Moorland to Herons Creek EIS

Working Paper No. 8
Geotechnical Assessment

Roads and Traffic Authority NSW

Pacific Highway Upgrade - Moorland to Herons Creek

Preferred Option - Working Paper - Geotechnics

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1. INTRODUCTION

The Roads and Traffic Authority of New South Wales (RTA) proposes to upgrade the existing Pacific Highway between Moorland and Herons Creek in the mid-north coast region of New South Wales. This section of the highway is approximately 36.6 km to the north of Taree and is shown in **Figure 1**. It is the intention of the RTA to upgrade the highway to dual carriageway in both north and southbound directions. Arup has been appointed by the RTA to carry out scheme design and the Environmental Impact Statement (EIS) for the preferred option shown in **Figure 1**.

This report summarises the findings of geotechnical studies, which have been carried out to support these objectives. This report also incorporates the geotechnical findings from the previous report *Geotechnical Investigations for Route Selection,* Arup May 2001.

1.1 Scope of work

This report describes the results of geotechnical and environmental investigations carried out to provide information for scheme design and EIS for the preferred option.

1.2 Format of the report

Section 2 of the report provides a general description of the area surrounding the preferred option including information on the soils and geology.

Section 3 outlines previous ground investigations that have been carried out for this project.

Section 4 describes the ground investigations that have been carried out as part of this study.

Section 5 summarises the findings of ground investigations by describing the materials along the preferred option.

Section 6 describes the investigations and results of an assessment of potential for soil and groundwater contamination.

Section 7 describes the investigation and results of an assessment for acid sulfate soils in the area of the Stewarts and Camden Haven Rivers.

Section 8 describes the results of the assessment for acid sulfate rock

Section 9 provides information for preliminary earthworks design. This information includes sections on topsoil, the quality of subgrade, sources of material, cut and fill slopes, excavations in rock, reactive soils and soil erosion, and material unsuitable for use in construction.

Section 10 summarises the analysis undertaken on the large fill embankments proposed on the Camden Haven and Stewarts Rivers flood plains.

Section 11 discusses scheme options for retaining wall locations along the preferred option.

Section 12 discusses major bridge locations and founding conditions.

Section 13 describes the location of existing groundwater abstraction and reviews the potential for abstraction for construction use.

2. THE SITE

2.1 Location

The Site is located on the mid-north coast of NSW and lies within Greater Taree and Hastings local government areas. Its southern end is located 36.6 km to the north-east of the town of Taree along the Pacific Highway and the northern end is about 50km south of Kempsey.

2.2 Desktop study

A desktop study was carried out during the previous phase of geotechnical investigations for route selection (Arup, 2001) to provide a preliminary assessment of the ground conditions at the site.

The findings of that desktop study have been supplemented with additional information and incorporated into this study in the following sections to focus the current preferred option investigation on certain areas and issues, in addition to providing an overall assessment of the prevailing ground conditions. The initial desktop study involved the following:

- review of geological literature;
- aerial photographic interpretation; and
- review of existing reports.

2.3 Topography, terrain and landforms

The topography of the preferred option is generally subdued with gently undulating country to the south of Stewart River and to the north of the Camden Haven River. The preferred option will cross the flat alluvial plains of these two rivers. To the west of the central section of the preferred option the topography becomes rugged where it crosses Middle Brother. Elevations seldom rise above 30m AHD within the area, and slope angles are generally low, rarely exceeding 20° except on the fringes of Middle Brother.

The local catchments within the preferred option region drain generally towards the east into Stewarts River, Camden Haven River and Herons Creek. Both the Stewarts River and the Camden Haven River ultimately drain into Watson Taylors Lake, immediately to the east of the central section of the preferred option. Herons Creek drains to Queens Lake. Twenty-two waterways in total cross the area, including these major drainages. Wetlands listed under the State Environmental Planning Policy No. 14 - Coastal Wetlands (SEPP14) are located within the study area for the project. The locations of these wetlands and other public land are shown in **Figure 2**. The project would have no direct impact on any SEPP14 Wetlands.

Between the Camden Haven River and Stewart River flood plain, and to the west of the existing highway, the terrain is generally rugged with rocky outcrops rising towards Middle Brother.

2.4 Regional geology

The preferred option lies predominantly within the Lorne Basin and partly within the Hastings Block, both of which are part of the New England Fold Belt.

2.4.1 Solid geology

The solid geology found within the preferred option is described below and is shown in **Figure 3**.

The Lorne Basin comprises Permian and Triassic sediments and volcanics, but only the Triassic sediments and volcanics are exposed within the preferred option. These Triassic sediments comprise red and grey mudstones, lithic sandstones and conglomerates, tuffaceous sandstones, felsic volcanics and minor coal units and form the Camden Haven Group. Additional unnamed rhyolite and rhyolitic volcanics also exist within the area.

The Hastings Block underlies the northern section of the Highway. This consists of lithic sandstones, siltstones, pebbly sandstones and conglomeratic units, which comprise the Carboniferous Byabbara Beds.

The Lorne Basin was intruded during the late Triassic and Tertiary by granite, granodirorite and microgranite.

2.4.1.1 Byabbara Beds (Carboniferous)

This unit (Cb) is exposed over the entire northern part of the preferred option from the Camden Haven River to Herons Creek. It comprises lithic sandstone, siltstone, tuff, shale and limestones, which dip 40°-50° towards the north-east and east.

2.4.1.2 Camden Haven Beds (Triassic)

This formation is composed of two units Rs and Rc.

This unit (Rs) is exposed over almost the entire southern half of the preferred option and comprises tuff, shale and tuffaceous sandstone. South of Johns River these beds dip 15°-20° towards the north and around Middle Brother they dip 40° towards the east.

This unit (Rc) is only exposed locally at Rossglen, just to the south of the Camden Haven River. It comprises conglomerate, sandstone and shale which dips 25°-30° towards the east.

2.4.1.3 Middle Brother Granite (Tertiary)

These Tertiary intrusive rocks comprise fine- to coarse-grained granite and syenite. They are exposed in three locations within the preferred option; immediately to the north of Stewarts River (this material is commercially extracted for aggregate nearby), about 1km north of the Camden Haven River and immediately south of the Camden Haven River at Rossglen.

2.4.2 Geological structure

The regional fault trend within the area is towards the north-west, with minor conjugate faults towards the south-west. The sedimentary and volcanic sequences are gently folded with a typically north to south synclinal fold axis. Dip angles are typically less than 50°.

2.4.3 Superficial deposits

Alluvium has been deposited during the Quaternary, in the low-lying areas of the Camden Haven and Stewarts River areas. Estuarine and alluvial deposition continues to the present in and around these rivers and Watson Taylors Lake to the east of the preferred option.

2.5 Soil landscapes

The Department of Land and Water Conservation (DLWC – now Department of Infrastructure Planning and Natural Resources) has carried out soil landscape mapping specifically for the RTA along a 10km wide corridor of the Pacific Highway between Hexham and Corindi, and includes the preferred option. The method used for producing this mapping was consistent with other soil landscape products published by DLWC. Detailed logging of 119 soil profiles was carried out by DLWC to assist with these interpretations. The location of these profiles is shown on **Figure 4**.

DLWC also provided information to the RTA on the likely limitations for major road construction of all of the soil landscapes identified. The results of this interpretation are shown on **Figure 4**. Typically, the limitations identified were:

- from the southern end of preferred option to Johns River: water erosion hazard, water logging and low strength soil;
- Stewarts River crossing: flooding, water logging, low strength soil, (shrink/swell);
- Stewart River to Camden Haven River: difficult excavation, water erosion hazard, low strength soil, (water logging);
- Camden Haven to Kew area: acid sulfate soil, flooding, water logging, low strength soil, (shrink/swell and water erosion);
- Kew area to Herons Creek: water erosion hazard, low strength soil, water logging.

Soil landscapes, and the soils that they contain, are a product of the prevailing climate, underlying geology and the topographic location. As such, it is reasonable to assume that the soil characteristics, such as reactivity and erosion, within each group are likely to be consistent.

The identified soil landscapes along the preferred option are explained in detail below.

2.5.1 Soil landscape descriptions

The soil landscapes identified by DLWC within the preferred option are described below.

ba BURRAWAN

Landscape: Undulating low hills with broad crests and drainage depressions, on lithic sandstones and conglomerates of the Lorne Basin (Rec). Local relief 10-30m, elevation 10-50m, slopes 5-10%. Tall open forests, partly cleared for grazing on native pastures, and some residential development.

Soils: 50-200cm, well drained Red or Brown Kurosols (red podzolic soils) on crests, with >200cm imperfectly drained Red Kandosols on flats.

Significant Soil and Land Qualities: Strongly acidic, sodic, erodible soils. Hardsetting surfaces, low wet bearing strength, high run-on, seasonal waterlogging, gully erosion hazard, low fertility.

bb BYABARRA

Landscape: Rolling hills, on lithic sandstones, mudstones and siltstones of the Byabarra beds (Cb). Local relief 50-100m, elevation 50-200m, slopes 10-33%. Open forests, partly cleared for grazing on native pastures.

Soils: 50-100cm moderately well drained Brown Kurosols (red podzolic soils) and Brown Kandosols (yellow earths), with <70cm Leptic Tenosols (lithosols) on ridges.

Significant Soil and Land Qualities: Strongly acidic, erodible, locally shallow, stony soils, low water holding capacity. Hardsetting surfaces, low wet bearing strength, water erosion hazards.

gh GRANTS HEAD

Landscape: Rolling low hills and hills on sandstones and conglomerates of the Lorne Basin (Rec). Local relief 50-300m, elevation <500m, slopes 15-33%. Open forests, partly cleared for grazing on native pastures, and some residential development.

Soils: 50-100cm moderately well drained stony Brown Kurosols (red podzolic soils), with occasional <70cm Leptic Tenosols (lithosols) on upper slopes.

Significant Soil and Land Qualities: Strongly acidic, erodible, locally shallow, stony soils, low water holding capacity. Hardsetting surfaces, low wet bearing strength, water erosion hazards.

Landscape Variant - gha: Colluvial fans and footslopes.

he HERONS CREEK

Landscape: Narrow to moderately broad floodplain and low terrace surfaces, with minor depressions and drainage lines and narrow braided channel, in mid to upper reaches of streams draining the Lorne Basin. Local relief <10m, elevation <100m, slopes <5% (up to 50% on banks). Extensively cleared riverine forests.

Soils: >300cm moderately well drained Brown Kandosols (brown earths and prairie soils) and Red Kandosols (red earths) and 50-200cm well drained gravelly Stratic Rudosols (alluvial soils).

Significant Soil and Land Qualities: Highly erodible, well drained, acid, gravelly soils, low available water holding capacity. Stream bank erosion, high run-on, foundation hazard, localised flood hazard, localised seasonal waterlogging.

ho HOLEY FLAT

Landscape: Undulating low hills on granite and rhyolite (Rlv, Rbg). Local relief 20-90m, elevation 40-200m, slopes 5% to 20% (generally <10%). Open dry sclerophyll forest and tall open forest, partly cleared.

Soils: 100-180cm, moderately well-drained Red Kurosols (red podzolic soils) on crests, with poorly drained Yellow Chromosols (yellow podzolic soils) in areas of poor drainage.

Significant Soil and Land Qualities: Highly erodible, acidic, hardsetting soils of low fertility and low water-holding capacity.

hv HANNAM VALE ROAD

Landscape: Rolling low hills and hills on granite and rhyolite (Rlv, Rbg). Local relief 40-200m, elevation 40-200m, slopes 15-33%. Open dry sclerophyll forest and tall open forest, partly cleared.

Soils: 100-180cm, moderately well-drained Red and Brown Kurosols (red podzolic soils).

Significant Soil and Land Qualities: Strongly acidic, erodible, stony soils, low water holding capacity. Water erosion hazards.

ji JONES ISLAND

Landscape: Level fluvial-deltaic floodplains with minor backswamps and flood basins. Local relief <3m, elevation <10m, slopes <2%. Wet meadow and sedgeland with open swamp sclerophyll forests.

Soils: >200cm poorly-drained Redoxic and Oxyaquic Hydrosols (humic gleys).

Significant Soil and Land Qualities: Low wet-bearing strength, acid sulfate soil hazard. Localised flood hazard, localised seasonal waterlogging.

la LAURIETON

Landscape: Undulating low foothills, scree slopes and fans derived from granite (Rbg), below steep hills. Local relief 10-50m, elevation <80m, slopes 5-20%, generally <10%. Open dry sclerophyll forest and tall open forest, partly cleared for pasture.

Soils: <80cm, moderately well-drained sandy Leptic Tenosols (lithosols) and Red Dermosols (red podzolic soils); 90-180cm poorly drained Yellow Chromosols (yellow podzolic soils) in areas of poor drainage.

Significant Soil and Land Qualities: Acidic, stony, hardsetting soils of low fertility and low water-holding capacity. Very high run-on.

me MELINGA

Landscape: Undulating rises and low hills, on lithic mudstones and lithic sandstones of the Byabarra Beds (Cb). Local relief 10-30m, elevation 10-50m, slopes 5-10%. Open forests, partly cleared for grazing.

Soils: 100-150cm, poorly drained Mottled Natric Brown or Yellow Kurosols (soloths and yellow podzolic soils).

Significant Soil and Land Qualities: Strongly acidic, sodic, erodible soils. Hardsetting surfaces, low wet bearing strength, high run-on, seasonal waterlogging, gully erosion hazard.

pi PIPECLAY CANAL

Landscape: Level estuarine backswamps and floodbasins. Local relief <1m, elevation <5m. Often extensively drained. Wet meadow and sedgelands with open swamp sclerophyll forests.

Soils: >200cm poorly-drained Sulphidic Hydrosols (humic gleys) and Sulphidic Organosols (acid peats).

Significant Soil and Land Qualities: Strongly to extremely acid, sodic, saline soils with high aluminium toxicity potential, high organic matter, low to very low wet bearing strength and slow subsoil permeability. Flood hazard, permanently high watertables, high foundation hazard, very high acid sulfate soil hazard.

pn PRINCES

Landscape: Narrow, often slightly convex alluvial fans on footslopes, grading to broad drainage plains, below hills of the Lorne Basin (Rec). Slopes <3%, local relief <2m, elevation 5-20m. Open dry sclerophyll and swamp forest, often partly cleared for grazing.

Soils: Very deep (>300cm), poorly drained, Mottled Brown or Grey Sodosols (solods) and Kurosols (gleyed podzolic soils).

Significant Soil and Land Qualities: Sodic, hardsetting soils, very low permeability, high erodibility, low wet bearing strength. Acidic, high aluminium toxicity potential, low fertility. High run-on, localised flood hazard, waterlogging, foundation hazards.

up UPSALLS CREEK

Landscape: Level elevated terrace surfaces on Early Pleistocene alluvium, in mid to upper reaches of streams draining the Lorne Basin. Slopes <5%, elevation 20-80m, local relief up to 3m. Open forests, mostly cleared for improved pastures.

Soils: 100-180cm imperfectly drained Brown Kurosols (red podzolic soils) and Brown Kandosols (yellow earths) overlying mottled alluvial loams.

Significant Soil and Land Qualities: Hardsetting, low fertility, acidic soils. Poor drainage, seasonal waterlogging, moderate foundation hazards, run on (localised).

wa WATSON TAYLORS

Landscape: Levees of fluvial-deltaic plains. Local relief <3m, elevation <8m, slopes <2% on backplains and upper surfaces, to 15% on side-slopes. Open forest and subtropical rainforest, mostly cleared for improved pasture, with gallery mangroves.

Soils: >300cm well drained Brown Kandosols (alluvial soils, chernozems, gradational yellow earths).

Significant Soil and Land Qualities: Low wet-bearing strength. High streambank erosion hazard, localised seasonal waterlogging, prime agricultural land.

xx DISTURBED

Landscape: Level to hummocky terrain, extensively disturbed by human activity, including complete disturbance, removal or burial of soil. Variable local relief and slopes. Includes quarries, tips, land reclamation and large cut and fill features. Original vegetation cleared, and weeds may be abundant.

Soils: Original soil has been removed, greatly disturbed or buried. Land-fill includes soil, rock, building and waste materials.

Significant Soil and Land Qualities: Highly variable; may include foundation hazard, unconsolidated low bearing strength materials, impermeable soils, poor drainage, very low fertility, toxic materials and wind erosion hazard. Sources of sediment and groundwater contamination.

2.6 Past, current and proposed mining activities

Information provided by the Department of Mineral Resources (DMR) indicates that there are five industrial mineral borrow areas or quarries within or near the preferred option. These sites have been designated as Section 117 sites by DMR and are shown on **Figure 5**. This figure shows both the identified resource location and a "buffer zone" drawn around it by DMR. Development applications, which cross into the buffer zone, will generally be referred to DMR for their comments as there may be implications for the resource.

All of these sites produce aggregate materials, consisting of granite, conglomerate and shale for the construction industry. The only facility, which was being operated during the course of this study was the Johns River Quarry operated by Boral. A summary of these quarries is given in the table below.

Table 1 Existing aggregate borrow pits

Name/Location	Operator/Operational Status	Product/material
Taylors Pit	Hurd Haulage Pty Ltd/Not operated	Road materials/conglomerate
Bethesda Quarry	NSW Roads and Traffic Authority/Not operated	Road materials/shale
Rossglen	State Rail Authority of NSW/Not operated	Railway ballast/granite
Stony Creek Road	State Forests of NSW/Not operated	Road materials/granite
Johns River	Boral Resources Pty Ltd/In operation	Hard rock aggregate/granite

DMR information indicates that there are no proposed mining locations within the preferred option.

2.7 Options development investigations

A number of previous investigations of the soils within the preferred option have been carried out by other organisations. The results of these studies have been reviewed and their findings incorporated into this report. These previous studies include:

Geotechnical investigations:

- Maunsell & Partners Pty Ltd, 1984, Geotechnical investigations for Camden Haven Bridge
- Department of Main Roads NSW, 1981, Geotechnical investigations for Stewarts River Bridge
- Arup, 2001, Geotechnical Investigations for Route Selection.

Acid Sulfate Soils investigations:

- DLWC, 1997, Soil landscapes of the Pacific Highway Corridor, Hexham to Corindi
- DLWC, Camden Haven, Acid Sulfate Soil Risk Map, 1:25,000
- Snowy Mountain Engineering Corporation, 1986, Rossglen Acid Sulfate Soil Management Plan

Aggregate studies

- Mitchell McCotter & Associates, 1995, Archaeological survey of a proposed quarry extension at Kew, Mid-North Coastal NSW

Existing pavement investigations:

- RTA, May 1999, investigations at Passionfruit Creek
- RTA, August 1999, investigations between Rossglen and Camden Haven River
- RTA, August 1999, investigations between Ocean Drive/Kendall Road Kew and Herons Creek
- RTA, May 1999, investigations at Walkers Creek
- RTA, September 1999, investigations between Walkers Creek and Herons Creek
- RTA, 1997, Deflectograph survey of Pacific Highway

3. PREVIOUS ARUP INVESTIGATIONS FOR ROUTE SELECTION

Arup was engaged by the RTA in 2000 to undertake a geotechnical investigation for Route Selection [1]. The investigation was conducted using a variety of techniques including boreholes, test pits, a ground conductivity survey, geological mapping, laboratory testing, petrographic analysis and a visual condition survey of the existing pavement. In addition, a contamination, acid sulfate and pedological survey were carried out in conjunction with the test pitting. The findings of these investigations are discussed in Section 4 of *Geotechnical Investigations for Route Selection,* Arup May 2001.

The intrusive geotechnical investigations incorporated drilling 28 boreholes, excavating 31 test pits and 9 cone penetration tests. These test locations were strategically positioned to provide broad information on the ground conditions to aid in the route selection process.

Of the above testing carried out, 11 boreholes, 16 test pits and 9 Cone Penetration Tests were undertaken along, or adjacent to the preferred option. The information obtained from these tests has been incorporated into this report. The location of existing test locations is shown in **Figure 6**.

For simplicity, in the text that follows, the investigations carried out for route selection will be referred to as the 'options development investigation" and the latest investigations will be referred to as the 'preferred option investigation".

4. PREFERED OPTION INVESTIGATIONS

4.1 General

The fieldwork carried out for the current phase of the project has been conducted using a number of investigative techniques. These are detailed below and include boreholes, test pits, seismic refraction surveys, continuous sampling, cone penetration testing, geological mapping, laboratory testing and petrographic analysis. In addition, a contamination and acid sulfate soil survey were carried out in conjunction with the test pitting, hand augering or continuous sampling. The findings of these investigations are discussed in Section 5 and the results are presented in appendices to this report. The locations of these investigations are given in **Figure 6**.

The drilling of boreholes was undertaken by APS drilling Pty Ltd, a Sydney based company. Test pits excavation was carried out by Delaforce Excavations of Port Macquarie. The University of Newcastle (NEWSYD) conducted the twelve cone penetration tests. Queensland-based Geocoastal Australia carried out continuous sampling boreholes. The seismic refraction survey was undertaken by Sydney based Earth Technology Solutions.

4.2 Field mapping

Field mapping particularly of existing cut slopes along the preferred option was carried out to supplement work that had been carried out as part of the route selection study. The results of this work, which largely confirmed the information provided by published geological maps, is discussed in detail in Section 5 of this report and are shown in **Figure 7**.

