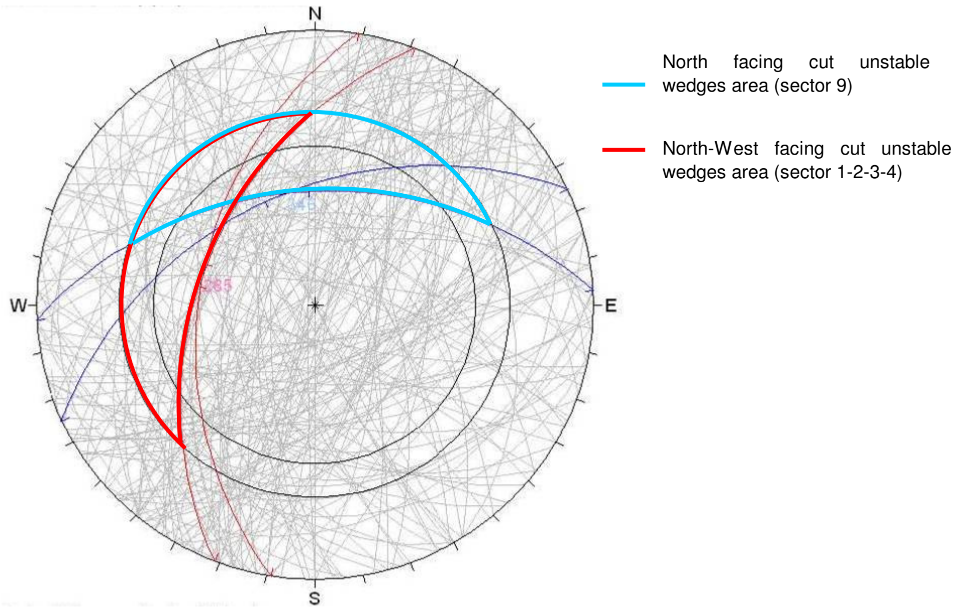

Illustration 3.16 - Stereonet for Wedge and Planar Failures on N & NW Dipping Cuts

Overall formation of the cuttings at a cut slope of about 45° to 50° will minimise the potential for planar failures due to the dominant shear surfaces that are steeper than about 45° to 50° (80%), see Illustration 3.13. Some 15% of shear zones that dip at between 20° to 45°, and in particular the significant proportion of those that dip towards the west-northwest (see Illustration 3.13) could give rise to some failures for the west-northwest facing cut slopes on the eastern flank of the Te Puka and Horokiri Valleys (for *In-designation Alignment 1A*), depending on the shear strength on the shear surfaces and crush zones.

There is no information on the shear strength of the shear surfaces / zones and crush zones, and testing is planned to be carried out in the next phase of the project to characterise the shear strength of these zones.

The shear zones that do not daylight or have sufficient shear strength may give rise to combined failures partially through the rock mass and partly along these weak shear zones, as discussed in Section 12.5.

12.5 Combined Rock Mass and Defect Controlled Instability

Failure in the rock mass with closely spaced joints, and the likely presence of dominant low strength shear surfaces (see Section 12.4) could potentially give rise to a combined failure along a dominant defect such as a shear zone and through a section of closely jointed rock (say a toe break out). This is a much more likely mode of failure in the rock mass in the Wellington Region and along Transmission Gully than a failure entirely through jointed rock mass. Such a mechanism needs to be considered during detailed design for specific cut slopes.

Stability analyses using SLIDE indicate that the presence of dominant defects that do not daylight in the proposed cuttings can still reduce the stability of the high cuttings in rock, see Illustration 3.17. The analysis indicates that dominant defects closely behind cut faces could lead to failure during construction or under static conditions, particularly if high groundwater pressures exist in the slope. The risk of such failures can be reduced by drainage measures, such as sub-horizontal drainage holes, and in some cases rock anchors. The sizes of failures from defects close to the face are expected to be small. The influence of combined failures and the associated risks would be further considered on a site specific basis for the large cuttings in rock.

Defects further back from the slope may not lead to failure during construction or under static conditions unless they are located close to the cut face. However, they could lead to failure under strong earthquake shaking, which would reduce the stability of such slopes. This confirms the precedent based assessment in Section 12.6.2 that some earthquake induced failures can still be expected.

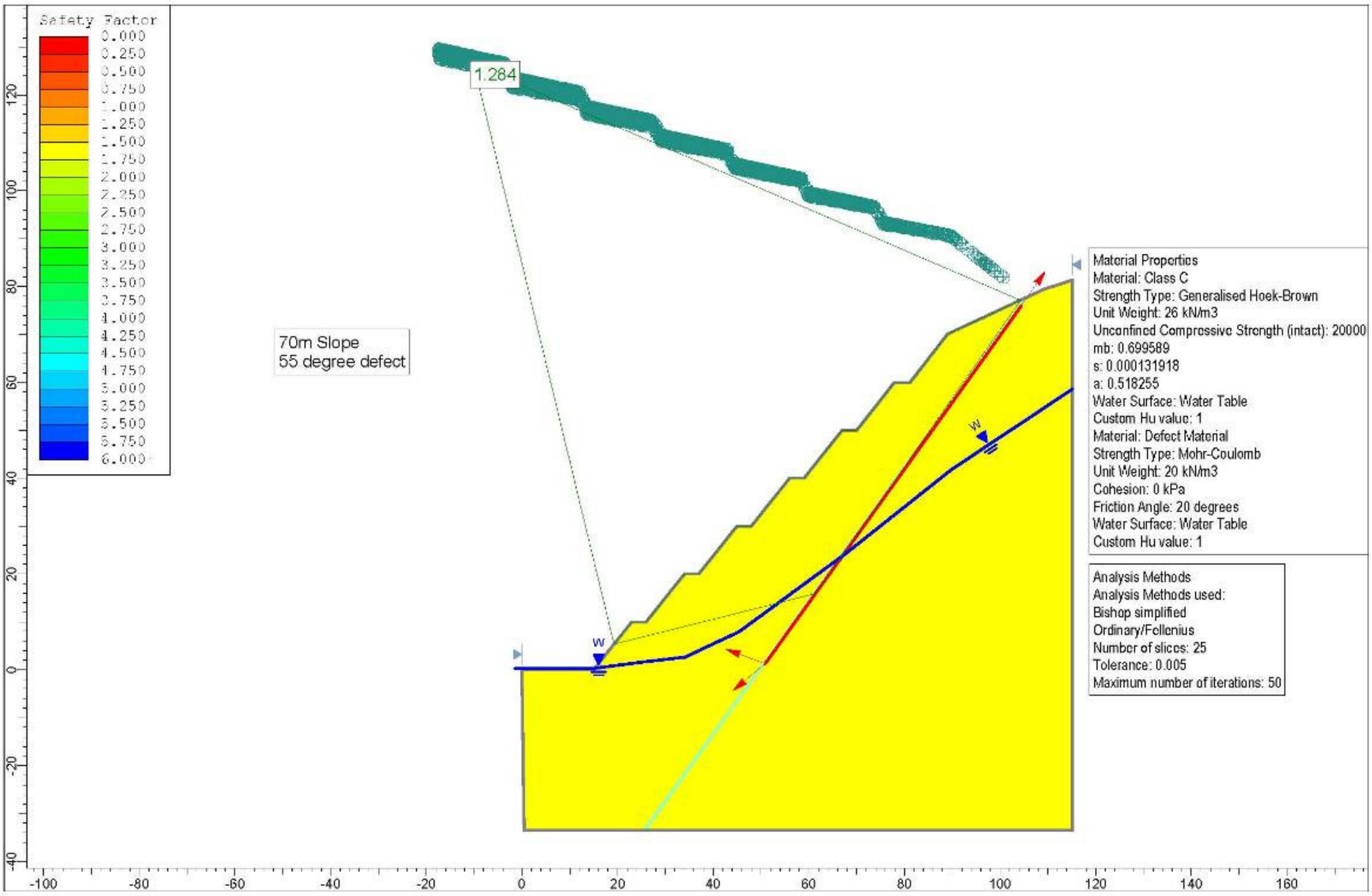

Illustration 3.17 - Typical Analysis of Combined Defect and Rock Mass Failures in Rock

12.6 Earthquake Stability of Cut Slopes

12.6.1 Historical Slope Failures Induced by Earthquakes

Historical landslides provide the most valuable information on the susceptibility of slopes to earthquake induced landslides. Historical landslides in the region have been considered to understand and present the potential for earthquake induced landslides along the Transmission Gully route.

Historical earthquake-induced landslides in the area resulted primarily from the Wairarapa earthquake in 1855 (Brabhaharan et al, 1994[19]; Hancox, 2005[20]), by which time small cuts had been made for roads. Slope failures caused by the 1855 earthquake were reported to be common along the Paekakariki Hill Road, which was constructed in 1849 utilising many cuts and fills and served as the main route north until the present SH 1 opened in 1939. Widespread slope failures are also reported along the Hutt Road and Rimutaka Hill Road in the 1855 earthquake event.

The large landslide reported at Paekakariki after the earthquake is inferred to have been the cause of the prominent scar high on the cliff above the old quarry near the Paekakariki railway station (Opus, 2007)[1]. It is unlikely that was the only large landslide on coastal slopes in the area – just the only one that was reported.

The shaking intensity from the two Masterton earthquakes in 1942 was only MM 6 to 7 in the Wellington area, which is not high enough to trigger large slope failures. However, a moderately large debris slide at Goat Point at Plimmerton during the 24 June 1942 earthquake blocked both lines of NIMT railway. A few small soil slides also occurred along the road and railway between Plimmerton and Paekakariki. These slope failures were minor, and the railway was reopened about 7 hours after the event.

The historical landslides caused by earthquakes indicate the potential for earthquake induced landslides in the steep greywacke slopes along the Transmission Gully route, particularly when modified by high steep cut slopes for road construction. However, these are likely to be limited and smaller in size than the large to very large landslides likely along the coastal cliffs, eg between Paekakariki Station and Pukerua Bay along SH 1.

12.6.2 Potential for Earthquake Induced Slope Failures

An earthquake induced slope failure hazard study for the Wellington Region was completed for the Greater Wellington Regional Council (Brabhaharan et al, 1994)[26].

On the Transmission Gully Route, south of Pauatahanui there are several areas rated as having a moderate susceptibility to earthquake induced slope failures (Opus, 2007)[1]. The northern sections of the Transmission Gully route from Battle Hill north have notably higher earthquake-induced landslide susceptibility, see Illustration 3.18.

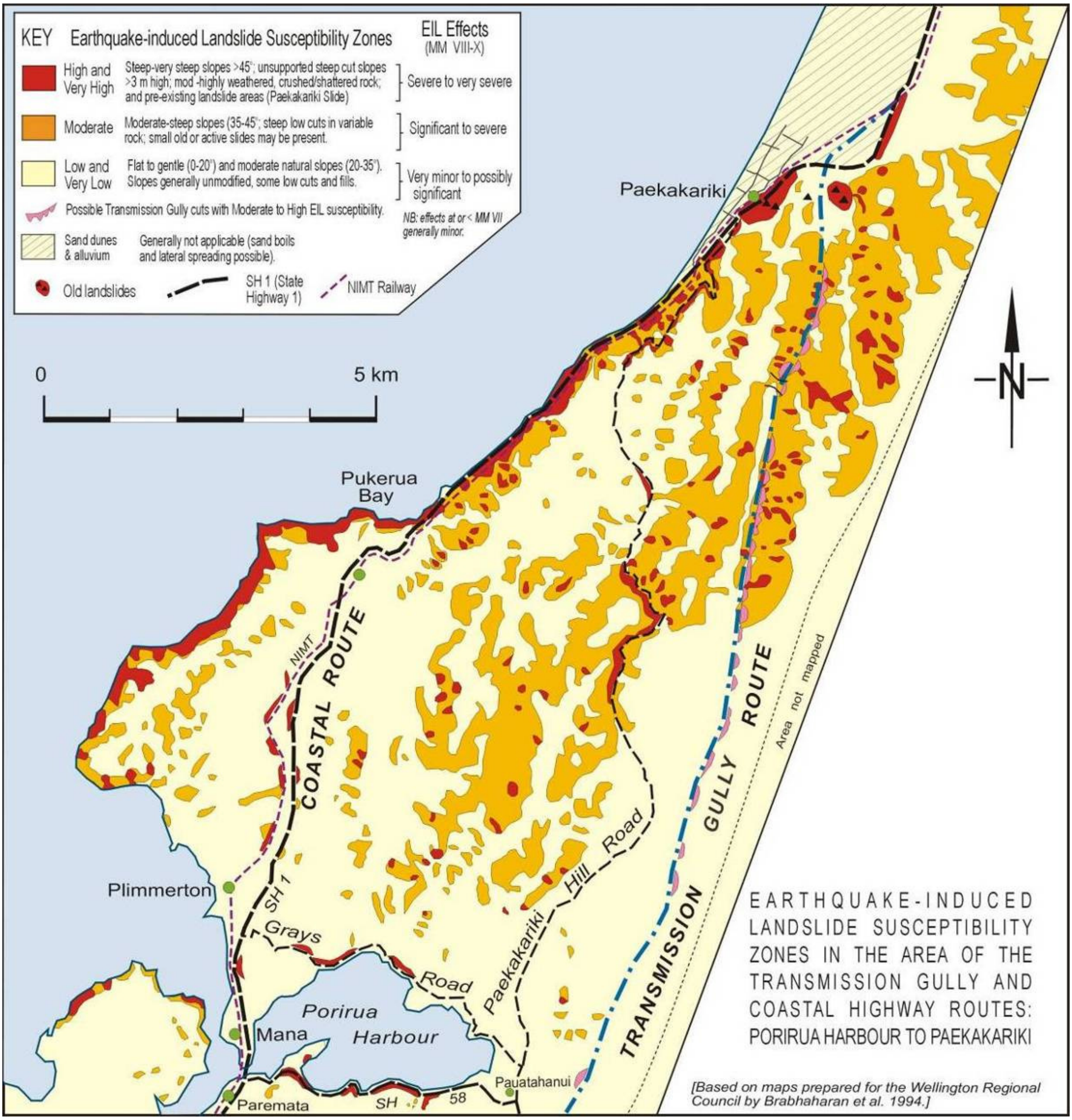

Illustration 3.18 - Map showing earthquake-induced landslide susceptibility

Shows earthquake-induced slope instability in the area of the Coastal Highway and proposed Transmission Gully route from Porirua Harbour to Paekakariki area (after Brabhaharan et al. 1994; Wellington Regional Council, 1995).

