Peka Peka to Ōtaki Expressway

Assessment of Environmental Effects

Technical Report
Geotechnical Engineering and Geology
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The New Zealand Transport Agency

Peka Peka to North Ōtaki Expressway

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Summary

This Technical Report on Geotechnical Engineering and Geology presents geotechnical issues of importance to the Assessment of Environmental Effects (AEE) for the Peka Peka to North Ōtaki section of the proposed Kāpiti Expressway (the "Expressway") including related elements such as local roads and a realigned section of the North Island Main Trunk railway. It is based on content from the geotechnical assessment report and factual reports prepared as part of the scheme assessment and subsequent geotechnical assessments as part of the assessment of environmental effects and route resilience.

The engineering geology and hydrogeology of the Expressway is described along with significant hazards which include earthquake ground shaking, liquefaction and active fault rupture. Precedent observations in the Wellington Region are an essential part of the engineering assessment.

A summary of the engineering assessment of geotechnical aspects for the Expressway is presented for:

- cut slopes in dune sand. Including erodibility and erosion protection;
- cut slopes in terrace alluvium;
- embankment fills including undercut of peat and other compressive swamp deposits;
- ground improvement and instrumentation;
- earthworks and construction materials; and
- effects on groundwater regime.

The proposed cuttings for the Expressway will involve lowering the groundwater levels at the terrace alluvium between Stations 3900 m and 5300 m locally by up to 2 m. Excavations to undercut and remove peat will require lowering of the groundwater levels locally by about 3 m over a short period of time of a few days. These have an insignificant effect on the groundwater regime in the area, and are not expected to have any adverse effects on the environment, or groundwater takes.

Settlements associated with construction and preloading of the embankments on soft ground will cause some settlement of the area immediately surrounding the preload areas. However these areas are rural farm areas and therefore the settlement will not cause any significant effects on the environment.

Resilience of the Expressway and the philosophy adopted to enhance resilience is presented. The Expressway will have a good level of resilience to major earthquakes that are possible in the region. Some limited subsidence of the Expressway embankments could occur due to liquefaction of isolated limited layers in the underlying ground, but the Expressway is likely to provide continued access with some uneven road surface. This is likely to be able to be reinstated in a few days.

In the event of rupture of the Northern Ohariu Fault, there is expected to be some displacement with distributed cracks over a wide area, but limited access is likely to continue to be available or can be restored within a few days by earthmoving machinery.

All proposed earthworks will be within the designations being sought for the Project.

1 Introduction

The New Zealand Transport Agency (the NZTA) is lodging a Notice of Requirement (NoR) and applications for resource consents for the construction of the Peka Peka to North Ōtaki (PP2O) section of the Kāpiti expressway project. The NoR for the re-alignment of about 1.2 km of the North Island Main Trunk (NIMT) railway through Ōtaki is also being sought, and is being undertaken on behalf of KiwiRail. In this application, "the Project" refers to:

- construction of the main road alignment;
- realignment of part of the NIMT; and
- associated local road connections.

The project is a proposal of national significance and the matters have been lodged with the Environmental Protection Agency (EPA).

The proposed Peka Peka to North Ōtaki section of the Kāpiti Expressway is an approximately 13 km long new route, which runs from Te Kowhai Road in the south to Taylors Road, north of Ōtaki. The Expressway forms one segment of the proposed road improvements along the Wellington Northern Corridor Roads of National Significance (RoNS), see Illustration 1.

Illustration 1 - Peka Peka to Ōtaki Expressway within the Wellington Northern Corridor

Actual and potential environmental effects relating to various geotechnical engineering aspects including effects to groundwater, aquifers and existing abstraction bores, rivers and ground settlement have been identified and assessed. Given the national and local significance of the Expressway and because it is located in an area of high seismicity, we have also studied the resilience of the road against natural hazards.

This Technical Report is based on the site investigation results presented in the geotechnical factual report (AECOM, 2011) and the geotechnical assessment presented in the geotechnical interpretative report (Opus, 2011).

This report presents the outcomes of the assessment of environmental effects and of road resilience and recommends mitigation measures for potential effects. Some background information such as site geology, ground conditions and geotechnical engineering of the Project, which includes cut slopes, embankments, bridges and ground improvement, is also provided in the report.

2 Location of the Route

The Expressway is located along the Kāpiti Coast, approximately 40 km north of Wellington (see Illustration 2). The Expressway stretches for 13.5 km from Peka Peka (Te Kowhai Road) in the south to Ōtaki (Taylors Road) in the north. The Expressway passes through the Te Horo and Ōtaki townships. It crosses the North Island Main Trunk (NIMT) railway line and a number of main watercourses including the Ōtaki River, Waitohu Stream and Mangaone Stream.

The NZMS 260 Map Grid Reference for the route is R26 860 386 at the Peka Peka Road intersection, S25 912 461 at the Ōtaki River crossing and S25 930 488 at the Taylors Road intersection.

Illustration 2 - Location of Peka Peka to Ōtaki Expressway

3 Geological Setting

3.1 Stratigraphy

The 1:250,000 QMap series Geological Map for Wellington area (IGNS, 2000a), see Illustration 3, indicates the area to be underlain predominantly by:

- aeolian dunes (Q1d) of Quaternary age (commonly known as sand dunes) to the south of Mary Crest, and north of Ōtaki;
- poorly to moderately sorted gravel with minor sand to silt underlying aggradational and degradational terraces (Q2a) of Quaternary age (commonly referred to as terrace alluvium) from Mary Crest to Ōtaki River;
- well sorted floodplain gravels (Q1a) of Holocene age along the Ōtaki floodplain to the north of Ōtaki River, and along the Waitohu Stream flood plain (commonly referred to as recent alluvium).

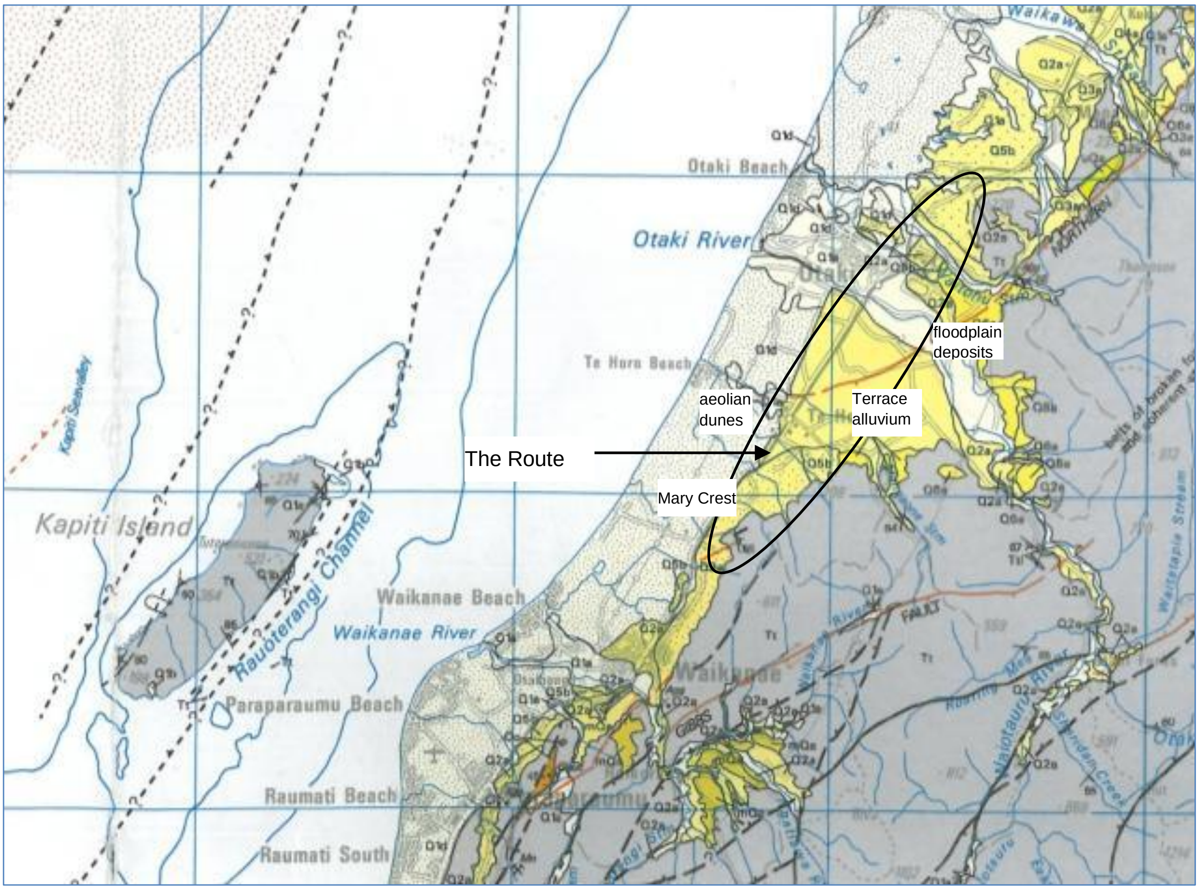


Illustration 3 - Geology of the area indicated on QMap (after IGNS, 2000a)

The Kāpiti District has been mapped at 1:25,000 scale Surface Geology Map by the Department of Scientific and Industrial Research (DSIR, 1992). The DSIR map shows similar and more detailed geological descriptions of the area compared with the QMap.

In particular, it differentiates between the dunes and inter-dunal peat/swamp deposits, see Illustration 6. The DSIR map indicates the area to be mainly underlain by:

- dune sand, inter-dunal deposits, old beach and dune deposits to the south of Mary Crest;
- terrace alluvium from Mary Crest to Ōtaki River;
- recent alluvium along the Ōtaki River floodplain and other river or stream locations along the expressway alignment; and
- localised inter-dunal deposits, terrace alluvium, recent alluvium and old beach and dune deposits towards the northern end of the alignment.

The ground conditions based on the site investigations are presented in Section 6.

3.2 Hydrogeology

The hydrogeology of the Kāpiti Coast was broadly classified into three groups of deposits based on their environment of deposition by Jones and Baker (2005), see also Illustration 4:

- glacial and inter-glacial deposits - a thick layered semi-confined to confined aquifer system of poorly sorted and stratified clay-bound gravels and sand overlying bedrock (see Q2a above)(note that the terrace alluvium at the surface to the east of the sea cliff is unconfined, but is underlain by semi-confined and confined aquifers);
- post-glacial beach and dune sand deposits - a low-yielding, unconfined aquifer which becomes semi-confined with depth towards the coast (up to 50 m depth)(see Q1d above). The aquifer system encompasses a coastal dune belt, which has resulted in the formation of a number of inter-dunal wetlands where drainage has been impeded; and
- recent river gravel deposits - relatively high-yielding unconfined aquifers in the alluvial flood plain around the rivers (see Q1a above). The gravels were reworked by rivers during the interglacial and postglacial period and have a direct hydraulic connection with surface water.

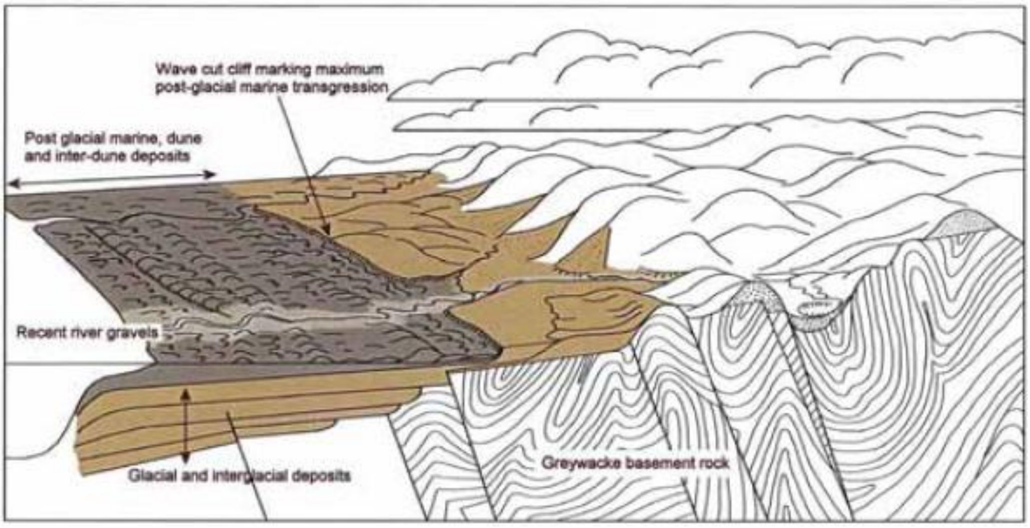


Illustration 4 - Geology and Aquifer Systems of the Kāpiti Coast (source: Jones & baker 2005)

3.3 Geomorphology

The Expressway runs through an area of relatively flat to rolling terrain, comprising recent sand dunes and inter-dunal deposits, the slightly raised terrace alluvial plateau, and the wide recent alluvial plains of Ōtaki River and Waitohu Stream.

The Expressway is about 1 km to 2.5 km west of the foothills of the Tararua Range and 3 km to 4 km east of the Te Horo- Ōtaki coast, see Illustration 5.

The Expressway crosses a number of watercourses including the Mangaone Stream (at Te Horo), Ōtaki River (south of Ōtaki township), Mangapouri Stream (near County Road, Ōtaki) and Waitohu Stream (north of Ōtaki).