4.3 Boreholes

A total of 14 boreholes were drilled between Johns River and Herons Creek during the preferred option investigation. They were drilled at locations along the preferred option in areas of cut, high fill (flood plains), geotechnical structure and at the river crossings of the Stewarts River and the Camden Haven River. Boreholes were drilled using a truck mounted Edson 3000 rig using rotary coring techniques. Auger drilling with Tungsten carbide bits was used above bedrock in cohesive soil to approximately 8m. Wash boring was undertaken to drill through cohesionless soils.

The boreholes were generally cased within the overburden material to prevent caving. All cored boreholes were drilled vertically with water flush and occasionally with approved mud where the core was too fractured to be recovered otherwise. NMLC core barrels were used and these produced a core of approximately 50mm in diameter. Boreholes were drilled to a depth of at least 4m below the proposed formation level at the time of the investigation. The borehole logs and core photographs are provided in Appendix A. Explanatory notes describing the terms and symbols used in their preparation are provided in Appendix A.

Standard penetration tests (SPTs) were conducted generally at 1.5m intervals in soils, although in gravels this interval was extended. In cohesive materials undisturbed U50 samples were recovered for laboratory testing. Point load strength index tests were carried out on recovered rock core at one metre intervals.

Standpipe piezometers were installed in boreholes 201, 204, 210, 211, 213 and 214 to allow monitoring of ground water levels and to provide access for ground water sampling. Piezometers installed in BH210 and 211 were sealed in gravel overlying bedrock. An additional piezometer was installed in both these boreholes 6m below ground level.

Point load strength index tests were carried out on rock samples at either 1.0m intervals or at apparent change of the rock strength. This test gives an indication of the Is(50) index strength of the rock.

The point load index strength test results are presented in Appendix N.

4.4 Test pits

A total of 63 test pits were excavated along the preferred option during the preferred option investigations. They were generally excavated in fill areas, at cut and fill interfaces, in cuttings, possible contamination areas, acid sulfate soil areas and in areas to provide general information on the soil/rock profile.

Test pits labelled TP were excavated specifically for geological/geotechnical and environmental sampling. Test pits labelled AS were primarily excavated for acid sulfate soil sampling, but were also logged as geological/geotechnical test pits.

All test pit excavations were undertaken using either a rubber wheeled backhoe or tracked excavator with a 600mm or 700mm wide bucket.

Pocket Penetrometer and hand shear vane testing was carried out within the walls of the test pits to give an indication of the consistency of cohesive soils. Soil Samples were taken at selected soil strata for testing purposes. The test pit logs and test pit photographs are provided in Appendix B. Explanatory notes describing the terms and symbols used in their preparation are provided in Appendix A

4.5 Hand augering

In addition to the 63 test pits, 6 holes were hand augered where sensitive environments did not allow the use of a backhoe or excavator. Of the 6 holes, 4 were augered to sample soil for acid sulfate testing and 2 were augered to sample soil for environmental testing.

4.6 Cone penetration tests

A total of 12 cone penetration tests with pore pressure measurement (CPTUs) were carried out along the preferred option within the flood plains of the Camden Haven and Stewarts Rivers. Pore Pressure Dissipation tests were undertaken at approximately 3 to 5 m intervals where positive pore pressures were generated.

CPTUs201 to 205, 207, 208, 211 and 212 were undertaken using the NEWSYD 18 tonne custom built rig. The remaining CPTUs were undertaken using portable equipment supplied and operated by NEWSYD staff. The portable equipment was transported to the testing location using a tracked excavator.

All CPT locations were determined by Arup. Test locations were generally selected in areas where deep soft soils were anticipated, mainly along embankments and at bridge sites on both the Camden Haven and Stewarts River flood plains. A 5 or 10 tonne electric cone was used for the penetration testing. The tests provide a continuous profile of the consistency of the subsurface soil and an interpretation of the soil type and pore pressures to the refusal depth of the probe. The test results are presented in Appendix C.

4.7 Seismic refraction survey

Four seismic lines, SL1 to SL4 were completed totalling 575m in length. The lines were located at points along the preferred option where it was anticipated that significant depth of cutting would need to be carried out. The work was carried out by Earth Technology Solutions Pty Ltd and was carried out in accordance with the RTA Model Brief. Details of this survey are provided in Appendix D

4.8 Laboratory testing

The samples recovered from boreholes and test pits were sealed in plastic sample bags and dispatched to Australian Soil Testing, a NATA registered laboratory. Both soil and rock samples were tested. The testing carried out included:

- Moisture Content
- Atterberg limits
- Particle Size Distribution
- Percent Dispersal
- Emerson Crumb Test
- Unconfined Compressive Strength (UCS)
- Petrographic Analysis
- X-ray Diffraction
- Linear Shrinkage
- Compaction
- California Bearing Ratio (CBR) 10 day soaked.
- Organic Content
- Unconsolidated Undrained Triaxial tests
- Consolidation.

The results of these tests are given in Appendix E.

Testing of soil samples for contamination and potential acid sulfate soils (PASS) was also carried out. These tests were carried out by Australian Laboratory Services, a NATA-registered facility.

Applied Petrographics carried out petrographic analysis on nine selected drill core samples.

Australian Soil Testing managed the NAP and NAG testing for acid rock. The testing was undertaken by Environmental Geochemistry international.

X- ray diffraction was undertaken by Workcover, and was managed by Australian Soil Testing.

5. GROUND CONDITIONS

This section of the report summarises the earthworks required along the preferred option and summarises the ground conditions that can be expected. This section has been subdivided into six sub-sections:

- Johns River Bypass CH0000 - CH3550
- Stewarts River Crossing CH3550 - CH4275
- Lake Section CH4275 - CH12300
- Camden Haven Crossing CH12300 - 13600
- Kew Bypass CH13600 - CH17100
- Northern Section CH17100 – 22200.

Additional geotechnical information on the proposed earthworks scheme is provided in Section 9. Tables for earthworks contain maximum cut and fill height and are not necessarily representative of the entire length of cut or fill. Major cuts and fills have been classified as those with 5m or more cut or fill. Minor cuts and fills have been classified as those with less than 5m of cut or fill. Chainages provided in earthworks tables are approximate due to the difference between the north and southbound profiles.

5.1 Johns River Bypass CH0000 - CH3550

5.1.1 Johns River Bypass

The earthworks that will be required for this part of the project are shown on **Figure 8** and are summarised in Table 2.

Table 2 Summary of earthworks for Johns River Bypass

Cut/Fill	Length (m)	Approximate Mid-point Chainage	Johns River Bypass	
			Max Cut height (m)	Max Fill height (m)
Minor Fill	1420	120		4
Major Cut	320	1550	14	
Minor Fill	853	2100		3
Minor Cut	200	2750	2	
Minor Fill	400	3100		4.5

The option crosses the gently rolling hills, which form the foot-slopes of South Brother. The land-use is generally woodland and pasture, with new sub-divisions being created adjacent to Stewarts River Road.

The geological map indicates that the entire length of the preferred option is underlain by tuff, shale and tuffaceous sandstone of the Camden Haven Beds. The soil landscapes identified by DLWC include the Princes, Laurieton, Burrawan and Upsalls Creek.

5.1.1.1 Field investigations

During the preferred option investigation seven test pits, TP201 to TP207 were excavated in this area. Test pits, TP1, and TP3 and three boreholes BH2, 2a and 3 were completed during the initial investigation. The locations of these points are given in **Figure 6**. A summary of the ground conditions generally encountered is given in Table 3 and shown in **Figure 8**.

Table 3 Summary of ground conditions for Johns River Bypass

Depth to top (m)	Typical description	Thickness range (m)	Origin
0	Soft-Firm, orange to dark brown-black silty clay/sandy clay or sandy Silt	0.1-0.55	Topsoil
0.1 to 0.55	Firm becoming very stiff with depth, grey or orange brown silty clay or sandy clay.	1.6-11.30	Residual soil
1.70 to 11.95	Extremely low strength, orange brown becoming, grey sandstone or moderate strength extremely weathered, off white unidentified igneous rock	-	Bedrock

Alluvium was encountered in TP205 to a depth of 2.9m. The test pit was excavated adjacent to a small creek, which has been dammed. The consistency and material types encountered are similar to the residual soil described in the table above.

Alluvium was also encountered in TP207 to a depth of 3.05m. This material can also be broadly categorised within the above-tabulated residual soil, consisting of predominantly firm to stiff grey silty clay with sand.

5.1.1.2 Laboratory testing

Classification testing of these soils has been carried out and the results are shown in **Figure 9** and **Figure 10**. These indicate that the residual soils are typically medium to high plasticity clays (liquid limit >35%) with occasional silts. Moisture contents range between 19% and 39% and plastic limits range from 16% to 35. Liquid limits range between 24% and 80% but are typically between 41% and 67%. Plasticity indices range between 7% and 52%.

Soaked CBR values for the silty clay residual soil generally range from 2 to 4 with a value of 8 recorded for the residual silty clay in TP203. Compaction tests indicate that maximum dry densities of 1.52 tonnes/m³-1.61 tonnes/m³ can be achieved at optimum moisture contents between 24 and 26.5% in this residual silty clay material.

5.1.1.3 In situ testing

The SPT N values are shown in **Figure 11** for BH2, 2a and 3. The chart indicates a general increase in strength with depth for the silty clay/sandy clay with consistency increasing from firm to very stiff. Undrained shear strengths in excess of 100 kPa were obtained from pocket penetrometer test results for the residual soils. High moisture contents could affect the strength of these materials and some localised softer areas may occur particularly at creeks. This is evident with localised soft saturated areas at TP202 and TP205.

5.1.2 Johns River ground water conditions

During the initial geotechnical investigation, water was encountered in BH2. During the current phase of investigations, water was encountered in test pits 205 and 207. The water table depths recorded during the investigations are presented in Table 4.

Table 4 Water table depth at Johns River

Investigation Location	Water Table Depth (m)
BH2	2.50
TP205	1.0
TP207	3.0

5.2 Stewarts River crossing CH 3550 - CH4275

This section of the preferred option includes duplication on the western side of the existing highway and construction of a new bridge across the Stewarts River for the new northbound carriageway.

The earthworks that will be required for the preferred option are shown on **Figure 12** and are summarised in Table 5.

Table 5 Summary of earthworks for Stewarts River crossing

Cut/Fill	Length (m)	Approximate Mid-point Chainage	Stewarts River crossing	
			Max Cut height (m)	Max Fill height (m)
Major Fill	725	3800		6.5

The preferred option crosses an alluvial plain and the Stewarts River to the north of Johns River. The land is used for dairy pasture.

The geological map indicates that the entire length of the preferred option in this section, is underlain by alluvium. The soil landscapes identified by DLWC include the Upsalls Creek, Jones Island and Watson Taylors. In addition, DLWC identified the soils in this area to have a low probability of acid sulfate risk. Tests carried out as part of this study indicate high potential for acid sulfate development. Acid sulfate issues are discussed in greater detail in a later section of this report.

5.2.1.1 Field investigations

The current fieldwork in this area comprised two boreholes (BH201 and 202), five CPTUs (CPT201, 202, 203, 204 and 205), seven test pits (TP208, 209,210 and AS201,202,203 and 204), two hand augers for agricultural environmental samples, one hand auger for acid sulfate samples (AS207) and two continuous sampling points (CSP201 and 202).

During the options development investigation, three boreholes (BH7, 8 and 9), 3 CPTs (CPT1, 2 and 3) and 2 test pits (TP8 and 46) were undertaken.

The locations of these points are provided in **Figure 6**. Thirteen boreholes had previously been bored for the design of the original road-bridge and this borehole information is provided in Appendix F. A summary of the ground conditions encountered is given in Table 6 and in the cross section shown in **Figure 12**.

Table 6 Summary of ground conditions Stewarts River crossing

Depth to top (m)	Typical description	Thickness range (m)	Origin
0	Very soft to stiff, brown silty and sandy clay, gravelly sand , organic clay and gravelly clay	0.15-1	Topsoil/Fill
0.15 – 1	Very soft to soft grey mottled red silty, sandy clay and organic clay and silt	2.75-9.4	Alluvium
2.9-5.5	Very loose to medium dense grey sand and sand with gravel	3-8.1	Alluvium
8.5-11	Medium dense to dense, brown and grey sand with gravel and sandy gravel	3.75-6.8	Alluvium
13.35 (BH8 only)	Grey, stiff silty Clay and dense to very dense silty sand	To end BH7	Residual
15.3-20.5	Medium to high strength Tuffaceous Sandstone and very low to medium strength Claystone		Bedrock

5.2.1.2 Laboratory testing

- **Fill**

  No laboratory testing was undertaken in the fill material. Fill is located both in the approach embankment and beneath the existing bridge. The fill consisted of variable material ranging in consistency from very soft to stiff, and classification from sandy clay to gravelly clay. Significant quantities of construction rubble were observed in the side slopes of the existing embankment. This rubble included boulder sized fragments of rock and concrete.

- **Silty, organic or sandy clay**

  Classification testing of these soils has been carried out and the results are shown in **Figure 13** and **Figure 14**. These indicate that the soils are typically high plasticity clays (PI greater than 50). Plasticity indices range between 9% and 60%. Organic matter contents of 2.52% and 0.81% were encountered at depths of 1.35m and 5.35m in BH201.

- **Sand**

  Particle size distribution tests from the options development investigation indicate that the material varies from gravelly to clayey sand and from well to poorly graded. The moisture contents in the sand samples during the options development investigations were approximately 24%. Close to the water table the sand is saturated.

- **Gravels**

  Classification testing of these soil materials has been carried out and the results are shown in Appendix E. Sieve analysis indicates that the material varies from well-graded clayey sandy gravel to poorly graded gravel. The gravel was generally saturated.

CBR results from sandy clay in TP7 indicate a value of 2.5 at 0.8m. Compaction tests indicate that a maximum dry density of 1.44 tonnes/m³ can be achieved at an optimum moisture content of 27% within the sandy clay material. As minimal material will be cut and used as fill from this part of the preferred option, CBR and compaction testing from test pits in this area was kept to a minimum. The soft clays will be unsuitable for use as fill.

Unsaturated undrained triaxial tests were carried out using U50 samples recovered from BH201. The results are tabulated in Table 7.

Table 7 Triaxial test results Stewarts River Crossing

Borehole	Depth	Material	Cu (kPa)
201	1.35-1.75	Silty CLAY	18
201	5.35-5.75	Sandy CLAY	41

Unconfined Compressive Strength (UCS) tests were undertaken on rock core recovered from BH201 and BH202. The results are summarised in Table 8.

Table 8 UCS Results Stewarts River Crossing

Borehole	Depth	Failure mode	UCS (MPa)	Rock Type
201	15.47-15.62	Through core	19.4	Tuffaceous sandstone
201	17.84-18	Through core	21	Tuffaceous sandstone
201	19.21-19.40	Through core	26.5	Tuffaceous sandstone
202	4.1-4.25	Through defect	3.8	Micro-granite
202	4.1-4.25	Through axis	35.6	Micro-granite

5.2.1.3 In situ testing

Boreholes

The SPT N values for BH7, 8 and 9 drilled during the options development investigation, together with BH201 drilled during the preferred option investigation are shown in **Figure 15**.

SPT N values from boreholes located on the lower flood plain terrace (BH7, 9 and 201), above approximately 5m below ground level are below 5, with no apparent increase within this depth range. In BH201 and 9, these low SPT N values can be attributed to very soft, high plasticity organic clays and silts encountered at these depths.

In BH7, the low SPT N values were encountered in loose to very loose clayey sands and sands.

Below 5m the SPT N values generally increase with depth in BH7,9 and 201 from loose to medium dense and dense clayey sands which overly medium dense to dense gravels.

SPT N values from BH8, located on the upper flood plain terrace, generally increase with depth from the surface in silty clay to approximately 9m below ground level.

Test pits

Undrained shear strengths in the range of 50 kPa to 100 kPa were obtained from pocket penetrometer test results in the clay deposits encountered in test pits.

Cone penetration testing (CPT)

The CPT test results from the options development investigation indicate the presence of fill to a depth of 0.4m at CPT1 and 1.3m at CPT12 overlying normally consolidated clays and silts to 4.0m and 6.0m at CPT1 and CPT12 respectively. The presence of soft normally consolidated clays and silts is indicated at CPT2 and CPT3 to depths below ground level of 3.0m-3.5m respectively. These materials are underlain by sensitive fine-grained soils to depths of approximately 6.0m and 8.0m below ground level. Sand was encountered at depths ranging from 5.0m to 8.0m below ground level to refusal in all the CPTs. The tests were terminated on dense sands or gravels generally where the tip resistance, Qc, was greater than 50 MPa.

CPTs from the preferred option investigations were undertaken with pore pressure measurement. On the upper flood plain terrace, three tests CPT201, 202 and 205 were undertaken. These tests generally produced negative pore pressures, which are indicative of dilation and associated with cohesionless soils such as silts and sands. Stiff to very stiff clay varying in depth from 2m to 5m was encountered at these CPTs, overlying interbedded loose becoming dense with depth, silty sands, sandy silts and sand. Interbedded clay, clayey silt and silty sand/sandy silt were encountered at CPT202 and 205 between depths of 8m and 10m.

CPT205 was terminated at a depth of 10.49m at a tip resistance greater than 45 Mpa. CPT201 and 202 were terminated at depths of 14.74m and 14.54m respectively, due to the presence of dense cohesionless material.

CPT203 and 204 were carried out on the lower flood plain terrace. At CPT204, approximately 80m from the Stewarts River, soft clays were encountered to a depth of 4m underlain by very soft or very loose sensitive fine-grained material to a depth of 8.5m, underlain in turn by dense sand and silty sands.

At CPT203 firm clay was encountered becoming stiff with depth to 3.7m overlying interbedded clayey silt/silty clay and silty sand/sandy silt, loose becoming dense with depth. CPT203 and CPT204 were terminated when tip resistance exceeded 45MPa.

5.2.2 Continuous sampling

Two continuous sample holes were put down in the area of the lower flood plain terrace.

At CSP201 very soft inter-bedded silty and organic clays were encountered to a depth of 3m overlying stiff clay to 5m, overlying gravel, sand and clay layers to refusal at a depth of 14.6m on gravel.

At CSP202 firm clay was encountered to a depth of 2.5m overlying very soft clayey silts and sands, overlying inter-bedded clayey sand, gravelly sand and silty sand. CSP202 was terminated on silty sandy gravel with refusal at a depth of 10.91m.

5.2.3 Stewarts River ground water conditions

During the preliminary geotechnical investigation, water was encountered in BH7 and BH9/9A and in TP46 within the Stewarts River area. Seepage was moderate in TP7 and TP8. During the current phase of investigations, water was encountered at BH201. The water table depths recorded during both investigations is presented in Table 9.

Table 9 Water table depth around Stewarts River

Investigation Location	Water Table Depth (m)
BH201	0.65
BH7	2.50
BH9	1.00
TP7	Seepage moderate from 2.8m
TP8	Seepage moderate from 1.6m
TP46	2.10

5.3 Lake Section CH4275 - CH12300

This section of the preferred option will involve duplication of the existing highway alongside the current alignment.

The earthworks that will be required for this preferred option are shown in **Figure 16** (a-c) and are summarised in Table 10.

The preferred option crosses the foot-slopes of Middle Brother and is bounded to the east for much of its length by the North Coast Railway and Watson Taylor's Lake and to the west by Middle Brother State Forest and Yoorigan National Park.

Table 10 Summary of earthworks for the Lake Section

Cut/Fill	Approximate Length (m)	Approximate Mid-point Chainage	Lake Section	
			Max Cut height (m)	Max Fill height (m)
Major Cut	225	4750	8	
Minor Fill	250	5000		2.5
Major cut	200	5250	8	
Minor fill	1290	5700		4
Cut	400	6200	3	
Minor fill	150	6500		1.5
Major cut	390	6650	6	
Minor fill	270	7150		2.5
Minor cut	300	7500	2	
Minor Fill	400	7800		3.5
Major cut	350	8250	12	
Fill	350	8800		4.5
Major Cut	300	9100	6	
Minor fill	675	9400		2
Major Cut	415	10050	5	
Fill/at grade	110	10700		2
Minor Cut	200	10900	1	
Fill	100	11050		2
Cut	200	11250	3	
Fill	100	11350		2
Cut	100	11450	2	
Minor Fill	250	11650		1.5
Fill	250	12000		2.5

The geological map indicates that from Stewarts River to Passionfruit Creek, the preferred option will encounter granite, with alluvium in the valley of Passionfruit Creek. North of Passionfruit Creek the preferred option passes into shale (Rs unit) of the Camden Haven Beds. Conglomerates and shales of the Rc unit of the Camden Haven Beds underlie the preferred option between Stony Creek and the Camden Haven River. Although not shown on the geological map, alluvial deposits would be expected at both Passionfruit and Stony Creeks. The soil landscapes identified by DLWC include Grants Head, Laurieton, Hannan Vale Road and Jones Island.

5.3.1.1 Field investigations

Preferred option investigation in this area incorporated 6 Boreholes (BH203 - 209), 17 Test pits (TP212 - 228) and 1 seismic line (SL01).

The fieldwork undertaken during the options development investigation in this area comprised three test pits (TP12, 13 and 19).

The locations of all investigation points are provided in **Figure 6**. A summary of the ground conditions encountered is given in the tables below.