The natural slopes along the Transmission Gully route in the Te Puka and upper Horokiri Stream valleys have moderate susceptibility, with only some small areas of high susceptibility. This is consistent with the generally lower overall slope angle and lack of deep seated landslides along the Transmission Gully route.

Studies of earthquake-induced slope failure in the Wellington area (Brabhaharan et al, 1994[26]; Wellington Regional Council, 1995[21]) and more recent studies in New Zealand (Hancox et al, 1997[22] and 2002[23]) suggest that any high cuts along the Transmission Gully route steeper than 45° will have high to very high susceptibility to earthquake-induced slope failure during MM 9 – MM 10 earthquake shaking.

Although there are currently no modified slopes along the Transmission Gully route (apart from farm tracks), construction of the proposed road would result in cuts with a higher susceptibility than the existing natural slopes.

Small to moderate failures (c.1,000 m^3 – 20,000 m^3) could occur along the Transmission Gully corridor, even on well designed cut slopes (no steeper than 45° and generally less than 50 m high), but few failures are expected on cuts of 35° or less.

12.6.3 Assessment of Seismic Performance

The seismic performance of cut slopes in rock can be considered using stability analyses using a software such as SLIDE, to assess the critical acceleration that leads to a factor of safety of 1, and then assessing the displacement of the slope using methods such as Ambraseys and Srubulov (1998)[24] or Jibson (2007)[25], for the expected peak ground accelerations for different return period earthquake events discussed in Section 8.3. Such an approach was used by O'Riley and Brabhaharan (2006)[26] to assess the slopes on the approach to the Terrace Tunnel in Wellington.

A displacement based design approach would need to be considered in the design of the cuttings along Transmission Gully. Based on the assessment of the Terrace Tunnel approaches with walls, stability of the overall slope relied on the large rock anchors that were installed and anchored to an anchor drive tunnel to support the cutting.

Given the presence of extensive dominant rock defects such as shear surfaces and crush zones, it is likely that stability of the cuttings would require extensive and heavy rock stabilisation to ensure earthquake stability, which could lead to a very high cost.

A more pragmatic approach would be to design the cuttings to ensure stability under normal conditions and limit the size of failures in large storm or earthquake events, and consider the potential for earthquake induced slope failures and their effect on the performance and availability of the highway after major events. This would enable the selection of cut slopes that would minimise the size and extent of failures likely, and the design of stabilisation measures at critical locations to ensure the required level of performance. Such an approach is consistent with the route security philosophy proposed for the project, see Section 9.

Using the methodology developed and used by Brabhaharan et al (1994)[26], the higher 50 m to 60 m high cuts at an average 45° slope would have a moderate susceptibility to earthquake induced slope failure. Moderate slope failure susceptibility would give rise to a minor slope failure potential in a MM VII to VIII ground shaking, and severe slope failure potential in a MM IX to X shaking intensity. The minor slope failure potential implies small (100 m^3 to 10,000 m^3) size failures, and a severe slope failure potential implies large size (> 10,000 m^3) failures. However, very large failures (>100,000 m^3) are not expected. Failures of the order of 10,000 m^3 can be cleared in a few days to a week or so, and hence such failures are consistent with the route security philosophy proposed for the project.

The maximum 70 m to 80 m high cuttings (predominantly along the In-designation Alignment 1A) formed at a slope angle no steeper than 45° are still likely to have a moderate slope failure potential in earthquakes. However, given the higher cut slopes, the volume of materials mobilised would be expected to be larger (say 100,000 m^3), particularly for the cuttings on the eastern flanks of the Te Puka Stream with thick overburden deposits.

Failures of 100,000 m^3 could take a few weeks to clear and may be at the upper end or somewhat higher than what is considered to be acceptable based on the route security philosophy. This is more applicable to the *In-designation Alignment 1A*.

The 70 m to 75 m high cuttings proposed at the Wainui saddle are recommended to be formed at a slope of 35°, and therefore are expected to lead to smaller failures in a large earthquake event, which can be expected to be cleared in a few days to 2 weeks, and hence are assessed to meet the route security philosophy.

In summary, the earthquake induced slope failure hazard study indicates that slopes steeper than 45° could lead to a high vulnerability to slope failures in earthquakes. The formation of cut slopes to an average cut slope of 45° or less as proposed for the Transmission Gully project, would limit the susceptibility to a moderate level with minor to severe potential and small to large failures, which can be cleared to provide limited access within days to 2 weeks.

The higher cut slopes in the range of 60 m to 80 m could lead to larger failures which could take longer to clear. Cut slopes higher than about 55 m to 60 m height have been minimised as part of the development of the preferred alignment. The few cut slopes that are higher than 60 m would be considered further at the next stage of design development, including the potential to reduce the height.

Steeper cut slopes, particularly for the high cuttings, are likely to lead to a poorer performance in earthquakes.

12.7 Cut Slope Configuration

The cut slope configuration widely adopted in the Wellington Greywacke rocks in the region is to use 3 m wide benches at about 10 m height intervals, for example at SH1 Newlands Interchange, see Illustration 3.19. The photograph shows a benched cut slope in Wellington Greywacke rock similar to that likely to be encountered at Transmission Gully, 10 years after construction, showing excellent performance. The 50 m high cut slope was formed at a slope of about 50°, with 3 m wide benches at 10 m height intervals, giving an overall slope angle of 45°. The benches had a slight outward fall to ensure that the benches are drained and do not lead to accumulation of surface water.

This has provided excellent performance of cut slopes and has helped manage rock fall hazards, minimise erosion, and helped establish vegetation on the slopes to provide an attractive landscape in the medium to long term.

Illustration 3.19 - Benched Cut Slope in Wellington Greywacke at SH 1 Newlands Interchange

The issue of the use of benches in relation to rock fall management is discussed further in Section 13. Alternative slope configurations can be considered, and these are discussed in further detail in Section 13.

12.8 Cut Slope Stabilisation Measures

Groundwater is a key factor that will influence the stability of slopes. This will be critical where the groundwater levels are high, particularly in relation to deep cuttings, and where faults are present leading to a retardation of natural drainage or preferential drainage along say more permeable crush zones. Sub-horizontal drainage holes are proposed in areas of high groundwater pressures to drain the cuttings to improve stability. It is important that the drains are maintained in the long term.

Some stability issues are likely to arise in the design and construction of the cut slopes, and may relate to potential planar or wedge failures, fault zones, weak fault disturbed rock, and instability of overburden deposits overlying bedrock. These will require stabilisation measures.

Table 3.13 - Cut Slope Stabilisation Measures

Issue No	Stability Issue	Possible Consequences	Stabilisation Measures
1	Groundwater levels are high	▪ Slope instability, particularly along weak defect surfaces	▪ Sub-horizontal drainage holes, and possibly pre-drainage holes in fault areas south of Wainui Saddle. ▪ 15 m to 25 m long, at 5 m to 20 m spacing, and one or two levels, drilled from benches as excavation proceeds.
2	Overburden soils are thick	▪ Slumping of overburden soils	▪ Round overburden soils above rock to a flatter 25° to 35° slope, where possible. ▪ Soil nail stabilisation of overburden, where rounding not possible due to steep hillside.
3	Sharp unstable edges at the ends of cut slopes	▪ Failures – rock slides, slumps and rock fall from ends of cuttings	▪ Horizontal rounding of the edges of cut slopes to blend into adjacent hillside.
4	Unstable wedges, blocks or slabs of rock	▪ Rock wedge failures, slabbing and rock fall hazard	▪ Scale to remove loose rocks after excavation of a maximum of each bench height. ▪ Rock bolting to secure wedges, blocks and slabs of rock identified during construction.
5	Unraveling of intensely fractured zones of rock	▪ Progressive unravelling, exposing rock to further wedge and block failures.	▪ Galvanised steel mesh or geogrid mesh pinned to secure intensely fractured area of rock face, using rock bolts.
6	Fault or shear zones prone to erosion	▪ Weathering and erosion of weak fault or shear zones, leading to undermining of rock and leading to failures, wedge / block failures / rock fall.	▪ Reinforced shotcrete with mesh and pins to cover and protect against degradation and erosion.
7	Rock slope Instability	▪ Medium to large scale failure of cut slope.	▪ Soil nailing or anchoring of unstable cut areas identified.

12.9 Landscaping, Revegetation and Erosion Control

Cut slopes are particularly prone to failure at the edges of cuttings, where the rocks and overburden soils are more weathered and the geometry leads to sharp corners. It is recommended that the edges of cut slopes are landscaped, rounded and blended into the adjacent natural slopes to avoid this.

Given that some weaker sheared rocks and weathered rocks are erodible, it is important that the slopes are revegetated as soon as possible during construction after they are formed, and the revegetation maintained during the early years after construction. Revegetation should encourage native grass and shrubs and avoid larger trees, the roots of which can penetrate rock joints and prise open them leading to ingress of water and rock fall hazards.

Surface cut off drains should be formed where there is a catchment above the cut slopes, to intercept and divert surface run-off away from the cut slopes. Controlled collection and discharge of all surface and sub-surface drainage off cut slopes should be provided.

Stepped or cascade drainages should be allowed for discharging the water off tributary gullies and streams, particularly where these are cut off by the road cuttings. The discharge from these should be channelled and discharged through appropriate culverts under the highway.

12.10 Summary

Cut slopes with an average cut slope of about 45° or less has been developed with 3 m wide benches at 10 m height intervals.

Earthquake performance of such cut slopes has been considered based on the Wellington Region slope failure hazard study (Brabhaharan et al, 1994)[26] and historical earthquakes in the region. The study indicates that cut slopes of 45° would give a moderate susceptibility, with the potential for failures up to the order 10,000 m^3 size failures in a large MM VII - VIII earthquake shaking and somewhat bigger > 10,000 m^3 failures in a MM IX to X shaking. Higher slopes of 60 m to 75 m, if adopted, could give local failures up to ~100,000 m^3 size.

Rock fall hazards can be better managed if the faces of cut slopes are kept within the range of 40° to 55°, leading to rolling behaviour of the rock fragments, rather than bouncing, which would limit the travel of the rock.

Considering all the above issues, and with a view to arrive at a cost effective solution, a slope of about 45° is considered appropriate for the majority of the cut slopes along Transmission Gully. Stabilisation measures are considered appropriate to deal with stability issues during construction.

13 Rock Fall Hazards and Management

13.1 Design Philosophy

Rock fall is a consequence of blocks of rock dislodging from cut slopes in bedrock and falling, rolling and bouncing down the slope to enter the road carriageway and posing a safety hazard to road users. It also leads to unravelling of the slope which can develop into larger slope failures.

The design philosophy adopted for rock fall hazard management is to minimise the potential for rock fall through appropriate selection of slope angle and providing for local stabilisation measures, adopting slope angles that would not exacerbate travel distances through local bouncing, and providing containment of any rock fall close to its source to minimise the safety hazard to road users. The philosophy is to incorporate and build on local knowledge and performance in the region.

13.2 Cut Slope Design and Effect on Rock Fall Hazards

The cut slope configuration has a significant effect on rock fall hazards associated with road cuttings.

Ritchie (1963)[27] developed a rock fall collection ditch design chart based on field tests (Wylie and Mah, 2004)[28], and considered the rock fall behaviour on different slopes as shown on the diagram reproduced in Illustration 3.20.

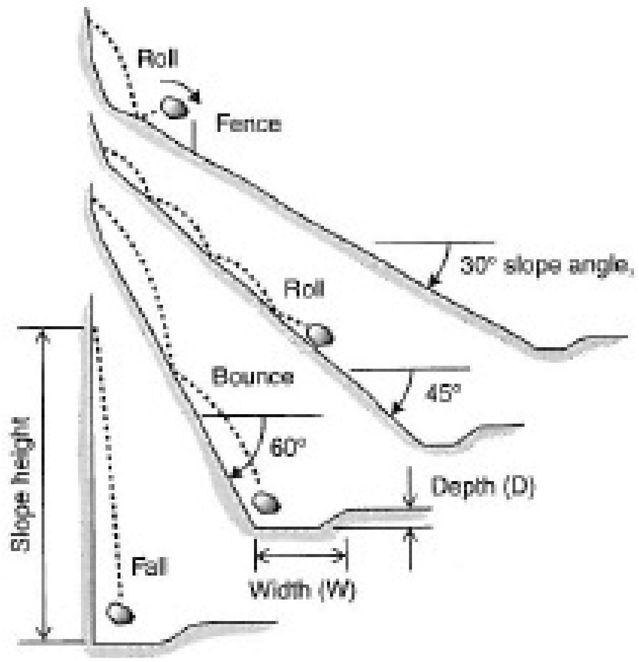

Illustration 3.20 - Rock Fall Behaviour on Slopes

After Ritchie (1963)[34] and Wyllie and Mah (2004)[35]

Wyllie and Mah (2004)[35] indicate the following behaviour of rocks on different slopes:

(a) Falling rocks tend to stay close to the face and land near the toe of the slope for slope angles steeper than 75°. This will not be applicable to cuttings at Transmission Gully, which will generally not be able to be formed to a steep 75° cut slope.

(b) Falling rocks are indicated to bounce and spin with the result that they can land a considerable distance from the toe, for slopes angles between about 55° and 75°. Avoiding cut slopes angles in this range would minimise rock fall travel and be beneficial in minimising rock fall ditches or rock fall management measures such as fences.

(c) Rocks will tend to roll down the face and into the ditch at the toe, with slope angles between about 40° and 55°. The cut slope angles adopted for the Transmission Gully project are in this range, and would be effectively minimise rock fall hazards to the highway, from the rock fall perspective. This would minimise the need for considerable rock fall management measures, such as wide ditches requiring large additional volumes of excavation or costly rock fall fences.

Typically rocks which dislodge tend to break up and the rock fall materials in greywacke are typically less than 200 mm in size. Locally larger blocks may occur particularly away from faults.