An abandoned sea cliff at Te Horo has been identified along the existing State Highway 1 and the NIMT railway line between south of Mary Crest and Peka Peka Road, and along Te Waka Road between Lethbridge Road and Te Horo Beach Road. The abandoned sea cliff is located along the surface boundary between the older terrace alluvium (Q2a) and the recent sand dunes and inter-dunal deposits. There are also alluvial terraces present, for example along the Ōtaki River.

Illustration 5: Oblique aerial photograph showing topography of the area (source : Google Earth)

3.4 Active Faults

Active faults in the area are discussed in Section 4.2.

4 Earthquake Hazards

4.1 General

The Expressway is located in an area of high seismicity. Primary geohazards identified include:

- active faults / fault rupture;
- ground shaking;
- earthquake induced slope failure; and
- liquefaction.

4.2 Active Faults

The Expressway is located in the Wellington Region, an area of high seismicity in New Zealand. There are a number of major active faults in the region, which are summarised in Table 1.

Table 1: Active Faults in the Area

Fault	Magnitude	Recurrence Interval (years)	Distance from site (km)	Direction
Northern Ohariu Fault	7.2 - 7.5	1,000 - 3,000	0.1	East
Ohariu Fault	7.1 - 7.5	2,200	1	Southeast
Gibbs Fault	unknown	3,500 - 5,000	5	Southeast
Southeast Reikorangi Fault	unknown	5,000 - 10,000	8	Southeast
Ōtaki Forks Fault	7.3 - 7.6	4,000 - 9,000	12	East
Pukerua Bay Fault	7.6	2,000 - 3,500	25	Southwest
Wellington Fault	7.6	610 - 1,100	27	East

Source: Heron et al. (1998); IGNS (2000 & 2003), Litchfield et al. (2004, 2006, 2010); Little et al. (2009, 2010); Palmer & Van Dissen (2002); Stirling et al. (2002); Van Dissen & Berryman (1996) & GNS NZ Active Faults Database (*http://maps.gns.cri.nz/website/af/viewer.htm*).

The Northern Ohariu Fault and Ohariu Fault are the two active faults closest to the site. The locations of the two faults are shown on Illustration 6 and in the Plan Set in Volume 5 (Figure GA05).

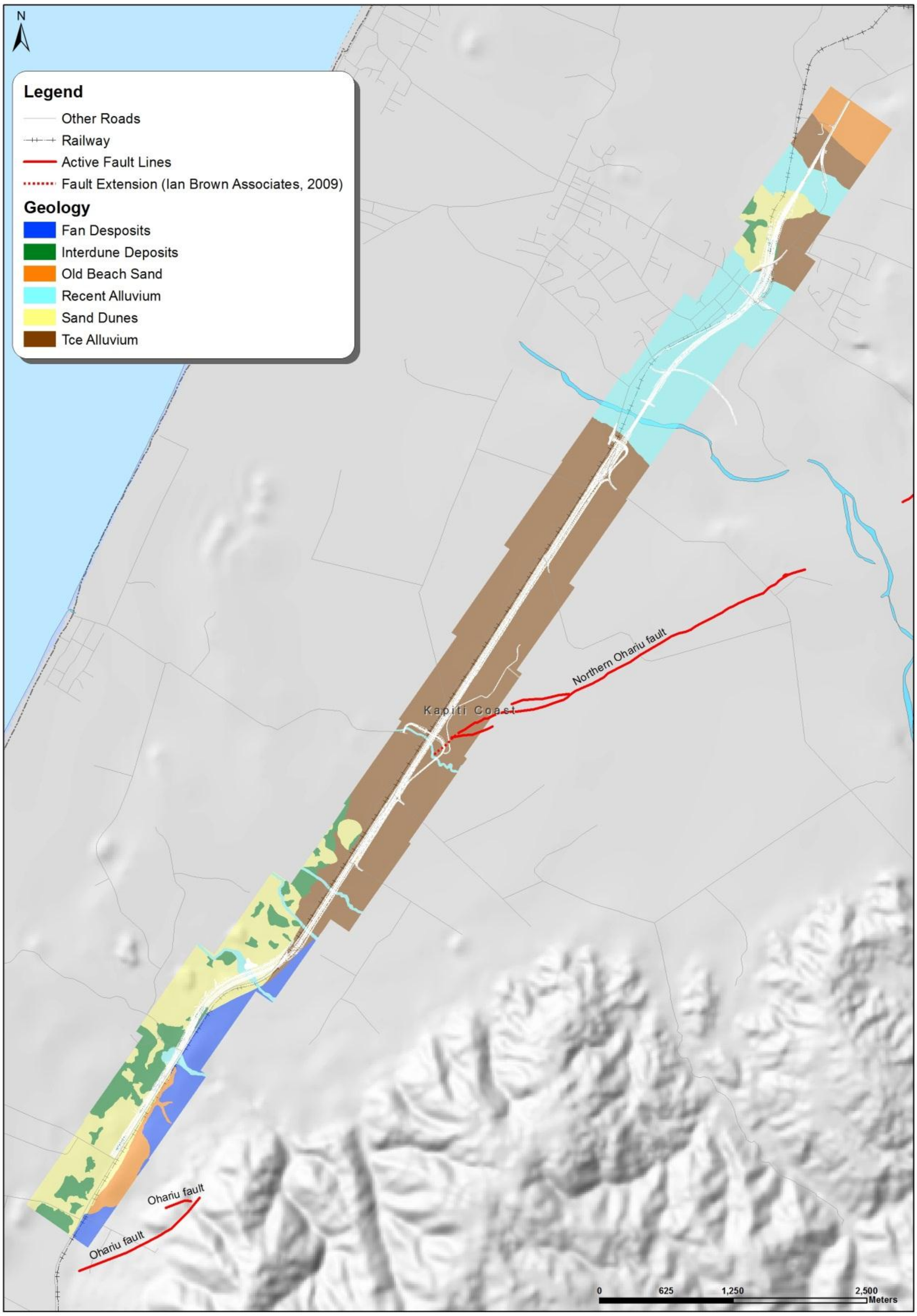


Illustration 6 - Geology and Active Faults along the Route

4.3 Fault Rupture

Rupture along active fault traces, due to local earthquakes, could lead to significant damage or deformation of structures built over or adjacent to the faults.

A rupture of the Ohariu Fault could result in between 3m and 5m of right-lateral displacement at the ground surface, with less and more varied vertical displacement. It is also expected that an individual surface rupture along the Northern Ohariu Fault could generate 3 m to 4 m of right-lateral displacement at the ground surface, with a lesser and variable amount of vertical displacement (Stirling et al., 2002).

Northern Ohariu Fault

The Northern Ohariu Fault trace is shown on the maps to be about 100m east of the Expressway at Te Horo. The southern section of the Expressway is indicated to be within the Fault Avoidance Zone of the Northern Ohariu Fault (GNS, 2003). The Northern Ohariu Fault has a characteristic magnitude of 7.2 to 7.5 with a recurrence interval of approximately 1000 to 3000 years (Palmer & Van Dissen, 2002).

The Northern Ohariu Fault trace has been mapped by the Institute of Geological and Nuclear Sciences (IGNS, 2003) and Ian Brown Associates (IRBA, 2009). Both studies record the presence of distinct fault scarps trending southwest across the alluvial terraces south of the Ōtaki Otaki River.

The original fault mapping by IGNS showed the fault trace becoming subdued and indistinct at the south western end of the fault, approximately 125 m from the Expressway. Extrapolating the trend of the fault trace suggests the fault could intersect the Expressway very close to the proposed bridge connecting School Road to Te Horo Beach Road.

The more recent study by Ian Brown Associates refines the fault location in this area, and shows the fault trace extends beyond the previously mapped extent but changes orientation, becoming more south-trending. This new mapping places the fault trace parallel to the Expressway, approximately 75 m to the east, and crosses the earth ramp leading to the overbridge. We have reviewed the available LIDAR data that the IRBA mapping was based on, to confirm the observations made in that report. This is shown in Illustration 6 and Figure GA05 in the Plan Set in Volume 5.

The hazard to the Expressway from fault rupture is discussed in Section 13.5.

Ohariu Fault

The Ohariu Fault is indicated to be about 1 km away from the alignment to the southeast of Peka Peka Road (GNS, 2003). This fault is capable of rupturing in a magnitude 7.1 to 7.5 earthquake with an average return period of 2200 years (Heron et al., 1998; Litchfield et al., 2004, 2006, 2010). The Ohariu Fault is likely to cross the MacKays to Peka Peka section of the Kāpiti Expressway outside the extent of the Peka Peka to North Ōtaki section of the Expressway, which is covered in this report.

4.4 Ground Shaking

There is potential for significant ground shaking during large earthquakes. The ground shaking is expected to be modified and exacerbated by the presence of deep soil deposits and soft ground in the area.

The design horizontal peak ground accelerations (PGA) to be used in assessing the stability of slopes and structures such as fill embankments and bridge structures have been derived according to the New Zealand Earthquake Loading Standard, NZS 1170.5: 2004 (Standards NZ, 2004) and the Bridge Manual (Transit NZ, 2003) and its Provisional Amendment in December 2004.

Given that the deep alluvial deposits are likely to exceed 100m in thickness over most areas along the proposed route, the site subsoil class has been assessed to be Class D (Deep or soft soil) according NZS 1170.5. For Waikanae and Ōtaki area, NZS 1170.5 provides a hazard factor, Z, of 0.4.

The Bridge Manual (Transit NZ, 2003) and its Provisional Amendments provides recommendations for the design of bridges and other highway structures. An Importance Level of 3 and a 100 year design life is assumed for bridge structures, resulting in an annual probability of exceedance of 1/2500. This means that they are designed to withstand a 1 in 2500 year earthquake event. For other structures such as free-standing walls, fill embankments and cuttings, which do not form an integral part of bridge structures, a return period factor of 1.5 is adopted (Table A4 in the Provisional Amendment (2004) of the Transit New Zealand Bridge Manual). This is equivalent to an annual probability of exceedance of 1/1500.

The assessed design Peak Ground Accelerations for analyses and design is presented in Table 2. These have been used in the preliminary geotechnical design.

Table 2 - Design Peak Ground Acceleration

Structure	Return Period Factor	Annual Probability of Exceedance	Design PGA
Bridge structures	1.8	1 / 2500	0.81g
Free standing structures (e.g. retaining walls, fill embankments and cuttings)	1.5	1 / 1500	0.67g

4.5 Earthquake Induced Slope Failure

The Regional Slope Failure Hazard Map (Wellington Regional Council, 1995) indicates a generally low susceptibility to earthquake induced slope failure along the Expressway. Localised narrow bands of high susceptibility to slope failure in earthquakes are identified along the abandoned sea cliff to the south of Mary Crest to Peka Peka Road, along Te Waka Road and the river terraces south of Ōtaki River, just north of Ōtaki township and along the stream opposite Te Hapua Road, see Illustration 7. There is also a high susceptibility to slope failure locally at a steep portion of the existing railway cutting just south of the Ōtaki River crossing.

4.6 Liquefaction

Liquefaction as a consequence of earthquakes could lead to subsidence and lateral spreading, which could affect any surface development.

According to the Regional Liquefaction Hazard Map (Wellington Regional Council, 1993), see Illustration 7, the majority of the Expressway is situated in areas which are not susceptible to liquefaction. In areas to the north of Ōtaki River, variable potential for liquefaction (from low to high) may be present associated with the recent alluvium, and moderate potential in the areas of dune sand and inter-dunal deposits.

There is very low or no potential for liquefaction between Ōtaki River and Mary Crest. A moderate potential for liquefaction is indicated in the areas underlain by sand dunes and inter-dunal deposits, south of Mary Crest. Results of the site investigations show that there are localised sand and silt layers within the site that could potentially liquefy.

The liquefaction hazard to the Expressway is discussed further in Section 13.3.