Table 11 Summary of ground conditions from Stewarts River to Passionfruit Creek

Depth to top (m)	Typical description	Thickness range (m)	Origin
0	Loose brown to black silty sand	0.5-0.6	Topsoil
0.5-0.6	Stiff orange brown sandy clay and loose to medium dense clayey and silty sand (extremely weathered Granite).	0.8-9.65	Residual
1.36 -11.05	Low to very high strength, grey to off white or red brown granite and micro granite	-	Bedrock

Table 12 Summary of ground conditions at Passionfruit Creek

Depth to top (m)	Typical description	Thickness range (m)	Origin
0	Loose brown sandy silt and soft silty clay	0.3-0.5	Topsoil
0.5	Firm, grey gravelly sandy clay	0.6	Fill
1.2	Soft to firm grey sandy silt and sandy clay	1.7	Alluvium
0.3	Soft to firm becoming hard, grey brown sandy clay	2.3	Residual soil
2.3	High strength grey granite		Bedrock

Table 13 Summary of ground conditions at Stony Creek

Depth to top (m)	Typical description	Thickness range (m)	Origin
0 (TP19)	Firm to stiff, grey orange sandy gravelly clay	2.0	Fill
0 (TP221)	Loose, black sandy silt	0.6	Topsoil
0.6 (TP221 & 19)	Loose to medium dense brown silty sand or soft or loose brown clayey sand/sandy clay	0.35 –1.1	Alluvium
1.7 (TP221)	Firm to stiff, brown silty clay or loose sand with cobbles	To end of TP	Alluvium
2.35 (TP19)	Firm to stiff, brown silty clay	To end of TP	Residual

Table 14 Summary of ground conditions from Passionfruit Creek to Camden Haven

Depth to top (m)	Typical description	Thickness range (m)	Origin
0	Loose grey brown silty sand or silty sand	0.1-0.6	Topsoil
0-0.6	Off white to dark grey, firm to hard sandy clay or silty clay, and medium dense clayey sand, some with high strength gravel and cobbles of granite	0.6- end of TPs	Residual
0.59-12.47	Extremely low to medium strength light grey or red brown sandstone and siltstone/mudstone and extremely low to very high strength light grey and red brown granite.	To end of BHs	Bedrock

Geological mapping of each of the existing cut slopes along the highway was carried out during the initial investigations. Further mapping has been undertaken during the current phase of investigations and is summarised in Table 15 and the location of the slopes is shown in **Figure 7**. Kinematic slope stability assessment is provided in Appendix G.

Table 15 Existing cut slopes

ID	Location	Height (m)	Angle (°)	Material	Observations/Kinematic stability results
4	Pacific Hwy (west side)	5	45	High to very high strength strong brown pink, slightly weathered fine- to medium grained granite, closely jointed. Becoming completely weathered northward.	Generally stable, localised spalling and water seepage.
5	Pacific Hwy (west side)	10	65	High to very high strength pink, slightly weathered fine- to medium-grained granite, closely jointed	Generally stable, localised spalling, minor water seepage. No kinematic instability identified
6	Pacific Hwy (west side)	7	60	High strength light orange brown, granite, very closely fractured	Localised spalling
7	Pacific Hwy (west side)	5	35	Very high strength, pale yellow, slightly weathered, medium-grained granite	Stable
8	Pacific Hwy (west side)	3	30	Extremely low strength to medium dense yellow brown slightly clayey fine sand (Residual Soil).	Vegetated and stable
9	Pacific Hwy (west side)	8	45	Low to very low strength distinctly to extremely weathered, silty mudstone	Erosion, past instability along bedding
10	Pacific Hwy (west side)	3	25	Low strength grey silty mudstone	Stable
11	Pacific Hwy (west side)	3.5	30	Saprolite	Stable, potential for bedding plane failure on slopes steeper than 40°
12	Pacific Hwy (west side)	2.5	20	Extremely weathered shale	Potential for planar failure on slopes angles steeper than 35°
13	Pacific Hwy (west side)	4	45		Potential for planar failure on slopes steeper than 36°
14	Pacific Hwy (west side)	8	45	Medium to high strength , light grey, distinctly to extremely weathered interbedded mudstone, and meta-siltstone	Generally stable, no kinematic instability
15	Pacific Hwy (west side)	4	35-50	Low to very low strength, distinctly weathered, silty mudstone	Generally stable, potential for toppling failure.
16	Pacific Hwy (west side)	4	25	Extremely low strength, distinctly to extremely weathered, massive conglomerate.	Stable
17	Pacific Hwy (west side)	5	60	High strength, pink, slightly weathered, granite. Close to medium spaced jointing	Potential for wedge, planar and toppling failure
18	Pacific Hwy (west side)	4	60	High strength, pink, slightly weathered, granite. Close to medium spaced jointing	Potential for wedge failure

5.3.1.2 Laboratory testing from Stewarts River to Passionfruit Creek

Classification testing of these soils has been carried out and the results are shown in **Figure 17** and **Figure 18**. These indicate that the residual soils are typically low to medium plasticity clayey sands and silts with some silty clays. Moisture contents range between 16% and 40% and plastic limits range from 24% to 33%. Liquid limits range between 40% and 81% and plasticity indices typically range between 14% and 48%.

Unconfined Compressive Strengths (UCS) tests were undertaken on rock core recovered from BH203 and BH204. The results are summarised in Table 16.

Table 16 UCS results Stewarts River to Passionfruit Creek

Borehole	Depth	Failure mode	UCS (MPa)	Rock Type
203	3.6-3.85	Through core	90.2	Granite
204	13.36-13.53	Through defect	6.5	Granite

5.3.1.3 Laboratory testing Passionfruit Creek

Classification testing of these soils during the options development investigations was carried out in this area. The results are shown in **Figure 19** and **Figure 20**. These indicate that the alluvial soils are typically medium to high plasticity clays and silts. Moisture contents range between 13% and 54% and plastic limits range from 17% to 27%. Liquid limits range between 18% and 60% and plasticity indices typically range between 18% and 33%, with one result (clayey sand) indicating a PI of 1%. The residual soils are typically low plasticity clays with moisture contents between 19% and 21%, and liquid and plastic limits of 38% to 46% and 20% to 21% respectively. Plasticity indices range between 15% and 28% for the residual soils.

Soaked CBR tests on the silty clay alluvial soil in TP12 and TP13 indicate values of 8. These relatively high values may be due to the relatively high percentage of sand. Compaction tests indicate that maximum dry densities of 1.64 tonnes/m³ can be achieved at an optimum moisture content of between 21 and 24.5%.

5.3.1.4 Laboratory testing Stony Creek

Classification testing of these soils has been carried out and the results are shown in **Figure 21** and **Figure 22**. These indicate that the alluvial soils are typically low to medium plasticity clayey sands and sandy clays. Moisture contents range between 18% and 24% and plastic limits range from 17% to 23%. Liquid limits range between 30% and 49% and plasticity indices typically range between 13% and 26%.

Sieve analysis (TP221) indicates the sand fraction in the sandy clay material is fine to coarse grained.

5.3.1.5 Laboratory testing from Passionfruit Creek to Camden Haven River

This section excludes data from Stony Creek.

Classification testing of these manly residual soils has been carried out and the results are shown in **Figure 23** and **Figure 24**. These indicate that the residual soils are typically low to high plasticity silty and sandy clays and low plasticity clayey sand.

Moisture contents range between 11% and 37% and plastic limits range from 17% to 35%. Liquid limits range between 30% and 115% and plasticity indices typically range between 13% and 85%. The liquid limit of 115% and the plasticity index of 85% came from TP227 at 0.65m.

Soaked CBR tests on the residual soils indicate typical values of 2 to 13. The relatively high variation in CBR values may be due to the variable presence of sand in these materials. One CBR test from a sample in TP227 indicates a value of 0.5 in high plasticity silty clay.

Compaction tests indicate that typical maximum dry densities of 1.50 tonnes/m^3 to 1.88 tonnes/m^3 can be achieved at an optimum moisture content of between 16% and 26.5%. An outlying maximum dry density value of 1.35 tonnes/m^3 was encountered in TP227, with an optimum moisture content of 34% in high plasticity silty clay. The low maximum dry density and CBR and high PI in the silty clay at this test pit indicate that this material is unsuitable for use as fill.

Sieve analysis indicates that the sandy clay is generally well graded through the sand fraction with some fine to medium gravel.

Unconfined Compressive Strengths (UCS) tests were undertaken on rock core recovered from BH205, 206A, 207,208 and 209. The results are summarised in Table 17.

Table 17 UCS results Passionfruit Creek to Camden Haven River

Borehole	Depth	Failure mode	UCS (MPa)	Rock Type
205	3.8-3.94	Through defect	2.3	Sandstone/Siltstone
205	5.85-6.00	Through defect	1.5	Siltstone
206A	14.0-14.15	Though defect	0.7	Sandstone
207	8.28-8.43	Through defect	14.5	Sandstone/Siltstone
208	4.50-4.70	Through core	25.6	Granite
208	6.4-6.55	Through core	29	Granite
209	2.38-2.53	Through core	65	Rhyolite
209	6.00-6.14	Through core	172	Rhyolite

5.3.1.6 In situ testing

SPTs were undertaken in boreholes along this section of the preferred option where the depth of soil exceeded 1.5m. This only occurred in BH204, 206 and 207. SPT N values from BH204 show a gradual increase with depth in the residual soil profile.

One SPT test was undertaken in each of BH206 and 207 shortly before auger refusal, with correspondingly high N values.

Undrained shear strengths of 55 kPa to 200 kPa were obtained from pocket penetrometer and hand shear vane tests in test pits in the residual soils.

A seismic refraction survey (SL1) was undertaken in the area to the west of the preferred option, where the Rossglen off/on ramp is planned. The following interpretation is provided of the results:

- Layer 1 460m/s Soil
- Layer 2 1900m/s Extremely weathered rock
- Layer 3 3300m/s Distinctly to slightly weathered rock
- Layer 4 4700m/s Slightly weathered to fresh rock

Results of the seismic survey are provided in Appendix D.

5.3.2 Lake section ground water conditions

During the preferred option investigation, ground water was encountered in BH204 in deep residual soil. Local seepage was encountered in test pits TP214 and 215. Groundwater was encountered in TP221, which was excavated on the floodplain of Stony Creek, and also in TP226. A standpipe piezometer was installed in BH204 for future ground water level and quality monitoring.

During the route selection geotechnical investigation, seepage was evident in TP13 adjacent to Passionfruit Creek at a depth of 2.1 m.

Table 18 Water table depth around Lake Section

Investigation Location	Water Table Depth (m)
BH204	7.45
TP214	Local seepage at 1.1m
TP215	Local seepage at 0.65m
TP221 (Stony Creek)	1.2
TP226	1.7

5.4 Camden Haven River Crossing CH12300 - CH13600

This section of the preferred option follows the existing highway alignment.

The earthworks that will be required are shown on **Figure 26** and are summarised in the table below. In addition to these earthworks new bridges will be required for the railway overpass and the Camden Haven River crossing.

Table 19 Summary of earthworks for Camden Haven River Crossing

Cut/Fill	Length (m)	Approximate Mid-point Chainage	Camden Haven River crossing	
			Max Cut height (m)	Max Fill height (m)
Major Fill	350 South	12400		7
Major Fill	550 North	13100		5

The preferred option passes through the foothills of Middle Brother and the alluvial plain of the Camden Haven River. The land is used primarily for dairy pasture. To the east of the preferred option, north of the Camden Haven River, the land has been designated as SEPP14 Wetland.

The geological map indicates that the preferred option will pass from Camden Haven Beds in the south, across alluvium in the flood plain and onto Byabbara Beds in the north. The soil landscapes identified by DLWC include the Jones Island, Watson Taylors, Pipeclay Canal and Melinga. In addition, DLWC has identified the soils in this area to have a high probability of acid sulfate risk. Tests carried out as part of this study confirm this and are described in greater detail in Section 7.

5.4.1 Field investigations

Preferred option investigations included 2 boreholes, 4 continuous sampling points, 5 test pits and 7 CPTUs.

During the options development investigation, two boreholes (BH21 and 23) five CPTs (CPT4, 6, 7, 10 and 11) and four test pits (TP26, 27, 28 and 50) were undertaken.

Previously four boreholes were bored for the design of the existing road bridge, and the information on these holes is provided in Appendix I.

The locations of test points are given in **Figure 6**. A summary of the ground conditions encountered is given in Table 20 and Table 22 and in **Figure 26**

Due to differing conditions north and south of the river, this section of the preferred option has been divided into Camden Haven River South and Camden Haven River North sections in the sections that follow.

5.4.2 Camden Haven River South

Table 20 Summary of ground conditions - Camden Haven River Crossing South

Depth to top (m)	Typical description	Thickness range (m)	Origin
0	Loose, brown silty sand or sandy gravel	0.3-0.58	Fill
0 - 0.58	Very soft organic/peaty clays and silts, silty and sandy clay, very loose to loose silt-sand mixtures and clay – sand mixtures.	2.6 – 7.6	Alluvium
2.6 - 7.6	Loose or soft becoming firm or medium dense, silty clays and clayey silts and organic clays and silts, with some clay-sand mixtures	2.0 - 7.6	Alluvium
9.0 – 13.0	Medium dense clayey sand or sand, trace of gravel	0.4-1.4	Alluvium
13.8 – 14.4	Interbedded loose to very dense sandy gravels, clayey gravels and gravelly sands.	1.9 - 10.2	Alluvium
12.6 (BH210 only)	Dark grey to black, highly organic, silt and peat	3.2	Alluvium
1.10 - 24.63	Extremely low to low strength, grey shale and sandstone and tuffaceous sandstone	To end of BHs	Bedrock

5.4.2.1 Laboratory testing Camden Haven River South

- **Fill**
  No investigation or laboratory testing was undertaken in the fill in the existing embankments.
- **Clays and silts**
  Classification testing of these soil materials has been carried out and the results are shown in **Figure 27** and **Figure 28**. These indicate that the soils are low plasticity to high plasticity clays and silts. Moisture contents range between 27% and 69% and plasticity limits range from 17% to 28%. Liquid limits range between 33% and 53% and the plasticity indices range between 12% and 25%.
- **Sand and Gravel**
  Classification testing of these soil materials has been carried out and the results are shown in Appendix E. Sieve analysis indicates that the material varies from well-graded sandy gravel to poorly graded gravel. The gravels were generally saturated.

Unsaturated undrained triaxial tests were undertaken on a U50 samples from BH210. The results are summarised in Table 21.

Table 21 Triaxial test results Camden Haven River South

Borehole	Depth	Material	Cu (kPa)
210	2.9-3.3	Silty CLAY	106

This value is not consistent with the SPT N values of zero above and below the U50 sample, and the dry density of 0.93 tonnes/m³. The high, undrained shear strength result is anomalous and can be attributed to the presence of fibrous organic material, which was noted.

5.4.2.2 In situ testing Camden Haven River South

Boreholes

The SPT N values for BH21 drilled during the options development investigation, together with BH210 are shown in **Figure 29**. These test locations are located on the southern flood plain of the river. The SPT N values, in general show no increase with depth to 8m, with values less than 5. This can be attributed to the very loose clayey silts and sands or very soft organic clays in this depth range. Below approximately 8m, the SPT N values increase with depth to approximately 16m through soft silty or sandy clays and loose to medium dense clayey sands and sandy gravels. SPT N values greater than 40 were encountered between 14 and 16m in dense gravels, overlying loose to medium dense sands and gravels or soft to firm organic clays and silts.

CPTs

During the initial investigation, two CPTs (CPT10 and 4) were undertaken south of the river. The CPT results indicate interbedded very loose to loose or soft to firm clay-silt mixtures to13.0m and 15.40m respectively. Both CPTs were terminated when the tip resistance exceeded 45MPa.

Two CPTUs (CPT206 and 207), were undertaken during the preferred option investigation. Adjacent to the highest section of the existing embankment, At CPT206 very soft to soft clay was encountered to a depth of 1.8m overlying very soft or very loose silt-clay mixtures to a depth of about 4.5m overlying very soft to loose interbedded silt, clay and sand mixtures to refusal at a depth of 6.44m with tip resistance exceeding 45MPa.

Results from CPT207 indicate firm clay to a depth of about 1.9m overlying interbedded very loose or soft clay-silt mixtures to a depth of about 10m, overlying very loose sandy or clayey silt. Refusal with tip resistance greater than 45 MPa occurred at a depth of 14.32m.

5.4.2.3 Continuous sampling Camden Haven South

At CSP203 clayey silt with scattered organics and shell fragments was encountered to a depth of 5.35m, overlying clayey sand with thin clay laminations to refusal at a depth of 9.42m on hard sandy clay.

At CSP204 organic silt and silty clay was encountered to a depth of 1.3m overlying silty and clayey sand to a depth of 5.6m overlying silty or sandy clay to a depth of 12.9m. Clayey sands and gravels were encountered between 12.9m and refusal at 16.3m.

5.4.3 Camden Haven River North

A summary of the ground conditions north of the Camden Haven River is given in Table 22.

Table 22 Summary of ground conditions - Camden Haven River Crossing North

Depth to top (m)	Typical description	Thickness range (m)	Origin
0	Loose or very soft sandy silt and organic silt with peat	0.25-0.4	Topsoil
0 (CPT214, CPT215 & CPT11)	Firm, brown, silty and sandy clay	3.2 –3.6	Fill (old highway embankment)
0 –3.6	Very soft organic clays and silts, silty and sandy clay, very loose to loose silt-sand mixtures and clay – sand mixtures.	6.06 -8.6 -	Alluvium
8.8-11.3	Interbedded loose or soft becoming firm or medium dense, silty clays and clayey silts and organic clays and silts, with some clay-sand-gravel mixtures	0- 6.2	Alluvium
9.66-14.6	Interbedded loose to very dense sandy gravels, clayey gravels and gravelly sands.	4.6-10.48	Alluvium
19.57 - 23.48	Extremely low to low strength, grey shale and sandstone, shale or conglomerate	To end of BHs	Bedrock

5.4.3.1 Laboratory testing Camden Haven River North

- **Clays and silts**
  Classification testing of these soils has been carried out and the results are shown in **Figure 31** and **Figure 32**. These tests indicate that there are both high and low plasticity clays and silts with some organics. Moisture contents range between 26% and about 48% and plasticity limits range from 15% to 30%. Liquid limits range between 32% and 64% and the plasticity indices range from 5% to 37%, with one sample in BH23 with a PI of 2%.

- **Sands and gravels**
  Classification testing of these soil materials has been carried out and the results are shown in Appendix E. Sieve analysis indicates that the material varies from well-graded clayey sand to poorly graded gravel. The gravels were generally saturated.

Unsaturated undrained triaxial tests were undertaken on U50 samples from BH211. The results are summarised in Table 23.

Table 23 Triaxial test results Camden Haven River North

Borehole	Depth	Material	Cu (kPa)
211	1.35-1.75	Silty CLAY	22
211	8.3-8.7	Sandy CLAY	105

5.4.3.2 In situ testing Camden Haven River North

Boreholes

The SPT N values for BH23 drilled during the initial investigation, together with BH211, are shown in **Figure 33**.

SPT N values in BH211 are 0 above 8m below ground level due to the presence of high plasticity organic clays and silts, while values of 1 and 2 were encountered in silty and clayey sands and silty clay in BH23.

Below approximately 8m, the SPT N values in both boreholes generally increase with depth in medium dense to very dense silty sands and sandy gravels.

Test Pits

Undrained shear strengths were extremely low and no strength could be recorded with a pocket penetrometer in test pits. Undrained shear strengths from hand shear vane tests of 35 kPa were encountered in TP233 at approximately 1.5m below ground level.

CPTs

CPT6 and CPT7 undertaken during the options development investigation, indicate the presence of soft normally consolidated clays and silts to depths of 12.5m and 11.5m below ground level respectively. Termination of the tests occurred on dense sands or gravels with tip resistance, Qc becoming greater than 50MPa or inclination increasing to its critical threshold.

At CPT208 and 209 similar profiles to CPT6 and 7 were encountered and were terminated at depths of 15.54m and 13.94m on dense sands or gravels. Termination of CPT209 occurred when the ground anchors, providing the reaction for the CPT failed in the soft ground. Termination of CPT208 occurred when the tip resistance, Qc exceeded 50MPa.

To the east of the present highway lies an embankment constructed for the old Pacific Highway alignment. To enable CPTs to penetrate this material containing zones of gravel and cobbles, pits were excavated to depths of approximately 3.5m and backfilled with loose fill to complete CPT214 and 215. Thus the material encountered above 3.2 to 3.5m in these tests can be regarded as disturbed fill. Below this depth predominately very loose or very soft sensitive fine grained material was encountered at CPT214 and CPT215, overlying medium dense to dense sands and gravels at depths of 9.66m and 11.3m respectively.

5.4.3.3 Continuous sampling Camden Haven North

At CSPs 205 and 206 very loose clayey sandy silt, sandy silt and silty sandy clay were encountered to depths of 4.3m and 4.9m respectively, overlying very loose, very fine to coarse silty and clayey sands with some organic matter and shell fragments and occasional organic silty clay horizons to depths of 14.2m and 12.7m respectively. Below this depth, gravelly sand and sandy gravel were encountered overlying hard clay at 15.8 and 15.7m where refusal occurred.