13.3 Rock Fall Hazards in the Wellington Region

The rock fall hazards in the Wellington Region, and in particular along Transmission Gully originate from the cuttings in fractured Wellington Greywacke rocks.

Rock falls from cut slopes are generally more dominant in the early years after construction when the rock has been freshly cut. Weathering, relaxation and rainfall can lead to rock slope failures exposing fresh rock and inducing further rock falls. Over a period of time the rock fall quantity reduces. The natural establishment of vegetation also tends to slow any rock fall after a period of time.

The Wellington Greywacke rock generally has closely to extremely closely spaced defects. This leads to the rock fragments being generally less than 200 mm in size (up to closely spaced joints), and often less than 60 mm in size (extremely closely to very closely spaced joints). Therefore, the risk to road users from rock fall is generally low, particularly when the cut slopes are formed at slopes of less than 50°.

Cut slopes up to 50 m high have been formed over the past 20 years in the Wellington Region, for example Rimutaka Hill corner improvements (25 m high), SH 1 Newlands Interchange (45 m high), SH 58 realignment (30 m high) and SH 1 Plimmerton to Pukerua Bay (30 m high). These recent cuttings have only caused a low level of rock fall hazard, comprising small size rock fretting from the slopes entering the road shoulder. A somewhat greater level of hazard has been observed from road cuttings formed at steeper slopes of 60° to 75°, where there is a greater potential for rock to become dislodged and the rock is more likely to bounce on the slope and reach the carriageway, for example on SH 2 Rimutaka Hill Road.

Flatter cut slopes (less than 50°) pose a lower level of rock fall hazard, as the rock is then generally likely to roll down the slope onto the road rather than bounce and pose a rock strike hazard to the road users, as indicated in Illustration 3.20.

A slightly higher (moderate) level of hazard has been caused by steep cuttings in older alluvial gravel deposits (eg Kaitoke Gravels) which comprise coarse gravel, cobble and occasional boulders in a sandy silt matrix. In some instances, rock fall protection netting has been installed to mitigate this hazard (eg SH 2 Kaitoke to Te Marua realignment).

Illustration 3.21 - Rock Fall Protection Netting at SH 2 Kaitoke to Te Marua

This would be relevant to cuttings formed in the dense gravels at the northern end of Te Puka Stream valley in Sector 2.

13.4 Assessment of Rock Fall Hazards

13.4.1 Cuttings in Pre-Holocene Alluvium

The older alluvium along TG (especially Sectors 5, 6 and 8) are generally fine grained based on the results of the site investigations to date (compared to the coarser Kaitoke Gravel) and hence pose a very low level of rock fall hazard.

An exception is the Te Puka gravel terrace immediately south of SH 1 in Paekakariki, where the cuttings in the coarse alluvial gravels may lead to a greater rock fall hazard.

These cuts are recommended to be cut at a slope of about 40°, and rocks are likely to roll down the slope at this angle, see Illustration 3.20.

13.4.2 Cuttings in Wellington Greywacke Bedrock

The geological formations, and in particular the bedrock, along Transmission Gully are generally similar to that present throughout the Wellington Region. The rocks are generally closely jointed due to the tectonic history of the region and the presence of local faults (active Ohariu and Moonshine Faults and other inactive faults such as the Horokiri Fault).

Where there are weaker rock features such as fault zones underlying more competent rock, degradation of the weaker zones may lead to initiation of rock fall from the overlying rock formation. These specific instances may lead to a greater localised rock fall hazard.

Small wedge failures are common in Wellington Greywacke given the closely jointed rock mass, and the rock mobilised could pose rock fall hazards. This is usually contained by 3 m wide benches at about 10 m height intervals that are commonly formed in the cuttings.

Given the similar rock conditions, the general rock fall hazards likely to be experienced in the Transmission Gully project are likely to be similar to those observed in the region, given that the cut slopes are of similar height or some 10 m to 20 m higher, and the cuts are formed to similar slope angles (say less than 50° reflecting recent cuttings in the region).

Cut slope angles recommended for the Transmission Gully project at this stage are similar to those formed recently for state highways in the Wellington Region, except for low height cuttings. A similar bench configuration to that commonly used in the Wellington Region (3 m wide at 10 height intervals) has also been adopted for preliminary design.

The heights of cuttings currently proposed for the Transmission Gully project are generally at the upper end of those recently constructed in the Wellington Region, ie up to 50 m high. However, some cuttings proposed are higher (~60 m) at localised locations on the *Preferred Alignment (1B)* in Sector 3. Where high cuttings are proposed a greater hazard to road users can be expected given the higher energy associated with rock fall originating at a greater height.

The cuttings required were up to 80 m in height for the *In-designation Alignment (1A)* on the eastern flanks of Te Puka and Horokiri Streams (Sectors 3 and 4), which has now been abandoned.

13.5 Integrated Rock Fall Management

A moderate level of rock fall hazard is assessed for Sectors 2 (Te Puka Terrace), Section 3 (Te Puka Stream), Sector 4 (Horokiri Stream), Sector 8 (Duck Creek) and Sector 9 (Ranui Forest), where alluvial gravel cuttings or high cuttings in bedrock are proposed based on the above considerations. Elsewhere, a low rock fall hazard is expected.

It is important that the rock fall hazard is considered and proactive measured developed to mitigate and manage the hazard. Rock fall can be managed through an integrated slope design and rock fall management approach.

13.5.1 Rock Fall Management Strategies

Rock fall hazard from road cuttings can be managed through a number of strategies:

(a) Form cut slopes at stable slope angles and minimise the potential for local failures.

(b) Manage rock fall mode by using a moderate cut slope angle in the range of 40° to 55°, where rock would slide or roll rather than bounce.

(c) Stabilise cut face and minimise rock fall by providing slope surface protection, such as using rock bolts, mesh and shotcrete. This would generally be very costly, except where used for short critical sections.

(d) Contain rock fall on the slope by using benches, if necessary supplemented by intermediate rock fall fences (access for maintenance needs to be considered).

(e) Control rock fall from bouncing and entering the road carriageway by using draped rock fall netting covering the slope. This will be effective for higher hazard sections, but costly for widespread use.

(f) Contain rock fall using rock fall barriers on slopes such as “Geobrugg” type rock fall energy absorbing fences. These are effective but involve a higher cost of installation as well as maintenance.

(g) Contain rock fall at base of slope using rock fall berm / ditch and rock fall fence at road level. This will involve additional road width and large additional excavation in the steep terrain at Transmission Gully.

Usually a combination of practices may be used, depending on the road configuration and the nature of the rock fall hazards.

All the above methods are commonly used in the Wellington Region for different circumstances, except for (f) above. Methods (d) (benches) and (g) (roadside berms / ditches) are the most widely used for highway cuttings in the region. Generally road side berms are narrow with no additional provision to enhance rock fall containment.

13.5.2 North American Rock Fall Containment Ditches

The North American approach involves use of uniform slopes with a wide and deep rock fall ditch at the toe of the slope. The rock fall containment ditch design chart by Ritchie (1963)[34] suggests that:

- a 40 m height of slope with a uniform slope angle of about 45° to 50° would require a ditch at the toe of the slope, about 5 m to 6 m wide, and 2.4 m deep.
- a 60° slope of 30 m height would require a ditch about 7.5 m wide and 2.4 m deep.

Adoption of this North American approach would involve using a uniform slope over the full height of the cutting, and provision of a 6 m plus wide and 2.5 m deep ditch at the toe of the cuttings. Such an approach would significantly widen the road platform and lead to a very large cost premium because of the need to excavate an additional 5.5 m to 7.5 m width, and a 2.5 m depth posing a safety hazard, and requiring a protection barrier from traffic. Because of the generally steep hillsides, the additional width will lead to slopes following up the hillside to the top of the ridge and giving an additional height and increased rock fall safety hazard.

It is noted that North American provisions are typically provided for much larger rock fall blocks (>1 m) than commonly experienced sized (< 200 mm) in the Wellington Region.

13.5.3 Australian Approach

The Australian approach is similar to the Wellington approach. Main Roads, Queensland's guidelines stipulate the unreinforced cuttings must provide minimum 4 m wide benches at 7 m height intervals. A similar approach with 4 m wide benches at 7 m height intervals was also provided for the Bulahdelah to Coolongolook Freeway in New South Wales (His & MacGregor)[29], managed by the Roads & Traffic Authority (RTA) in New South Wales.

The Australian approach is consistent with and is more conservative than the Wellington approach (and they generally have rocks of larger size reflecting wider spaced joints than in Wellington).

13.5.4 Proposed Approach based on Wellington Precedent

The approach proposed is based on the Wellington precedent is to use a 3 m to 4 m wide berm (including road shoulder) at the toe of cut slopes.

A typical cut slope configuration indicating the terminology used is shown on Illustration 3.22.

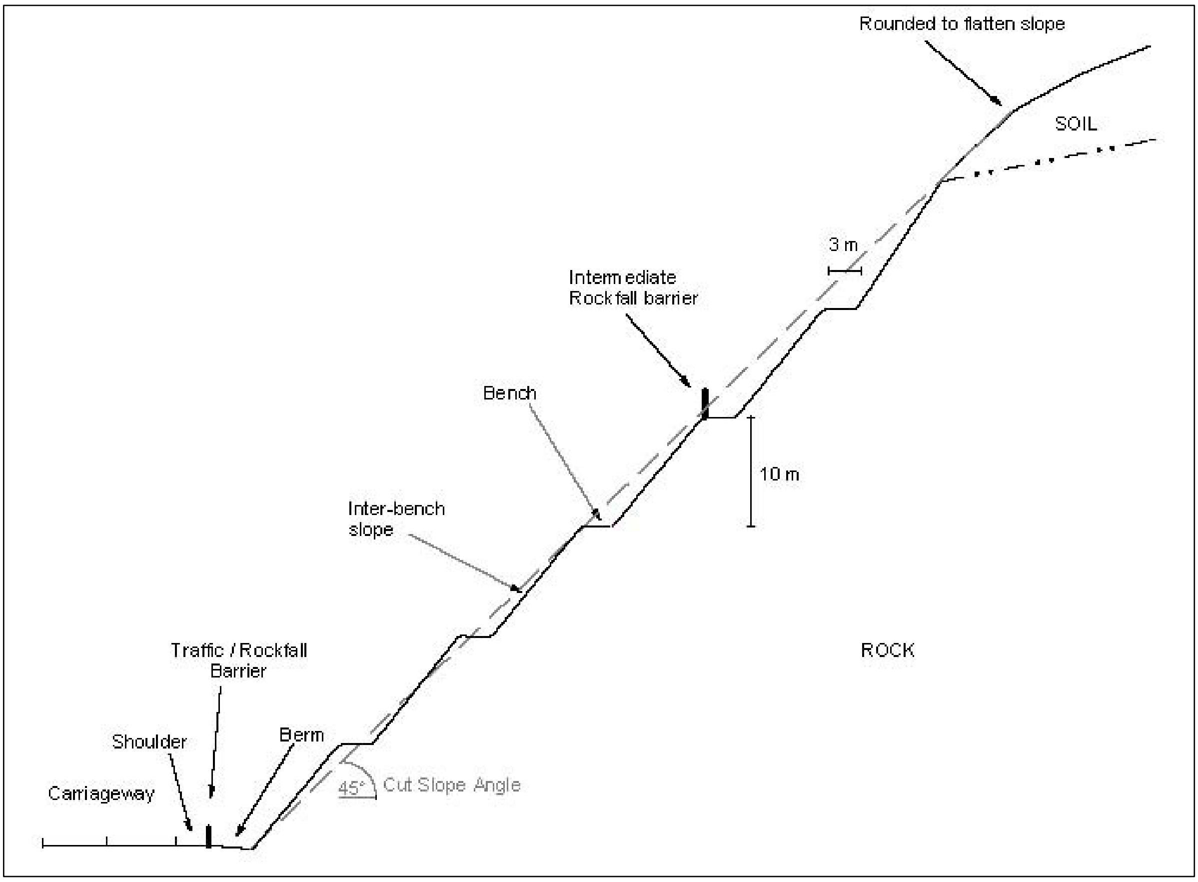

Illustration 3.22 - Typical Cut Slope Configuration

Part of the berm, outside the sealed shoulder, could be set down 0.5 m below the road surface, and / or a concrete traffic barrier provided to prevent rock fall from entering the shoulder and carriageway, to supplement the 3 m wide benches at 10 m height intervals as part of the slope configuration.

This arrangement will have the benefit that the rock fall is contained close to its point of origin by the intermediate benches, and very little rock fall will reach the bottom of the slope. The benches will also facilitate the natural generation of small vegetation.

A disadvantage of this approach is that the benches are narrow and difficult to access after a period of time, for maintenance. However, the long experience of the use of benches in the Wellington Region, shows that this has not been a major issue. For example Illustration 3.23 shows a 1950's cutting over 50 years old and over 50 m high showing bench configuration with only significant rock accumulation at one location, and which is fully contained by the benches. Note that lower benches will provide additional capacity even if a bench overflows.

Illustration 3.23 - 1950's Cut Slope in Ngauranga Gorge, Wellington

13.5.5 Discussion of Alternate Rock Fall Management Approaches

Alternative approaches to rock fall management are presented in Table 3.14.

Intermediate benches are not currently commonly used in North America as they are considered to provide more sources for rock fall, and can also lead to rocks bouncing off benches and landing into the road carriageway (Wyllie and Mah, 2004)[35].

However, there is no evidence of the small fractured greywacke blocks having bounced off the benches in the Welllington Region, and also the small fragments in the closely fractured greywacke are generally well contained on the benches. It should be noted that for the 45° to 50° cut slopes proposed for the Transmission Gully route, the rocks are more likely to slide or roll than bounce. The slab or wedge shape of the rock fragments from the closely fractured rock also promotes sliding than roll or bounce.