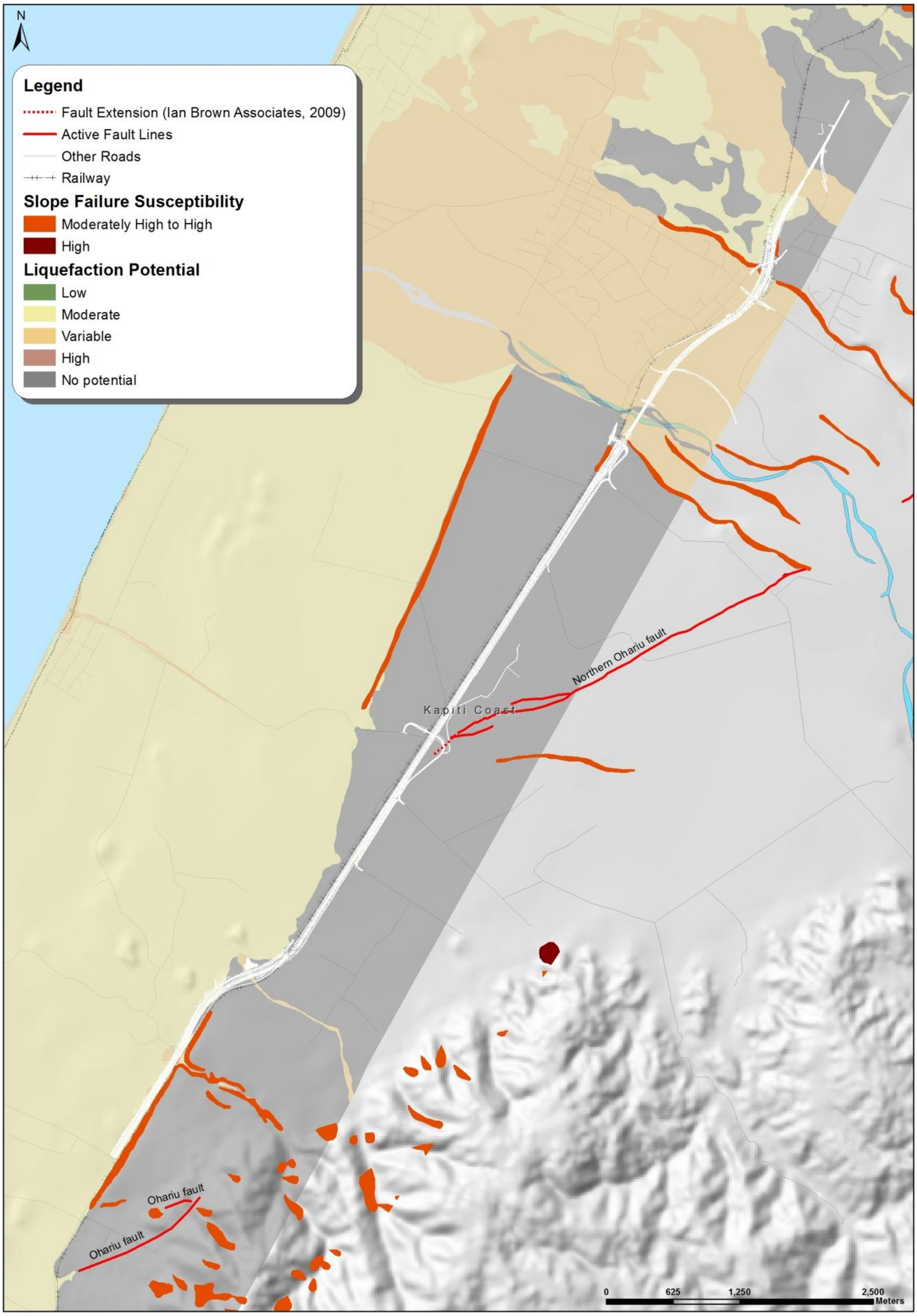


Illustration 7 - Earthquake Induced Slope Failure and Liquefaction Hazards along the Route

5 Geotechnical Investigations and Assessment

5.1 Engineering Geology Mapping

Field engineering geological mapping was carried out by Opus in November 2010 to confirm and map the geology and geomorphic features and the fault traces identified in the geological maps and aerial photographs. An engineering geological map was produced after the mapping and was refined after more information was gathered during the site investigations. The resultant engineering geological map is presented in Volume 5 Plan Set (Figures GI01 to GI08).

5.2 Site Investigations

A programme of geotechnical site investigations was developed by Opus following the engineering geological mapping, and was implemented and reported by AECOM (2011) with overview by Opus.

The programme of investigations completed are summarised in Table 3, and the locations of the site investigations are included in in the Plan Set in Volume 5 (Figures GI01 to GI08).

Table 3 - Programme of Site Investigations

Investigation type	Number	Depth
Boreholes	15	13 m to 40 m
Piezo-cone Penetration Tests (CPTu)	26	1 m to 15 m
Trial Pits	34	2 m and 5.5 m
Dynamic Cone Penetration Tests	40	2.4 m and 3.9 m

Piezometers for monitoring groundwater levels were installed in 14 of the boreholes and some Piezo-cone Penetration Test holes, and the locations are also identified in the Plan Set in Volume 5. The piezometers are being monitored by Opus.

Laboratory tests comprising soil classification tests, compaction tests, CBR tests, consolidation tests and triaxial tests were carried out by AECOM and Opus laboratories.

The results of the investigations are presented in the Geotechnical Factual Report (AECOM, 2011).

5.3 Geotechnical Assessment

An assessment of the geotechnical information for the scheme design of the expressway was carried out by Opus and is presented in the Geotechnical Interpretative Report (Opus, 2011). This interpretative report and the factual report referred to above have formed the basis for the assessment presented in this technical report on geotechnical engineering and geology.

6 Ground Conditions

6.1 Alluvial floodplain deposits (recent alluvium as indicated in (DSIR, 1992))

Alluvial floodplain deposits are found in the Ōtaki River floodplain and along other watercourses including the Mangaone Stream the Waitohu Stream. The floodplain alluvium comprises well sorted sub-angular to rounded gravel and cobbles with some boulders in a sand and/or silt matrix. The deposits are generally loose to medium dense near surface and become dense to very dense with depth. The clasts are predominantly formed from Greywacke sandstone. Interbedded layers of dense to very dense sand and firm to stiff silt and clay are commonly found within the gravels and cobbles.

Typical alluvial floodplain deposits found at the ground surface are shown in Illustration 8.

Illustration 8 - Alluvial floodplain deposits at Waitohu Stream

6.2 Aeolian sand deposits (dune sand) and old beach deposits

Dune sand is located at the southern part of the proposed route (south of Mary Crest) and at the northern end (between Waitohu Stream and Rahui Road). The dune sand is generally fine to medium sand with some or trace of silt and is generally loose at the surface and becomes denser with depth. The sand dunes generally have side slopes of between about 5° and 25° and rise up to 25 m above the surrounding flat terrain.

Typical sand dunes are shown in Illustration 9.

The Expressway crosses the old beach / dune deposits at the northern end of the alignment and to the east of the existing state highway south of Mary Crest.

Illustration 9 - Typical sand dunes adjacent to the alignment

6.3 Swamp deposits (inter-dunal deposits)

The swamp deposits are commonly encountered within isolated and sometimes inter-connected inter-dunal depressions between sand dunes. The swamp deposits generally comprise organic silt, clay, peat and sand. The peat is generally soft, fibrous and spongy and sometimes consists of decomposing fine rootlets and wood fragments. The silt and clay materials have a variable plasticity from low to high. The groundwater level is high and is commonly observed as seepages within the swamp deposits in trial pits. In winter, some of the swamp areas become inundated with stagnant water.

Typical swamp deposits are shown in Illustration 10. The location and indicative thickness of swamp deposits along the proposed route are shown in Table 4.

Table 4 - Inter-dunal Deposits along the Proposed Route

Location	Approx. Station	Indicative Thickness of Swamp Deposits
North of County Road	1,450 - 1,700 m	Up to 3 m
South of Mary Crest	10,500 - 11,550 m	3 m to 4.5 m
South of Mary Crest	11,550 - 12,200 m	1 m to 2 m

Illustration 10 - Swamp deposits in between sand dunes

6.4 Alluvial terrace deposits (terrace alluvium)

A significant length of the proposed route (from south of Ōtaki River to north of Mary Crest) is underlain by terrace alluvium. The terrace alluvium is comprised of well graded sub-angular to sub-rounded gravel, cobbles and boulders in a sand and/or silt matrix. The clasts are predominantly formed from Greywacke sandstone. Typical alluvial terrace deposits are shown in Illustration 11.

Illustration 11 - Typical alluvial terrace deposits along the route

7 Road Form of the Proposed Route

7.1 Expressway

The Expressway runs parallel with the existing SH1, from Taylors Road on the northern side of Ōtaki through to the northern end of the proposed Peka Peka interchange. The Expressway runs through rolling terrain comprising raised sand dunes and inter-dunal depressions, alluvial flood plains and raised terrace alluvium. It crosses two main watercourses, the Ōtaki River and Waitohu Stream. A few crossings of the Expressway will be required to provide linkage between the existing local roads, including the current SH1.

Based on the road alignment design as shown in the Plan Set in Volume 5 (Figures GM01 to GM08), the Expressway will require the following forms of road construction:

- cuttings of up to 20 m high in sand dunes and up to 8 m high in terrace alluvium;
- embankments of up to 8 m high over inter-dunal deposits, river alluvium and terrace alluvium (the higher sections of embankments are generally at the approaches to bridges and grade separated interchanges);
- bridges: several bridges will be required across roads (generally local roads over the Expressway), railways (both local roads and the Expressway over the railway) and main watercourses, including the Ōtaki River and Waitohu Stream;
- roundabouts, which have been proposed at the interchange to the south of Ōtaki River; and
- culverts, where the proposed route dissects small water courses.

All the earthworks for the expressway will be within the proposed designation area. There are shared stormwater facilities (i.e. shared swales) with KiwiRail which are outside the NZTA / Expressway designation, but within the existing and proposed KiwiRail designation. There is an agreement between NZTA and KiwiRail in this regard.

7.2 Realignment of NIMT Railway

The project includes realignment of the NIMT railway line through the Ōtaki area. These works will include:

- cuttings of about 20 m high through sand dunes north of Ōtaki;
- low embankments up to 4 m high;
- construction of new culverts;
- construction of a ballast formation for the railway tracks; and
- modifications to the Ōtaki Railway Station.

8 Cut Slopes

8.1 Cut Slope Locations and Configuration

High cuttings up to 20 m high will be formed in three main areas (see Plan Set in Volume 5 - Figures GM01 to GM08, for the actual location of the cuttings):

- south of Waitohu Stream;
- south of Ōtaki River; and
- north of Te Hapua Road.

Based on the materials in which the cuttings will be formed, and their performance under static conditions and during earthquake events, the following slope configurations, as shown in Table 5, have been adopted for road design. Since there is a demand for fill materials for the construction of embankments to form the Expressway, the cuttings may be formed with flatter slope angles and a wider berm during detailed design, where there is sufficient space, in order to generate more fill materials.

Table 5 - Cut Slope Configurations

Major Cut Location	Station (m)	Ground Characteristics	Approximate Max. Height (m)	Slope Angle	Benches
South of Waitohu Stream	1100 – 1500	Partly dune sand, partly terrace alluvium	20	22° (2.5H : 1V) or less	3 m wide benches at 10 m maximum height intervals
South of Ōtaki River	3900 - 5300	Terrace alluvium (gravels, cobbles and boulders)	8	40° (1.2H : 1V) or less	Wider berm at the base of the cutting
North of Te Hapua Road	10100 - 10500	Dune sand	12	22° (2.5H : 1V) or less	Not necessary

A typical cross section showing the adopted cut slope configuration in dune sand is shown on Illustration 12.

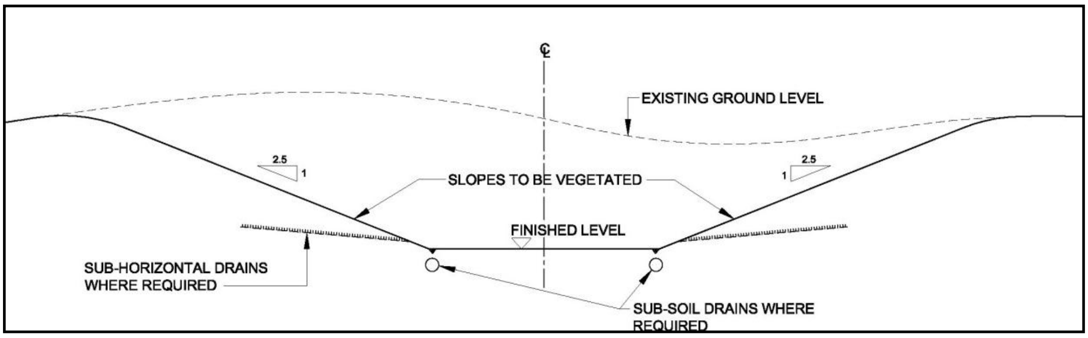


Illustration 12 - Typical Cut Slope Configuration

Benches

Benches will be formed in cuttings higher than 10 m to minimise rilling and gully erosion from surface run-off and rock fall from boulders and cobble in the terrace alluvium.

The benches will have an outward cross-fall to shed water rather than allowing the accumulation of water on the benches and hence destabilisation of the slope or causing localised erosion of the slope. Longitudinally, horizontal benches should be provided to avoid flow along benches leading to infiltration and erosion.

Rounding

It is proposed that the top of the cuttings and edges of benches be rounded in the vertical plane and the ends of the cuttings be rounded horizontally in plan to ensure the stability of near surface loose soils and also to provide a natural appearance which blends into the natural landscape.

8.2 Precedent Behaviours of Slope

Previous observations and experience in the design, construction and observation of the performance of cut slopes, particularly for state highways and railways in the region, provide understanding and knowledge of the characteristics and behaviour of cut slopes of similar nature and the common issues affecting cut slopes.

The proposed cut slopes will be formed in fine to medium dune sand and terrace alluvium.

The proposed cut slopes of 22° through dune sand are consistent with the natural sand dune slopes prevalent in the district, which generally stand naturally between 15° and 25° and probably reflects the angle of repose of the dune sand. Some dune slopes stand at a slightly steeper angle of not more than 30° and are probably marginally stable with the assistance of natural vegetation and partial saturation. There are some existing dune sand open cuttings in the area and they are generally small and stand at similar angles to the proposed cut slope of 22° in fine dune sand. Based on the above observations, the cut slopes formed at 22° have been adopted in the dune sand.