5.4.4 Camden Haven River ground water conditions

During the preferred option investigations water was encountered in TP231 and TP232 as well as in TP233, where moderate inflow was encountered. Water was also encountered in BH210 and BH211. Recorded levels are shown in Table 24.

Table 24 Water table depth around Camden Haven River

Investigation Location	Water Table Depth (m)
BH210	1.87
BH211	1.57
AS213	Seepage from 1.8m
TP233	Seepage from 1.0m
TP232	Seepage from 0.6m
TP231	Seepage from 1.3m
BH21	1.00
BH23	2.50
TP27	0.00 (At surface)
TP28	0.00 (At surface)
TP26	Seepage at 2.10

5.5 Kew Bypass CH13600 - CH17100

This section includes an upgrade of part of the existing highway alignment south of Kew and a new bypass to the east of the town. The bypass re-joins the existing alignment to the north of the town.

The earthworks that will be required for the preferred option are summarised in Table 25.

Table 25 Summary of earthworks for Kew Bypass

Cut/Fill	Length (m)	Approximate Mid-point Chainage	Kew Bypass	
			Max Cut height (m)	Max Fill height (m)
Fill	1000	13500		2.5
Major Cut	400	14700	19	
Minor Fill	200	15050		3.5
Cut	150	15300	3.5	
Minor Fill	100	15350		3
Major Cut	450	15750	10	
Major Cut	400	16250	10	
Fill	400	16600		5
Fill	200	17000		1.5

The preferred option passes from the Camden Haven Flood Plain onto moderately steep, to steep hills to the south-east and north-east of Kew. The land-use is generally woodland and pasture, with new housing sub-divisions being created to the east to Kew. The preferred option passes through the disused Bethesda Quarry to the south of the town.

The geological map indicates that the preferred option crosses the Byabbara Beds for almost the entire length, except for a minor outcrop of granitic rock 1.5km north of the Camden Haven River. The soil landscapes identified by DLWC include Melinga and Grants Head.

5.5.1 Existing cut slope mapping

Table 26 summarises the conditions of existing cut slopes, which were mapped in the Kew area.

Table 26 Existing cut slopes

ID	Location	Height (m)	Angle (°)	Material	Observations/Kinematic stability results
19	Pacific Hwy (west side)	20	45	High strength, pale grey granitic rock. Very closely to closely jointed, Carbonaceous shale and mudstone interbeds.	Generally spalling. No berms
20	Pacific Hwy (east side)	10	35	High strength, pale grey granitic rock. Very closely jointed, Carbonaceous shale and mudstone interbeds.	Generally stable
21	Pacific Hwy (west side)	3	40	High strength, moderately weathered, lithic sandstone. Closely bedded and folded	Sheared, generally stable
22	Pacific Hwy (east side)	5	45	High strength, moderately weathered, lithic sandstone. Closely bedded and folded	Generally stable
23	Kendall Road	5	20	High to medium strength, fine-grained meta-sandstone	Fretting
24	Bethesda Quarry	2	45	Medium strenght, distinctly weathered, interbedded felsic tuff, mudstone and quartz conglomerate	Conglomerate clasts very strong
24	Ocean Drive	2.5	76	Low to medium strength interbedded shale and meta-siltsone and felsic tuff	Generally unstable (fretting, block, wedge, slip)
25	Pacific Hwy (east side)	5	50	Medium stength, interbedded mudstone and sandstone	Generally stable

5.5.1.1 Field investigations

During the preferred option investigations, seventeen test pits (TP236-252), two boreholes (BH212 and 213), and three seismic lines (SL01 to SL03) were completed. Of the 17 test pits, 6 were excavated for environmental sampling around nurseries near Bethesda Road and Ocean Drive. These environmental sampling locations were also logged as geotechnical test pits.

The fieldwork undertaken during the initial investigations along the preferred option incorporated four boreholes (BH26, 36, 38 and 31) and three test pits (TP50, 49 and 33). The locations of all points are given in **Figure 6**.

A summary of the ground conditions encountered is given in the Table 27 and shown in Figure 34.

Table 27 Summary of ground conditions for Kew Bypass

Depth to top (m)	Typical description	Thickness range (m)	Origin
0	Soft - firm black sandy silt to soft brown sandy clay/clayey sand with gravel (loose brown clayey gravelly sand FILL in TP50).	0.20-0.55	Topsoil
0.- 0.8	Soft - hard, grey/brown/red/yellow mottled orange sandy clay and silty clay or loose to dense light grey-orange clayey or silty sand, sandy silt or clayey silt.	0.2 - > 3m	Residual Soil
0.3 - >3m	Very low to high strength sandstone, medium to extremely high strength tuffaceous sandstone, low strength shale/siltstone and very low to extremely high strength tuff.	To end of Boreholes (>19.63m)	Bedrock

5.5.1.2 Laboratory testing Kew Bypass

Classification testing

Classification testing of these soils has been carried out and the results are shown in **Figure 35** and **Figure 36**. The test results indicate that the residual soils are typically high plasticity silty or sandy clays and clayey sand. Moisture contents of the residual soils range between 11% and 34% and plastic limits range from 16% to 32%. Liquid limits for residual soils typically range between 30% and 82%. Plasticity indices typically range between 30% and 89%.

Laboratory testing on material from TP50 indicated a soaked CBR value of 6 from a silty clay residual soil. Compaction tests indicate that maximum dry densities of 1.49 tonnes/m^3-1.58 tonnes/m^3 can be achieved at an optimum moisture content of 23%-27% with the silty clay material. A soaked CBR value of 8 was obtained from a clayey sand slope-wash from TP33. Compaction tests for this soil indicate that maximum dry densities of 1.82 tonnes/m^3 can be achieved at an optimum moisture content of 14.5%. Also, soaked CBR values of 4 and 21 respectively were obtained from the sandy clay residual soil from TP33 and TP49. Compaction tests for these soils indicate that maximum dry densities of 1.52 tonnes/m^3-1.67 tonnes/m^3 can be achieved at an optimum moisture content of between 19.5%-24.5%.

California bearing ration and compaction testing

Soaked CBR results of 3.5, 6 and 3 from TP238, 245 and 252, respectively, were recorded during the current ground investigation from silty and sandy residual clays. Maximum dry densities of 1.35, 1.83 and 1.57 tonnes/m^3 were recorded at optimum moisture contents of 32.5, 15.0 and 23% in TP238, TP245 and TP252 respectively.

Soaked CBR results of 16 and 9 from TP241 and TP244, respectively, were recorded during the current ground investigation in residual clayey or silty sands. Maximum dry densities of 1.75 tonnes/m^3 and 1.66 tonnes/m^3 were recorded at optimum moisture contents of 17.5% and 20.5% in TP241 and TP245, respectively.

Sieve analysis on sandy clay from TP245 indicates that the material has a sand fraction of approximately 40% and that this fraction is well graded. This may account for the relatively high soaked CBR values compared with other residual sandy to silty clays along the preferred option.

Sieve analysis on silty sand encountered in TP244 indicates that the sand is well graded.

Unconfined compressive strength test results

Results for the Unconfined Compressive Strength (UCS) testing carried out on the rock core are summarised in the table below.

Table 28 UCS results Kew Bypass

Borehole	Depth	Failure mode	UCS (MPa)	Rock Type
BH26	3.21	Though core	204.8	Tuffaceous Sandstone
BH26	9.5	Though defect	85.1	Tuffaceous Sandstone
BH36	5.73	Through core	57.4	Sandstone
BH31	5.66	Through core	43.8	Tuffaceous Sandstone
BH31	8.78	Through core	120	Tuffaceous Sandstone
BH31	14.31	Through core	31.6	Tuffaceous Sandstone
BH212	7.8-7.95	Through defect	15.5	Sandstone
BH212	11.64-11.74	Through core	76.5	Sandstone
BH213	6.65-6.80	Through defect	23.3	Siltstone
BH213	11.30-11.55	Through defect	6.3	Siltstone

5.5.1.3 Insitu testing

Only one SPT in BH38 was taken in the area due to the shallow depth to bedrock in each of the boreholes. The one SPT was located in stiff sandy clay.

Undrained shear strengths for the residual soils ranging from 85 kPa-200 kPa were obtained from pocket penetrometer and hand shear vane tests.

A seismic refraction survey was undertaken at three locations within this section: at the proposed cutting location near the Bethesda Nursery and to the north and south of the proposed crossing of Ocean Drive. The results of this survey indicate a seismic velocity profile consisting of four layers, which are provided with some indicative interpretation below:

Bethesda Nursery

- Layer 1 750m/s Soil
- Layer 2 1750m/s Distinctly to extremely weathered rock
- Layer 3 2900m/s Slightly weathered rock
- Layer 4 4500m/s Fresh rock

South of Ocean Drive

- Layer 1 700m/s Soil
- Layer 2 1400m/s Distinctly to extremely weathered rock
- Layer 3 2550m/s Distinctly weathered rock
- Layer 4 3900m/s Slightly weathered to fresh rock

North of Ocean Drive

- Layer 1 750m/s Soil
- Layer 2 1850m/s Distinctly to extremely weather rock (many defects)
- Layer 3 3550ms/ Distinctly weathered rock (few defects)
- Layer 4 4300m/s Slightly weathered rock

Results of the seismic survey are provided in Appendix D.

5.6 Northern Section CH17100 - CH 22200

The northern section includes an upgrade of the existing highway to link into the existing dual carriageway north of Herons Creek.

Only minor earthworks (cut and fills less than 4m high) will be required within this area. These are summarised in Table 29 and are shown in **Figure 37**.

Table 29 Summary of earthworks for Northern Section

Cut/Fill	Length (m)	Approximate Mid-point Chainage	Northern Section	
			Max Cut height (m)	Max Fill height (m)
Minor Fill	350 (cont)	17000		4
Minor Cut	300	17500	3	
Minor Fill	230	17800		4
Minor Cut	200	18000	3	
Minor Fill	150	18200		3
Minor Cut	180	18400	4	
Minor Fill	170	18550		3
Minor Cut	200	18750	1.5	
Minor Fill	750	119200		2
Minor Cut	225	19800	1	
Minor Fill	325	20050		2
Minor Cut	300	20450	3	
Minor Fill	550	21000		4
Minor Fill	250	21400		4
Minor cut	150	21750	6	

The preferred option generally follows the existing highway and crosses low rolling hills from Kew to Herons Creek.

The geological map indicates that the preferred option crosses Byabbara Beds for the entire length. The soil landscapes identified by DLWC include Melinga, Princes Upsalls Creek and Herons Creek.

5.6.1.1 Field investigations

During the current field investigation, nine test pits (TP253-261) and one borehole (BH214) were undertaken. Of the nine test pits, two were excavated for environmental sampling at a disused petrol station.

During the initial investigation, two boreholes (BH34 and 45) and four test pits (TP35, 60, 39 and 42) were excavated along the preferred option. The locations of all investigation points are given in **Figure 6**.

A summary of the ground conditions encountered is given in Table 30 and shown in **Figure 37**.

Table 30 Summary of ground conditions for Northern Section

Depth to top (m)	Typical description	Thickness range (m)	Origin
0.0-0.2	Soft-firm dark grey-black sandy clayey silt/ silty sand	0.10-0.20	Topsoil
0.00-0.60	Very soft to firm grey-dark brown mottled orange sandy clay, gravelly clay and loose to medium dense orange- light brown silty sand/sandy silt.	0.2 - 8.5	Alluvial (Herons Creek)
0.00-8.50	Firm-stiff light grey-orange mottled off white sandy clay to very dense clayey sand. Medium dense to very dense silty sand/clayey sand. Stiff to hard grey silty clay/clayey silt	1.00-6.79	Residual Soil
1.30-10.58	Low strength sandstone, very low to low strength inter-bedded sandstone/shale and very low to very high strength blue-grey granite	To end of Boreholes	Bedrock

5.6.1.2 Laboratory testing North Section

Classification testing of these soils has been carried out and the results are shown in **Figure 38** and **Figure 39**. These indicate that the residual soils are typically low and high plasticity clays. Moisture contents range between 12% and 33% and plastic limits range from 16% to 48%. Liquid limits range between 29% and 87%. Plasticity indices range between 9% and 55%.

Soaked CBR testing undertaken on samples within the preferred option included TP35, 60 and 42, 254, 256 and 260. Soaked CBR values from sandy clay residual soil in TP35 and TP60 indicate values between 3.5 and 12. Compaction tests indicate that maximum dry densities of 1.49 tonnes/m^3 -1.83 tonnes/m^3 can be achieved at an optimum moisture content of 14.5%-26% within the sandy clay material. A soaked CBR value of 2.5 was obtained from a silty clay (residual soil) from TP60. Compaction tests for this material indicate that a maximum dry density of 1.47 tonnes/m^3 can be achieved at an optimum moisture content of 27.7%. A soaked CBR value of 12 was obtained from a silty sand (residual soil) from TP42. Compaction tests for this material indicate that a maximum dry density of 1.77 tonnes/m^3 can be achieved at an optimum moisture content of 16.5%. Soaked CBRs of 2.5 were recorded for silty clays from TP254 and TP256 (residual soil). Compaction tests indicate that maximum dry densities of 1.45-1.47 tonnes/m^3 at an optimum moisture content of 27% can be achieved from these samples. A soaked CBR of 2.0 was obtained for residual silty clay from TP261. The compaction test indicates that dry densities of 1.68 tonnes/m^3 can be achieved at an optimum moisture content of 21.5%.

5.6.1.3 Laboratory testing Herons Creek

Classification testing of these soils has been carried out and the results are shown in **Figure 40** and **Figure 41**. The test results indicate that the alluvial soils encountered at Herons Creek are typically low plasticity sandy clays and silts overlying sand and gravelly sands at greater depths. Moisture contents range between 18% and 25% and plastic limits range from 22% to 27%. Liquid limits typically range between 31% and 43%. Plasticity indices range between 9% and 16%.

Unconfined Compressive Strengths (UCS) tests were undertaken on rock core recovered from BH214 and 34. The results are summarised in Table 31.

Table 31 UCS results Herons Creek

Borehole	Depth	Failure mode	UCS (MPa)	Rock Type
BH214	9.0-9.15	Through core	61.0	Siltstone
BH34	13.56	Through core	1.0	Sandstone

5.6.1.4 In situ testing

The SPT N values for BH34, drilled during the initial investigation, together with BH214, drilled during the preferred option investigation, are shown in **Figure 42**. This figure indicates that the SPT N values from BH34 in the alluvium do not increase with depth. However, when residual soils are encountered, the SPT N values increase significantly. Similarly in BH214, when residual soils are encountered, the SPT N values are generally higher.

Undrained shear strengths ranging from 50 kPa-175 kPa were obtained from pocket penetrometer tests for the residual soil and 25 kPa-50 kPa for the alluvial soils at Herons Creek.

5.6.2 Northern Section ground water conditions

During the initial geotechnical investigation, water was encountered in BH34 at a depth of 2.5 m. Water was encountered at 4.98m in BH214 during the preferred option investigations. Water was not encountered in any of the test pits excavated in this section.

Table 32 Water table depth for Northern Section

Investigation Location	Water Table Depth (m)
BH34	2.5
BH214	4.98

6. POTENTIAL FOR CONTAMINATION

A review of past land use activities which might have led to ground contamination was carried out as par of the options development studies. This information was used to establish targets for investigation during the preferred option investigation.

6.1 Site investigation

6.1.1 Method

Following a review of current land use and the presence of buildings (occupied and disused) a series of locations were identified where it was deemed that soil and groundwater samples should be taken in order to analyse for potential contamination. The locations of the trial pits and boreholes are illustrated in **Figure 6**. The locations and their associated land use are described in the table below.

Table 33 Land-use description at sampling locations

Sample Point	Land Use Description
TP231	Agricultural (for improved pasture)
TP232	Agricultural (for improved pasture)
TP233	Agricultural (for improved pasture)
TP236	Agricultural (for improved pasture)
TP237	Agricultural (for improved pasture)
TP209	Agricultural (for improved pasture)
TP210	Agricultural (for improved pasture)
TP240	General (for garden nursery)
TP242	General (for garden nursery)
TP243	General (for garden nursery)
TP246	General (for garden nursery)
TP247	General (for garden nursery)
TP248	General (for garden nursery)
TP259	Old petrol station
TP260	Old petrol station
BH201	Groundwater Stewarts River
BH210	Groundwater Camden Haven River
BH211	Groundwater Camden Haven River
BH214	Groundwater Herons Creek

Fifteen trial pits (TPs) were excavated at locations along the route. All excavations were undertaken using a rubber wheeled backhoe with a 600mm wide bucket. Soil samples were taken at selected soil strata for testing purposes. The trial pit logs are provided in Appendix B. Explanatory notes describing the terms and symbols used in their preparation are provided in Appendix A.

Four boreholes (BHs) were drilled to test groundwater in the vicinity of the main watercourses in the area. The boreholes were drilled using either a Ute mounted Explorer rig, a truck mounted Edson 3000 or Hydrapower 5000 rigs, using rotary coring techniques. Auger drilling with Tungsten carbide bits was used in soils above bedrock. The boreholes were generally cased within the overburden material to prevent caving. All boreholes were drilled vertically with water flush and occasionally with drilling mud where the core was too fractured to be recovered otherwise. NMLC core barrels were used and these produced a core of approximately 50mm in diameter. Boreholes were drilled to a depth of at least 2.5m below the proposed formation level at the time of the investigation. Copies of the borehole logs are provided in Appendix A. Explanatory notes describing the terms and symbols used in their preparation are provided in Appendix A.

Standpipe piezometers were installed in each borehole to allow monitoring of ground water levels and to provide access for ground water sampling.

Groundwater samples were taken for laboratory analysis at the time of standpipe installation (July 2002) and again during October 2002 when it was found that the initial samples had been contaminated with drilling mud.

The trial pit and borehole locations were chosen to achieve a broad spread along the preferred option and to be representative of the land uses of the area. Soil samples were taken from the trial pits and densely packed into glass jars. The samples were labelled and then stored in ice-chilled eskies until delivery to the laboratory. Samples delivered to the laboratory were accompanied by chain of custody documentation to specify the analyte range. A total of 24 samples were collected.

A geotechnical engineer supervised all site investigation works.

6.1.2 Laboratory analysis

Based on the potential contaminants identified in Section 3 of this report, 24 soil samples and four groundwater samples were selected for chemical analysis. The samples were submitted to ALS Environmental in accordance with the following schedule:

- TPs 231, 232, 233, 236, 237, 209 and 210 – organochlorine pesticides (OCP), organophosphorous pesticides (OPP) and polychlorinated biphenyl (PCB).
- TPs 240, 242, 243, 246, 247 and 248 – total petroleum hydrocarbons (TPH), BTEX, polynuclear aromatic hydrocarbons (PAH), OCP, PCB, arsenic, cadmium, chromium, copper, nickel, lead, zinc and mercury.
- TPs 259 and 260 – TPH, BTEX and PCB.
- BHs 201, 210, 211 and 214 – arsenic, cadmium, chromium, copper, nickel, lead, zinc, mercury, total cyanide, fluoride, phenols, OCP, TPH, BTEX and PAH.

ALS is NATA accredited for all the analyses undertaken. Certificates detailing test results together with Chain of Custody and Receipt Advice forms are included in Appendix H.

6.2 Assessment of results

6.2.1 Assessment criteria

6.2.1.1 Soil

Guideline values, referred to as Soil Investigation Levels (SILs), are published in *Contaminated Sites: Guidelines for the NSW Site Auditor Scheme*, (NSW EPA 1998). These values have been developed for use when assessing urban redevelopment sites and do not apply to land being, or proposed to be, used for agricultural purposes. The publication *Contaminated Sites: Guidelines for the Vertical Mixing of Soil on Former Broad-Acre Agricultural Land* (NSW EPA 1995) refers to the use of those criteria contained in the *Australian and New Zealand Guidelines for the Assessment and Management of Contaminated Sites* (ANZECC/NHMRC, 1992). Finally, soil investigation levels are also published in the National Environment Protection (Assessment of Contamination) Measure (NEPM) 1999 *Schedule B(1) Guideline on the Investigation Levels for Soil and Groundwater*.

For the purposes of this assessment the NEPM SILs for land use A ('Standard' residential with garden/ accessible soil), which is identical to the NSW SILs for the same end use, have been adopted. This is more stringent than required for the proposed end use but represents a conservative assessment such that provided all substances tested are below these values then no further action is deemed necessary. These assessment criteria are presented together with the results of the soil testing analyses in Table 34, Table 35 and Table 36

6.2.1.2 Groundwater

Due to the proximity of the groundwater monitoring boreholes to rivers and to the rural nature of the area, the groundwater investigation levels published in the National Environment Protection (Assessment of Contamination) Measure (NEPM) 1999 *Schedule B(1) Guideline on the Investigation Levels for Soil and Groundwater*, Aquatic Ecosystems (fresh waters) would be appropriate for use to assess potential groundwater contamination. However, the NEPM 1999 (Schedule B(1)) is based upon the use of the 1992 ANZECC criteria but these have since been updated (ANZECC, 2000). Consequently these latter guidelines have been used for the assessment of groundwater issues in this report. In particular, the trigger values for freshwater aquatic ecosystems with a 95% and 80% level of protection have been used. These assessment criteria are presented together with the results of the groundwater chemical analyses in Table 37.