Table 3.14 - Comparison of Rock Fall Management Measures

No	Primary Rock Fall Management Approach	Advantages	Disadvantages	Possible Mitigation of Disadvantages
1	Intermediate and Toe Bench 3 m wide at 10 m height intervals.	▪ Proven in Wellington Region, with no known rock fall incidents in past 50 years. ▪ Little rock fall reaches the toe berm. ▪ Economical because of use of narrower berm at toe. ▪ Low maintenance.	▪ Incompatible with North American Practice which considers increased potential for rock fall generation off edge of benches and risk of rocks bouncing off bench. ▪ Difficult to access for maintenance.	▪ Slope angle adopted ensures rocks slide or roll rather than bounce. ▪ Consider small rock fall containment fences on some or all intermediate benches for the very high cuttings.
2	No Benches, 6 m wide, 2.5 m deep ditch at toe level.	▪ Rock fall reaches road level and can be easily removed. ▪ Containment in wide toe ditch.	▪ Rock fall accelerates and gains momentum as it travels down slope over up to about 75 m height. ▪ Small obstruction could lead to large bounce given momentum. ▪ Large cost penalty to form very wide and deep ditch. ▪ Need to manage safety hazard posed by ditch.	▪ Adopt barrier at road level to enhance road safety.
3	Wider Intermediate Benches, say 5 m to 8 m wide at 15 m height intervals.	▪ Access may be possible for small machines for maintenance to a wider bench if connecting roads are able to be formed in the adjacent hills.	▪ Rock fall gains momentum compared to Option (1). ▪ Cost penalty of providing wider bench. ▪ Maintenance may mobilise rock and destroy vegetation and prevent the cutting from healing, and thus prolonging rock fall hazard. ▪ Any intermediate bench fences may be damaged by maintenance equipment access.	▪ Incorporate higher or energy absorbing rock fall barriers on intermediate benches.

Adoption of an alternative of wider benches would overcome the concerns regarding access and maintenance of the benches by machinery, but this approach will still retain the potential for rock fall generation and rock bouncing off benches. Increased width benches would increase the volume of earthworks, or increased inter-bench heights to compensate for this would increase rock fall acceleration between wider spaced benches. Given that natural vegetation occurs on slopes, access to intermediate benches would be a difficult proposition, and could serve to destroy vegetation and disturb the slope leading to loosened rocks and triggering rock falls.

The Australian approach provides for slightly wider (4 m) benches to allow for erosion control, rock fall control and maintenance purposes, but at 7 m height intervals. This would provide a much flatter overall cut slope and hence a more costly design. Wellington's much wetter climate leads to relatively quicker regeneration of vegetation compared to say in New South Wales, Australia. Vegetation also serves to promote erosion and rock fall control compared to unvegetated slopes, and reduces the need for maintenance access.

13.5.6 Rock Fall Analyses of Proposed Benched Solution

Analysis of rock fall using the software RocFall (RocScience, 2007)[30] has been used to assess typical rock fall protection arrangements for a typical 50 m high Transmission Gully rock cutting. It is recognised that rock fall analysis programs typically overestimate the rock fall hazards and lead to significant costs of rock fall mitigation (Wyllie – pers comm.)[31]. Nevertheless rock fall analyses are useful to simulate the type of rock fall behaviour and consider the function of types of mitigation solutions. The analyses allows for a rough surface in greywacke rock with no vegetation or debris on the benches at the initial state.

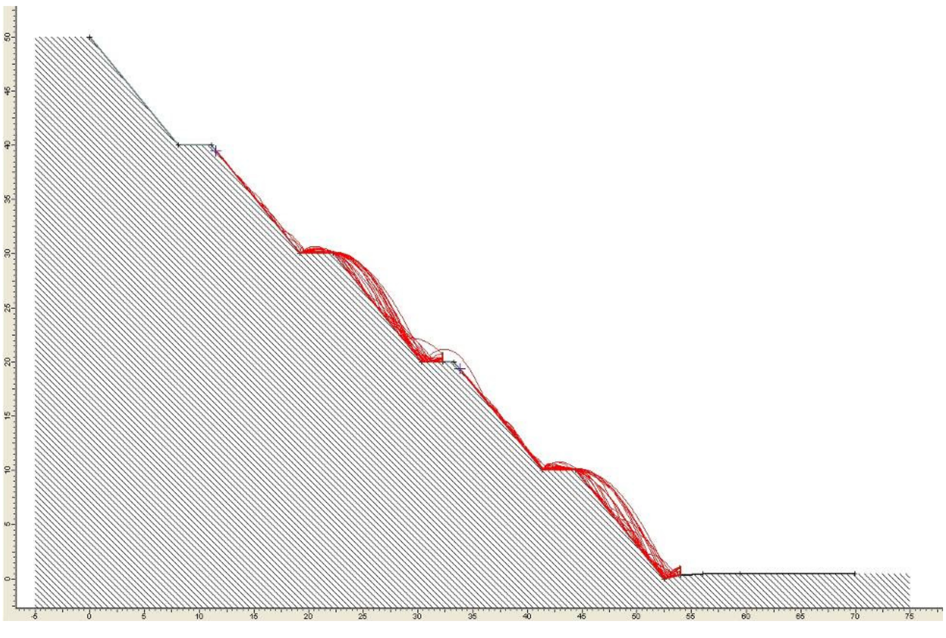

Illustration 3.24 - Cut Slope Configuration with Intermediate Benches, and Rock Fall Barriers

This shows that a 1 m high concrete barrier between the rock cut toe and the shoulder and on an intermediate bench would arrest rock falls from reaching the road. The scenario where one of the benches is substantially filled with debris has also been simulated and this shows that the configuration continues to be effective. The barriers would be effective in arresting most if not all rock falls.

One of the key uncertainties in rock fall simulation analyses is the coefficient of restitution of the slope materials which are generally assumed based on past testing and experience. Recommendations are given by RocScience (2007)[37]. The above simulations assume that rock is present on the slopes and benches. In reality the rock is somewhat weathered along the Transmission Gully route and after a period of time some debris is present on the benches and also some vegetation takes hold.

Assuming weathered rock surfaces and talus and vegetation on the benches, reduces the rock fall hazard considerably as shown in Illustration 3.25. This shows that the rock fall barrier fences are no longer required to be functional in the long term.

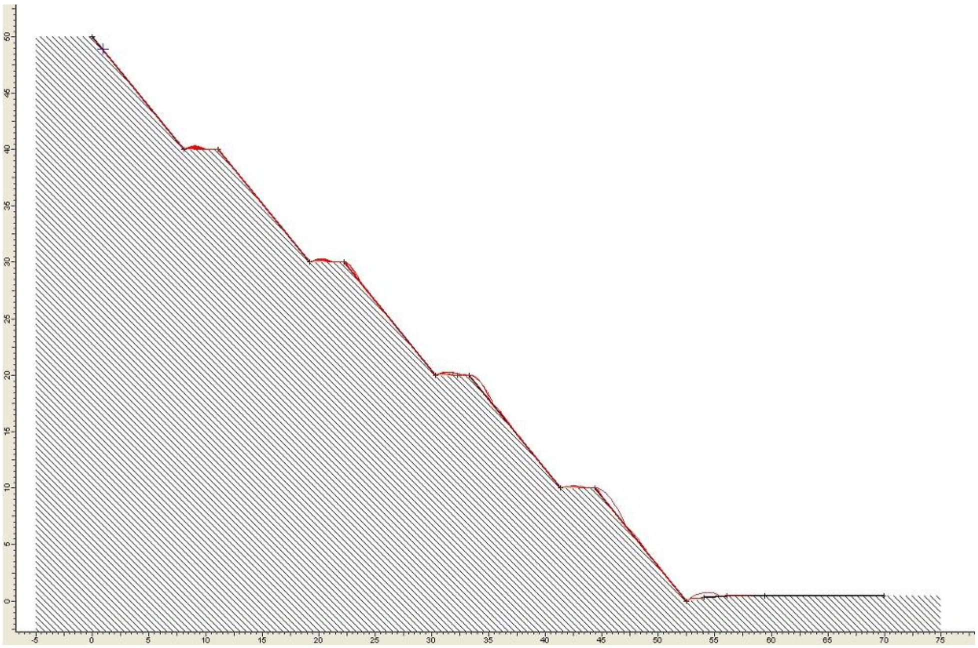

Illustration 3.25 - Rock Fall with talus and Vegetation on Benches

This represents the situation some years after construction. This shows that there is a very low hazard from rock fall as vegetation takes place and the surfaces are weathered. This analysis confirms the low rock fall hazard is only a short term hazard and an intermediate fence if required is not required to be functional in the longer term. The results from the analyses are consistent with the observed long term rock fall performance of benched cut slopes in the Wellington Region.

For comparison, a uniform cutting based of the North-American approach has also been analysed and indicates that even with the wide ditch at the base, it still leads to some residual rock fall hazard, and requires a rock fall fence at the base to make it effective. This is illustrated in Illustration 3.26.

Even with weathering in the long term, the uniform slope configuration is indicated to require a rock fall barrier at the base in the long term. Given that the ditch at the base will be maintained it has been assumed to not be vegetated in the long term.

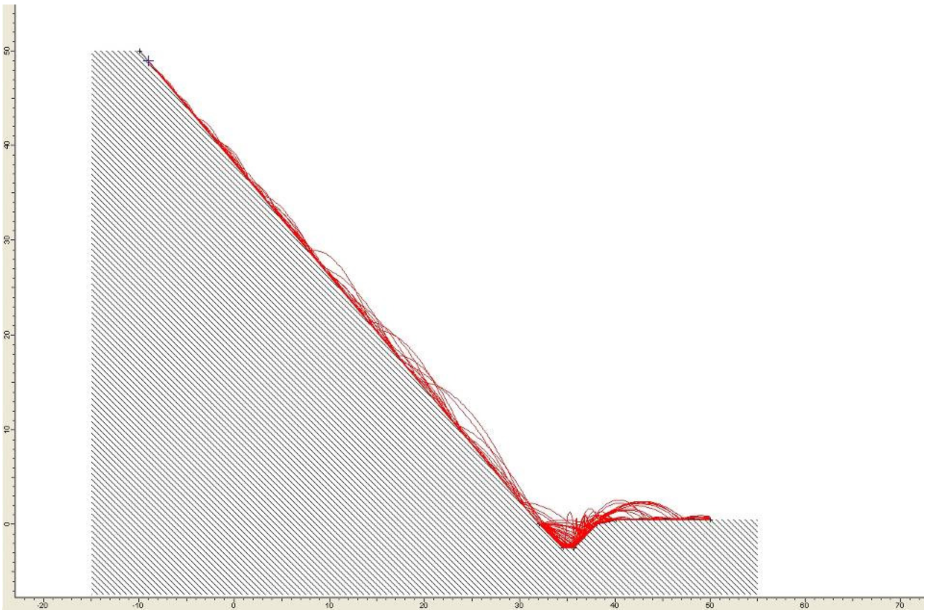

Illustration 3.26 - Rock Fall Analysis with a Uniform Cutting with No Benches

13.6 Maintenance of Rock fall Protection Measures

The experience in the Wellington Region is that the rock fall protection measures such as benches require little maintenance even with the initial post-construction high intensity rock fall period. Later the cuttings tend to become covered with vegetation providing further protection from active rock fall hazards. Maintenance of the road level berm should be provided for. This should be considered further in the design.

Illustration 3.27 shows a cutting in Wellington Greywacke rock, with intermediate benches, more than 40 years after construction. The cutting is well vegetated and there is no evidence of rock fall hazards. It shows that in practice, accessing the benches for frequent maintenance in the Wellington environment would not only be required, but would affect the beneficial established vegetation and has the potential to remove vegetation and hence initiate rock fall.

Illustration 3.27 - Rock Cutting with Benches along State Highway 2, after 50 Years

13.7 Summary

Preliminary rock fall management measures based on the experience in the region, and the expected geology and ground conditions along the Transmission Gully Route, are proposed to manage rock fall hazard:

(a) Given that the closely jointed and fault disturbed rocks in the region require flatter cut slopes than what would be possible in more competent rock, form high cuttings greater than 20 m to no steeper than 45° to 50° slope angle (this is consistent with the cut slope recommendations in Section 12) to minimise rock bounce and reduce rock fall hazard to road users;

(b) Adopt a preliminary configuration of minimum 3 m wide benches at 10 m height intervals, as shown in Illustration 3.22 (this is consistent with cut slope recommendations in Section 12);

(c) Adopt a rock fall catch berm at the base of cuttings, say 3 m to 4 m wide (including shoulder), preferably with the berm set 0.5 m below road level to facilitate rock fall collection);

(d) Adopt a rock fall fence / barrier at the base for cut slopes greater than 20 m height – this could also be a solid traffic barrier (say NJB) that can be moved to facilitate removal of collected rock fall during maintenance. This may not be required in the long term.

(e) Adopt a supplementary rock fall fence 0.5 m to 1 m high on the benches at 30 m height intervals, where a greater potential for rock fall is identified. This does need to be maintained in the long term as it becomes redundant as the slope becomes revegetated.

(f) Provide draped rock fall netting for the cuttings in alluvial gravel in Sector 2, and make provision for this at other cuttings where greater localised rock fall hazard is identified.

(g) Allow for localised rock bolting and shotcrete, where localised rock blocks pose a rock fall hazard, as discussed in Section 12.8.

14 Soil Cut Slopes

14.1 Design Philosophy

Preliminary design recommendations for cut slopes in soil have been developed to provide stable cut slopes through selection of appropriate cut slope angles and geometry, with provision for implementing localised stabilisation measures during construction, and accepting some failures in large hazard events – storm or earthquake, as discussed in the route security philosophy in Section 9.

Three important issues influence the selection of cut slopes in soil (see Illustration 3.):

- stability under normal conditions
- route security under storm events
- route security under large earthquake events

The stability of cut slopes has been considered using three approaches:

(a) Precedent cut slopes

(b) Behaviour of slopes in historical earthquakes

(c) Stability analyses

Consideration of these approaches for the type and height of cut slopes proposed for the Transmission Gully route alignments has enabled the development of cut slope recommendations for scheme assessment level design.

14.2 Proposed Cut Slopes and Geometry

Cut slopes appropriate for the different soils encountered along the Transmission Gully route have been developed based on precedent cut slopes in the region, and representative stability analyses, and are summarised in Table 3.15.