Cuttings were formed in the Pre-Holocene gravels and alluvium at 26° to 55° slopes at the SH 2 Kaitoke to Te Marua realignment and the Silverwood sub-division during the period of 2002-2006 and 2006-2008 respectively. The cuttings are stable with only localised failures, and performing well even after several large storm events over the years. It is considered that the proposed maximum cut slopes of 40° in the dense terrace alluvium is appropriate as it is within the range of the cut slopes formed in the previous projects. Localised areas with weaker soils may be encountered, and may need to be formed at a flatter slope of say 25° to 35°. During detailed design, consideration may be given to forming the cuts in the terrace alluvium at 26° to obtain additional good fill materials for the embankments, which would also further reduce the risk of instability in weaker areas of terrace alluvium.

8.3 Stability Analysis

Stability analysis for the proposed cuttings has been carried out based on representative material strength parameters to check for long term stability with a factor of safety against failure of 1.5 under static conditions. The material strength parameters were derived using the site investigation results as well as Opus' experience in the Kāpiti area (Opus, 2011).

Earthquake induced slope displacements were limited to no more than 150 mm as assessed using Newmark's method (Ambraseys & Srbulov, 1995). The assessed shallow slope displacements of this order will only lead to small volumes of soil deposits mainly on the road shoulder which can be easily cleared within days.

The analyses confirm that cut slopes of up to 22° (1V : 2.5H) in dune sand and up to 40° (1V : 1.25H) in alluvium are appropriate. A typical cut slope stability analysis is shown in Illustration 13.

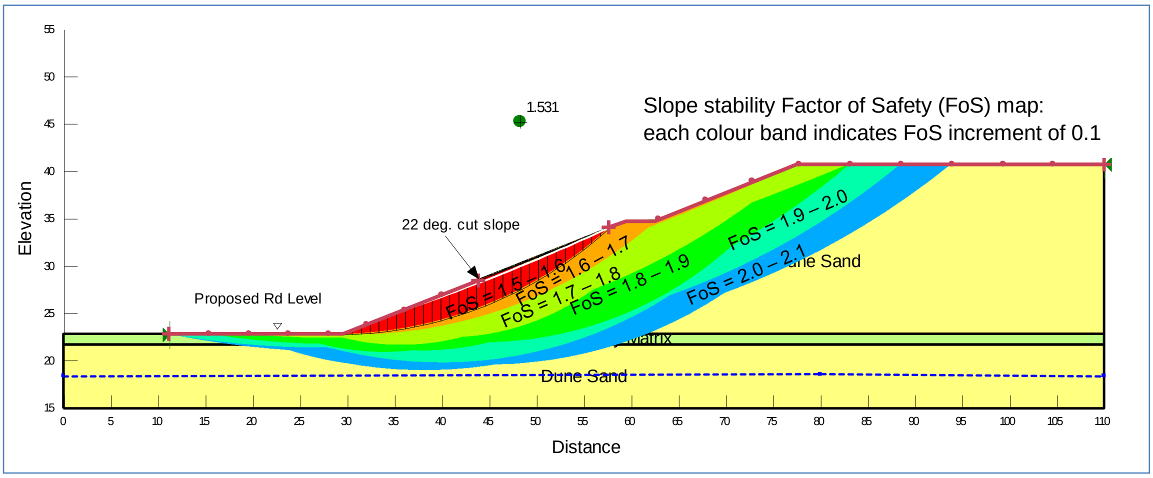


Illustration 13 - Typical Stability Analysis of Cut Slopes

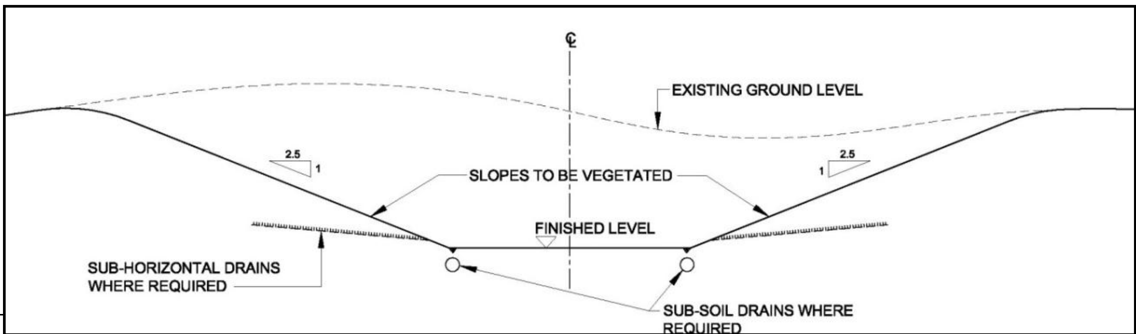
8.4 Drainage

To ensure the stability of the cut slopes and reduce erosion, adequate drainage measures will be provided in the cuttings.

Where the existing groundwater table is high, sub-horizontal drainage holes will be installed in the lower part of the cuttings to maintain low groundwater levels for slope stability and to minimise the risk of erosion or instability in storm events. Discharge from the drainage holes will be managed, such as by installing detachable flexible HDPE pipes, to prevent erosion of the cut slopes from the outflow. However, the ground water levels are generally low where cuttings are proposed in the sand dunes and the terrace alluvium, and these drainage measures will be installed where locally higher water levels are encountered.

Sub-soil drains will be installed between the toe of the cut batters and the pavement to keep the slope free of water seepages and to keep the water level well below the pavement subgrade. Sub-soil drains will be at a depth of at least 1.5 m at the toe of the cuttings and comprise geotextile wrapped free draining aggregates with a slotted sub-soil pipe. Schematic drainage details proposed for the cut slopes are shown in Illustration 14.

Illustration 14 - Schematic Drainage Details at Proposed Cut Slopes



8.5 Re-vegetation and Erosion Control

Cut slopes will be re-vegetated as soon as possible after the formation and the re-vegetation will be maintained during the early years after construction. Vegetation is usually grass or small plants to provide protection to the slope surface against erosion. The type of vegetation will be selected in conjunction with landscape architects to suit the local environment.

Erosion protection measures such as covering the slopes with topsoil or peat, installing surface drains and geotextile erosion matting are proposed for the erodible dune sand cut slopes.

9 Embankments

9.1 Configuration

Fill embankments of up to 8m high will be formed mainly over alluvium and interdunal deposits, see Figures GM01 to GM08 in the Plan Set in Volume 5.

The fill embankments will be formed at a maximum angle of 26° (2H : 1V). Steeper reinforced soil slopes may be considered using geogrid reinforcement if there is a constraint of space or for other reasons during detailed design. The recommended maximum fill slope angle is provided based on our experience in the design and construction of embankments and observation of the performance of embankments, particularly for state highways and railways in the region. Examples of the relevant projects involving embankment slopes include the SH2 Kaitoke to Te Marua realignment, MacKays Crossing and MacKays to Waikanae railway double tracking.

Placement of shoulder fill comprising undercut unsuitable materials and peat would help improve the stability of the structural embankment and reduce deformation in earthquakes by providing a buttress.

9.2 Sources of Embankment Fill Materials

The dune sand and terrace alluvium materials excavated from the cuttings will be generally suitable for construction of the fill embankments. The current road geometric design model shows that the volume of embankment fill required is higher than the amount of materials obtained from the cuttings.

There is an opportunity during detailed design to reduce the difference between cut and fill volumes, by forming the cuts through dune sand or terrace alluvium at flatter slopes and/or offset from the Expressway with a wider berm where space is available. This could be a more economical way of generating fill within the corridor than sourcing fill off-site. There is some space in the proposed designation to accommodate such widening or flatter slopes.

Potential borrow sites at the eastern foothills and potential quarry sites in the Kāpiti area could provide suitable quality fill materials for construction. There are also existing aggregate extraction plants along the Ōtaki River close to the Expressway, and road and concrete aggregate and possibly bulk fill materials may be available from this source.

9.3 Fill Construction

Given the generally dry nature of the dune sand and terrace alluvium materials, water is likely to be required for compaction to bring the materials up to optimum moisture content. Some soils excavated from below the groundwater level may require drying prior to compaction. Although the dune sand is uniform, it has been successfully used for the formation of embankments in the Kāpiti District for subdivisions as well as transportation projects. Local contractors have good experience in the placement and compaction of these sands. Compactions trials were carried out during development of the Kāpiti Western Link Road project which confirmed that the dune sand materials can be effectively compacted (Opus 2008).

9.4 Flood Retention Embankments

Some of the Expressway embankments will also retain flood water in the event of storms and flooding from the rivers. These embankments will be designed so that they can retain the floodwater over a short period of time during flood events, without internal erosion and significant seepage. These embankments may require a low permeability section to minimise seepage flows.

Some embankments may cross flood plains and will need to be designed to allow water flow through pipes / culverts provided in the embankments to allow flow across the Expressway alignment in flood events. This will require the selection of appropriate embankment fill materials and detailing of the embankment - pipe interface to prevent erosion of the embankment in flood events. This will be carried out during detailed design.

9.5 Embankment Foundations

Embankments in terrace alluvium areas

In the terrace alluvium areas, topsoil and/or weak materials of 0.2 m to 0.3 m thickness are generally present over dense to very dense gravel, cobbles and boulders. The groundwater table was observed to be about 5m or more lower than the embankment foundation level in these areas.

Based on the interpreted ground conditions, undercut of 0.2 m to 0.3 m of topsoil or weak materials is likely to be required where embankments are founded on the terrace alluvium.

Embankments in floodplain alluvium areas

Floodplain alluvium generally consists of a top layer of weak materials such as loose to medium dense sand/gravel and soft to firm silt/clay. The thickness of this top weak layer varies and generally ranges from 0.3 m up to 4 m. The groundwater table is generally dependent on the level of the nearby watercourse levels.

Undercut of generally up to 0.3 m of weak alluvial materials is likely to be required where embankments are founded on the floodplain alluvium. Undercut of 1m to 3m and special measures may be required at some locations where there are concerns of slope instability and substantial settlement of the embankment due to the presence of weak foundation materials of considerable thickness as discussed in Section 11.

Embankments in interdunal deposit areas

Some of the fill embankments are located in the low lying interdunal areas underlain by soft and compressible peat, silt and clay of about 0.5m and 4. m thickness over dense to very dense sand. The groundwater table is generally close to the ground surface in the interdunal areas.

The following factors have been taken into account when considering the amount of undercut and other ground improvement measures for fill embankments in interdunal areas:

- cost effectiveness - this relates to the cost involving undercutting and replacement of competent fill and other measures such as preloading and soil mixing;

- stability and permanent earthquake-induced displacements of the fill embankments; and
- settlement of the fill embankment due to consolidation and decomposition of organic peat materials.

The distribution and thickness of inter-dunal deposits below fill embankments are summarised in Table 6. Comparison and discussion of each type of ground improvement measures such as excavation of inter-dunal deposits, preloading, and monitoring instrumentation is covered in Section 11.

Table 6 - Embankments in Inter-dunal Areas

Location	Approximate Station	Embankment Height	Indicative Thickness of soft soils
North of County Road	1450 - 1700 m	Up to 3 m	Up to 3 m
South of Mary Crest	10500 - 11550 m	Up to 8 m	Up to 3 m to 4.5 m
South of Mary Crest	11550 - 12200 m	Up to 1 m	1 m to 2 m

9.6 Embankment Stability

Fill embankment stability is largely dependent on the following factors:

- height of the embankment;
- side slope of embankment;
- strength of compacted embankment fill materials; and
- strength of founding material on which the fill is placed.

Embankments have been checked for long term stability with a design factor of safety against failure of 1.5 under static conditions. During large earthquake events, some slope displacements (say 100 mm to 200 mm) are considered acceptable, but complete failure of the road carriageways will not occur. This is acceptable because such small displacements will not restrict access and can be quickly reinstated after such major events.

The analysis indicates that fill slopes of 26° (2H:1V) are appropriate, assuming that any soft materials underneath the embankments are undercut and replaced with competent materials. Specific stability analysis will be carried out once there is better knowledge of the founding material properties to confirm the slope angle, amount of undercut and whether side berms are required to provide buttress. A typical fill slope stability analysis is shown in Illustration 15.