6.2.2 Soil contamination

The soil analytical results are presented in tabular format in Table 34, Table 35 and Table 36. Laboratory test certificates are presented in Appendix H. Without exception all the results are below the SILs adopted for this assessment and almost all, with the exception of some of the heavy metals, are below the limit of recording (LOR) for the laboratory test methods used.

Table 34 Analytical results for soils (mg/kg)

Substances	LOR	HIL A	TP240	TP240	TP242	TP242	TP243	TP246	TP247	TP248	TP248
			0.1-0.2m	0.9-1.0m	0.1-0.2m	0.9-1.0m	0.1-0.2	0.1-0.2m	0.1-0.2m	0.1-0.2m	0.5-0.6m
Arsenic	0.1	100	6	6	6	5	8	3	4	5	9
Cadmium	1	20	<1	<1	<1	<1	<1	<1	<1	<1	<1
Chromium	1	120,000	2	4	3	2	2	2	1	1	3
Copper	1	1000	4	3	5	<1	<1	<1	<1	<1	<1
Nickel	1	600	<1	<1	<1	<1	<1	<1	<1	<1	<1
Lead	1	300	12	13	15	13	17	5	7	5	10
Zinc	1	7000	12	32	30	21	9	1	3	2	10
Mercury	0.1	15	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.2	<0.1

Table 35 Analytical results for soils (mg/kg)

Substances	LOR	HIL 'A'	TP209	TP210	TP231	TP232	TP233	TP236	TP237	TP240	TP240	TP242	TP242	TP243	TP246	TP247	TP248	TP248
			0.1-0.2m	0.1-0.2m	0.1-0.2m	0.1-0.2m	0.1-0.2m	0.1-0.2m	0.1-0.2m	0.1-0.2m	0.9-1.0m	0.1-0.2m	0.9-1.0m	0.1-0.2m	0.1-0.2m	0.1-0.2m	0.1-0.2m	0.5-0.6m
Polychlorinated Biphenyl	0.1	10	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
Organochlorine Pesticides																		
alpha-BHC	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
HCB	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
beta-BHC & gamma-BHC	0.1	-	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
Delta-BHC	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Heptachlor	0.05	10	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Aldrin	0.05	10	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Heptachlor epoxide	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Chlordane-trans	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Endosulphan	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Chlordane-cls	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Dieldrin	0.05	10	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
DDE	0.05	200	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Endrin	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Endosulfan 2	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
DDD	0.05	200	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Endrin aldehyde	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05

Substances	LOR	HIL 'A'	TP209	TP210	TP231	TP232	TP233	TP236	TP237	TP240	TP240	TP242	TP242	TP243	TP246	TP247	TP248	TP248
			0.1-0.2m	0.1-0.2m	0.1-0.2m	0.1-0.2m	0.1-0.2m	0.1-0.2m	0.1-0.2m	0.1-0.2m	0.9-1.0m	0.1-0.2m	0.9-1.0m	0.1-0.2m	0.1-0.2m	0.1-0.2m	0.1-0.2m	0.5-0.6m
Endosulfan sulphate	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
DDT	0.2	200	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2
Endrin ketone	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Methoxychlor	0.2	-	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2
Organophosphorous Pesticides																		
Dichlorves	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Demeton-S-methyl	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Monocroptophos	0.2	-	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2
Dimethoate	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Diazinon	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Chlorpyrifos-methyl	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Parathion-methyl	0.2	-	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2
Malathion	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Fenthion	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Chlorpyrifos	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Parathion	0.2	-	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2
Pirimphos-ethyl	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Chlorfenvinphos E	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Chlorfenvinphos 2	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Bromophos-ethyl	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Fenamiphos	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Prothiofos	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Ethion	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Carbophenothion	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Azinphos-methyl	0.05	-	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05

Table 36 Analytical results for soils (mg/kg)

Substance	LOR	HIL 'A'	TP240	TP240	TP242	TP242	TP243	TP246	TP247	TP248	TP248	TP259	TP259	TP259	TP259	TP260	TP260	TP260	TP260
			0.1-0.2m	0.9-1.0m	0.1-0.2m	0.9-1.0m	0.1-0.2m	0.1-0.2m	0.1-0.2m	0.1-0.2m	0.5-0.6m	0.1-0.2m	0.9-1.0m	1.7-1.8m	2.6-3.0m	0.1-0.2m	0.8-0.9m	1.6-1.7m	2.8-2.9m
Total Petroleum Hydrocarbons																			
C6-C9	2	-	<2	<2	<2	<2	<2	<2	<2	<2	<2	<2	<2	<2	<2	<2	<2	<2	<2
C10-C14	50	-	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50
C15-C28	100	-	<100	<100	<100	<100	<100	<100	<100	<100	<100	<100	<100	<100	<100	<100	<100	<100	<100
C29-C36	100	-	<100	<100	<100	<100	<100	<100	<100	<100	<100	<100	<100	<100	<100	<100	<100	<100	<100
BTEX																			
Benzene	0.2	-	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2
Toluene	0.2	-	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2
Chloroenzene	0.2	-	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2
Ethylbenzene	0.2	-	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2
Meta- & para- Xylene	0.2	-	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2
Ortho-Xylene	0.2	-	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2
Polynuclear Aromatic Hydrocarbons																			
Naphthalene	0.5	-	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Acenaphthylene	0.5	-	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Acenaphthene	0.5	-	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Fluorene	0.5	-	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Phenanthrene	0.5	-	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Anthracene	0.5	-	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Fluoranthene	0.5	-	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Pyrene	0.5	-	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Benz(a)anthracene	0.5	-	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Chrysene	0.5	-	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Benzo(b)fluoranthene	0.5	-	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Benzo(k)fluoranthene	0.5	-	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Benzo(a)pyrene	0.5	1	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Indeno(1.2.3.cd)pyrene	0.5	-	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Dibenz(a.h)anthracene	0.5	-	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Benzo(g.h.j)perylene	0.5	-	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5

6.3 Groundwater contamination

The groundwater analytical results are presented in Table 37. Laboratory test certificates are presented in Appendix C.

6.3.1 Metals and metalloids

The concentrations of metals in the groundwater bores varied greatly between the two sampling periods (29 July 2002 and 10 October 2002). Concentrations of cadmium, copper, lead and zinc above the 80% protection limits were recorded in the majority of the four boreholes. In addition, concentrations of nickel above the 95% protection level were recorded in boreholes 201 and 214.

Concentrations of these metals had in all cases fallen when sampling took place some three months later and were well below those recorded in July. The concentration of cadmium was slightly higher than the 80% protection level in BH201 as was zinc. Zinc concentrations were also higher than the 80% protection level in all the other boreholes. Lead was higher than the 80% protection level in BH214 as was copper in BHs 210 and 214.

These results contrast with surface water testing undertaken in the nearby watercourses in February and August 2001 (Appendix H). Tests for cadmium and lead in Herons Creek (the location of BH214), Camden Haven River (BH210 & BH211) and Stewarts River (BH201) were all below the limit of recording. So in spite of some slightly elevated concentrations of these metals being recorded in the groundwater, this is not being expressed in the surface waters.

6.3.2 Hydrocarbons

There are no assessment criteria for total petroleum hydrocarbons (TPH). However, the samples from BHs 201 and 211 were to all intents and purposes below the limit of recording. The concentrations of the C6-C9 and C10-C14 fractions in BH210 were elevated and together provide a total of 575μg/L, which is indicative of some potential contamination. The concentrations of TPH in BH214 were high, particularly for the C10-C14 and C15-C28 fractions. BH214 was located in the vicinity of former petrol station site and the concentrations recorded indicate a high probability of contamination associated with this past use.

Elevated PAH concentrations were detected in the samples from BH210 taken in both July and October 2002. In particular, the naphthalene concentration in BH210 was above the 80% protection level recorded in the October sample. The concentration of 2-methylnaphthalene was also highly elevated. Elevated concentrations of these two PAHs were also detected in BH211 and BH214. BH214 also recorded elevated concentrations of acenaphthene, fluorine and phenanthrene. Slightly elevated concentrations of fluorine and phenanthrene were also detected in BH210. None of these substances have been allocated a protection level trigger value.

PAHs are primarily the result of man made (anthropogenic) sources and it is noted that both BHs 210 and 214 exhibit the highest concentrations and ranges of PAHs while at the same time exhibiting elevated TPH concentrations. In this regard, it seems likely therefore that the PAHs detected in the groundwater in BH214 are associated with the previous use of the site for a petrol station. No such link exists, however, for BH210 located near Camden Haven River and this is the borehole with the highest concentrations of PAH.

Most PAH occurring naturally in the environment is caused by combustion such as wildfires. However, wildfire combustion is extremely fast and hot and discussion with the chief organic chemist at the testing laboratory indicates that the nature of the TPH chromatograph for BH210 indicates that this has been generated under ‘slow bake’ conditions. However, it is not possible to conclude one way or the other from the available data whether or not the PAHs in BH210 are from a natural source or not.

The TPH chromatograph for the sample from BH214 shows some resemblance to weathered kerosene, perhaps mixed with a small amount of weathered diesel, providing more evidence that this is the result of contamination caused by the operation of the old petrol filling station.

6.3.3 Others

All the tests for organochlorine pesticides and polychlorinated biphenyls gave results below the level of recording.

Table 37 Analytical results for groundwater (μg/L)

Substances	LOR	Trigger Values Freshwater Aquatic Ecosystems		July 2002				October 2002			
		95% protection level	80% protection level	BH201	BH210	BH211	BH214	BH201	BH210	BH211	BH214
Metals & Metalloids											
Arsenic	1	24	360	1	4	<1	1	2	<1	7	<1
Cadmium	1	0.2	0.8	6	4	<1	3	1	<1	<1	<1
Chromium	1	ID	ID	7	1	4	3	<1	<1	<1	3
Copper	1	1.4	2.5	18	4	8	129	<1	4	1	6
Nickel	1	11	17	14	6	10	14	<1	<1	1	5
Lead	1	3.4	9.4	346	65	199	181	<1	<1	<1	31
Zinc	1	8.0	31	114	225	92	311	44	47	42	73
Mercury	0.1	0.6	5.4	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
Non-Metallic Inorganics											
Cyanide	5	7	18	8	<5	<5	<5	<5	6	12	-
Fluoride	100			300	700	400	700	200	100	300	-
Phenol	50	320	1,200	<50	<50	<50	<50	100	<50	<50	-
Organochlorine Pesticides											
alpha-BHC	0.5			<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
HCB	0.5			<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
beta-BHC & gamma-BHC	1			<1	<1	<1	<1	<1	<1	<1	<1
delta-BHC	0.5			<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Heptachlor	0.5			<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Aldrin	0.5	ID	ID	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Heptachlor epoxide	0.5	0.09[1]	0.7[1]	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Chlordane-trans	0.5	0.08[2]	0.27[2]	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Endosulfan 1	0.5	ID	ID	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Chlordane-cls	0.5	0.08[2]	0.27[2]	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Dieldrin	0.5	ID	ID	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
DDE	0.5	ID	ID	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Endrin	0.5	0.02	0.06	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Endosulfan 2	0.5	ID	ID	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
DDD	0.5			<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Endrin aldehyde	0.5			<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Endosulfan sulphate	0.5			<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
DDT	2	0.01	0.04	<2	<2	<2	<2	<2	<2	<2	<2
Endrin ketone	0.5			<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Methoxychlor	2	ID	ID	<2	<2	<2	<2	<2	<2	<2	<2

Substances	LOR	Trigger Values Freshwater Aquatic Ecosystems		July 2002				October 2002			
		95% protection level	80% protection level	BH201	BH210	BH211	BH214	BH201	BH210	BH211	BH214
Total Petroleum Hydrocarbons (TPH)											
C6-C9	20	ID	ID	c/s	c/s	c/s	c/s	<20	259	<20	365
C10-C14	50	ID	ID	c/s	c/s	c/s	c/s	53	316	<50	8,160
C15-C28	100	ID	ID	c/s	c/s	c/s	c/s	<100	<100	<100	2,590
C29-C36	50	ID	ID	c/s	c/s	c/s	c/s	<50	<50	<50	<50
Polynuclear Aromatic Hydrocarbons (PAH)											
Napthalene	2	16	50	<2	19	2	<2	<2	103	**4**	<2
2-Methylnaphthalene	2			<2	**39**	**11**	<2	2	**176**	**5**	**31**
2-Chloronaphthalene	2			<2	<2	<2	<2	<2	<2	<2	<2
Acenaphthylene	2			<2	<2	<2	<2	<2	<2	<2	<2
Acenaphthene	2			<2	<2	<2	<2	<2	2	<2	**5**
Fluorene	2			<2	3	<2	**3**	<2	**9**	<2	**8**
Phenanthrene	2	ID	ID	<2	2	<2	**3**	<2	**5**	<2	**11**
Anthracene	2	ID	ID	<2	<2	<2	<2	<2	<2	<2	<2
Fluoranthene	2	ID	ID	<2	<2	<2	<2	<2	<2	<2	<2
Pyrene	2			<2	<2	<2	<2	<2	<2	<2	<2
N-2-Fluorenylacetamide	2			<2	<2	<2	<2	<2	<2	<2	<2
Benz(a)anthracene	2			<2	<2	<2	<2	<2	<2	<2	<2
Chrysene	2			<2	<2	<2	<2	<2	<2	<2	<2
Benzo(b) & (k) fluoranthene	4			<4	<4	<4	<4	<4	<4	<4	<4
7.12-Dimethylbenz(a)anthracene	2			<2	<2	<2	<2	<2	<2	<2	<2
Benzo(a)pyrene	2	ID	ID	<2	<2	<2	<2	<2	<2	<2	<2
3-Methylcholanthrene	2			<2	<2	<2	<2	<2	<2	<2	<2
Indeno(1.2.3-cd)pyrene	2			<2	<2	<2	<2	<2	<2	<2	<2
Dibenz(a.h)anthracene	2			<2	<2	<2	<2	<2	<2	<2	<2
Benzo(g.h.i)perylene	2			<2	<2	<2	<2	<2	<2	<2	<2
Polychlorinated Biphenyl (PCB)	1	ID	ID	<1	<1	<1	<1	<1	<1	<1	<1

Notes:
1. Trigger for Heptachlor
2. Trigger for Chlordane

ID = Insufficient data to derive a reliable trigger value.
LOR = Limit of Reporting
- = not measured
c/s – contaminated sample (drilling mud)
Bold text indicates an elevated concentration
An empty cell = this substance is not specifically referred to in the appropriate section of ANZECC, 2000
Highlighted cells indicate exceedances of one or more trigger values

6.4 Discussion of results

6.4.1 Soils

The results of the soil testing indicate that no contamination has occurred at these locations and no further action is required.

6.4.2 Groundwater

At a general level, the results of the groundwater testing indicate that of the four boreholes tested could potentially be contaminated.

The results of the groundwater testing in BH214 close to the old petrol filling station provide a strong indication that the groundwater beneath the site has been contaminated by this past use. However, it is noted that none of the soil testing in this area showed any indication of such contamination. Consequently, it is recommended that a more detailed and targeted chemical site investigation be undertaken to clarify the situation. Neither TPH or PAH were tested for in the surface water testing undertaken at the nearby Herons Creek but a test for oil and grease in August 2001 gave results below the limit of recording. As a result of the contamination levels found in BH214, further investigations were carried out and these are reported in Appendix L.

The issue of the elevated TPH and PAH concentrations associated with BH210 is less clear. However, it is unlikely that dewatering works would be required for the upgrade of the highway at this location. Any localised dewatering would need to be disposed of appropriately.

6.5 Conclusions and recommendations

The following recommendations and conclusions can be made as a result of the investigations carried out during the course of this study.

- The results of the soil testing indicate that there has been no contamination due to human activity at the locations selected and no further action is required.
- Contamination has been identified in the ground water near the former petrol station. This is the topic of a separate investigation and report.

7. ACID SULFATE SOIL

7.1 Background

Acid sulfate soils (ASS), including both actual acid sulfate soils (AASS) and potential acid sulfate soils (PASS), are commonly encountered in alluvial soils below RL5m AHD. Disturbance to these soils could include the excavation of significant volumes of alluvial clay or the potential lowering of the water table through construction processes, excavation of service trenches and piling works.

When PASS material is exposed to the atmosphere the iron sulfides contained within the soil matrix begin to release iron and sulfate ions that in the presence of water form sulfuric acid. At this stage the soil is said to have transformed from PASS to AASS. When surface runoff or groundwater flows through AASS material, the sulfuric acid is transported and becomes a potential risk to various environments.

It is important to note that excavation of ASS material is not the only process by which oxidation and acid production can occur. Artificial lowering of existing water tables may lead to previously submerged PASS becoming exposed to oxygen within the soil structure, and hence result in oxidation and production of AASS material.

7.2 Desktop studies

A desktop study of existing published and unpublished information listed above was carried out to determine the likelihood and location of any potential acid sulfate soil (PASS). The soil landscape and acid sulfate risk maps were the primary sources for delineating PASS. In addition, aerial photographic interpretation of project specific photography was carried out to enhance the resolution of the information provided by these published maps. (In general this additional interpretation was in agreement with the published maps and only minor changes to soil landscape boundaries were necessary). The following criteria have been used to assist with the determination of PASS:

- sediments of recent geological age (Holocene);
- soil horizons less than 5 m AHD;
- marine or estuarine sediments and tidal lakes;
- coastal wetlands, back swamps waterlogged or scalded areas; and
- areas where the dominant vegetation is mangrove.

7.3 Previous ASS investigations

A number of previous studies of soils in the area have been referenced, including:

- DLWC, 1997, Soil landscapes of the Pacific Highway corridor, Hexham to Corindi (unpublished for the RTA);
- DLWC, Camden Haven, Acid Sulfate Soil Risk Map; and
- SMEC, 1986, Rossglen Acid Sulfate Soils Land Management Plan.
- Arup, 2001, Geotechnical Investigations for Route Selection (Options Development).

Areas of high probability ASS were identified within the study area adjacent to the Camden Haven River, and areas of low probability ASS were identified adjacent to Stewarts River.

The Arup investigation for route selection concluded that field sampling results largely confirmed the findings of the desk study and DLWC Acid Sulfate Risk Maps. The exception was the flood plain of the Stewarts River, where a high risk of ASS was identified in an area shown as low risk in the published maps.

7.4 ASSMAC requirements

The ASSMAC Guidelines (ASSMAC, 1998) provide 'Action Criteria' for varying soil types and volumes of soil to be disturbed, as presented in the table below.

Table 38 ASSMAC guidelines for ASS action criteria

Type of Material		Action Criteria Less than 1000 tonnes disturbed		Action Criteria More than 1000 tonnes disturbed	
Texture Range	**Clay Content (%)**	**Sulfur Trail S_{POS} (%)**	**Acid Trail TPA or TSA (kg H_2SO_4 /tonne)**	**Sulfur Trail S_{POS} (%)**	**Acid Trail TPA or TSA (kg H_2SO_4 /tonne)**
Coarse Texture Sands to loamy sands	<5	0.03	0.883	0.03	0.883
Medium Texture Sandy loams to light clays	5-40	0.06	1.766	0.03	0.883
Fine Texture Medium to heavy clays and silty clays	>40	0.10	3.041	0.03	0.883

For the purposes of this section of the report, soil volumes in excess of 1000 tonnes have been presumed to be required to be excavated, and subsequently the Action Criteria limits are:

- **0.03** percent oxidisable sulfur for the Sulfur Trail; and
- **0.833** kg H_2SO_4/tonne (which is equivalent to 18 mole H^+/tonne) for the Acid Trail.

Works in soils that exceed these Action Criteria require preparation of an ASS investigation, an ASS management plan.

7.5 Preferred option investigation

7.5.1 Report scope

This report comprises an ASS Investigation for two locations along the route. These locations had been identified during the preferred option selection stage to contain PASS.

- a length of 530m just south of the Stewarts River crossing;
- a length spanning (but not including) the Camden Haven River, including 330m just south of the river and 600m north of the river.

The objectives of the ASS site investigation are to:

- determine the extent and severity of ASS in these two locations; and
- recommend possible management options.

The investigation was carried out with reference to the following documents:

- Acid Sulfate Soils Planning Guideline (ASSMAC, 1998);
- Acid Sulfate Soil Guidelines (RTA, 1996);
- Acid Sulfate Soils Policy and Procedures (RTA, 1995);
- Draft Procedure TSS-SOP-409015 Procedure for the Sampling and Testing of Acid Sulfate Soils for Road Construction.

7.6 Field sampling

Soil samples were collected from test pits and continuous sampling locations. Test pits were excavated to a maximum depth of 3 m or to the water table. Continuous sampling was undertaken by GeoCoastal Australia. In general samples were obtained when a change in soil horizon was encountered. Where no distinguishable horizons exist samples were obtained at 0.5 m intervals down to 2 m, and then at every 1 m in depth.

Samples were immediately bagged, excess air removed, sealed and then frozen. Samples remained frozen until they were tested.

The locations of testing points are shown in **Figure 6**.

7.7 Laboratory testing

Based on the bore log records, 38 samples were analysed for detailed Peroxide Oxidation Combined Acidity and Sulfate (POCAS), in accordance with ASSMAC guidelines, as follows:

- South of Stewarts River: 16 samples;
- South of Camden Haven River: 9 samples; and
- North of Camden Haven River: 13 samples.