Table 3.15 - Proposed Cut Slopes in Soils

Sector	Soil Unit	Description	Cut Slope Angle, Geometry & Performance	Additional Stabilisation Measures as requried
1	Dune Sand	Fine to medium sand	22° 3 m wide benches at 10 m height intervals	▪ Sub-horizontal drainage holes ▪ Sub-soil drains at toe
2	Te Puka Terrace Gravel	Sandy Gravel with cobble, boulders	40° 3 m wide benches at 10 m height intervals	▪ Sub-horizontal drainage holes ▪ Sub-soil drains at toe ▪ Rock fall draped netting, see Section 13.

Sector	Soil Unit	Description	Cut Slope Angle, Geometry & Performance	Additional Stabilisation Measures as requried
3 & 4	Colluvium (at top of cuttings)	Silty sandy Gravel	35°	▪ Soil nailing if required to steepen slope ▪ Sub-horizontal drainage holes if appropriate
5	Western Ridge Pre-Holocene Alluvium	Sandy Silt	25° 3 m wide benches at 10 m height intervals	▪ Sub-horizontal drainage holes ▪ Sub-soil drains at toe ▪ Possibly additional measures such as counterfort drains
6	Pre-Holocene Alluvium	Clayey and sandy silt	25° to 35° 3 m wide benches at 10 m height intervals	▪ Sub-horizontal drainage holes ▪ Sub-soil drains at toe ▪ Possibly additional measures such as counterfort drains
8 & 9	Completely weathered Greywacke	Silty and Sandy Clay	35° 3 m wide benches at 10 m height intervals	▪ Sub-horizontal drainage holes

14.3 Precedent Cut Slopes

Observations and experience in the design, construction and observation of the performance of cut slopes, particularly for state highways in the region, provides knowledge of stable cut slopes and common stability issues affecting large cuttings in soil.

The most relevant recent information on the behaviour of cut slopes in the region comes from the SH 2 Kaitoke to Te Marua realignment and MacKays Crossing projects. Cuttings in these projects were constructed through Pre-Holocene deposits (including Kaitoke Gravels and Pakuratahi deposits at Kaitoke), and are relevant given the presence of similar Pre-Holocene deposits along parts of the Transmission Gully route, particularly Sectors 2, 5 and 6. The cut slopes at Kaitoke were formed at slopes as summarised in Table 3.16.

The Silverwood sub-division was constructed during 2006-2008 period immediately to the south of State Highway 58, and west of the Transmission Gully alignment. The limited information available from this sub-division is included in Table 3.16.

The precedent cut slope information supports the slope configuration developed for the soil slopes (Table 3.15) particularly in Sectors 2, 5 and 6.

Table 3.16 - Precedent Cut Slopes in Pre-Holocene Soils in Wellington Region

Project	Soil Unit	Description	Cut Slope Angle, Geometry & Performance	Probably Similar TG Unit
*SH 2 Kaitoke to Te Marua	Kaitoke Gravels (Coarse) (west of summit)	Sandy Gravel Cobble & Boulders	45° (required rock fall draped netting to manage rock fall hazards)	Te Puka Terrace (Sector 2)
	Kaitoke Gravel (Finer) (west of Black Beech Gully)	Sandy Silty Gravel	50° to 55° with 3m wide benches at 10 m heights (failures in steeper cuts with silt layers)	
	Pakuratahi Deposits (east of summit)	Sandy / Clayey Silt	25° to 35° with 3 m wide benches at 10 m heights (failures in steeper cuttings, and localised failures in cuts formed at these slopes confirm that these slopes are not conservative)	Sector 5 (west ridge) & Sector 6
Silverwood Subdivision	Pre-Holocene Gravel	Sandy Gravel, cobble & boulders	45°	Te Puka Terrace (Sector 2)
	Pre-Holocene Alluvium	Clayey Silt	26° ?	Sector 6 and 8

Note: *After Opus (2002)[32].

14.4 Behaviour in Historical Earthquakes

There is no precedent information readily available on significant slope failures during large earthquakes in cut slopes formed at shallow angles such as 25° to 35°. Steeper cuttings are likely to have failed.

14.5 Stability Analyses

Stability analyses have been carried out assuming 15 m to 30 m high slopes formed at 35° slope with 3 m wide benches at 10 m height intervals in the Pre-Holocene alluvium in Sectors 5 and 6, and completely weathered greywacke in Sector 8. Groundwater level profile was at 5 m to 10 m depth based on observations. Representative soil parameters were chosen based on the laboratory test results. The analyses indicated that the commonly accepted factor of safety of 1.5 would be achieved under normal static conditions.

Illustration 3.28 shows results from a typical slope stability analysis in Pre-Holocene alluvium.

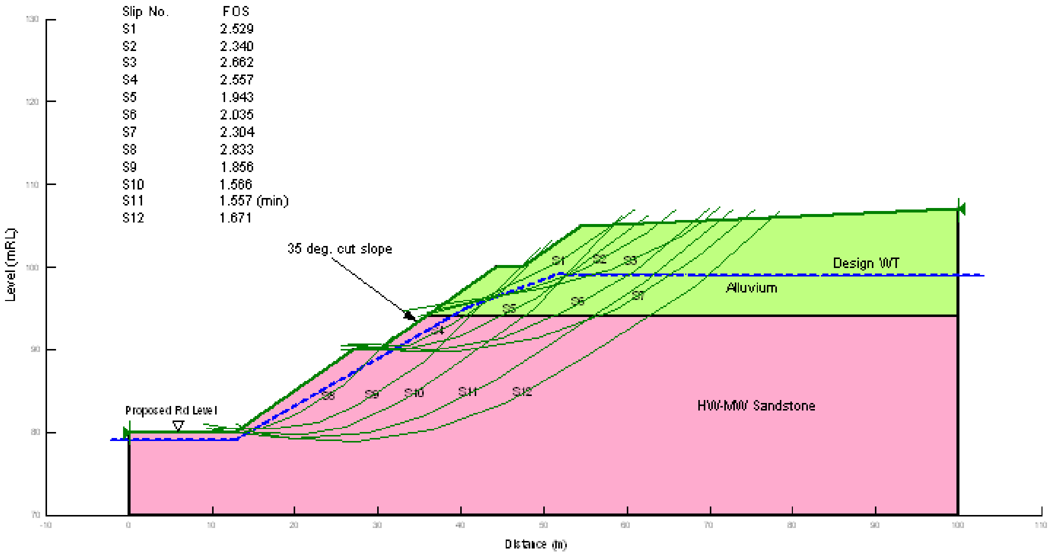

Illustration 3.28 - Typical Slope Stability Analysis of Cut Slopes in Soil

Should the groundwater level rise to 3 m depth reflecting wet or storm conditions, the factor of safety is indicated to drop to slightly less than 1.5 for the most critical surfaces, which is acceptable based on the design philosophy adopted, see Section 9.4. Existing landslides in Sector 6 confirm the potential for failure of slopes, which are likely to have been triggered by high groundwater conditions. Incorporation of sub-horizontal drainage holes and sub-soil drains at the toe of slopes, and in some cases at intermediate levels, is therefore considered to be important to maintain stability where high groundwater levels are present. Counterfort trench drains may also be appropriate in some cases with very high groundwater levels and weak soils. It may be necessary to form sections of the cuttings to a shallower 25° slope where weaker soils are encountered as can be expected from the variability of some of the alluvium units, or where higher cut slopes greater than 30 m height are proposed.

Stability analyses have been undertaken using a peudo-static approach to derive the critical horizontal acceleration necessary to reduce the factor of safety of the cut slope to unity (1). Then slope displacements were assessed using the critical accelerations and design accelerations, with the aid of charts presented by Ambraseys and Srbulov (1995)[31]. These analyses indicated slope displacements of less than 100 mm in a 1000 year return period earthquake event (0.52g peak ground acceleration), and up to 250 mm in a larger event giving a peak ground acceleration of 0.75g, with an 84 percentile level of confidence. Such limited displacements in large earthquakes are considered to be acceptable.

When adopting a displacement based design approach for cut slopes, lower soil shear strengths are adopted to reflect large strain soil strengths (rather than peak strengths). However, there is a risk in some cases that the displacement could lead to larger reductions in strength, leading to slope failure. This scenario is acknowledged in the route security philosophy, as discussed in Section 9.3.3.

14.6 Areas of Existing Landslides

A number of existing slope failure features exist in Sectors 6 and 8. Based on the limited site investigations undertaken these have been found to be shallow. Provision should be made to excavate these features where they are important to the alignment, and replaced with engineered drainage and embankment fill.

14.7 Landscaping, Revegetation and Erosion Control

Cut slopes are particularly prone to failure at the edges of cuttings, where the soils are more weathered and the geometry leads to sharp corners. It is recommended that the edges of cut slopes are landscaped, rounded and blended into the adjacent natural slopes to avoid this.

Given that some soils are erodible, it is important that the slopes are revegetated as soon as possible during construction after they are formed, and the revegetation maintained during the early years after construction. The dune sands at the north end between MacKays Crossing and the intersection of TG with SH 1 near Perkin's property (Sector 1) and soils near the James Cook and Waitangirua Link roads are particularly vulnerable to erosion and should be top soiled, and/or protected with an erosion protection matting prior to revegetation.

Surface cut off drains should be formed where there is a catchment above the cut slopes, to intercept and divert surface run-off away from the cut slopes. Controlled collection and discharge of all surface and sub-surface drainage off cut slopes should be provided.

Stepped or cascade drainages (with lining and erosion protection systems where the soils are erodible) should be allowed for discharging the water off tributary gullies and streams, particularly where these are cut off by the road cuttings. The discharge from these should be channelled and discharged through appropriate culverts under the highway.

14.8 Cut Slope Stabilisation Measures

Cut slope stabilisation measures are essential to ensure stability, particularly in the Pre-Holocene fine alluvium deposits encountered in Sectors 5 and 6. The existing shallow instabilities and landslides and the high groundwater conditions in Sectors 6 and 8 indicate the potential for slope instability in cuttings formed in the soils in these areas. Therefore, cut slope stabilisation measures are likely to be required to enhance stability. This is confirmed from the stability analyses undertaken for typical cuttings in these soils, recognising the likelihood of low strength soil layers being present.

Cut slope stabilisation measures would include:

(a) Sub-horizontal drainage holes, up to 20 m long and at 20 m height intervals at about 5 m spacing, to draw down the groundwater levels to improve stability, and minimise the risk in storm events by maintaining low groundwater levels in the slope.

(b) Sub-soil drains to 1.5 m depth at the toe of the cut slopes to draw down the groundwater level near the toe and also keep water levels below pavement subgrade.

(c) Counterfort trench drains down the slope, where necessary to further remove shallow groundwater near the slope surface and strengthen the slope.

(d) Local reprofiling of the slope to a flatter slope.

(e) Soil nailing at critical locations, particularly where thick colluvium is encountered at the top of cut slopes and cannot be practically rounded off to a flatter slope because of the steep terrain.

Some of the cut slope stabilisation measures can be designed and specified (eg sub-horizontal drainage holes), and provision should be made for additional stabilisation measures to be implemented during construction as the actual conditions are exposed.

14.9 Summary

Cut slopes are required in a variety of soils along the Transmission Gully Route. Appropriate cut slopes have been considered based on the performance of precedent cut slopes in similar soils in the region, and stability analyses based on soil properties from laboratory tests undertaken to characterise representative strengths of some fine grained materials encountered during the site investigations.

The following measures are proposed:

(a) Cut slopes proposed are:

▪ Dune sand (Paekakariki)	Sector 1	20° to 25°
▪ Coarse Alluvium (Te Puka Terrace)	Sector 2	40°
▪ Colluvium, fine Pre-Holocene Alluvium	Sectors 5&6	25° to 35°
▪ Completely weathered greywacke	Sectors 8&9	35°

(b) Stabilisation measures comprising sub-horizontal drainage holes and sub-soil drains will be incorporated to lower and maintain low groundwater levels in the slopes with high groundwater pressures, and provision should be made for additional stabilisation measures as necessary during design and construction.

(c) Erosion protection measures would include surface drains to redirect surface flows from catchments away from cut slopes, and measures such as geotextile erosion matting to protect erodible materials (eg dune sands, loess, fine alluvium).

(d) Rock fall protection draped netting may be appropriate for the cut slopes in coarse alluvium with cobble and boulders, such as at the Te Puka terrace immediately south of SH 1 at Paekakariki.

(e) Investigations are recommended for all significant cut slopes to determine soil profiles, properties and groundwater conditions for the design of individual slopes.

15 Embankments

15.1 Design Philosophy

Preliminary design recommendations for embankments have been developed to provide generally stable embankments through selection of appropriate slope angles and geometry, and accepting some limited displacement in large hazard events – storm or earthquake, as discussed in the route security philosophy in Section 9.1, and design philosophy in Section 9.4.

Three important issues influence the selection of embankment slopes:

- stability under normal conditions
- route security under storm events
- route security under large earthquake events

The stability of embankment slopes has been considered using three approaches:

(a) Precedent embankment slopes

(b) Behaviour of slopes in historical earthquakes

(c) stability analyses

Consideration of these approaches for the type and height of embankment slopes proposed for the alignments in Transmission Gully has enabled the development of embankment recommendations for scheme assessment level design.

15.2 Fill Materials

The embankment fill materials that would be used for the construction of embankments along the Transmission Gully project will vary, depending on the materials available from cuttings in each area.

Rock derived fill materials would be the main fill in Sectors 2, 3, 8 and 9, whereas soils from Holocene or Pre-Holocene soils will be the main fill from Sectors 1, 2, 5 and 6.

Fill materials that will be available from cuttings, and the proposed embankment configuration developed for the Transmission Gully project are summarised in Table 3.17. The proposed slopes take into consideration performance under static conditions as well as storm and earthquake conditions.