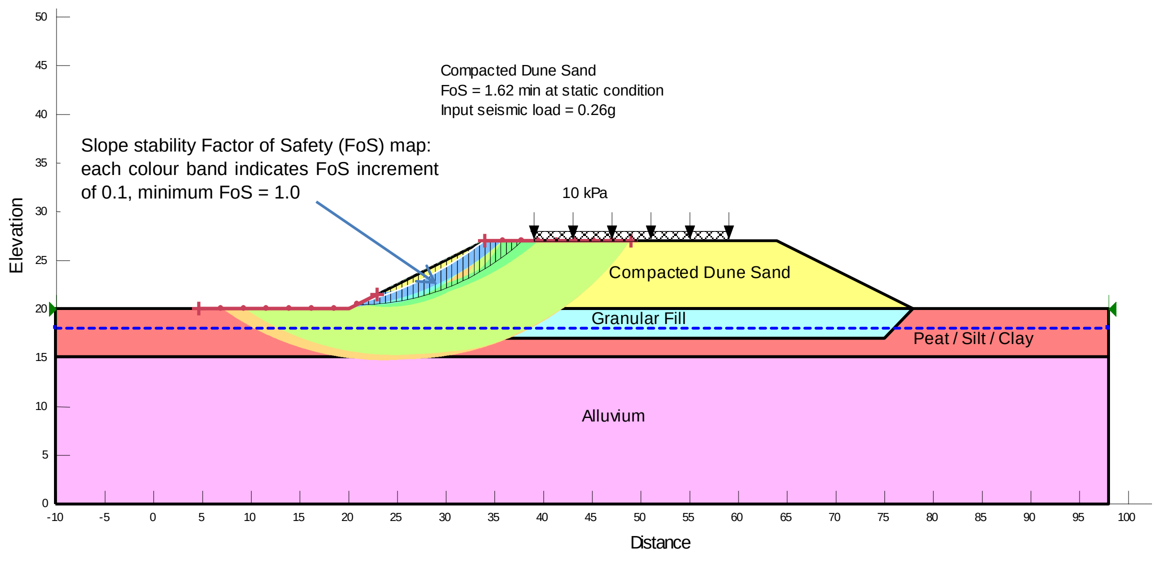


Illustration 15 - Typical Stability Analysis of Fill Slopes

9.7 Drainage

The following measures will be used to minimise the risk of embankment instability and slope erosion due to surface water and groundwater:

- construction of swale drains on the embankment will be avoided. Swales will be formed outside the toe of the embankments;
- drainage blankets wrapped in geotextile with outlet sub-soil drains and geogrid reinforcement will be provided below fill embankments in inter-dunal areas where soft compressible deposits are present;
- subsoil drains will be installed at the interface between new fill and existing slopes to collect and dispose of potential seepages in wet winter conditions; and
- toe protection will be considered during detailed design to protect embankments that are proposed in a gully or valley with a stream.

9.8 Landscaping and Vegetation

The fill embankment slopes will be vegetated during, or as soon as possible after, construction. Where the embankment construction happens over a period of time, revegetation of sections of partially completed embankments may be required before the full height of the embankment is completed. Due to the erosion-prone nature of dune sand, embankments formed by dune sand may need to be protected by geotextile or geo-membrane until the vegetation is established. Peat excavated from the low lying areas can be used to providing an organic layer to help vegetation. Grass or small plants would provide protection to the slope surface against erosion but not large trees, which could destabilise the slope. The type of vegetation will be selected jointly by the geotechnical engineer and the landscape architect to suit the local environment.

10 Bridges

10.1 Proposed Bridges

New bridges are proposed at the following locations, from north to south:

- Waitohu Stream bridge.
- Ōtaki ramp bridge across the NIMT railway.
- Ōtaki ramp bridge across the proposed Expressway.
- Overbridge over the NIMT railway and the proposed expressway at Rahui Road.
- Ōtaki River bridge (twin bridges).
- Overbridge across the proposed Expressway at Ōtaki Gorge Road.
- Overbridge across the existing rail corridor at Ōtaki Gorge Road.
- Overbridge over the existing SH1, NIMT railway and the Expressway at Te Horo.
- Bridges at Mangaone Stream.
- Mary Crest Underpass across the existing state highway and NIMT railway line.

10.2 Ground Conditions at the Bridge Sites

The ground conditions at each of the bridge sites have been assessed based on the results of the site investigations, and are summarised in Table 7.

Geotechnical investigations have not been carried out at every bridge site, and therefore the ground conditions have been estimated based on the geology and the investigations to characterise the different geological units encountered along the proposed expressway route. Further geotechnical investigations will be carried out before the commencement of design of the structures along the Expressway.

Table 7 - Ground Conditions at Bridge Sites

Bridge Location	Geological Unit	Comments on Ground Conditions
Waitohu Stream Bridge	▪ Recent (floodplain) alluvium	▪ Mainly silty gravel and cobbles with interbeded sand and clay layers. ▪ Medium dense at the top 1 m to 2 m; becoming denser with depth.
Ōtaki ramp Bridge (NIMT railway)	▪ Dune sand	▪ No investigations carried out at bridge site.
Ōtaki Ramp Bridge (expressway)	▪ Inter-dunal deposits / terrace alluvium at the eastern side ▪ Dune sand at the western side	▪ Soft to firm organic silt encountered at up to 1.5 m depth overlying silty gravel / cobble and sand at the eastern end.
Rahui Road Overbridge	▪ Recent (floodplain) alluvium	▪ Dense to very gravel with interbedded sand and silty clay layers.
Ōtaki River Bridge	▪ Recent (floodplain) alluvium	▪ Silt and medium dense to dense sandy gravel and cobbles to 2 m depth; gravel

Bridge Location	Geological Unit	Comments on Ground Conditions
		and cobbles becoming denser at depth. ▪ Interbedded sand/silt/clay layers.
Ōtaki Gorge Road Expressway Overbridge	▪ Terrace alluvium	▪ Very dense gravel and cobbles in a sandy matrix.
Ōtaki Gorge Road Railway Overbridge	▪ Terrace alluvium	▪ No investigations carried out at bridge site.
Te Horo Overbridge	▪ Terrace alluvium	▪ Organic gravelly silt to 1 m depth; overlying very dense gravel and cobbles.
Bridges at Mangaone Stream	▪ Recent (floodplain) alluvium	▪ No investigations carried out at bridge sites.
Mary Crest Underpass	▪ Terrace alluvium at eastern abutment ▪ Dune sand at western abutment	▪ No investigations carried out at bridge site. ▪ Thicker weak deposits may be encountered along the western side of the Mary Crest Underpass.

10.3 Types of Abutments and Foundations

Given the ground conditions at the bridge sites, large diameter bored pile foundations and reinforced soil wall (RSW) abutment walls are generally considered to be appropriate for most of the proposed bridges. Spill-through abutments are also appropriate where space is available.

Large diameter bored piles are considered to be appropriate because of their:

- simple form of foundation which would not require complex pile caps close to the live road and railway environment and is easy to construct even in a river environment;
- robustness against lateral displacement during earthquake events;
- ability to provide high end bearing capacity in very dense gravel and cobbles;
- suitability to be installed through dense layers to the required depths to provide adequate lateral capacity;
- ability to be advanced through very dense gravel and cobbles, on the other hand, the gravel, cobbles and boulders would possibly retard the penetration of driven piles.
- inherent lower noise during construction, compared to driven piles.

Driven piles are also considered suitable in a dune sand environment which generally permits easy penetration and provides sufficient strength.

Shallow foundations such as spread footings have the fundamental advantages of simplicity and low cost. Nevertheless the tolerance against vertical and lateral displacements of bridges supported on spread-footing would need to be checked if this type of foundations is considered.

Shallow foundations may be considered in terrace alluvium which is not prone to liquefaction or earthquake deformation. The use of shallow foundations should however be avoided where soil scour is a design consideration such as for river-crossing bridges.

10.4 Design Considerations

The following design issues will be taken into consideration in the detailed design of bridge foundations and abutments:

- any weak or liquefiable materials will be removed and replaced by compacted fill materials before construction of the bridge abutments. In general, the amount of undercut at the bridge abutments along the alignment is about 1 m to 2 m. A higher depth of undercut may be required at certain locations such as the western abutment of the Mary Crest Underpass;
- the type of foundations will need to be carefully chosen and the design of foundations carried out to ensure adequate load capacity and tolerance against both lateral and vertical displacements; and in some cases the allowance for scour effects;
- allowance of additional pile length will need to be made in the design due to the presence of intermediate weak lenses of silt/clay/sand layers, especially in an alluvial environment;
- the design of pile foundations in river channels will need to allow for the depth of general and local scour which could be a significant depth in the large rivers such as the Ōtaki River;
- the effects of negative skin friction due to liquefaction will need to be considered in the design of pier foundation where liquefiable materials may be present in the top few metres of the site soils; and
- piles / columns constructed through reinforced soil wall abutments or approach fill will need to be sleeved to minimise the effects of earthquake-induced displacement of the reinforced soil wall or approach fill to the piles and the bridge structure.

10.5 Liquefaction Potential

Liquefaction of saturated soils such as loose sand, gravel and silt could cause differential settlement of the abutments, instability of the abutment walls, lateral spreading of the abutment slopes, negative skin friction on the abutment and pier piles and, ultimately, may affect the bridge structures.

Earthquake induced liquefaction at the bridge abutments can be avoided at most locations along the route by removing the upper 1 m to 2 m thickness of loose or soft soils before construction of the abutments. The groundwater level at the Mary Crest Underpass was found to be at about 8 m below ground level, therefore we consider that liquefaction is unlikely to occur although firm silt / clay and medium dense gravels were encountered to 5 m depth. It is possible that detailed site investigations may indicate some liquefaction hazard at some bridge abutment sites.

If detailed investigations identify liquefaction susceptible soils, ground improvement such as stone columns can be constructed to minimise adverse effects on bridge abutments.

At the Ōtaki River bridges and the Waitohu Stream bridge, although there could be a potential for liquefaction of some loose liquefaction susceptible layers at the bridge piers, given the flat river bed topography at the pier locations, lateral spreading is unlikely to occur and the localised liquefaction and consequent subsidence is unlikely to be critical for the structure.

The induced negative skin friction on the pier piles due to liquefaction would need to be considered in the design of the bridge structure. However, the negative skin friction is not expected to have a significant effect on the structure if large diameter bored piles are founded in dense gravels.

10.6 Active Fault Crossing

The present road form shows that the Te Horo Overbridge is about 75 m away from the Northern Ohariu Fault trace and does not cross the fault trace. It would be prudent to confirm that any subsidiary fault traces do not cross the bridge site by trenching, prior to detailed design.

11 Ground Improvement

11.1 Ground Improvement Solutions

Ground improvement will be required for foundations of fill embankments and other structures such as pavements, bridges and culverts to avoid adverse effects of settlement, instability or liquefaction and consequent lateral spreading.

Based on the site conditions and properties of the site foundation materials and the proposed road form, several types of ground improvement together with some other engineering solutions are considered to be appropriate and are summarised in Table 8. Ground improvement is carried out under the structure, and is not expected to require any additional area beyond the proposed designation boundary.

Table 8 - Ground Improvement & Other Engineering Solutions to Poor Ground Conditions

Poor Ground Conditions	Possible Consequences if not Treated Properly	Engineering Solutions
Unsuitable materials (such as wet, weak, compressible, or organic soils) under fill embankments and pavements	▪ Failure of embankment ▪ Differential settlement and cracking of pavement	<u>Ground Improvement</u> ▪ Undercut / excavate the unsuitable materials and replace with competent fill. ▪ Preload the compressible soils, with wick drains. ▪ Soil mixing using lime or other cement materials. <u>Other Engineering Solutions</u> ▪ Install geogrid below the embankment / pavement.
Liquefiable materials (such as loose silt, sand, or gravels below groundwater table) under bridge abutments and high embankments	▪ Failure or lateral spreading of embankment / abutment slopes ▪ Additional load on bridge foundation due to negative skin friction ▪ Differential settlement and cracking of pavement and culverts	<u>Ground Improvement</u> ▪ Undercut / excavate the unsuitable materials and replace with competent fill ▪ Solutions such as vibro-compaction, vibro-replacement, stone columns <u>Other Engineering Solutions</u> ▪ Stronger bridge foundation to resist the additional load.

11.2 Undercutting

Undercut of weak and compressible soils has been allowed for preparing the foundations below fill embankments, bridge abutments, culverts and pavements. The thickness of undercut will generally be up to 0.3 m and is expected to be more at the floodplain alluvium close to the water courses and in the inter-dunal peat areas, as discussed in Section 11.6.

11.3 Complete Excavation and Replacement

Some of the fill embankments and bridge abutments are in areas underlain by weak, compressible, or liquefiable alluvial or inter-dunal deposits comprising peat, soft silt and clay of considerable thickness (generally up to 3 m but can be locally up to 5 m).

Complete excavation of the unsuitable materials and replacing them with competent fill would eliminate the risk of liquefaction and can avoid long term issues such as ongoing settlement of embankments due to consolidation of compressible materials and decomposition of organic peat. Settlements usually cause road deformation and result in poor road quality and high road maintenance costs. Therefore, complete removal of the unsuitable materials would reduce long term maintenance costs. However, the cost of excavation would increase with depth. It would usually become difficult for depths greater than 3 m, where careful control of groundwater and lateral support is likely to be required. Partial excavation and pre-loading may be better in such areas as outlined in the section below.

11.4 Partial Excavation and Preloading

General

Another approach, rather than complete excavation, is to excavate and remove part of the unsuitable materials and preload the remaining compressible soils. Preloading can accelerate the process of consolidation and reduce the amount of post-construction settlement to an acceptable level. Additional drainage such as wick drains would help accelerate the settlement during the preload period. This approach can be cost effective especially for cases where deep excavation is required; however, the remaining peat and compressible materials will cause on-going settlements. A balance of initial cost, maintenance cost and accepted level of road performance will be considered carefully to determine the depth of undercut and preloading.

In some situations, the presence of remaining peat or weak materials will increase the risk of embankment slope instability. Solutions such as berms/fill buttresses at the embankment toes or flattening the fill slopes can minimise the risk of instability. Compaction of fill over compressible materials can be difficult and, therefore, the methodology of fill compaction would need to be considered carefully.

Preloading

Based on the thickness of compressible materials and the proposed road form, it is estimated that preloading using a surcharge (of additional soil weight) would be required for a period of about 6 to 9 months. Typical preloading with surcharge is shown on Illustration 16. Installation of wick drains will be considered during detailed design to facilitate dissipation of porewater pressures and accelerate consolidation. The progress of preloading will be monitored by suitable instrumentation.