The POCAS test aims at standardising procedures and combining two commonly used peroxide oxidation methods to analyse the acidity of soils, using both the Acid Trail and Sulfur Trail analytical techniques. This includes the following indicators:

- pH before oxidation pH_{KCL}
- pH after oxidation pH_{FOX}
- Sulfur Trail S_{POS} (%)
- Acid Trail TPA, TAA, and TSA (mole/tonne).

Test results are summarised in Table 39 and copies of the laboratory certificates are provided in Appendix I. It should be noted that laboratory results for TPA, TAA and TSA are given in moles H^+/tonne. Conversion rates are as follows:

- to convert moles H+/tonne to kg $H_2 SO_4$ /tonne, multiply by 0.049; and
- to convert kg $H_2 SO_4$ /tonne to moles H+/tonne, multiply by 20.39.

Results are discussed in the following paragraphs.

Table 39 Summary of laboratory analysis

Test Location	Sample Depth (m)	Material Type	Description	PH_{KCL}	pH_{OX}	pH_{KCL} - pH_{OX}	S_{POS} (%S)	TPA (mol H^+ /tonne)	Equiv. TPA (%S)	TAA (mol H^+ /tonne)	TSA (mol H^+ /tonne)
LOR:				0.1	0.1	-	0.02	2	-	2	2
South of Stewarts River:											
AS201	0.8-0.9m	Clayey SILT	Light brown/grey with orange mottling	3.3	2.9	0.4	<0.02	127	0.20	73	54
AS201	1.8-1.9m	Clayey SILT	Orange and grey with red brown mottling	3.3	2.8	0.5	<0.02	92	0.15	73	19
AS202	0.5-0.6m	Clayey SILT	Light brown/orange with mottling	4.2	3.6	0.6	<0.02	20	0.03	12	8
AS202	2.9-3.0m	Silty SAND	Red/orange and grey	3.6	2.9	0.7	<0.02	61	0.10	41	20
AS203	0.3-0.4m	Clayey sandy SILT	Dark grey, rootlets noted	3.7	3.6	0.1	<0.02	56	0.09	53	3
AS203	1.1-1.2m	Silty Clayey SAND	Grey, strong sulphurous smell	3.3	2.9	0.4	0.02	77	0.12	59	18
AS204	0.6-0.7m	Clayey SILT	Light brown/orange with grey mottling	3.4	2.7	0.7	<0.02	91	0.15	61	30
AS204	2.9-3.0m	Sandy SILT	Light grey, light orange/yellow/red mottling	3.5	3.0	0.5	<0.02	48	0.08	30	18
CSP201	0-0.2m	Gravelly clayey FILL	Dark grey brown	4.1	3.6	0.5	0.02	63	0.10	20	43
CSP201	2.8-3.0m	SAND	Dark brown to grey band	3.8	2.6	1.2	0.26	170	0.27	18	152
TP209	0.9-1.0m	Silty CLAY	Grey mottled orange	3.7	3.2	0.5	<0.02	107	0.17	69	38
TP209	2.5-2.6m	Sandy CLAY	Grey, trace orange	3.6	3.5	0.1	<0.02	96	0.15	82	14
CSP202	0-0.2m	Clayey SILT	Very dark brown	4.8	3.5	1.3	0.03	103	0.17	12	91
CSP202	1.3-1.5m	Silty CLAY	Grey with bright red mottle	3.6	2.8	0.8	0.02	109	0.17	81	28
TPAS207	0.4-0.5m	Silty CLAY	Grey brown, trace of orange iron staining	4.0	3.9	0.1	<0.02	61	0.10	49	12
TPAS207	0.8-0.9m	Silty CLAY	Grey brown, trace of orange iron staining	3.9	3.5	0.4	<0.02	79	0.13	63	16
South of Camden Haven River											
CSP203	0-0.2m	Clayey SILT	Very dark grey	4.1	2.7	1.4	0.12	435	0.70	25	410
CSP203	1.3-1.5m	Clayey sandy SILT	Dark grey	4.2	3.6	0.6	<0.02	46	0.07	21	25
CSPAS209	0-0.2m	Clayey SILT	Very dark grey brown	4.3	4.0	0.3	0.10	139	0.22	15	124
CSPAS209	2.8-3.0m	Sandy SILT	Grey brown	3.3	1.6	1.7	2.43	1410	2.26	94	1320
CSPAS21	0-0.2m	Clayey SILT	Very dark grey brown	4.2	3.8	0.4	0.04	144	0.23	48	96
CSPAS21	1.3-1.5m	Sandy Silty CLAY	Very dark grey	3.8	2.5	1.3	0.12	181	0.29	45	136
CSPAS21	2.8-3.0m	Silty Clayey Sand	Very dark grey	4.4	2.0	2.4	1.24	663	1.06	10	653
TP231	2.3-2.4m	Silty SAND	Grey	4.0	2.6	1.4	<0.02	81	0.13	23	58
TP232	0.7-0.8m	Clayey SILT with sand	Dark brown/grey	3.9	1.9	2	1.58	974	1.56	122	852

Test Location	Sample Depth (m)	Material Type	Description	PH_{KCL}	pH_{OX}	pH_{KCL} - pH_{OX}	S_{POS} (%S)	TPA (mol H^+ /tonne)	Equiv. TPA (%S)	TAA (mol H^+ /tonne)	TSA (mol H^+ /tonne)
LOR:				0.1	0.1	-	0.02	2	-	2	2
North of Camden Haven River											
AS213	0.8-0.9m	Clayey sandy SILT	Light brown/orange with grey mottling	4.0	3.4	0.6	<0.02	46	0.07	36	10
AS213	1.9-2.0m	Clayey silty SAND	Light grey, trace rootlets, sulfur smell	3.7	3.0	0.7	0.06	64	0.10	35	29
TPAS215	0.4-0.5m	Sandy CLAY	Grey mottled orange, trace of white	4.2	3.6	0.6	0.26	531	0.85	77	454
TPAS215	0.8-0.9m	Sandy CLAY	Grey mottled orange, trace of white	4.0	2.3	1.7	0.29	403	0.65	68	335
CSP205	0-0.2m	SILT	Very dark grey	3.8	3.6	0.2	0.19	247	0.40	82	165
CSP205	1.3-1.5m	Silty clayey SAND	Very dark grey to dark grey	3.5	1.8	1.7	1.98	1070	1.72	68	1000
CSP206	0-0.2m	PEAT	Black	3.8	2.5	1.3	0.47	801	1.28	125	676
CSP206	1.3-1.5m	Silty clayey SAND	Very dark grey to dark grey	3.9	1.7	2.2	2.55	1250	2.00	84	1170
CSP206	2.8-3.0m	Silty clayey SAND	Very dark grey to dark grey	3.2	1.8	1.4	1.95	1140	1.83	61	1080
TPAS218	0.4-0.5m	Sandy SILT	Dark brown-black	4.1	2.9	1.2	0.43	565	0.91	97	468
TPAS218	0.8-0.9m	Silty gravelly SAND	Grey, iron-stained orange red	4.0	2.8	1.2	0.49	575	0.92	111	464
TPAS219	0.4-0.5m	Sandy CLAY	Red brown	4.0	3.6	0.4	<0.02	44	0.07	20	24
TPAS219	0.8-0.9m	Sandy CLAY	Red brown	4.1	3.3	0.8	0.36	303	0.49	38	265

Notes:
Source: Laboratory Results, ALS
Shading indicates the value is above the ASSMAC Action Criteria (S_{POS} > 0.03 %S, Equivalent TPA > 0.883 kg H_2 SO_4 /tonne or 18 moles H^+ /tonne)
TSA = TPA – TAA. For TSA, negative results are treated as zero (such results are possible on soil materials with high organic matter and no sulfidic material). Brackets indicate results that are negative or zero.
From discussions with Laboratory ALS, any anomalies in TSA results explained by significant figures and rounding errors.
Limits of Reporting as follows: TAA, TPA, TSA = 2 mole/tonne; pH_{KCL} = 0.1.

7.7.1 South of Stewarts River

The bore log records and laboratory results indicate that this area contains acidic material, however 15 of the 16 S_{POS} results show negligible sulfur related acidity. It is concluded that the materials are not acid sulfate soils, but contain organic acidity (reflected in the TAA and TSA results).

Sample CSP201 (sand) was the only exception from the samples tested at this location. Both S_{POS} and TSA results trigger the ASSMAC criteria. It is concluded from the results that this sample does comprise an acid sulfate soil, comprising both actual and potential acidity. With reference to the bore log, this sample was taken from a 200mm band of fine sand at a depth of 2.8-3.0m. Extrapolating from this result and given the location of the sample within the floodplain of the Stewarts River, it is possible that other bands of similar material in the area could also contain acid sulfate soils. This material would require management for acidity should it be disturbed.

7.7.2 South of Camden Haven River

Bore log records and laboratory results from this section of the route indicate that generally, the clayey and sandy silts contain acid sulfate soils comprising varying levels of both actual and potential acidity that would require management should they be disturbed. It appears from the results that the materials also contain organic acidity, in particular samples CSP203 (1.3-1.5) and TP231 (2.3-2.4).

7.7.3 North of Camden Haven River

As for south of the Camden Haven River, bore log records and laboratory results from this section of the route indicate that generally, the clayey and sandy silts contain acid sulfate soils comprising varying levels of both actual and potential acidity that would require management should they be disturbed. It appears from the results that the materials also contain organic acidity, in particular samples AS213 (0.8-0.9) and TPAS219 (0.4-0.5).

7.8 Conclusions and recommendations

The following conclusions are made from this investigation:

- the area tested south of Stewarts River can be considered low probability for acid sulfate soils, but does contain organic acidity; and
- the areas tested south and north of the Camden Haven River can be considered high probability for acid sulfate soils.

These areas will therefore require consideration for acidity should they be disturbed, to ensure that runoff pH levels do not constitute a potential risk for environmental harm. The following bullet point list commonly accepted ASS management techniques:

- Avoid disturbance of areas containing soils with high levels of oxidisable sulfur, including minimising the extent of lowering of the water table.
- Control surface water drainage and where necessary provide aggregate in drains to capture fines and or limestone to buffer acid runoff. Consideration should also be given to the use of silt and vegetation sediment fences.
- Reduce oxidation of ASS material by placing the material below the water table.
- Neutralise ASS material with calculated quantities of lime.
- Remove pyritic material via techniques such as sluicing.
- Control oxidation followed by treatment of the acidic leachate (ie during landfarming exercises).

For developments with small amounts of ASS disturbance (ie less than 1000m^3) over a relatively short time frame (ie over a few months), it is usual to implement a management strategy which consists of ASS avoidance, neutralisation of any disturbed ASS material with lime at a specified rate, and burial of the material onsite or disposal offsite.

However, if significant volumes of ASS material would likely be disturbed over a relatively long construction period, these characteristics would require a staged ASS management approach that considers the fluctuating periods and locations of construction activity. In addition, due to the heterogeneity of ASS material identified, the treatment strategy would need to consider the following, to provide a solution that is able to be effectively implemented onsite:

- visual identification of ASS strata within soil profiles (using investigation results and bore log information); and
- treatment locations; and
- testing and validation work.

8. ACID SULFATE ROCK

8.1 Introduction

The oxidation of the sulfide minerals in rocks is a natural process resulting from their exposure to atmospheric conditions. The subsequent creation of sulfuric acid by this oxidation and its mobilisation by runoff or seepage causes acid drainage. This acid drainage can impact on the immediate and wider environment.

Metal sulfides are common accessory minerals in all forms of rock and are not restricted to any particular rock type, depositional environment or age. However, significant economic concentrations are typically found in some igneous and metamorphic rock. Coal is also associated with higher sulfide concentrations.

In any engineering project where rock is excavated, rock fill placed or the water table lowered, there is potential for the formation of acid rock drainage. The increase in the surface area of the rock following excavation accelerates the oxidation process. It is this increase in the rate of sulfide oxidation, which will largely determine whether there is a significant potential for acid drainage.

8.2 Implications of Acid Sulfate Rock

The immediate result of this process is clearly a decrease in soil and water pH. This in turn can result in the mobilisation of heavy metals.

Once established, acid rock drainage may persist for tens to hundreds of years and may be very difficult and costly to remediate. The potential problems caused by acid rock drainage include the following:

- impact on the quality of both surface water (rivers, streams) and groundwater (particularly shallow aquifers);
- impact on aquatic ecosystems in downstream environments resulting from acidity and dissolved metals;
- impact on engineering and landscaping works, including the type of concrete and steel required, the design of roads, buildings, embankment and drainage systems;
- impact on agricultural practices, including suitability of crops, liming practices, fertilizer requirements and drainage systems;
- impact on riparian communities along the downstream drainage channels (for example, tree deaths); and
- impairment of the beneficial use of waterways downstream of the construction or mining operations for purposes such as live stock watering, recreation, fishing, or irrigation.

For construction sites, it is particularly important to determine the following:

- whether excavations and formations have potential for acid rock drainage, and how this can be managed or mitigated;
- the effect of acid rock drainage on construction materials and buried structures, particularly concrete and steel, and how this can be mitigated; and
- whether excavated materials should be re-used or disposed of, and the management of its disposal.

8.3 Investigation for Acid Sulfate Rock

As part of the preferred option investigation, the following assessment for Acid Sulfate Rock was carried out:

- observations in the field for indications of actual acid generation such as low pH of residual soils and surface water, jarosite development, and iron staining;
- observations in the field of the potential for acid generation, particularly the existence of sulfide minerals such as phyrite in rock core; and
- sampling of rock core and testing for Net Acid Generation (NAG) and Net Acid Producing Potential (NAPP).

8.4 Assessment criteria

The following criteria for the assessment of NAG and NAPP results have been adopted based upon information published by the Australian Minerals and Energy Environmental Foundation (Miller, 1998).

Table 40 Acid rock assessment criteria

Primary Geochemical Type	Final NAG pH	Static NAG Value (kg H2SO4/t)	NAPP (kg H2SO4/t)
Potentially Acid Forming	<4.5	>5*	Positive
Potentially Acid Forming – Low Capacity (PAF-LC)	<4.5	<5*	Positive
Non Acid Forming	>4.5	0	Negative
Acid Consuming	>4.5	0	Less than –100
Uncertain **	>4.5	0	Positive
	<4.5	>0	Negative

* This criterion can range up to 10 depending on site environmental geochemistry.
** Further testing required to confirm material classification including mineralogy, sequential/kinetic NAG tests.

In addition, a Total Sulfur Percentage of >0.1% (equivalent to 3.06kg/tonne of $H_2 SO_4$) has been used to identify rock with a potential to produce significant quantities of acid. Rock with a Total Sulfur content lower than <0.1% is considered to have insufficient sulfur to be a problem (Miller, 2003, personal communication).

8.5 Results of investigation

During the course of the preferred option investigation, metal sulfides, particularly phyrite was idendified in very low quantities in some of the drill core. The results of the NAG and NAPP testing carried out on this core is summarised in Table 41.

Table 41 Results of Acid Sulfate Rock testing

Sample	Total S%	MPA* (kg/t)	ANC* (kg/t)	NAPP	NAG	NAGpH
BH205 at 5.09m	0.02	0.6	0.5	<0.5	8.7	4.5
BH206a at 15.7m	0.06	1.8	<0.5	1.8	10.1	4
BH207at 9.2m	0.02	0.6	0.8	<0.5	8.1	4.7
BH208 at 2.05m	<0.02	<0.5	12.7	-12.7	5.9	5.9
BH208 at 8.05m	<0.02	<0.5	17.8	-17.8	4.9	6.1
BH209 at 8.34m	<0.02	<0.5	12.6	-12.6	5.9	6.2
BH212 at 11.05m	<0.02	<0.5	4.0	-4.0	1.9	6.7
BH213 at 9.7m	<0.02	<0.5	9.4	-9.4	5.0	6.0
BH214 at 10.69m	<0.02	<0.5	7.3	-7.3	3.0	6.0

* MPA = Maximum potential acidity
* ANC = Acid neutralising capacity

8.6 Conclusions

All of the samples tested have Total Sulfur Percentage levels below the threshold (0.1%) at which acid generating potential could become a problem.

The material tested from HB206a at 15.7m could, based upon the NAG/NAPP assessment criteria, be classified as “potentially acid forming”. However, the total sulphur percentage is below the threshold at which acid generation could be a concern.

9. EARTHWORKS

This section of the report summarises the results of soil and rock testing that will affect earthworks and provides recommendations for scheme design.

9.1 Topsoil

The topsoils along the preferred option are generally soft to firm silty clays/clayey silts and sandy silts. During the initial investigation it was found that the topsoil is susceptible to becoming waterlogged. During rain tracks to some of the investigation locations were inaccessible. The average depth of topsoil based on the investigation information for each section of the preferred option is tabulated below.

Table 42 Summary of topsoil depths

Preferred option section	Average Topsoil Depths (m)
Johns River Bypass	0.24
Stewarts River	0.28
Middle Brother	0.39
Camden Haven River	0.34
Kew Bypass	0.28
Northern Section	0.27

At cuts, the topsoils may be stripped, stockpiled and reused as topsoil. Topsoil should as far as possible be reused in the area in which it was stripped. In fills, a number of options are available:

- Strip topsoil and stockpile for reuse as topsoil on embankments. This removes potentially weak materials from the base of the fill and ensures that the same soil type is replaced in the same area. This may require working in waterlogged soils.
- Strip topsoil along edges of embankments. This reduces the risk of toe failure due to weak materials beneath the embankment without stripping all the topsoil. Stockpiled topsoil could be reused as topsoil over embankments, although additional topsoil would probably need to be imported.
- Leave topsoil in place. Place geotextile separation/reinforcing layer over topsoil and construct embankment directly on topsoil. This type of construction is best suited to embankments on soft soil foundations where geotextile reinforcement and other special construction techniques would be required to provide stability.

9.2 Subgrade material

Cohesive material makes up the majority of soils, which will comprise the subgrade for the preferred option. An assessment of the suitability and quality of these materials has been made using the results of soaked CBR tests, the results of which are summarised in Table 43. These results show great variability, with values of 2.5% being estimated for high plasticity silty clays to value of >15% for sandy clay and clayey sand.

Table 43 Summary of California Bearing Ratio test results

TP	Depth (m)	Soil material	Soil Landscape	Soaked CBR %	PI
203	0.3-0.4	Silty Clay	la	8	14
204	0.4-0.5	Silty clay	pn	2.0	40
206	0.8-0.9	Silty clay	ba	3.5	28
213	1.3-1.4	Clayey silty sand	hv	3	22
217	0.3-0.4	Sandy silty clay	hv	8	-
219	0.5-0.7	Silty clay	hv	2	53
219	1.6-1.7	Sandy clay	hv	3.5	43
222	0.4-0.5	Clayey sand	hv	6	20
223	0.5-0.6	Sandy clay	la	13	9
225	0.9-1.0	Sandy clay		13	40
227	0.6-0.7	Silty Clay	gh	0.5	85
238	0.5-0.6	Silty clay	ho	3.5	56
241	0.5-0.6	Clayey sand	me	16	-
244	1.7-1.8	Silty sand	me	9	12
245	0.4-0.5	Sandy clay	gh	6	13
252	0.4-0.5	Sandy silty clay	me	3	47
254	0.8-0.9	Sandy silty clay	me	2.5	37
256	0.7-0.8	Silty clay	me	2.5	36
261	1.8-1.9	Silty clay	ba	2.0	49
12	1.8-1.9	Sandy clay	hv	8	31
13	0.15-0.3	Sandy clay	ji	8	25
33	0.2-0.4	Clayey sand	me	8	-
33	1.4-1.6	Sandy clay	me	4	53
35	0.6-0.7	Sandy clay	me	3.5	41
48	0.7-0.8	Clayey sand	me	45	5
50	0.9-1.1	Sandy clay	me	6	30
50	1.6-1.7	Silty clay	me	6	55
1	0.6-0.7	Silty clay	pn	4	43
1	1.05-1.15	Silty clay	pn	3	47
42	0.5-0.6	Silty sand	up		14
49	1.1-1.3	Clayey sand	me		21
60	0.6-0.7	Sandy clay	me	13	14
60	1.4-1.5	Silty clay	me	2.5	56

To assist with the determination of the likely quality of subgrade, a comparison has been made of soaked CBR values and plasticity index (PI). The results of this comparison are shown in **Figure 43** on which a lower bound straight-line correlation between the two variables is shown. This relationship suggests subgrade CBR values of greater than 3 can be achieved where the PI of cohesive soils is less than 35 and when the soil is compacted to its maximum dry density (Standard Compaction Method). Typically, soils with CBR results 2% or less will need to be stabilised.

The soil landscape groups identified by DLWC and PI results from boreholes and test pits taken within about 3 m of the existing ground surface have been used to identify the likely locations of poorer quality subgrade materials. Soil landscapes and the soils that they contain, are a product of the prevailing climate, the underlying geology and the topographic environment. As such, it is reasonable to assume for this assessment that soil characteristics, within each soil landscape are reasonably consistent. Three classes of soil landscape have been established from the testing carried out, to predict the likely presence of high plasticity clay and hence low CBR and the need for stabilisation. These are outlined below:

- High — Soil landscapes, which are likely to have a high proportion of clay PI >35 (or silts)
- Moderate — Soil landscapes, which are likely to have a moderate proportion of clay PI >35
- Low — Soil landscapes, which are likely to have a low proportion of clay PI >35

The distribution of these subgrade classes within the preferred option is shown in **Figure 44** and summarised in Table 44.