Table 3.17 - Fill Materials and Embankment Configuration

Fill Source Sector	Embankment Fill Materials	Description	Embankment Slope Angle, Geometry & Performance	Additional Stabilisation Measures
1	Dune Sand	Fine to medium sand	22° No berms given low height embankments	▪ Erosion protection matting, top soiling and revegetation to minimise erosion.
2	Pre-Holocene Gravel	Sandy Gravel with cobble, boulders	25° 3 m wide berms at 10 m height intervals	▪ Rock fall netting unlikely to be required given that there is usually no access to toe of embankment.
3 & 4	Wellington Greywacke derived fill	Silty sandy Gravel	25° 3 m wide berms at 10 m height intervals 45° Reinforced soil embankment	▪ No additional measures envisaged. ▪ Erosion protection matting on 45° reinforced soil embankments.
3 & 4	Fault disturbed Wellington Greywacke derived fill	Gravelly sandy Silt and Clay	25° 3 m wide berms at 10 m height intervals	▪ Possibly intermediate coarse gravel / rock fill layers to ensure drainage during construction
5 & 6	Pre-Holocene Alluvium	Clayey and sandy silt	25° 3 m wide berms at 10 m height intervals	▪ Intermediate coarse gravel / rock fill layers to ensure drainage during construction
8 & 9	Wellington Greywacke derived fill including completely weathered Greywacke	Silty sandy Gravel and Silty and Sandy Clay	25° 3 m wide berms at 10 m height intervals 45° Reinforced soil embankment	▪ Possibly intermediate coarse gravel / rock fill layers to ensure drainage during construction, if completed weathered rock derived fill.

15.3 Precedent Embankment Slopes

Observations and experience in the design, construction and observation of the performance of embankments, particularly for state highways in the region, provides knowledge of stable embankment configurations.

The most relevant recent information on the behaviour of embankment slopes in the region comes from the SH 1 Newlands Interchange, SH 2 Kaitoke to Te Marua realignment and SH 1 MacKays Crossing projects. The Newlands and Kaitoke projects involved the construction of embankments up to 30 m high, with 26° slopes with 3 m wide berms at 10 m to 15 m height intervals in similar materials. The embankment slopes have performed well as expected. Similar embankment configurations have been adopted for the state highway projects in the region, where the embankment is on stable ground.

The embankment configuration adopted for Transmission Gully project is supported by the precedent information on the construction and performance of embankments in the region.

15.4 Behaviour in Historical Earthquakes

There is no precedent information on the earthquake performance of embankment slopes in the Wellington Region. There is also no readily available New Zealand evidence of failure of well engineered road embankments in earthquakes, except where they are constructed on liquefiable soils. Road embankments on liquefiable soils failed extensively through lateral spreading in the 1931 Hawkes Bay Earthquake, see Illustration 3.29.

Illustration 3.29 - Failure of Road Embankments due to Liquefaction in 1931 Hawkes Bay Earthquake

There are only localised instances of known liquefaction risk along the Transmission Gully route at the Paekakariki intersection with SH 1 and possibly at the Kenepuru Link as discussed in Section 7.7.

Bray (2000)[33] indicates that fill slopes do experience settlement and associated cracking during large earthquakes based on experience in the United States. They also suggest that the settlement magnitude was dependent on the materials and standard of compaction. The fills studied by Bray were residential fills constructed many years ago, and hence the standard of compaction was variable.

The Transmission Gully project will be constructed to modern day earthworks standards to NZTA earthworks specifications, which will provide well compacted embankments. Any settlements are therefore expected to be small (say less than 100 mm) and are considered to be acceptable given the route security philosophy proposed for the project.

15.5 Stability Analyses

Stability analyses have been carried out assuming 20 m to 40 m high embankment slopes formed at 25° slope with 3 m wide berms at 10 m height intervals using the fill materials derived from the Pre-Holocene alluvium in Sectors 5 and 6 and completely weathered greywacke, and soil parameters assumed based on the laboratory test results. The analyses indicated that the commonly accepted factor of safety of 1.5 would be achieved under normal static conditions. Illustration 3.30 shows the output from a typical slope stability analysis undertaken for an embankment fill.

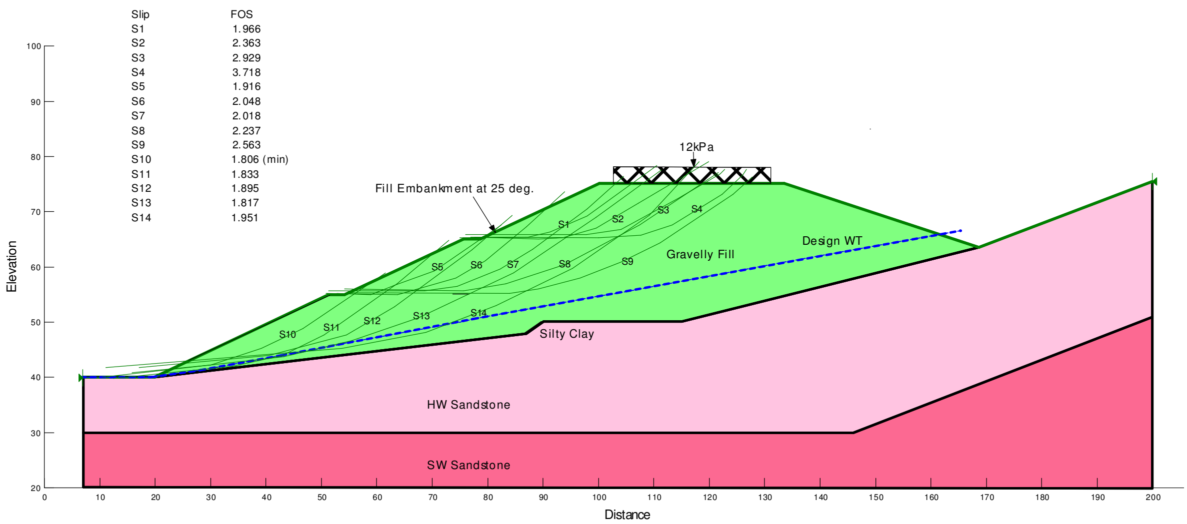

Illustration 3.30 - Typical Slope Stability Analysis of Soil Embankment

Incorporation of drainage blankets at foundation level and sub-soil drains at the interfaces between the fill and the hillsides is considered important to ensure that the embankment is drained and maintains stability. Any low strength layers at the base of the fill embankment should be undercut and removed and the embankment founded on competent ground.

Stability analyses have been undertaken using a peudo-static approach to derive the critical horizontal acceleration necessary to reduce the factor of safety of the embankment slope to unity (1). Then slope displacements were assessed using the critical accelerations and design accelerations, with the aid of charts presented by Ambraseys and Srbulov (1995)[31]. These analyses indicated slope displacements of less than 100 mm in a 1000 year return period earthquake event (0.52g peak ground acceleration), and up to 300 mm in a larger event giving a peak ground acceleration of 0.75g, with an 84 percentile level of confidence. Where sandy silt materials are used as fill the displacement could be larger in a 0.75g ground shaking representing a 2,500 year return period event. Such limited displacements in large earthquakes are considered to be acceptable.

15.6 Settlement

Embankments founded on competent ground, or on foundation levels undercut to competent ground are proposed to ensure that settlements are small.

The soft ground (peat) in the low lying areas at Sector 1 is proposed to be undercut to remove peat to competent underlying dense sandy gravel and sand. If there are some residual peat layers or compressible clay / silt layers at depth, preloading is proposed to minimise post-construction settlements to less than 50 mm as discussed in Section 15.7. Similarly soft ground may be encountered along Kenepuru Link adjacent to the Porirua stream and possibly locally at the Waitangirua and James Cook Drive links that have not been fully investigated. Such soft ground can be dealt with in a similar manner.

15.7 Ground Improvement

Ground improvement for embankments will generally comprise:

- Undercut of soft, compressible, organic materials at foundation level.
- Construction of a minimum 500 mm thick drainage blanket wrapped in geotextile with outlet sub-soil drains.
- Benching into the natural ground interface to remove loose, soft or organic materials, and keying in the new embankment fill into competent natural ground.
- Construction of sub-soil drains along the interface between the natural ground and the fill embankment.
- Construction of drainage layers within the embankment fill, where the fill is fine grained soils, and particularly where the fill may be wet of optimum moisture content.

Additional ground improvement measures may be required where there are significant thicknesses of soft compressible layers below foundation level, which cannot be practically or economically excavated and removed. These areas should be preloaded, with wick drains to accelerate settlement if necessary, to minimise post-construction settlements.

Also ground improvement may be required where there is a potential for liquefaction which may affect the route security or affect the performance of adjacent structures such as bridges and retaining walls. This would then require ground improvement using measures such as stone columns to improvement strength / density of the ground and provide drainage in earthquake events. The need for such ground improvement is localised, for example at the intersection between Transmission Gully and the existing SH1 at the northern end of the Te Puka Stream valley.

15.8 Fill Slope Configuration

Appropriate fill slopes and fill slope configurations are presented in Table 3.17 for the various fill materials that are likely to be encountered along the route.

The fills should have berms at 10 m height intervals. The presence of the berms will serve to reduce the velocity of water flow and minimise erosion and gully erosion on the fill surface. The 3 m wide berms should slope outwards to shed rain water rather than concentrate it and increase the risk of gully erosion or infiltration of water into the slope due to blockage and stagnation.

Any surface water from the road surface should be collected and either piped or run down lined cascade drains to minimise the potential for embankment erosion.

15.9 Reinforced Soil Embankments

15.9.1 Concept and Application

Reinforced soil embankments (RSE) are proposed to enable the construction of embankments with steeper slopes, with the potential to reduce the footprint of the embankments and minimise the impact on existing features such as streams. These will also provide a robust reinforced embankment at or close to active faults. In particular, these reinforced soil embankments have been adopted in Sectors 3 and 4 to prevent or minimise the encroachment of the fills into the Te Puka and Horokiri Streams.

Reinforced soil embankments with a face slope not exceeding 45° have been chosen given their ability to be constructed like normal embankments, but incorporating geogrid reinforcement layers, and without the need for temporary or permanent facing. The reinforced soil block is also expected to perform as a “rigid” block and accommodate displacement without significant damage in strong earthquake shaking. In addition, given that the RSE may be located close to and in some sections straddling the Ohariu and the splinter fault south of the saddle, this rigid block is likely to displace as a block or the fault may rupture at the ground surface around the block.

Careful design and construction quality control will be required given that reinforced soil embankments to such heights (30 m to 40 m) have not been commonly constructed in New Zealand.

15.9.2 Reinforcement

It is proposed that geogrid made of high density polyethylene (HDPE) (such as Tensar geogrids) be used as the reinforcement, which would provide a relatively low cost and durable reinforcement for the embankment fill. These are also more robust against damage than polyethylene (PE) geogrids, given the coarse angular fill derived from rock cuttings. Selected fill (see Section 15.9.3) will be required for the RSE and given that this will be sourced from excavated rock, some allowance for damage will be required.

The geogrids are predicted to provide an adequate design life exceeding 100 years (FHWA, 2001)[34], although these geogrids have not been in use for such a long period so far, as they are a relatively recent development.

The reinforced soil embankments will require adequate reinforcement length to ensure stability. This varies depending on the ground conditions, fill materials and the configuration as well as the earthquake performance expectations. The proposed earthquake design philosophy is to allow controlled displacement of these reinforced embankments without failure through the reinforced soil block. To achieve an adequate reinforcement length, some excavation into the natural hillside is likely to be required.

15.9.3 Fill Materials

Reinforced soil embankments can be constructed using a wide range of fill materials. The height of the reinforced fill embankments may be up to 40 m high along the upper reaches of the Te Puka and Horokiri Stream valleys. These high RSE are required to perform in large earthquakes given the high seismicity of the region. Therefore, select granular fill materials sourced from cuttings in competent slightly to highly weathered greywacke rock materials (ie silty sandy gravel) is proposed to provide the necessary fill strength properties, drainage characteristics and ensure that the fill strength does not degrade with displacement for the displacement based design approach.

In addition, the select fill materials would need to be screened to remove the large size particles that may cause undue damage to the geogrids and also remove the fine materials that would reduce permeability, impede free drainage and reduce the strength performance of the fill.

15.9.4 Stability Analyses

Stability analyses have been undertaken using reinforced slope stability assessment software RESSA[35] to assess the stability and preliminary configuration of reinforced soil embankments. A typical output from the analyses showing the reinforced soil embankment configuration is presented in Illustration 3.31.

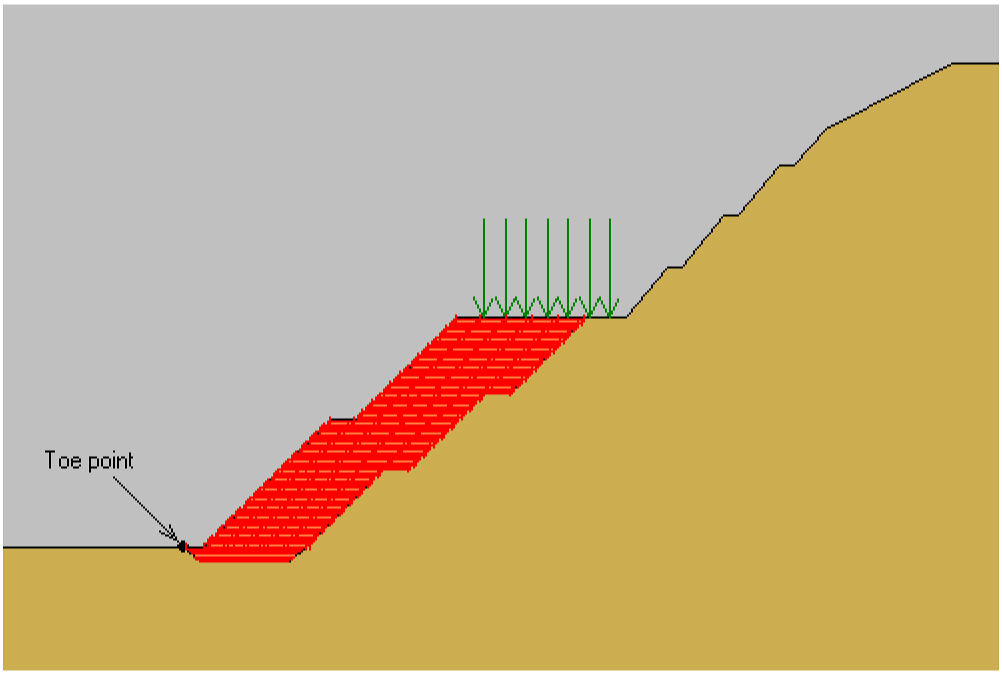

Illustration 3.31 - Typical Analysis of a Reinforced Soil Embankment

15.10 Landscaping and Revegetation

It is important that the RSE slopes are vegetated during or as soon as possible after construction, with the aid of erosion protection matting, and the vegetation maintained during the life of the RSE. Vegetation is usually grass and low vegetation to provide protection to the slope surface but not large trees that could destabilise the slope.