Where preloading is adjacent to the existing SH1 or other facilities, it is likely to cause differential settlement of the existing carriageways. Minor road repair may be required to maintain the performance of the road carriageways during construction and the pavement may need to be reinstated on completion of preloading and construction in this area.

Illustration 16 - Preloading adjacent to State Highway

11.5 Other Ground Improvement Methods

Other ground improvement methods, such as soil mixing using lime or other cement materials, vibro-compaction/replacement and stone columns, may be suitable options if unforeseen ground conditions such as substantially thick unsuitable materials are encountered after further site investigations or during construction.

11.6 Recommended Ground Improvements

At this stage, we recommend excavating any unsuitable materials up to 3 m depth and treating any remaining compressible soft materials by preloading. Other ground improvement methods as mentioned in Section 11.5 may be required if more difficult ground conditions are encountered.

The thickness of undercuts will typically be 0.3 m along the route. At certain embankments and bridge abutment locations, thicker undercuts are expected, see Table 9.

Table 9 - Thickness of Undercut

Location	Proposed Structures	Thickness of Undercut (m)
North of County Road (Sta. 1450 – 1700 m)	▪ Embankments (up to 3 m high)	▪ Up to 3 m
South of Mary Crest (10500 – 11550 m)	▪ Embankments (up to 8 m high)	▪ Up to 3 m (Preloading is likely to be required)
South of Mary Crest (11550 – 12200 m)	▪ Embankments (up to 1 m high)	▪ 1 m to 2 m
Various bridge locations (refer to Plan Set in Volume 5 - Figures GM01 to GM08)	▪ Bridge abutments	▪ 1 m to 2 m
Mary Crest Underpass western abutment	▪ Bridge abutment	▪ Potentially undercut of greater than 2 m thick may be required.

11.7 Instrumentation and Monitoring

Types of monitoring instrumentation

The performance of the new fill embankments and progress of preloading will be monitored during and after construction in order to determine the preload period and to detect any signs of distress, settlement or lateral movement.

Monitoring instrumentation listed in Table 10 will be installed at the locations of new fill embankments and the preload areas. Monitoring will be carried out on a regular basis during and for a period after construction.

Settlement stations will be used to gauge the total settlement of fill and the foundation soils. Deep settlement plates and vibrating wire piezometers are required to monitor the settlement and porewater pressures of the foundation soils during preloading. These will be used in determining the duration the surcharge weight of soil needs to be in place as part of the preloading. Shear probes and inclinometers will be used to monitor lateral movement of large embankments which may be caused by lateral deformation of the compressible materials or potential instability of the embankment.

Settlement stations and deep settlement plates will be installed at intervals along the preload area and shear probes will be installed at high embankments over thick compressible soils.

Illustration 17 shows typical details of instrumentation at the preload areas.

The results and frequency of monitoring will be reviewed by a geotechnical engineer during the period of monitoring. If the monitoring survey indicates potential distress then appropriate corrective actions will be taken.

Table 10 - Instrumentation and Monitoring

Instrumentation	Type of Monitoring
Settlement stations	Total settlement
Deep settlement plates	Foundation settlement
Piezometers (vibrating wire type)	Porewater pressures
Shear probes	Detection of embankment deformation
Inclinometers	Measurement of embankment deformation

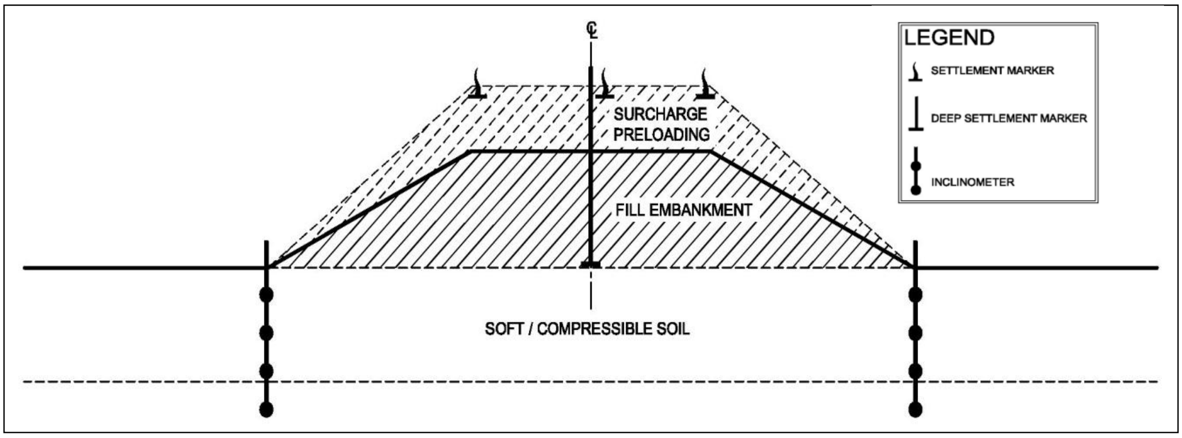


Illustration 17 - Typical Details of Instrumentation at Preload Areas

11.8 Reuse of Excavated Peat

The excavated peat can be used as non-structural fill such as to form noise bunds or gently-sloping shoulders over fill embankments where settlement is not an issue. Trial vegetation of peat material may be carried out during construction to determine the effectiveness of the peat to facilitate grassing.

It is proposed that the peat material be spread out and drained to reduce the moisture content after excavation. Once the moisture content is reduced (the level of moisture content reduction for the purpose of construction should be determined at site), the peat can be placed in shoulders and bunds or spread on slopes. Consideration may be given to mixing peat with sand to improve material performance.

12 Environmental Effects

12.1 Groundwater and Aquifers

12.1.1 Groundwater Conditions

Standpipe piezometers were installed in boreholes and CPT holes, and groundwater levels have been monitored (see Plan Set in Volume 5 - Figures GI01 to GI08 - for locations of the piezometers). They generally show the groundwater levels in the shallow unconfined aquifers along the road route. The measured groundwater levels so far are summarised as follows.

Interdunal areas

Groundwater level was measured to be at shallow depths of up to 2 m in the inter-dunal areas. Groundwater seepage was commonly observed at 1 m to 2 m depths within layers of inter-dunal swamp deposits during excavation of trial pits in the dry summer conditions. Some low-lying inter-dunal areas are commonly water-logged with standing water during the wet winter periods.

Dune sand areas

Groundwater levels in the sand dune areas are generally dictated by the groundwater conditions at the inter-dunal depressions; thus the sand dunes are generally dry with reduced groundwater levels similar to that in the adjacent inter-dunal areas.

Floodplain alluvial deposit areas

The groundwater levels within the alluvial floodplain deposits are generally determined by the level of the adjacent watercourses such as rivers and streams, and are typically at 2 m to 5 m depth below ground level.

Terrace alluvial deposit areas

Groundwater levels were found to be fluctuating between 5 m to 10 m below surface in the alluvial terrace deposits. The groundwater at the terrace south of Ōtaki River is likely influenced by the fluctuation of the river levels.

12.1.2 Potential Groundwater Drawdown

Permanent Drawdown due to Cuttings

The potential of groundwater drawdown due to formation of cuttings has been reviewed. It is assessed that the excavations in dune sand areas (Station 1,000 m to 1,750 m and Station 10,250 m to 10,600 m) will generally be formed above the measured groundwater level and thus would have negligible localised effects on the groundwater table. For excavations within terrace alluvium to the south of Ōtaki River (Station 3900 m to 5300 m), the measured highest groundwater level in the area is about 1 m above the excavation level. This implies that groundwater table could potentially be drawn down up to 2 m along a short section between Stations 4,100 m and 4,400 m due to the installation of sub-horizontal drains at the cut slopes and sub-soil drains below the road pavement.

This drawdown will be in an unconfined aquifer with low storage and hence low potential for water yield given the silty sand bound gravel formation as can be seen in Trial Pit 111 (AECOM, 2011). This has not warranted a more detailed study because this drawdown has negligible consequences as discussed in Sections 12.1.3, 12.1.4 and 12.2.1.

Temporary Drawdown due to Undercutting

Undercutting in the order of 3 m depth is expected at some of the wet inter-dunal areas. These areas will be backfilled with well compacted materials. During excavation, groundwater in the excavation will be drained temporarily by sumps or drains and thus, the groundwater will be temporarily drawn down for a period of a few days to a few weeks. This will lead to localised draw down up to about 3 m depth below ground surface, in the road corridor and the inter-dunal low lying paddocks. The zone of effect is based on the limited extent of the inter-dunal areas that will be affected by such drawdown of groundwater and the experience of the author in similar earthworks projects requiring undercut. This has not warranted a more detailed assessment given the negligible consequences of such groundwater drawdown as discussed in Sections 12.1.3, 12.1.4 and 12.2.1.

The effect on wetlands is discussed in Section 12.3.

12.1.3 Effects on Groundwater Flow and Direction

Groundwater in the Project area generally flows from the east towards the sea coast. Since the permanent groundwater drawdown as discussed above is local (at some locations between Station 3900 m and 5300 m) and of relatively small order (up to 2 m depth), its effects on groundwater flow and direction are considered to be insignificant. Temporary groundwater drawdown would have negligible influence on the long-term groundwater flow and direction.

12.1.4 Effects on Groundwater Use

The distribution of the latest consented water takes (including a water take for the Arcus Road Water Scheme) within 1 km from the Expressway has been obtained from GWRC, and are shown in the Plan Set in Section 5 (Figures GI01 to GI08). The water takes are generally located in the following areas:

- north of Waitohu River;
- in the vicinity of Ōtaki River;
- sparsely scattered along the State Highway from south of Ōtaki River to Te Horo; and
- south of Peka Peka.

The long term effects of the Expressway to the consented water takes have been reviewed. The Expressway drawdown affects the shallow groundwater less than 10 m below the ground surface, whereas the groundwater extraction from the bores in the vicinity of the Expressway cuttings is generally from greater depths.

The closest extraction well from the proposed cuttings in terrace alluvium just south of the Ōtaki River are indicated in the Greater Wellington database to be located 15 m away from proposed cutting and used for extraction of water as part of the Arcus Irrigation works. This is located on the high terrace with a ground surface reduced level of about 22 m. The indicated depth of the well is 10 m, and if located on the high terrace will be just below the groundwater level indicated by the boreholes, and being in the terrace alluvium is not likely to yield much water. Local discussions indicate that the well from which water is pumped for the Arcus irrigation scheme is actually located further towards the river with a ground surface reduced level of about 12 m, and some 300 m away from the proposed cuttings for the Expressway, see Illustration 18. At this location the well will be located within the recent alluvium associated with the Ōtaki River flood plain, and will draw water from the river gravels, and indirectly from the river. Therefore, the up to 2 m groundwater drawdown at the cuttings in the lower permeability terrace gravels, at a distance of 300 m from the well, and from an elevation significantly above the well used for water abstraction is likely to have a negligible, if any, effect on the yield in the licenced groundwater bores.

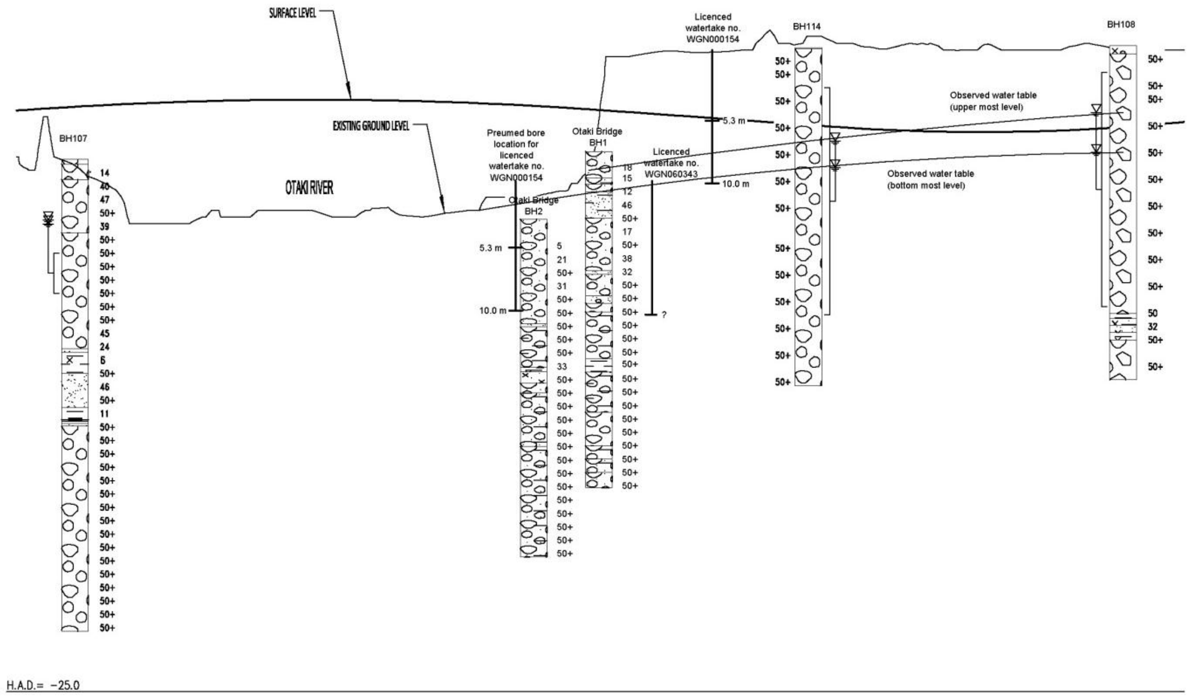


Illustration 18 – Location of Well for Extraction of Water for the Arcus Irrigation Scheme in relation to Expressway and Groundwater Drawdown

Greater Wellington Regional Council has requirements that all wells should have consents – bore permits for construction and water permits for abstraction. Greater Wellington Regional Council indicates that there are many permitted takes which do not require resource consent, and these bores are likely to be shallow. Given that these may be permitted takes, and the poor accuracy of the location of these bores as discussed above, it is recommended that these bores within 500 m of the cutting (Station 3900 m to 5300 m) be surveyed and their depth ascertained by inquiry prior to construction. There could be about 20 unconsented bores within this area, but many of these bores are deep (~ 60 m) and are not likely to be affected by the 1 m to 2 m drawdown. Using this information, a list of shallow bores within proximity of the drawdown should be compiled and monitored during construction to assess any effects on these bores. In the unlikely event that any bores that have permitted or consented takes are affected, they could be replaced with deeper bores that would provide the same water yield.