Table 44 Soil landscape subgrade classes

Soil group	Number of samples	No. group with PI> 35	% group with PI>35	Subgrade Class		
				Low (<20%)	Medium	High (>40%)
Ji	25	0	0%	Low		0
me	15	7	47%	0		High
Hv	13	2	15%	Low		0
Ba	11	2	18%	Low		0
La	10	2	20%	0	Medium	0
Up	10	3	30%	0	Medium	0
Pn	7	4	57%	0		High
He	6	0	0%	Low		0
Wa	5	0	0%	Low		0
Gh	4	2	50%	0		High
Bb	3	1	33%	0	Medium	0
Ho	2	2	100%	0		High
Pi	2	0	0%	Low		0

In addition to these areas, it should be anticipated that high plasticity clays (PI>40) are likely to be found in and immediately around creeks and preferred drainage paths.

Only limited testing has been carried out on some of the soil landscape types, and additional testing will be required during the detailed investigation phase to improve the reliability of these results.

9.3 Sources of material

9.3.1 General fill

It is intended that wherever possible site won material will be re-used as fill. For soil, the RTA specification for earthwork materials recommends cohesive soils with a PI of greater than 25 should not be used as subgrade or fill without suitable stabilisation. Soils with a PI greater than 25 underlie large areas of the preferred option. However, from tests carried out as part of this study and plotted in **Figure 43**, it can be seen that CBR values of greater than 3 can generally be achieved following compaction, in soils with PI of up to 35. Given this information it should be possible to re-use soils with PI less than 35 in general fill, provided that adequate moisture control and compaction procedures are followed.

The results of compaction testing of soil materials obtained from test pits are summarised in Table 45.

Table 45 Summary of compaction test results

Test pit	Depth (m)	Description	MDD (t/m^3)	OMC (%)	Soil landscape
TP203	0.3-0.4	Silty Clay	1.53	24.5	la
TP204	0.4-0.5	Silty clay	1.52	26.5	pn
TP206	0.8-0.9	Silty clay	1.54	25.5	ba
TP213	1.3-1.4	Clayey silty sand	1.57	23.5	hv
TP217	0.3-0.4	Sandy silty clay	1.73	16.0	hv
TP219	0.5-0.7	Silty clay	1.50	26.5	hv
TP219	1.6-1.7	Sandy clay	1.53	27.0	hv
TP222	0.4-0.5	Clayey sand	1.67	21.0	hv
TP223	0.5-0.6	Sandy clay	1.88	13.50	la
TP225	0.9-1.0	Sandy clay	1.58	24.0	
TP227	0.6-0.7	Silty Clay	1.37	34.0	gh
TP238	0.5-0.6	Silty clay	1.35	32.5	ho
TP241	0.5-0.6	Clayey sand	1.75	17.5	me
TP244	1.7-1.8	Silty sand	1.66	20.5	me
TP245	0.4-0.5	Sandy clay	1.83	15.0	gh
TP252	0.4-0.5	Sandy silty clay	1.57	23.0	me
TP254	0.8-0.9	Sandy silty clay	1.45	27.0	me
TP256	0.7-0.8	Silty clay	1.47	27.5	me
TP261	1.8-1.9	Silty clay	1.68	21.5	ba
TP1	0.6-0.7	Silty CLAY	1.61	23.50	pn
TP1	1.05-1.15	Silty CLAY	1.54	24.00	pn
TP12	1.3-1.4	Clayey SAND	1.92	12.00	hv
TP12	1.8-1.9	Sandy CLAY	1.70	21.00	hv
TP13	0.15-0.3	Sandy CLAY	1.64	24.50	ji
TP33	0.2-0.4	Clayey SAND/Sandy CLAY	1.82	14.50	me
TP33	1.4-1.6	Sandy CLAY	1.52	24.50	me
TP35	0.6-0.7	Silty CLAY	1.49	26.00	me
TP42	0.5-0.6	Silty SAND	1.77	16.50	up
TP49	0.3-0.5	Sandy CLAY	1.67	19.50	me
TP50	0.9-1.1	Silty CLAY	1.58	23.00	me
TP50	1.6-1.7	Silty CLAY	1.49	27.00	me
TP60	0.6-0.7	Sandy CLAY	1.83	14.20	me
TP60	1.4-1.5	Silty CLAY	1.47	27.70	me

The silty clays from the Grants Head (gh) and Holey Flat (ho) soil landscapes should be regarded as unsuitable material due to their low maximum dry densities, high optimum moisture content and PIs and low CBR.

Rock won from cuttings will generally be suitable for re-use. However, shale, mudstone and siltstone from both the Camden Haven and Byabbara Beds should be used with caution as these will deteriorate rapidly to a silty clay when exposed. They should not be used where high strength engineered fill is required unless their durability and long term strength characteristics have been determined.

9.3.2 Pavement quality material

Two types of potential pavement quality rock, granite and quartz conglomerate have been identified within the preferred option. The granite, which is generally located in the area, has already been exploited for rail ballast by SRA and by State Forests of NSW for road base. The Johns River Quarry operated by Boral currently exploits this material and could be a source of supply for the project.

A quartz conglomerate bed approximately 5 m thick exists within the Byabarra Beds and probably forms the more prominent hill tops to the east of Kew. This material has already been exploited by the RTA at the Bethesda Quarry and by Hurd Haulage at Taylors Pit. However, it is likely that these sources will not be sufficient to provide adequate supply alone for the northern part of the highway upgrade and may need to be supplemented with material from Johns River.

9.4 Slopes

9.4.1 Cut slopes

It is anticipated that cut slopes will need to be formed in both soil and rock and an assessment has been made of suitable batter angles for these new slopes along the preferred option. This assessment has been based upon information from the ground investigation, geological mapping and kinematic analysis and observation of the performance of existing cut slopes in the area.

From stability considerations alone, cut slopes in soil of 2:1 (horizontal:vertical) could be achieved however, due to the potential erodibility of the soil in the study area, it is recommended that batters of 3:1(horizontal:vertical) be used for soil slopes unless surface protection is provided. It is also recommended that benches be installed at approximately 7 m intervals in the deeper cuts and at the rock/soil interface.

A summary of the recommended batter angles is given in Table 46.

Rock cut batters are generally relatively flat (flatter than 1:1 Horizontal:Vertical) due in many instances to the fractured nature and adverse jointing present in rock mass.

The cut batters given above are based on slope stability and erodibility criteria, and it may be desirable to flatten the slope batters for maintenance or environmental reasons.

Table 46 Summary of major cut batters

Approx mid Chainage	Approx cutting length (m)	Approx cutting height (m)	Depth of soil within cutting (m)	Proposed soil cut angle (H:V)	Depth of rock within cutting (m)	Proposed rock cut angle (H:V)
1550	320	14	10	3:1	4	1:2
2750	200	2	2	3:1	-	-
4750	150	8	Est 2	3:1	7	1.5:1
5250	200	8	1.7	3:1	7	1.5:1
6200		2		3:1		
6650	390	6	1.1	3:1	5	2:1
7500	300	8	Est 1	3:1	7	1.5:1
8250	350	10	2.2	3:1	7.8 (XW)	2:1
9100	300	6	2.15	3:1	3.85(XW)	2:1
10500	415	5	0.6	3:1	4.5	1.5:1
11650 offset	350	7	1.3	3:1		2:1
13930	330	6.5	0.3	3:1	6.2	2:1
14700	400	19	2.1	3:1	17.9	1.5:1 to 9m 1:2 to 19m
15300	150	3.5	>3	3:1	-	-
15750	450	15	1.15 - 2.5	3:1	7.5 - 8.85	*Variable:* 1:1 around CH15650 2:1 around CH15800 1:2 around CH15925
16250	400	10	1.1-1.6	3:1	9.1	1:1.5
17500	300	3	>2.9	3:1	-	-
18000	200	3	1.4	3:1	1.6	2:1
18400	180	4	>3m	3:1	-	-
20450	300	3	2.2	3:1	0.8	2:1
21750	150	6	6	2:1	-	-

9.4.2 Fill slopes

A number of new fill embankments will need to be constructed along the preferred option The suitability of material won from excavation for use as fill will depend on the type of material, which has been discussed earlier. The method of excavation will also have an impact on the suitability of the material. Fine-grained material of shale or mudstone origin will generally be low strength and, if moisture contents are high due to the excavation process or stockpiling, will be unsuitable for use as structural fill. Coarse material can be used in fills, provided fines and moisture contents are controlled.

The stability of the embankments at the Stewarts and Camden Haven River crossings is discussed in Section 10. Steeper batters could be used if crushed rock is used and the embankment is founded on competent soil or rock.

An assessment has been made of suitable batter angles for these new slopes. This assessment has been based upon information from the ground investigation. From stability considerations alone, maximum fill slopes should not exceed 2:1.

Where environmental or other constraints require steeper batters, these could be achieved if crushed rock is used and the embankment is founded on competent soil or rock. In this case batters of around 1.5:1 (H:V) can be considered, provided detailed stability analyses are carried out to confirm safe batter slopes. Laboratory testing to confirm suitable strength parameters will also need to be carried out to determine embankment fill strength parameters and quality control testing will be required during construction to ensure suitable material is used in the embankments.

However, due to the potential for erosion and the dispersivity of the soils which have been identified within the preferred option, it is recommended that fill slopes should be flattened to 3:1 (H:V). Alternatively, slopes could be grassed or otherwise protected to reduce the likelihood of long-term degradation due to erosion.

Stability and settlement of embankments is likely to be an issue at a number of locations where soft, alluvial soils, make up the founding stratum. At these locations, flatter slopes or stabilising berms may be required. RTA have identified settlement of the existing embankment, on the approach to the south side of the Camden Haven Bridge.

9.5 Excavations in rock

Rock will be encountered in a number of the proposed highway cuttings. The ground investigation has provided initial information on the type and quality of rock that can be expected in these excavations. Using this information, together with seismic velocities from the seismic refraction survey, a preliminary estimate can be made of the locations where blasting may be required to assist with the excavation.

9.6 Reactive/expansive soils

Reactive soils swell when they become wet and shrink on drying out, which can lead to significant damage to structures built on them. To estimate the potential soil reactivity within each soil landscape, the characteristic surface movement y_s was calculated using an interpolated shrinkage index (Ips) (Mitchell et al 1984; Cameron 1989; Longmac 1993) on samples collected during the previous route selection investigation. The surface movement calculation enables the soil to be classified in accordance with AS2870 "Residential slabs and footings construction", the details of which are given in Table 47.

Table 47 General definitions of site classes (AS 2870)

Class	Foundation
A	Most sand and rock sites with little or no ground movement from moisture changes
S	Slightly reactive clay sites* with only slight ground movement from moisture changes
M	Moderately reactive clay or silt sites, which can experience moderate ground movement from moisture changes
H	Highly reactive clay sites, which can experience high ground movement from moisture changes
E	Extremely reactive sites, which can experience extreme ground movement from moisture changes
A to P	Filled sites (see Clause 2.4.6 of AS 2870-1996)
P	Sites which include soft soils, such as soft clay or silt or loose sands; landslip; mine subsidence; collapsing soils; soils subject to erosion, reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise

* For examples of clay sites classified as Class S, refer to Appendix D of AS 2870-1996.

The soil landscapes groups identified by DLWC, and the calculated soil movement, have been used to identify the locations along the preferred option where different classes of reactive soils are likely to be found.

Soil landscapes and the soils that they contain, are a product of the prevailing climate, underlying geology and the topographic environment. As such, it is reasonable to assume that the soil characteristics, such as reactivity, within each group are likely to be reasonably consistent.

The results of these soil movement estimates are shown in the table below. It should be noted that the reactivity class listed in the Table below takes into account the thickness of the soil profile and that on the basis of the PIs of the soils alone, the soil groups would have different classifications.

Table 48 Summary of soil reactivity results

Soil landscape group	Class	Typical Surface Movements, y_s
bb, ba, la, pi, pn	H	40mm<y_s≤70mm
Up	M-H	20mm< y_s ≤70mm
gh, gha, hv, ji, he	M	20mm< y_s ≤40mm
Me	S-M	0mm< y_s ≤40mm
wa, ho, cn	S	0mm< y_s ≤20mm

Notes:

- Typical Surface Movements, y_s based on Table 2.3 in AS2870
- The DLWC 'Soil Landscapes and Physical Limitations' map was used to determine the reactiveness for soil groups gh, ho and cn.
- Soil group ba: variable results obtained (moderate to extreme)

9.7 Potential for soil erosion

Soil erodability is the susceptibility of a soil to the detachment and transport of soil particles by water and wind. Dispersivity is closely linked to erodibility and is a measure of the instability upon wetting of a soil.

Information obtained from the DLWC was used to make an initial assessment of the erodability and dispersivity of the soils within the area of the preferred option . This information was then supplemented with specific data collected from field observations, and from samples collected during the current and initial ground investigations.

9.7.1 Dispersive soils

Dispersive soils are those, which become unstable upon wetting and as a result are susceptible to gully erosion and tunnelling of earthworks. Dispersive soils are typically sodic, and contain >6% exchangeable sodium. The dispersivity of a sodic soil will also depend on the total salt content, organic matter, clay content, and type of cementing agent.

Field observation, simple field-tests and laboratory testing can readily identify dispersive soils. Within trial pits, a combination of the following attributes is an indication of potential for dispersivity (Charman and Murphy, 2000):

- coarse columnar or prismatic structure;
- very high strength when dry;
- very low permeability;
- very high bulk density (>1.8g/cm³).

Samples that indicated dispersion in the field together with a general sampling scheme to broadly cover the preferred option, were tested in the laboratory using the Emerson Aggregate Test. The results of these tests are given in Table 49.

The results of these tests indicate that these soils are generally non-dispersive or have low dispersivity.

Table 49 Summary of Emerson Aggregate tests

BH/TP	Test depth	Emerson Class	Soil group
TP201	2.5-2.6	6	La
TP206	0.8-1.0	5	Ba
TP203	0.3-0.4	5	La
TP213	1.3-1.4	6	Hv
TP219	0.5-0.7	6	Hv
TP219	1.6-1.7	6	Hv
TP222	0.4-0.5	5	Hv
TP226	1.4-1.5	6	Gh
TP227	0.6-0.7	6	Gh
TP228	0.8-0.9	5	Gh
TP238	0.5-0.6	6	Ho
TP245	0.4-0.5	5	Gh
TP245	1.4-1.5	6	Gh
TP249	0.4-0.5	6	Gh
TP254	0.8-0.9	6	Me
BH23	1.0-1.45	5	Wa
BH23	5.5-5.95	5	Wa
BH23	8.5-8.95	5	Wa
BH34	1.0-1.45	5	He
BH45	1.0-1.45	5	Bb
BH45	4.0-4.45	5	Bb
TP1	1.05-1.15	6	Pn
TP3	0.70-0.80	6	Ba
TP3	1.40-1.60	6	Ba
TP4	0.70-0.90	5	Ba
TP7	0.50-0.65	5	Up
TP11	0.80-1.0	5	Ba
TP19	2.0-2.2	5	La
TP35	0.6-0.7	6	Me
TP39	1.4-1.5	5	Me
TP48	0.3-0.4	5	Me
TP50	0.9-1.1	5	Me?
TP51	0.6-0.8	5	Pn
TP51	1.0-1.2	6	Pn
TP56	0.5-0.7	5	Up
TP58	0.5-0.6	6	Gha
TP59	0.5-0.6	6	Pn

9.7.2 Erosion hazard

The DLWC's method of soil erosion hazard assessment from major road construction has been used to assess the preferred option. DLWC define erosion hazard as "the susceptibility of a parcel of land to erosion if the soil is left exposed and no erosion control management is employed. It is a function of the intrinsic attributes of the land contributing to potential soil loss including rainfall erodibility, soil erodibility and slope angle. It is independent of cover and soil management practices. As such it can be used to determine the relative risk of erosion of a soil landscape, should it be disturbed, when compared with other soil landscapes.

Based on the method proposed by Murphy (Charman and Murphy, 2000), shown in Table 50 the results of the Emerson tests suggest that the soils within the project area fall into the moderate to low erosion hazard classes.

Table 50 Soil erosion classes (Murphy, 1984)

Erodibility	Topsoil	Subsoil
Low	• High organic matter content (>3%) (soils have a dark colour and feel greasy when textured) • High coarse sand content	• Cemented layers including silcrete, orstein and laterite, iron, manganese and silicon pans. • High coarse sand content • Well structured, non-dispersible clay and clays having aggregates that do not slake in water to particles less than 2 mm (Emerson Aggregate Classes 4, 6, 7 and 8)
Moderate	• Moderate organic matter content (2-3%) • Moderate fine sand and silt content • Well structured clay loams and clays that slake in water to particles less than 2 mm (Emerson Aggregate Classes 3 to 6)	• Stable, non-dispersible loams and clay loams • Non-dispersible or slightly dispersible clays with particles that slake to finer than 2 mm (Emerson Aggregate Classes 3 to 6)
High	• Low (1-2%) to very low (<1%) organic matter content • High to very high silt and fine sand content (>65%)	• Dispersible clays (Emerson Aggregate Classes 1 and 2) • Unstable, dispersible clayey sands and sandy clays. • Unstable materials high in silt and fine sand, such as unconsolidated sediments and alluvial materials

9.8 Unsuitable materials

Materials along the preferred option can be considered unsuitable for general earthworks for a number of reasons and these are presented below:

9.8.1 High plasticity, low CBR material

This material includes the high plasticity clays discussed in earlier sections of this report. Materials with a PI over 35 or a CBR of less than 2 would fall into this category. These are generally low strength materials unsuitable for use in subgrades. The ability to compact these materials is very sensitive to changes in moisture content, and these materials are generally unworkable at moisture contents above the optimum moisture content. It is possible that these materials can be used within the core of high embankment fills where long-term moisture conditions are generally stable and high strength is not required.

9.8.2 Material affected by moisture

This material would generally include soils with a PI less than 35 where the natural moisture content is more than about 2% above the optimum moisture content. The materials would generally be suitable for use as general fill if the moisture content can be reduced. This can be done by scarifying and drying, stabilising with lime, mixing with dry material or other methods. The occurrence of these materials would be dependent on seasonal changes in weather, surface drainage, location of the water table and other factors affecting their moisture content and is difficult to predict.

9.8.3 Material with high organic content

These materials are generally present in the topsoil layer and within the alluvial deposits in the Camden Haven River and Stewarts River flood plains.

9.8.4 Potential Acid Sulphate Soils

These soils are discussed in more detail in the section Acid Sulphate (PASS) Soil. Essentially areas in the flood plain south of the Stewarts River can be considered low probability for acid sulphate soils although they contain organic acidity. The flood plain north and south of the Camden Haven River can be considered high probability for acid sulphate soils.

10. EMBANKMENTS

As part of the preferred option major embankments will be constructed on the flood plains of the Camden Haven and Stewarts Rivers. Due to the nature of the soft or loose alluvial deposits encountered in these areas, global stability and settlement issues have been reviewed to assess the viability of construction.

10.1 Stability

Preliminary analysis has been undertaken to assess the stability of proposed embankments at critical sections on both the north and south side of the Camden Haven River floodplain, and on the southern Stewarts River floodplain.

Geotechnical soil strengths were modelled based on correlations from SPT values and Cone Penetration Testing together with laboratory test results (UU triaxial tests). The stratigraphy is tabulated in the following sections.

The soil profiles have been developed from the investigations undertaken to date at the chainages specified. A nominal embankment slope of 2.5:1 (h:v), has been modelled. This batter angle is based purely on engineering stability constraints. Issues including erosion and maintenance have not been considered, but may control the slope geometry resulting in a flatter slope such as 3:1 (h:v).

The soil profile used in the stability analyses, together with the slope geometry are shown in Appendix J. Stability analyses were undertaken using SLOPE, an Oasys Limit State Equilibrium software package, adopting a circular slip circle failure surface.

Three sections on the Camden Haven River and one section on the Stewarts River flood plain have been analysed using both undrained and drained geotechnical strengths. The undrained analysis does not incorporate excess pore pressures, and has been undertaken to model the undrained stability of the embankment after construction

Excess pore pressures, which will be generated in strata with low permeabilities such as clays and silts during construction, have not been incorporated into the modelling as they will be critically dependent on construction staging, construction methodology and subsoil drainage methods adopted. These excess pore pressures will have a destabilising effect on embankment stability, and given the low factors of safety in the undrained condition, some embankments are likely to require adoption of one or more of the options below.

1. Flattening the embankment angle to 1:3
2. Construction of embankments in several lifts, allowing pore pressures to dissipate after each construction stage
3. Inclusion of high strength basal geotextile reinforcement within the embankment
4. Construction berms at the toe of the embankment.

The second option may be accelerated using the following techniques:

a) Installation of wick drains to increase pore pressure dissipation rates
b) Vacuum consolidation techniques to increase pore pressure dissipation rates

During construction, the interaction between consolidation settlements, excess pore pressures and embankment stability is complex. The analysis process is iterative and specific to the adopted construction methodology and is beyond the scope of this report

The purpose of the analyses undertaken and presented in this report is to establish whether it is possible to construct embankments that will be stable in the long term against slope failure, when constructed on the ground conditions encountered during the investigations. The results of the analyses are presented in the following sections of this report.