15.11 Summary

Embankments are required to be constructed using a variety of soils from cuttings along the Transmission Gully Route. Appropriate fill slopes have been considered based on the performance of precedent slopes in similar soils in the region, and stability analyses.

Reinforced soil embankments have been proposed to reduce the foot print of the embankments in critical areas and avoid or minimise encroachment into features such as streams, particularly in Sectors 3 and 4.

(a) The following embankment slopes have been developed for preliminary design:

▪ Dune sand (Paekakariki)	Sector 1	20° to 25°
▪ Coarse Alluvium (Te Puka Terrace)	Sector 2	25°
▪ Sandy and silty gravel (greywacke rock fill)	Sectors 3&4	25°
▪ Colluvium, fine Pre-Holocene Alluvium	Sectors 5&6	25°
▪ Completely weathered greywacke	Sectors 8&9	25°

(b) The embankments would have 3 m outward draining berms at 10 m height intervals for slopes exceeding 15 m height.

(c) The following ground improvement and stabilisation measures would be considered and incorporated into the embankment design and construction:

- Drainage blanket of free draining gravel wrapped in geotextile at foundation level.
- Benching into the natural slopes to key the new fill into competent natural ground.
- Subsoil drainage along the benches to pick up seepages and keep the embankment free of groundwater pressures.
- Excavation and removal of any soft compressible, weak or otherwise unsuitable materials encountered in the foundations of embankments.
- Preloading, with wick drains as necessary, where thick or deep compressible soils are present.

- Construction of drainage layers within the embankment fill, where the fill in fine grained, and particularly where the fill may be wet of optimum moisture content.
- Any surface water from the road surface should be collected and either piped or run down lined cascade drains to minimise the potential for erosion.
- Erosion protection matting for particularly erodible soils such as dune sands
- Revegetation of slopes soon after construction.

(d) Reinforced soil embankments should be designed and constructed with the following considerations:

- Geogrids made of HDPE (such as Tensar geogrids)
- Geogrid lengths shall be accommodated by excavation into the hillside as necessary to develop a suitable RSE block.
- Select granular fill is proposed given that the reinforced soil embankments are high and need to be designed to resist large earthquake shaking with some controlled displacement of the reinforced block.
- The select fill materials would need to be screened from the rock excavation derived fill to obtain materials without large size rock and fines.

16 Earthworks and Construction Materials

16.1 Unsuitable Foundation Materials

Excavation and replacement of unsuitable materials is proposed below all structure foundations, retaining walls, reinforced soil embankments and embankments. In some instances the materials may be suitable for reuse when recompacted.

In some exceptional circumstances, it may be impractical to remove all the materials present, and it may be necessary to undertake ground improvement such as by preloading, surcharge and wick drains.

Unsuitable materials excavated and removed would need to be disposed of at disposal areas, as discussed in Section 17.

16.2 Excavation Characteristics

The general excavation characteristics of the cut materials have been considered for the route, and are summarised in Table 3.18.

Table 3.18 - Excavation Characteristics

Sector	Materials to be Excavated	Excavation Characteristics	Additional Comments
1	Dune Sand	Easily excavated using earthmoving machinery without need for any ripping.	▪ Excavated dune sand is likely to require some wetting to facilitate placement.
2	Pre-Holocene Gravel	The gravels can be very dense with cobble and boulders. While can be excavated with dozer blades, may benefit from light ripping to facilitate excavation.	▪ Excavated sandy gravels may require some wetting to facilitate placement and compaction.
3 & 4	Wellington Greywacke	Rocks are closely jointed and given the proximity of the fault generally have shear and crush zones. The majority of the materials are likely to be able to be excavated by ripping. Very hard ripping is likely to be required for some of the rock. It is possible that blasting may be required to help facilitate excavation, particularly for the In-designation alignment (1A) on the eastern flank and south of the Wainui Saddle.	▪ Seismic refraction survey indicated compression wave velocities of 2,500 m/s to 3,000 m/s below about 10 m. ▪ The high velocities are considered to be related to high groundwater conditions in the area. ▪ Rippability assessed using approach of Weaver, and construction experience in the region. ▪ For example cuttings in slightly to highly weathered rock at Newlands Interchange was entirely by ripping. ▪ Blasting appropriate given rural environment, but would require safety control and measures to minimise damage to the remaining rock.

Sector	Materials to be Excavated	Excavation Characteristics	Additional Comments
5 & 6	Pre-Holocene soil deposits	The largely fine grained soil with some gravelly sandy silts can be relatively easily excavated by earthmoving machinery. Some light ripping may facilitate excavation of dense gravelly materials at depth and localised areas of rock.	▪ The fine grained soils are likely to require some drying.
7	Alluvium and Estuarine Deposits	No excavation proposed in the sector, other than undercut of the soft materials below embankments and retaining walls.	▪ Some excavation in pre-Holocene deposits at the ends similar to Sectors 5 & 6.
8	Wellington Greywacke rock, with completely to highly weathered near surface	The closely jointed rocks are likely to be able to be excavated by ripping. Some localised heavy ripping or blasting may be required.	▪ Rippability assessment using the approach of Weaver, and experience with construction projects in the region.
9	Wellington Greywacke rock, with completely to highly weathered near surface	Completely and highly weathered greywacke rock predominates in the upper excavation, particularly in the area north of Cannon's Creek. The rocks north of Cannon's Creek are likely to be able to be excavated with ripping, and perhaps some heavy ripping and blasting near the base of the box cutting. The area south of Cannon's Creek has more competent bedrock materials, but is still closely jointed. These rocks may require very hard ripping and some blasting, particularly between Cannon's Creek and the Gun Club.	▪ Rippability assessment using the approach of Weaver, and experience with construction projects in the region. ▪ Blasting may be an issue given adjacent residential areas.

There are a number of approaches to consider the rippability of rock for excavation purposes. The most common approach is an empirical approach based on the rating chart developed by Weaver (1975)[36]. Weaver's approach uses the following parameters to assess the rippability of rock:

- seismic velocity
- rock hardness (actually UCS strength)
- rock weathering
- defect spacing
- defect continuity
- defect separation / gouge

- strike and dip orientation

Weaver's approach has been used to predict the general rippability of the rock in different sectors. A more detailed assessment of rippability should be undertaken during detailed design, when more information is available on the variability of rock conditions along the cuttings proposed and with depth, and the alignment and extent of cuttings are better defined.

16.3 Suitability of Cut Materials

The soils and rocks excavated on site are likely to be able to be used for the fills. Some of the materials from the soils in Sectors 5 and 6 may not be suitable for use as fill, primarily because the natural moisture content is higher than the optimum moisture content (see Section 16.4), and hence drying may be time consuming. Also some of the materials from the rock cuttings near the fault zone at the saddle area and adjoining areas may be too cohesive or wet. These wet materials may be more readily disposed in favour of the more suitable materials, and will possibly be a more cost-effective approach given the general surplus of materials from the cuttings.

Select fill materials will be required for the reinforced soil embankments and the reinforced soil walls proposed in Sectors 3 and 4 and at the bridge and interchange sites elsewhere. These can be derived from the rock cuttings but may need to be hauled some distance as the materials available locally may not be suitable. For example, the cut materials in the northern part of Sector 4 may be of poorer quality because of disturbance by the Active Splinter of the Ohariu Fault, and may not be suitable for the select reinforced block fill, and select fill may need to be carted from north of the Wainui Saddle.

16.4 Compaction Characteristics

The compaction properties of the soils are discussed in Section 6.2. The soils in Sector 6 are generally 5% to 15% wet of optimum, and would require significant drying to use them as embankment fill. It is important that this section of the project would need to be carried out during favourable seasons where drying would be possible. Similar materials appear to have been encountered in the Silverwood subdivision, south-west of the proposed interchange with SH 58. It is understood that the materials were dried by discing at the cut and the fill sites. Some of the more wet and clayey soils were cut to waste. A similar approach would be suitable for the Transmission Gully project given the general surplus of cut materials indicated by preliminary geometric design.

Other soils along the route are generally 2% to 4% wet of optimum, and are more readily dried if the earthworks are undertaken during favourable earthworks seasons.

The fill from the excavated rock and the coarse gravels from the Te Puka terrace just south of SH 1 at Paekakariki are likely to require wetting to facilitate compaction, and would require sources of water to facilitate this. These materials are generally able to be placed and compacted across a much longer construction season. An exception to this may be the near fault materials with seepages along the splinter fault south of the saddle, which may be wet and may cause some difficulty with earthworks. This area may require prior drainage by the drilling of drainage holes to drain the area.

16.5 On-site Material Sources

16.5.1 Bulk Fill

The on site excavations for cuttings will provide suitable fill materials for the bulk fill and subgrade as discussed in Section 15.2.

16.5.2 Select Fill for Reinforced Soil Embankments and Walls

The materials from the rock cuttings are likely to provide suitable materials for the reinforced soil embankment select fills and also the sub-base materials. The select materials would need to be screened to provide materials of appropriate quality and grading as discussed in Section 15.9.3. Similarly select fill materials will be required for the reinforced soil walls.

16.5.3 Basecourse and Concrete Aggregate Sources

Much of the rock materials along the TG route are weathered, intensely fractured and sheared or weak, as a result of the faults along the route (Ohariu, Active Splinter of Ohariu, Horokiri and Moonshine Faults) and the effects of deep weathering.

There are two areas identified along the route that has better quality rock which is slightly weathered to unweathered, strong and still with closely spaced joints. These are the materials that may be suitable as base course aggregate and possibly concrete aggregate. These areas are:

(a) The eastern flank of Horokiri Valley, south of Wainui Saddle, between about Stations 6,000 m and 7,500 m along the TG Designation (1996), see GSI values (Opus, 2008c) and borehole logs for BH 24 and BH 29 (Opus, 2008b)[3]. This is along the In-designation Alignment (1B).

(b) In the vicinity of Cannons Creek and particularly south of Cannon's Creek, see GSI values (Opus, 2008c), and borehole logs for BH 23, BH 40 and BH 45 (Opus, 2008b)[3]. This is along the preferred alignment.

Some basecourse quality materials may be able to be sourced locally from other cuttings, particularly near the base of cuttings below the weathering zone.

There are two options to exploit these materials to win basecourse and possibly concrete aggregate materials. These are:

(a) Widen the road corridor or use flatter the rock cut slopes to win more materials where good quality rock is available. This has the advantage that the work will at the same time provide a reduced slope stability risk to the highway through improved stability. Such an approach would be suitable for the area south of Cannons Creek.

(b) Open a borrow area to source the aggregate materials, and landscape or use as a disposal site for unsuitable or surplus bulk fill materials. This approach may be considered for the eastern flanks of Horokiri Stream south of the Wainui Saddle.

16.6 Developing Aggregate Sources

The potential sources of aggregate are referred to based on the rock quality from the site investigation results and mapping. If these are to be pursued further, it would be necessary to undertake further investigations to:

(a) Assess the quantum of suitable rock that may be available through additional site investigations that may comprise further cored boreholes, seismic refraction surveys.

(b) Assess the material quality through engineering geological assessment of recovered cored samples and laboratory aggregate quality tests.

(c) Assess the production techniques necessary to win the materials and process the materials for use.

The above work will require the input from geotechnical engineers, geologists as well as people with aggregate production experience.

Further testing investigations are essential to confirm the viability of winning aggregates on site. This will also require engineering, production, environmental and planning considerations.

16.7 Summary

Unsuitable materials shall be excavated at all structure foundations, retaining walls, reinforced soil embankments and embankments and disposed of at disposal sites.

The rock materials can generally be excavated by normal earth moving machinery, with ripping to heavy ripping as required particularly in the rock and dense gravel. Some blasting may be appropriate to facilitate excavation.

Cut materials are likely to be generally suitable as fill materials for the fill embankments, and with some selection and processing for the reinforced soil embankments and walls. The materials are likely to be able to be placed and compacted, with some wetting required for the dune sand and rock fill / sandy gravel materials, and some drying for the pre-Holocene fine alluvium. Some of the wet fine grained alluvium and residual weathered rock materials may be too wet to be economically dried and may be cut to waste, which is appropriate give the surplus fill materials expected to be available.

There may be suitable higher quality rock present at locations along the route, including on the eastern flank south of the Wainui Saddle and south of Cannons Creek. The suitability and viability of these sources for base course and concrete aggregate should be further investigated.

The following issues would be taken into consideration:

(a) Allowance should be made for ripping and possibly blasting in the earthworks for the project. The consents for blasting, and appropriate methods and patterns of blasting to facilitate excavation without causing damage to the remaining rock cuttings should be investigated, with trial excavations.

(b) The compaction properties of materials at depth were not able to be confirmed from the small size samples available from the boreholes. Further investigations should include some deep large diameter boreholes, to obtain samples for compaction and CBR testing.

(c) Further investigation of the suitability of rock materials for aggregate at selected sites and the viability of deriving adequate quantities for use as all or part of the road and concrete aggregate requirements should be carried out.

17 Disposal Sites

17.1 Disposal Philosophy

The philosophy proposed for disposal of surplus fill materials and unsuitable materials is to:

(a) create disposal space by road design, such as separation of the carriageways to provide a wider median, which will accommodate surplus fill

(b) dispose of surplus materials through the placement of fill on the flanks of the fill embankments beyond the recommended structural fill envelope, to achieve undulating more natural looking landscaped areas

(c) utilise near-site disposal sites to create landscape features through the use of surplus fill, where space is available

(d) identify disposal areas that are close to the alignment, which can be developed and used without creating a hazard to the highway, land owners or natural features such as water courses, and where possible create usable land (eg farming or subdivision).

(e) off-site disposal where necessary, including offering materials for use by landowners where they have consent to fill.

(f) On site disposal on the flanks of the formation should be developed as part of the next stage of design for the project.