As for the potential temporary groundwater drawdown of about 3 m depth at some of the inter-dunal areas, the closest groundwater extraction wells are located outside the zone of influence of the proposed undercut areas for removal of the inter-dunal peat. Therefore the effects to the existing abstraction wells are insignificant.

The groundwater level and pressure will be monitored during construction to ensure there is no unexpected water drawdown that will affect water abstraction in the vicinity.

12.1.5 Water Abstraction for Construction

It is estimated that about 300 cu.m / day (300,000 litres / day) of water will be required during construction for activities such as dust suppression and fill compaction. The water is proposed to be sourced from groundwater sources.

Such wells will be located at least 250 m away from existing groundwater abstraction wells. This is to avoid any adverse effects on the existing abstraction wells. The proposed locations of the groundwater abstraction wells are south of Mary Crest, in the vicinity of the proposed Te Horo overbridge near Mangaone Stream, north of the Ōtaki River, and south of the Waitohu Stream, see Illustrations 19, 20, 21 and 22 where the locations are shown in blue. Bore permits for construction and water permits for abstraction are sought for these abstraction wells.

It is also proposed that the consented bores to be constructed to provide water for construction of the MacKays to Peka Peka project, be also used to provide construction water at the south end of this Project.

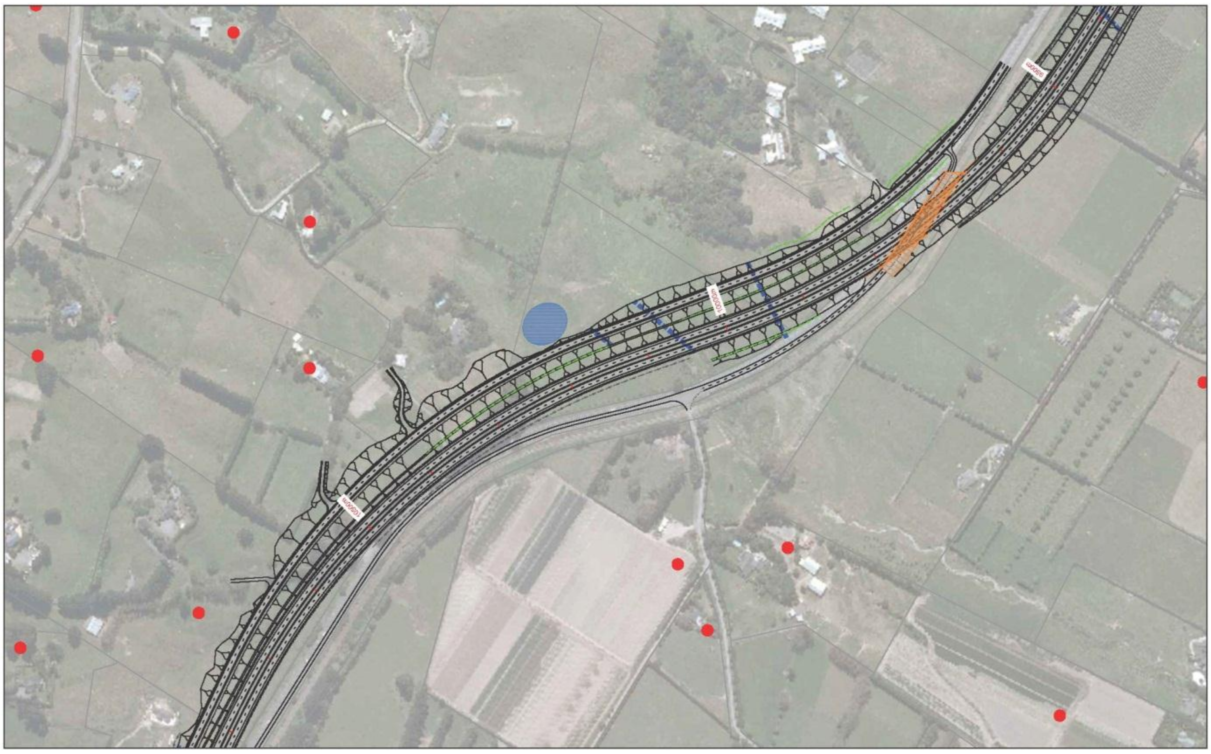


Illustration 19 – Proposed location for Water Abstraction Well south of Mary Crest

Illustration 20 – Proposed location for Water Abstraction Well opposite Te Horo Beach Road

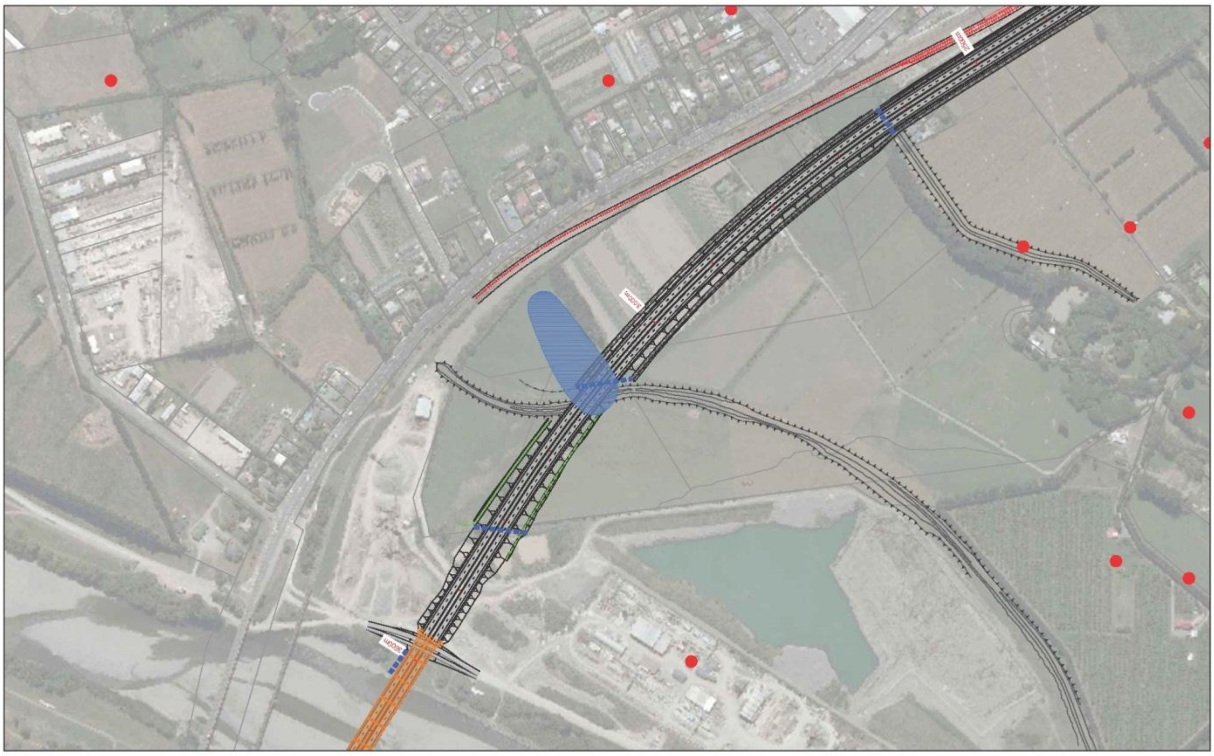


Illustration 21 – Proposed location for Water Abstraction Well north of Ōtaki River

Illustration 22 – Proposed location for Water Abstraction Well near Waitohu Stream

12.2 Ground Settlement

Ground settlement can potentially be caused by consolidation of ground due to groundwater drawdown and construction of fill embankments on compressible ground.

12.2.1 Consolidation of ground due to groundwater drawdown

Since the permanent groundwater drawdown at the cutting areas is local and of small order (about 1 m to 2 m), and the ground at these areas (dense terrace alluvium) has low compressibility, the resultant ground settlement is likely to be negligible.

Temporary excavation and dewatering at some of the interdunal areas could potentially cause groundwater drawdown to about 3 m depth. Because the groundwater drawdown is temporary, and for no more than a few days to few weeks at a time, the ground settlement during this period will be small. Also these areas are generally farming areas with no significant buildings in the vicinity and therefore any small ground settlement will have no adverse effects on the environment.

12.2.2 Consolidation of ground due to fill embankments

Soft and compressible materials such as peat in inter-dunal areas will be removed / undercut either completely or as much as practicable before placing of fill. There are some areas along the route, e.g. south of Mary Crest, where soft materials of more than 3 m thickness are present and it becomes costly to completely undercut. In such instances partial undercutting to 3 m depth followed by preloading is proposed, with extra surcharge to accelerate consolidation during construction. However, secondary consolidation is expected to continue even after construction. This will result in ground settlement within the road footprint and a short distance (about 10 m) in the surrounding area.

The adjacent existing state highway and railway in the area south of Mary Crest could potentially be affected by ground consolidation. The settlement of the embankment as well as the state highway and railway will be monitored during construction, and the state highway reinstated and the railway line re-levelled using ballast, as necessary, see section on mitigation below.

Interdunal deposits are also present north of Rahui Road. Current investigations show that the soft materials are up to 3 m thickness and thus, can be removed completely before placing of fill. However, should the soft materials be deeper and some soft materials remain below a nominal 3 m depth of undercut, ground settlement due to construction of embankment on soft ground will occur.

For locations where there is a potential for ground settlement e.g. south of Mary Crest and possibly north of Rahui Road, there are no buildings in the surrounding area that may be affected by such settlements and therefore the risk is low. However, settlement will be monitored to ensure that post-construction settlements are low, and also to manage the effects on adjacent facilities, especially state highway 1 and the NIMT railway line south of Mary Crest.

Mitigation

Ground settlement will be closely monitored by deep seated settlement plates during and after preloading period to assess the effect to settlement of the adjacent ground, especially the state highway. Any development of cracks on the seal should be recorded and monitored.

Minor road repair such as sealing of cracks may be required during construction and the pavement may need to be reinstated on completion of the preloading and construction. Any settlement of the railway line will need to be rectified by re-levelling the railway tracks using ballast, where the tolerances are exceeded.

12.3 Wetlands

The groundwater level near the existing Ōtaki Railway Wetland will be temporarily lowered during construction to facilitate undercut and removal of the peat and other soft deposits to enable construction of embankments for the Expressway. The Technical Report on Terrestrial Ecology describes how this wetland is likely to have been formed by poor drainage through the existing railway and road embankments and its limited quality. The report also identifies that this wetland will be largely affected by construction of the Expressway. Moreover, the wetland has already been highly modified and it will suffer significant on-going degradation due to the highly modified nature of the catchment and its small size. A small narrow wetland strip adjacent to the railway wetland and another one to the west of County Road will be created. Again the temporary groundwater drawdown should not have a significant effect on the long term viability of these two new wetlands. Not ensure that these wetlands are not drained by potentially permeable soils used construction of the new embankments, it is proposed that a zone of low permeability soil be placed upstream, between the embankment and the wetlands. The low permeability peat and clayey silt excavated as part of the undercut for the embankments can be used for this purpose.

Another new area of wetland will be created adjacent to the Mary Crest bush. Low embankments will be constructed adjacent to this proposed wetland. The groundwater level in the area will only be lowered temporarily during construction and this will not affect the long term viability of the proposed wetland to be created at this location after the road embankment is constructed. Therefore the temporary groundwater drawdown during construction is not relevant to the proposed Mary Crest wetland due to be formed after construction of the Expressway.

Because the ground level falls to the west where the water course currently drains this area, a low embankment is likely to be required with weir to ensure water is retained in the wetland area and surplus water can overflow through the weir during wet weather conditions.

12.4 Effect of Bridge Foundation Works on the Environment

The latest bridge scheme design shows that bridges will generally be founded on bored piled foundations and abutments on either reinforced soil walls or spill through embankments.

Some potential effects due to piling works are:

- Ground settlement could be caused by vibration and consolidation of the ground surrounding the piles when pile casings are driven through loose soils (eg sand). There are no buildings adjacent to the proposed bridges and, therefore, effects are unlikely.
- Spilling of concrete and grout leading to contamination of watercourses. Adequate care should be taken when constructing piles, particularly adjacent to waterways to prevent any spillage that could contaminate waterways.
- Change of aquifer system, such as piling into a confined aquifer, resulting in the pressurized artesian water flowing through the aquifer(s) above. The site investigations have not shown the presence of confined or artesian aquifers that may be affected within the depth of the proposed piles. Therefore, such risk to the groundwater systems is low. This will be confirmed during detailed design.