10.2 Settlements

Preliminary analyses have been undertaken using a combination of elastic analysis and one-dimensional consolidation theory to investigate likely vertical deformation during the life of the embankments to be constructed on the Camden Haven and Stewarts River floodplains.

Total settlements have been estimated as follows:

Total Settlement = $S_I + S_C + S_S$

S_I - immediate elastic settlement – estimated using VDISP from the OASYS suite of programs.

S_C - consolidation settlement – estimated using one-dimensional consolidation theory

S_S - secondary consolidation – no allowance has been made for secondary consolidation at this stage as there is insufficient data to make this prediction.

Two sections have been analysed on the Camden Haven River flood plain and one on the Stewarts River flood plain with profiles as identified for the stability analyses. Material properties from correlations with CPT, SPT and consolidation laboratory tests are tabulated in the tables that follow.

10.3 Camden Haven River North

10.3.1 Performance of existing embankment

Settlement of the existing embankment is noticeable, particularly at the existing culvert crossing at about CH13,100. This embankment is about 20years old.

Stability and settlement analyses were undertaken at CH12900, where the depth to bedrock is approximately 19.6m and the height of the proposed new embankment is approximately 4m.

10.3.2 Material properties

Material properties adopted for consolidation, elastic settlement and stability analyses are summarised in Table 51.

Table 51 Material properties CH12900

Depth to soil layer (m)	Soil Description	ϕ' °	c′ kPa	Cu kPa	E’ MPa	ν’	Eu MPa	ν	C_c	e_o	C_v m²/ year
0	Organic silty Clay and silty Clay	25	0	12	-	-	2	0.5	0.174	0.652	1.75
4.75	Silty Sand	30	0	-	3	0.3	-	-	-	-	-
6.45	Sandy Clay	26	0	8.0	8.0	0.5	3.5	0.5	0.174	0.652	1.75
10	Clayey Sand	32	0	-	20	0.35	-	-	-	-	-
12.5	Sandy gravel and gravelly sand	34	0	-	10 to 25	0.3	-	-	-	-	-
19.57	Bedrock			-	-	-	-	-	-	-	-

10.3.3 Stability analyses

The Factors of Safety (FOS) from the stability analyses are summarised in Table 52.

Table 52 Embankment FOS at CH12900

	Undrained FOS	Drained FOS
CH 12900	1.27	1.63

10.3.4 Elastic settlements

The calculated immediate elastic settlement at CH12900 is approximately 0.19m under the four metre high embankment.

10.3.5 Consolidation settlements

Consolidation analysis has been undertaken at CH12900 and is based on the soil profile presented in Table 51. Consolidation coefficients and compressibility were adopted from one-dimensional consolidation testing from U50 samples recovered from BH211, together with correlations from CPTU testing.

The results are tabulated in Table 53.

Table 53 Consolidation at CH12900

	Consolidation (mm)	Time to 50% average consolidation (months)	Time to 90% average consolidation (months)
CH12900	240	8	33

The total settlement (elastic plus consolidation) is approximately 430mm.

Analysis has also been undertaken on CPT data based on correlations of the constrained modulus and cone tip resistance. This analysis suggests that the total settlement (consolidation and elastic) would be between 110mm and 240mm.

10.4 Camden Haven River South

10.4.1 Performance of existing embankment

The embankment in this are is known to have settled significantly and in particularly noticeable at a culvert crossing at the southern end of the embankment. This culvert is generally underwater year round. Highway pavement problems observed by the RTA in this area, may also be partly due to this settlement.

Stability and settlement analyses were undertaken at CH12400 and CH12620. At CH12400, the depth to bedrock is approximately 9.5m and the height of the proposed new embankment is approximately 11.5m.

At CH12620, the depth to bedrock is approximately 24.6m and the height of the new embankment is approximately 7m.

10.4.2 Material properties CH12400 and CH12620

Material properties adopted for consolidation, elastic settlement and stability are summarised in Table 54and Table 55.

Table 54 Material properties CH12400

Depth to soil layer (m)	Soil Description	ϕ' °	c′ kPa	Cu kPa	E' MPa	ν'	Eu MPa	ν	C_c	e_o	C_v m²/ year
0	Organic clayey silt	26	0	-	1.0	0.3	-	-	-	-	-
2.65	Silty clay	26	0	15	3 to 5		3 to 5	0.5	0.322	0.533	2.69
5.35	Clayey Sand	30	0	-	8.0	0.5			-	-	-
9.42	Bedrock				-	-	-	-	-	-	-

Table 55 Material properties CH12620

Depth to soil layer (m)	Soil Description	ϕ' °	c′ kPa	Cu kPa	E' MPa	ν'	Eu MPa	ν	C_c	e_o	C_v m²/ year
0	Organic Silty CLAY	25	0	12	-	-	1	0.5	-	-	-
2.5	Clayey Sand	30	0	-	2	0.3	-	-	-	-	-
5.5	Silty Clay	25	0	15	-	-	2	0.5	0.322	0.533	2.69
8.3	Silty Clay	26	0	50	-	-	14	0.5	0.322	0.533	2.69
13	Sandy Gravel	34	0	-	30 to 80	0.3	-	-	-	-	-

10.4.3 Stability Analyses

The Factors of Safety (FOS) from the stability analyses are summarised in Table 56.

Table 56 Embankment FOS at CH 12400 and CH12620

	Undrained FOS	Drained FOS
CH 12400	1.25	1.71
CH 12620	1.31	1.68

10.4.4 Elastic Settlements

The calculated immediate elastic settlement at CH12400 is approximately 510mm under the 11.5m high embankment and 450mm at CH12620.

10.4.5 Consolidation Settlements

Consolidation analysis has been undertaken at CH12400 and 12620 and is based on the soil profile presented in Table 54 and Table 55. Consolidation coefficients and compressibility were adopted from one-dimensional consolidation testing from U50 samples recovered from BH210, together with correlations from CPTU testing.

The results are tabulated in Table 57.

Table 57 Consolidation at CH12400 and CH12620

	Consolidation (mm)	Time to 50% average consolidation (months)	Time to 90% average consolidation (months)
CH12400	120	6	28
CH12620	560	12	53

The total settlement (elastic plus consolidation) at CH12400 and CH12620 is approximately 630mm and 1010mm, respectively.

Analysis has also been undertaken on CPT data based on correlations of the constrained modulus and cone tip resistance. This analysis suggests that the total settlement (consolidation and elastic) is between 100 and 370mm in this area.

10.5 Southern Stewarts River Flood Plain

Stability and settlement analyses were undertaken at CH3950, where the depth to bedrock is approximately 14.2m and the height of the new embankment is approximately 6m.

10.5.1 Material Properties

Material properties adopted for consolidation, elastic settlement and stability are summarised in the Table 58.

Table 58 Material properties CH3950

Depth to soil layer (m)	Soil Description	ϕ' °	c′ kPa	Cu kPa	E' MPa	ν'	Eu MPa	ν	C_c	e_o	C_v
0	Silty Clay	25	0	18			3	0.5	0.209	0.615	5.04
5.0	Sandy Clay	26	0	40			10	0.5	0.209	0.615	5.04
6.5	Sandy Gravel/ Gravely Sand	35	0	-	30	0.3	-		-	-	-
14.22	Bedrock	-	-	-	-	-	-	-	-	-	-

10.5.2 Stability analyses

The Factors of Safety (FOS) from the stability analyses are summarised in Table 59.

Table 59 Embankment FOS Stewarts River South

	Undrained FOS	Drained FOS
CH 3950	1.30	1.64

10.5.3 Elastic settlements

The calculated immediate elastic settlement at CH3950 is approximately 130mm under the 6m high embankment.

10.5.4 Consolidation settlements

Consolidation analysis has been undertaken at CH3950 and is based on the soil profile presented in Table 58. Consolidation coefficients and compressibility were adopted from one-dimensional consolidation testing from U50 samples recovered from BH201, together with correlations from CPTU testing.

The results are tabulated in Table 60.

Table 60 Consolidation at CH3950

	Consolidation (mm)	Time to 50% average consolidation (months)	Time to 90% average consolidation (months)
CH3950	**310**	**8**	**32**

The total settlement (elastic plus consolidation) is approximately 440mm.

Analysis has also been undertaken on CPT data based on correlations of the constrained modulus and cone tip resistance. This analysis suggests that the total settlement (consolidation and elastic) is approximately 50mm.

10.6 Summary

During the investigations it was noted that the thickness of materials that are likely to undergo time dependent settlements (consolidation) varies significantly within each flood plain (**Figure 12**, **Figure 26** and **Figure 30**). Consequently, the analyses undertaken for consolidation settlements only apply to the specific areas analysed.]

The presence of a high proportion of predominantly silty material in both the Camden Haven and Stewarts River flood plains will have some effect in delaying immediate settlements. However, it is considered that settlements should still occur during the construction period and as such does not present significant time dependant settlement issues, although there may be some stability issues.

The majority of elastic settlement will occur during construction, as the embankment material is placed. In this case, displacements that occur can be built out during construction. An additional volume of fill to account for the settlement will be required to obtain finished levels.

The results of the analysis undertaken, together with field results from CPTUs indicate that consolidation of the embankment on the southern side of Stewarts River will not significantly affect construction or embankment performance, although the rate of pore pressure dissipation may affect the stability of the embankment during construction. Where embankments are to be constructed alongside existing embankments additional differential settlements of the existing embankments will occur. This is likely to lead to deterioration of the pavement over the existing embankment or to changes in cross fall over the existing pavement.

11. RETAINING WALL CONCEPT DESIGN

In the area north of Stewarts River, split-level carriageways of the preferred option would be constructed within close proximity to the existing North Coast Railway line.

As a result of these geometric constraints, the ground would need to be cut near vertical, or retained by means of retaining structures or other support.

Two areas have been identified as requiring retaining structures or steep cuttings.

Several scheme options are discussed in the following sections.

11.1 Concept options

These concepts have been developed based upon purely geotechnical issues.

11.1.1 Cut slope

Where the rock strength is sufficient to support a vertical cut face and defect frequency and orientation permits, a near vertical cut slope would be a feasible option in place of, or in conjunction with, a retaining system. The strength of fresh rock encountered in the region of the proposed retaining walls would most likely facilitate the use of blasting for excavation. To prevent erosion or localised instability, localised shotcrete and rock bolting may also be required. If defect orientation has the potential to initiate a larger block slide failure, rock anchors may be required.

11.1.2 Cantilever bored pile wall

Cantilever bored concrete piles, either spaced apart with shotcrete infill (soldier piles) or contiguous, are an appropriate solution to retain highly weathered rock and soils where up to approximately 4m of material is to be retained. Casing or continuous flight auger piles may be required if the soil is not free standing in the short term. Preferably the piles would be socketed into rock at the base.

11.1.3 Anchored bored pile wall

Anchored bored concrete piles, either spaced apart with shotcrete infill (soldier piles) or contiguous, are an appropriate solution to retain soil and highly weathered rock where the height of retained soil exceeds approximately 3 to 4m. Permanent multi strand anchors at 2 to 3m vertical spacing are likely to be required. Horizontal anchor spacing will depend on construction method, with anchors through each pile or at 2m to 3m intervals along a waling beam. Anchors would generally be a minimum of 2m below ground level to avoid existing and future services. Casing or continuous flight auger piles may be required if the soil is not free standing in the short term. Preferably the piles would be socketed into rock at the base.

11.1.4 Soil nail walls

Soil nail walls, consisting of grouted nails with a reinforced shotcrete face, are appropriate to retain cohesive soils and highly weathered rock. This system may be appropriate where a combination of fresh rock and extremely weathered rock/soil is encountered.

11.1.5 Gravity walls

Gravity walls may be appropriate where the retained height is less than 2m to 3m and may be useful for retention of shallow residual soils overlying high strength rock. Typically, these walls would be reinforced concrete ‘L’ shaped elements or gabion walls.

11.2 CH 4260 – CH 5140

11.2.1 Ground conditions

During the preferred option investigations, two boreholes (BH203 and 204) were drilled within this section. The locations of boreholes are provided in **Figure 6**.

Preliminary analysis of defect orientation on the cut slope immediately behind the proposed wall indicates that the slope could be cut at a maximum batter slope of 1.5:1 (h:v) in fresh rock.

At BH203 medium silty sand was encountered to 1.36m overlying very high strength Granite. At BH204 residual soils were encountered consisting of firm silty clay or medium dense clayey sand and silt overlying very high strength granite at 11.05m. Water was encountered in BH204 at a depth of 7.45m below ground level.

Inspection of the cut slope immediately behind the proposed location of the retaining wall indicates varying rock quality decreasing from high strength to extremely low strength granite to the north of the cutting.

From the investigation, it is evident that the depth of residual soil varies significantly along the length of proposed wall.

11.2.2 Recommendations

From the investigations carried out to date, the extent of highly weathered material cannot be quantified and it is recommended that further boreholes be drilled along the length of the wall to establish soil/rock profile in more detail at detailed design stage.

It is likely that a combination of a retention system and cut slope may be suitable. Pending further investigations, the most suitable retention structure would be a soil-nailed wall or anchored bored pile wall.

11.3 CH 7360 – CH 7580

11.3.1 Ground conditions

One Borehole, BH205 was drilled within the proposed location of the retaining wall. At this borehole sandy silt and sandy gravel was encountered to 1.5m overlying interbedded predominantly medium strength siltstone and sandstone.

11.3.2 Recommendations

Based on the information obtained from BH205, it is unlikely that an unsupported steep cut slope is feasible due to defects and weathered zones, however this would need to be confirmed with mapping to determine defect orientation. A bored pile wall or soil nail wall would be appropriate for this area.

12. BRIDGE CROSSINGS

A number of new bridge structures will be required for the preferred option. These will be bridges across Stewarts River Road, Stewarts River, Stony Creek, North Coast Railway, Camden Haven River, Ocean Drive, Herons Creek and Herons Creek Floodplain.

12.1 Stewarts River Road

The ground conditions at Stewarts River Road consist of residual soil over rock.

It is proposed that the Stewarts River Road will pass over the preferred option on over-bridge, with the approaches supported by fill. The abutments for this new bridge would be supported on bored or driven steel piles founded in weathered rock. Piers could also be supported on piles but shallow foundations may also be suitable

12.2 Stewarts River

The ground conditions at Stewarts River consist of alluvial sediments over rock. The existing bridge has been founded on piles, bearing on rock.

It is proposed that the piers and abutments for the new bridge should be supported on driven steel or concrete piles founded on rock at about –15 m AHD. The selection of steel for the piles, should take into account the acidic nature of the soil near the ground surface. A sacrificial thickness of steel, to allow for corrosion may need to be considered.

Excavations for pile caps will expose acid sulfate soils, requiring a management plan dealing with these materials to be in place prior to construction.

Due to the expected consolidation of the embankments adjacent to bridge abutments, lateral and vertical ground movements can be expected. As a result, piles for bridge the abutments will experience additional lateral loads and negative skin friction. These loads could be incorporated into the design, or the construction sequence of the abutment and embankment could be programmed to allow for the majority of consolidation to take place before installation of the piles.

12.3 Stony Creek

The existing bridge at this location is believed to be founded on piles bearing on weathered rock. The ground conditions identified in TP19 and TP221 comprise greater than 2m of alluvium over weathered rock. Given these conditions its is proposed that the piers and abutments of the new bridge would be supported on driven steel or concrete piles founded on weathered rock.

12.4 Bridge over the North Coast Railway

The ground conditions at this location, which were previously investigated for the existing bridge consist of weathered and fresh granite. The existing bridge is founded on shallow spread footings founded on the rock. A new structure at the same location would likely be founded in a similar manner.

12.5 Camden Haven River

The ground conditions at the Camden Haven River consist of alluvial sediment over rock. The existing bridge is founded on 560 mm octagonal pre-stressed concrete piles bearing on rock.

It is proposed that the piers and abutments for the new bridge be supported on driven steel or concrete piles founded on rock at about –25 m AHD. The selection of the steel for the piles, should take into account the acidic nature of the soil near the ground surface.

As with bridge the abutments on the Stewarts River, expected consolidation of the embankments adjacent to bridge abutments will result in substantial lateral and vertical ground movements. Consequently piles for bridge abutments will experience additional lateral loads and negative skin friction. These loads could be incorporated into the design, or the construction sequence of the abutment and embankment could be programmed to allow for the majority of consolidation to take place before installation of the piles.

12.6 Ocean Drive

The ground conditions at this location (BH38) comprise 2.5m of residual soil over weathered rock. It is proposed that the abutments and piers would be founded on shallow footings bearing onto weathered rock.

12.7 Herons Creek and Herons Creek Floodplain

BH34 and BH214 were drilled to assess ground conditions at the Herons Creek bridge site. At BH34, drilled at approximately CH21460, alluvium consisting of 8.5 m of alluvial silty clay, gravelly sand and silty clay overlying 2.08 m of clayey silt residual soil was encountered. These soils overlay inter-bedded shale and sandstone of the Byabarra Beds. The ground conditions identified at BH214, which is adjacent to the northern abutment of the existing bridge, consist of residual soils (sandy silts) overlying granite bedrock at 7.3m.

This substantial change in geological conditions may indicate the presence of a geological feature and further investigations would be required at detailed investigation stage to determine the nature of this feature and its effect on the bridge substructure.

The UCS of 61MPa from a cored sample of the igneous rock encountered in BH214 indicates that end bearing piles could accommodate substantial vertical loads on the northern bridge abutment. However a UCS value of 1.0MPa from Sandstone encountered in BH34 during the options development investigation, indicates a low bearing capacity for end bearing piles. The adopted piling system and structure will need to address the differential settlements that are likely to arise from the differing stiffness of the bedrock encountered at opposing bridge abutment locations.

The existing bridges across the Herons Creek flood plain are founded on piles. The founding stratum is unknown, but is likely to be rock.

It is proposed that the piers and abutments for the new bridge should be supported on driven steel piles founded on rock at about -8 m AHD.

13. GROUNDWATER

Information on the ground water conditions at the time of the investigation is given in Section 5.

13.1 Existing groundwater abstraction

Twenty-two existing groundwater abstraction wells are registered by DLWC within a 1km distance of the preferred option. The location of these wells is shown in **Figure 45**. Details of these wells, as provided by DLWC are given in Appendix K. Abstraction from these wells is primarily for domestic, stock and irrigation purposes. Well yields are all low, generally less than 2l/s.

13.2 Potential for abstraction for construction purposes

Given the low permeability of the prevailing residual soils (silty clays) and the low yields of existing wells in the area, it is probable that the potential for the abstraction of ground water for construction purposes is generally low, except from sand and gravel lenses within the alluvial profiles at Stewarts River and Camden Haven River. Some potential may also exist within the more porous elements (Sandstones) of the Byabarra and Camden Haven Beds.

13.3 Affects effects of the project on groundwater levels

The only likely effect of the development of the preferred option on the existing groundwater could be draw-down of groundwater table around major cuttings. Given the depth of the proposed cuts and the depth to the water table, this is only likely to occur in the vicinity of Ocean Drive, in Kew. However, this is unlikely to have an effect on any existing abstraction given the distance to the nearest registered abstraction well shown in **Figure 45**. Plants which rely on moisture from this deep source, may be affected in the short term

Localised draw-down of very shallow water perched within residual soils could occur. The effects however, would be very localised and would be expected to not extend more than about 10m from the cut.

14. FURTHER GEOTECHNICAL INVESTIGATIONS

The following additional field investigations will be required for the detailed geotechnical investigation phase of the project:

14.1 Boreholes

Boreholes will be required at bridge sites, pier locations and abutments, cuttings and in soft soil areas. Over water boreholes will be required at pier locations for river crossings. Boreholes may be required within the rail reserve for the bridge over the North coast Railway.

14.2 Test pits

Test pits will be required at regular intervals along the preferred option to characterise the soil for general earthworks, subgrade and pavements, and to assess acid sulphate potential in the river flood plains. Test pits will be from 2m to 4m deep.

14.3 Seismic refraction surveys

Seismic refraction surveys will be required in cuts to assess the depth and excavatability of rock and to assess sources of construction materials in the cuttings. Seismic refraction surveys may also be required on the river beds at bridge sites to determine bridge geometry (span lengths) before over water holes are drilled.

14.4 Continuous sampling

Continuous sampling will be required in the Camden Haven and Stewarts River flood plains to determine subsoil conditions at embankment locations and to further assess the extent of acid sulphate soils.

14.5 CPTs

CPT test will be required in the Camden Haven and Stewarts River flood plains to determine the subsoil profile below embankments and at bridge sites.

14.6 Trial embankments

Due to the relatively low consolidation settlements predicted in the investigations to date it is considered that a trial embankment will not be required for the detailed geotechnical investigations, unless some particular technology or construction methodology is to be trailed.

14.7 Pavement condition assessment

A detailed pavement condition assessment would be required in those sections where the existing pavement is to be retained.

14.8 Access tracks

Access tracks will be required for various boreholes, test pits, CPTs, continuous samples and seismic refraction surveys. These tracks may require the cutting down of trees and shrubs, or the crossing of wetlands, depending on the location of the test positions.

14.9 Piezometers

Piezometers will be required at various boreholes to monitor water levels and water quality. These piezometers will be monitored at regular intervals, requiring the provision and ongoing maintenance of suitable access.

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