Disposal of surplus materials as part of the project development along the road should be encouraged as it has the potential to significantly reduce the risks to the road through the use of flatter embankment slopes and non-structural shoulder fills, in addition to disposing of materials. Also use of a wider median to accommodate surplus fill (for example at Battle Hill Area) would have additional environmental and road safety benefits.

17.2 Selection of Potential Disposal Sites

Potential disposal sites were identified as a joint activity between the SAR team and the geotechnical team, through:

(a) Identification of surplus materials and locations by the road designer

(b) Workshop with road designer, landscape and environmental specialists and geotechnical engineers to assess and select sites along the route.

(c) Field visit to identify and screen potential disposal sites.

17.3 Geotechnical Issues for Consideration of Disposal Sites

The following geotechnical considerations have been developed for screening disposal sites and to assist with future design of disposal sites:

(a) Disposal sites should be developed at locations that do not pose a hazard to the highway, landowners or the environment, either through potential slope failure affecting the highway or through erosion and discharge of mud and sediments onto the highway following heavy rainfall. Disposal sites in gullies above the highway should therefore not be selected. If they are chosen, mitigation measures should be adopted to effectively avoid the above hazards.

(b) Disposal sites need to be founded on competent ground to avoid failures, though settlement may be accommodated by appropriate design.

(c) Disposal sites should incorporate good drainage provisions, including:

- channelling surface water from natural water courses through culverts
- cut off surface runoff from the catchment above using surface drains avoiding the flow entering the disposal area, and collect and discharge to water courses appropriately
- collection and disposal of surface run-off from the disposal area, with appropriate treatment
- subsoil drainage and drainage blankets to manage groundwater pressures and possible destabilising effects.

(d) Earthworks and compaction control is important to ensure that disposal sites are stable and are not just end tipped without any compaction or earthworks control.

(e) Unsuitable and wet materials disposed need to be contained behind engineered and well constructed bunds.

(f) Adequate landscaping and revegetation of disposal areas to minimise erosion and ensure rehabilitation.

17.4 Construction Management

It is important to ensure that disposal sites are well managed during construction, and any geotechnical hazards are identified and mitigated as part of the design and construction process.

Construction should allow for managing the disposal sites during storm events, to avoid destabilisation during construction as well as in the long term.

17.5 Summary

(a) The proposed disposal philosophy is to use the surplus and unsuitable materials as non-structural fill and for landscaping through incorporation into the road by appropriate road design. Disposal could also be used as landscaping fill adjacent to the highway to achieve a natural and attractive environment.

(b) Disposal sites that pose a risk to the highway or the environment as a consequence of slope failure or erosion or flow of mud should be avoided. Therefore the use of gullies above the proposed highway to dispose of spoil should be avoided.

(c) Disposal sites should be engineered to identify and mitigate geotechnical hazards during the design and construction processes. Site suitability and stability, foundations, earthworks, and drainage issues will need to be addressed.

18 Route Security Performance

18.1 The Regional Context

The regional and national importance of route security for the Transmission Gully route which provides an alternative to the current highly vulnerable SH 1 route was presented in Section 9, under design philosophy.

18.2 Route Security Philosophy

A route security philosophy (Section 9) for Transmission Gully was proposed as follows:

(a) Highway is open for full access with minimum structural damage in small hazard events with a short return period

(b) Highway suffers limited repairable damage in moderate hazard events, with continued limited access, or highway reopens after a short period of closure, say 12 hours to 3 days.

(c) Highway suffers major damage, but does not collapse, in large, long return period events, and limited access can be restored within a reasonable period (say 3 days to 2 weeks).

18.3 Route Performance

The expected performance of the route has been conceptually assessed considering the proposed current "*Preferred Alignment (1B)*", the size of cuts, fills, retaining walls and bridge structures proposed, and the likely performance of these based on current knowledge. It is possible that the likely performance will somewhat change as further investigations and design is undertaken and construction is completed.

The broad level performance of the route after a large earthquake on the Ohariu Fault has been assessed in terms of:

(a) Damage State

(b) Availability State

(c) Outage State

The performance states show that:

- there are a few locations of route closure and
- these areas are generally likely to be able to be opened relatively quickly, with appropriate emergency response planning by NZTA.

The performance states show that the Preferred Alignment (1B) generally meets the route security philosophy proposed for the project. There are a few areas of improvement necessary to meet the route security philosophy and these relate to two very high cuttings along the Te Puka Stream. This section should be investigated further to mitigate the risks in the area, e.g. by reducing the height of cuttings so that the size of potential landslides and the closure times can be reduced.

19 Geotechnical Engineering Risk Management

19.1 Geotechnical Engineering Uncertainties and Risks

The geotechnical investigations and assessment has enabled a better appreciation of the ground and groundwater conditions along the route and remaining uncertainties and associated risks.

The geotechnical risks arise from uncertainties in:

- ground and groundwater conditions
- route alignment and concept design
- design, procurement and construction.

The key geotechnical uncertainties, risks and recommended risk management measures are summarised in Table 3.19. The order in which the uncertainties are listed is not indicative of the level of risk or priority.

Table 3.19 - Key Geotechnical Uncertainties and Risk Management

No	Key Uncertainties	Risks	Risk Rating	Risk Management Actions
1	Knowledge of peat distribution, thickness and properties (Sector 1).	▪ Increased costs of ground improvement. ▪ Failure / settlement of embankments	▪ Moderate	▪ Additional site investigations. ▪ Organic content, triaxial strength and consolidation tests. ▪ Adopt 1A alignment where ground known and peat thickness is small.
2	Liquefaction potential of ground at SH 1 Paekakariki (Sector 1).	▪ Embankment and abutment failure	▪ High	▪ Additional site investigations at bridge site. ▪ Develop ground improvement options.
3	Alignment in the vicinity of pre-historic landslide at Paekakariki (Sector 1).	▪ Potential for destabilising landslide, although current alignment buttresses landslide toe.	▪ High	▪ Confirm alignment and effect on landslide. ▪ Avoid landslide or develop stabilisation measures.
4	Preferred Alignment 1B straddles Te Puka Stream downstream section with embankment in Te Puka Stream (Sector 2)	▪ If stream realigned to west, erosion destabilises unstable hillside. ▪ If stream culvert blocked in debris flow, creation of reservoir behind embankment.	▪ High	▪ Locate alignment through Te Puka terrace and avoid stream. ▪ Provide large box culvert that can pass debris flows.
5	Location of Ohariu Fault on eastern flank of Te Puka Stream at north end of valley (Sector 3).	▪ Fault zone affects cut slopes. ▪ Fault location affects viaduct abutment foundations and seismic design across fault	▪ Moderate	▪ Engineer In-designation Alignment 1A to minimise risk, if chosen. ▪ Investigate the fault with fault trenches and inclined boreholes.

No	Key Uncertainties	Risks	Risk Rating	Risk Management Actions
		(In-designation Alignment 1A)		
6	Dominant rock defects not identified in specific cut slopes or foundations. (Sectors 3, 4, 8 and 9)	▪ Failure of cuttings in rock due to unfavourable defects. ▪ Affects performance during construction, long term and route security in hazard events.	▪ High	▪ Geotechnical investigations – mapping defects in exposures, boreholes (including inclined boreholes, acoustic televiewer surveys), trial cuts.
7	Shear strength of shear, crush, fault zones not known. (Sectors 3, 4, 8 and 9)	▪ Failure of cuttings in rock due to unfavourable defects. ▪ Affects performance during construction, long term and route security in hazard events.	▪ High	▪ Geotechnical investigation – boreholes to collect samples. ▪ Laboratory shear strength tests in shear box / ring shear. ▪ Develop and implement mitigation measures.
8	Activity of landslides at Paekakariki, Stn 5,100 on western flank of Te Puka Stream. (Sectors 2 and 3)	▪ Movement of landslide poses a risk to bridge piers for alignment 1A.	▪ High	▪ Early Installation of survey points and inclinometers and monitoring over a period of time (seasons and rainfall) ▪ Develop mitigation measures if required.
9	Retaining walls on steep slopes (Sectors 3 and 4)	▪ Instability of slopes undermining walls and leading to failure. ▪ Expensive solutions. ▪ Route security compromised.	▪ High	▪ Early Investigation of ground conditions at wall sites. ▪ Consider alignment to avoid / minimise walls. ▪ Early investigation of alternate solutions (reinforced embankments and effect on stream, if any).
10	Very high cuttings in variable rock conditions (Sector 3)	▪ Route security performance reduced by potential for large failures in earthquake.	▪ High	▪ Early investigation of the alignment and cut details to consider improvements to route security.
11	High reinforced soil embankments (Sectors 3 and 4)	▪ Instability due to optimistic design.	▪ High	▪ Further investigate reinforced soil embankments and design concepts. ▪ Develop solutions.
12	Nature and inclination of fault zones at Wainui Saddle area. (Sector 3)	▪ Extent of poor ground causing cut slope failures	▪ High	▪ Drill inclined boreholes to investigate fault zones. ▪ Laboratory shear strength tests in shear box / ring shear. ▪ Adjust alignment to suit conditions. ▪ Develop stabilisation measures
13	Extent of Active Splinter of Ohariu Fault south of Wainui Saddle not well defined. (Sector 3 & 4)	▪ Poor fault disturbed ground leading to failure or expensive solutions. ▪ Groundwater conditions affecting stability. ▪ Route security affected if inappropriate road concepts	▪ High	▪ Further fault trench excavation and assessment. ▪ Dril boreholes (including inclined boreholes) to investigate nature and width of zone. ▪ Develop design / stabilisation concepts to suit.

No	Key Uncertainties	Risks	Risk Rating	Risk Management Actions
		adopted.		
14	Inadequate characterisation of pre-Holocene soil deposits (Sectors 5 and 6)	▪ Cut and embankment failure. ▪ Poor route security. ▪ Pavement failure. ▪ Earthworks difficulties during construction.	▪ High	▪ Additional geotechnical investigations – deep larger diameter boreholes to sample soils ▪ Laboratory strength, compaction and CBR tests. ▪ Develop earthworks and pavement strategy.
15	Characteristics and activity of landslides not known (Sectors 6 and 8).	▪ Failure during construction. ▪ Route security affected in long term in storm events.	▪ High	▪ Early investigation of specific landslides. ▪ Install inclinometers early and monitor over a period of time (seasons and rainfall) ▪ Develop stabilisation measures if required.
16	Inadequate knowledge of ground conditions along James Cook, Waitangirua and Kenepuru links, where no preliminary geotechnical site investigations undertaken. (Sectors 8 and 9).	▪ Discovery of conditions lead to escalation of construction costs. ▪ Alignment changes, designation changes and land purchase necessitated by unfavourable ground conditions.	▪ High	▪ Carry out further geotechnical mapping, site investigations (trial pits, boreholes) and laboratory testing. ▪ Geotechnical assessment of conditions. ▪ Undertake investigations early given large uncertainties.
17	Knowledge of rock characteristics and extent of good rock for aggregate sources are uncertain.	▪ Unable to take advantage of the opportunity to locally win aggregate. ▪ Unable to gain designation or consents for winning local aggregate. ▪ Increased construction costs. ▪ Increased environmental effects due to transport over large distances an depleting regional quarry resources.	▪ High	▪ Early investigation of aggregate potential along the route. ▪ Drilling boreholes to characterise rock quality and extent. ▪ Laboratory testing for aggregate quality. ▪ Investigate production viability and logistics.
18	Knowledge of selected disposal areas poor.	▪ Selection of disposal areas without adequate information resulting in increased costs. ▪ Inability to gain designation, landowner approval or consents for suitable disposal areas.	▪ Moderate	▪ Early investigation of disposal area. ▪ Screen disposal areas for geotechnical risks.
19	Poor knowledge of ground conditions at structure sites.	▪ Inability to confirm length of bridge and form and extent of foundations depending on ground condition. ▪ Bridge failure from inadequate geotechnical knowledge of	▪ High	▪ Geotechnical investigations at structure sites.

No	Key Uncertainties	Risks	Risk Rating	Risk Management Actions
		conditions.		
20	Insufficient characterisation of ground conditions for selected route.	▪ Concepts cannot be confirmed. ▪ Inadequate understanding leads to failures during construction and long term. ▪ Route security compromised because conditions not known when alignment and concepts could be changed. ▪ Further land take and designation changes as geotechnical conditions are discovered late.	▪ High	▪ Early geotechnical investigations.
21	Poor knowledge of groundwater conditions and fluctuations over time and seasons.	▪ Poor design decisions based on insufficient knowledge on groundwater level changes. ▪ Failure or increased costs during construction due to inadequate knowledge.	▪ Moderate to High	▪ Ongoing monitoring on groundwater conditions in piezometers installed and future piezometers over a period of time and across different seasons. ▪ Measure fluctuation of water levels with rainfall in key boreholes.
22	Procurement methods do not deliver project route security objectives and leads to increased operational maintenance costs.	▪ Poor alignment with Principal's objectives on route security. ▪ Failures arise from insufficient investigations. ▪ Route security compromised because lower cost solutions chosen to win project based on poor subjective decisions particularly on cut slopes and fills. ▪ Poor route security outcomes. ▪ Lower cost project but greater ongoing maintenance costs.	▪ High	▪ Discuss and select appropriate procurement strategy through consultation and discussion with key technical specialists in addition to procurement advisors.
23	Delayed geotechnical investigations and assessment.	▪ Designation needs to change due to geotechnical conditions. ▪ Land purchase and landowner consultation changes as geotechnical conditions uncovered. ▪ Designation, land purchase, consultation decision lead to poor design decisions affecting performance and route security.	▪ High	▪ Early geotechnical investigations of key uncertainties. ▪ Ongoing geotechnical inputs to alignment engineering, designation, land purchase and consultation decisions.

19.2 Risk Management

Risk management initiatives to manage the geotechnical risks should be undertaken on an ongoing basis without delaying actions until later stages. Past experience shows that if risk management actions are undertaken in a timely manner, then there are more opportunities and options to manage the risks. As time goes by, the risks become more difficult and expensive to manage and some risks cannot be mitigated and become residual risks compromising performance. Risk management actions recommended to manage the identified geotechnical risks are also outlined in Table 3.19.

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