Mitigation

The following mitigations are appropriate and are recommended:

- Site investigation results did not show significant thick layers of loose soil that could result in settlement due to vibration. However, ground settlement should be carefully monitored when piling adjacent to the existing railway and state highway;
- Care must be taken during pile construction particularly adjacent to waterways to prevent spillage and contamination; and
- The presence of artesian groundwater within the depth of piles will be confirmed during detailed design and construction. Any artesian water encountered should be managed through raised casing above ground level. The holes should also be double-cased to minimise the risk of leakage along the pile casing, should investigations during detailed design indicate that piling into artesian aquifers may occur. Artesian water can be expected at the Ōtaki River at about 35 m depth, but the piles are not likely to extend below this depth.

12.5 Erosion Control at New Cut Slopes

The newly-formed cuttings in the sand dune areas are susceptible to erosion. Adequate measures are required to protect the cuttings from erosion. Appropriate measures include:

- re-vegetation as soon as possible after formation of cut, and maintenance of the vegetation during the early stages after construction. The type of vegetation should be carefully selected to suit the local coastal dune sand environment;
- installing erosion protection geotextile mats; and
- placing topsoil or peat on the slope surface.

With appropriate management and measures, the effects of proposed excavations on sand dune erosion can be kept to minimum and adverse effects are unlikely.

13 Route Security

13.1 The Regional Context

State Highways 1 and 2 provide access into the Wellington Region along the western corridor from the north, and from the eastern corridor from the northeast. A regional road resilience study (Opus, 2012) has assessed the resilience of state highway and local arterial roads in the region. The study indicates that sections of State Highway 2 are highly vulnerable to closure from landslides and retaining wall failures through the Rimutaka Hills, particularly in major earthquake or storm events. Sections of State Highway 1 are also vulnerable, particularly between Pukerua Bay and Paekakariki, and Porirua and Paremata. These sections will be bypassed by the proposed Transmission Gully expressway which forms part of the Northern Roads of National Significance, and has been developed to enhance resilience of that section. It is important that other sections of the RONS also provide resilient access into Wellington.

The Peka Peka to Ōtaki section of the current state highway is vulnerable to disruption of access at the Ōtaki River Bridge in a large earthquake event, and liquefaction and lateral spreading along some sections built on inter-dunal peat and sand deposits.

13.2 Route Security Philosophy

A route security philosophy for the Expressway is proposed as follows:

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

(b) the Expressway suffers limited repairable damage in moderate hazard events, with continued limited access; and

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

13.3 Liquefaction Hazard

The regional liquefaction hazard maps shows that the route crosses an area of moderate liquefaction hazard south of Mary Crest, and an area of variable liquefaction hazard north of the Ōtaki River.

The geotechnical investigations show that there is potential for some liquefaction within the inter-dunal swamp deposits but given that this will be substantially or fully removed by undercut and replaced with well compacted fill, and the underlying sands are dense, the potential for damage to the Expressway due to liquefaction will be limited to minor subsidence and associated cracking in a large earthquake. The Expressway is likely to continue to provide access although with some difficulty until this can be repaired. Repairs can be achieved within a few days to 2 weeks.

The abutments associated with bridge structures will be protected by removal of any near surface liquefiable materials by undercut, and if necessary ground improvement if any deeper liquefiable deposits are encountered. The bridge structure itself will be supported by piles extending below any liquefiable layers.

The liquefaction hazard maps also shows a variable potential for liquefaction in the area north of the Ōtaki River in the recent alluvial deposits associated with the Ōtaki River and Waitohu Stream. Geotechnical investigations indicated that these deposits are largely coarse gravels, cobbles and boulders with only localised limited layers of sand that may be susceptible to liquefaction. With the proposed undercutting of the shallow loose layers, any liquefaction of the isolated loose layers is not likely to cause significant damage, but perhaps only minor cracking of the road surface. The Expressway is likely to remain open and provide access in a large earthquake.

13.4 Earthquake Induced Slope Failures

The hazard from slope failures is low. While limited shallow failures may occur in the cuttings through terrace alluvium (Station 3,900 m to 5,300 m), these are likely to be of a small size and are not expected to close the Expressway, due to the absence of high steep slopes.

13.5 Fault Rupture

In the event of a rupture of the Northern Ohariu Fault, it is possible that the Expressway will be closed due to road deformation or rupture of the fault. Given that the fault trace becomes indistinct in the lidar based topography, it is possible that the fault rupture where the Expressway crosses the fault is distributed deformation rather than concentrated rupture. This will reduce the extent of damage and disruption, in which case the Expressway is likely to be able to opened for limited access by earthmoving machinery. The Northern Ohariu Fault crosses the earth ramp leading to the Te Horo overbridge over the Expressway. Any fault rupture displacement of the earth embankment is likely to disrupt access to the bridge. However, access can be quickly reinstated by earth moving machinery in the event of fault displacement. This is also a subsidiary access compared to the Expressway, which provides primary north-south access into the Wellington Region.

The objective of the Project has been to avoid crossing the Northern Ohariu Fault on structures which may be severely damaged and will take a long time to reinstate, but rather to cross the fault on earthworks which would enable quick restoration of access. This objective has been achieved by the proposed design.

13.6 Route Performance

The expected performance of the Expressway route has been conceptually assessed considering the proposed alignment, the size of cuts, fills 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 preliminary assessed broad level performance of the Expressway route after a large Magnitude 7.5 local earthquake in the Region is presented in Illustration 23 in terms of Availability State - the degree of access available on the route after a major earthquake event in the region.

The performance state presented for the Expressway shows that:

- the Expressway is likely to remain open for access, perhaps with some reduced level of service due to road deformation associated with liquefaction and subsidence; and
- these areas are generally likely to be able to be reinstated quickly within 3 days to 2 weeks.

The performance state shows that the Expressway meets the route security philosophy proposed for the Project.

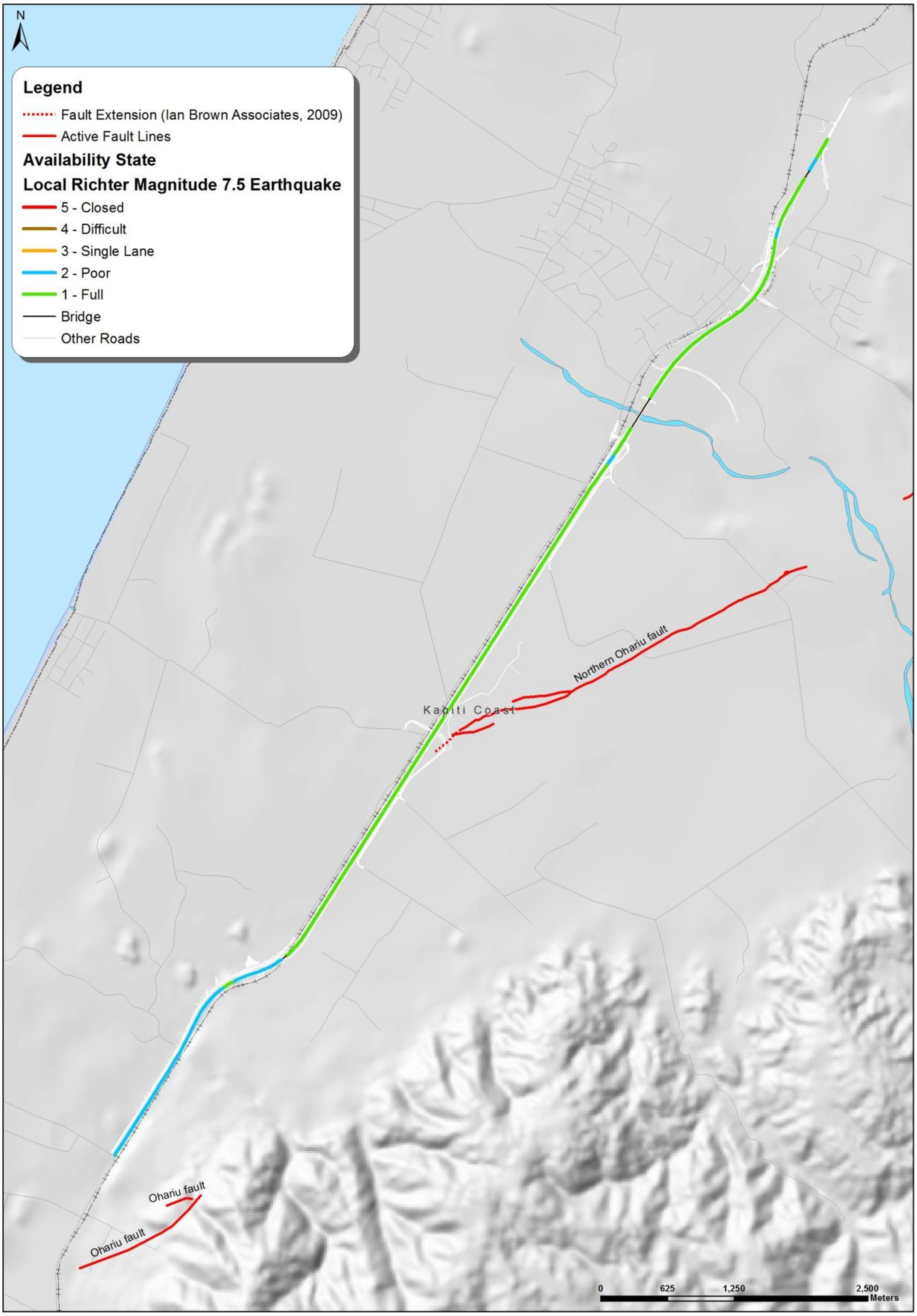


Illustration 23 - Availability (State) of Expressway after a Major Earthquake in the Region

14 Conclusion

14.1 Environmental effects

An assessment of environmental effects related to geotechnical engineering and a study of route security has been carried out for the Expressway route.

Groundwater and aquifers

1. At some of the cuttings in terrace alluvium (Stations 3900 m to 5300 m), the groundwater table is likely to be drawn down 1 m to 2 m locally across the new road.

2. During construction, temporary groundwater drawdowns in the order of 1 m to 3 m are expected at locations of excavation / undercutting in inter-dunal areas.

3. The effects of groundwater drawdowns on groundwater flow and direction are considered to be insignificant.

4. The long term effects due to the Expressway to the consented water takes are insignificant. The unconsented water bores with shallow water take within 500 m of the excavation for the expressway, between Stn 4,100 m and 4,400 m south of the Ōtaki River, should be surveyed prior to construction, and monitored prior to and during construction to assess if there are any adverse effects on these bores. If affected they could be replaced with deeper bores providing the same yield.

5. The effects of temporary groundwater drawdown on existing groundwater abstractions are not likely to be significant.

6. The groundwater wells proposed to extract water for use in construction will be located at least 250 m away from existing water abstract wells, so that they do not affect the existing wells.

7. Groundwater level and pressure will be monitored during excavation and dewatering to ensure there is no unexpected water drawdown that will affect water abstraction in the vicinity.

Ground settlements

8. Since the permanent groundwater drawdown at the cutting areas is local and of small order (about 1 m to 2 m), the resultant ground settlement should be minimal.

9. Preloading at inter-dunal areas will result in ground settlement within the road footprint and a short distance further. Areas of concern are south of Mary Crest and possibly north of Rahui Road. There are no buildings in the surrounding area that may be affected by such settlements and therefore the risk to buildings is low. However, the ground settlement will be monitored to ensure that post-construction settlements are low, and also to manage the potential effects on adjacent facilities, especially State Highway 1 and NIMT railway line south of Mary Crest. Settlement of the road and railway line will be reinstated as necessary.

Wetlands

10. The Ōtaki Railway wetland will be affected by the construction of the Expressway including the embankment and any undercut to facilitate construction, as described in the Ecology Report. The new wetland to be formed after the construction of the Expressway, at the Mary Crest bush area, will not be affected by the undercut of soft materials and any associated groundwater drawdown during construction.

Bridge Foundation Works

11. With appropriate care and precaution, the effects of bridge foundation works to the environment are insignificant.

Erosion Control at New Cuts

12. The effects of proposed cuttings to sand dune erosion can be kept to minimum with implementation of appropriate management and measures as recommended in this report.

14.2 Route Security

The Expressway will have a good resilience against earthquakes and other natural hazards.

The Expressway is likely to remain open for access in a large local Magnitude 7.5 earthquake event in the Region, perhaps with some reduced level of service due to road deformation associated with localised liquefaction and subsidence. These areas are generally likely to be able to be reinstated quickly with 3 days to 2 weeks.

The objective of the Project has been to avoid crossing the Northern Ohariu Fault on structures which may be severely damaged and will take a long time to reinstate, but rather to cross the fault on earthworks which would enable quick restoration of access. This has been achieved.

The Northern Ohariu Fault trace becomes indistinct in the lidar based topography, and it is possible that the fault rupture where the Expressway crosses the fault is distributed deformation rather than concentrated rupture. This will reduce the extent of damage and disruption, and the Expressway is likely to be able to remain open for limited access or access can be quickly reinstated by earthmoving machinery.

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