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Department of Transport and Regional Services

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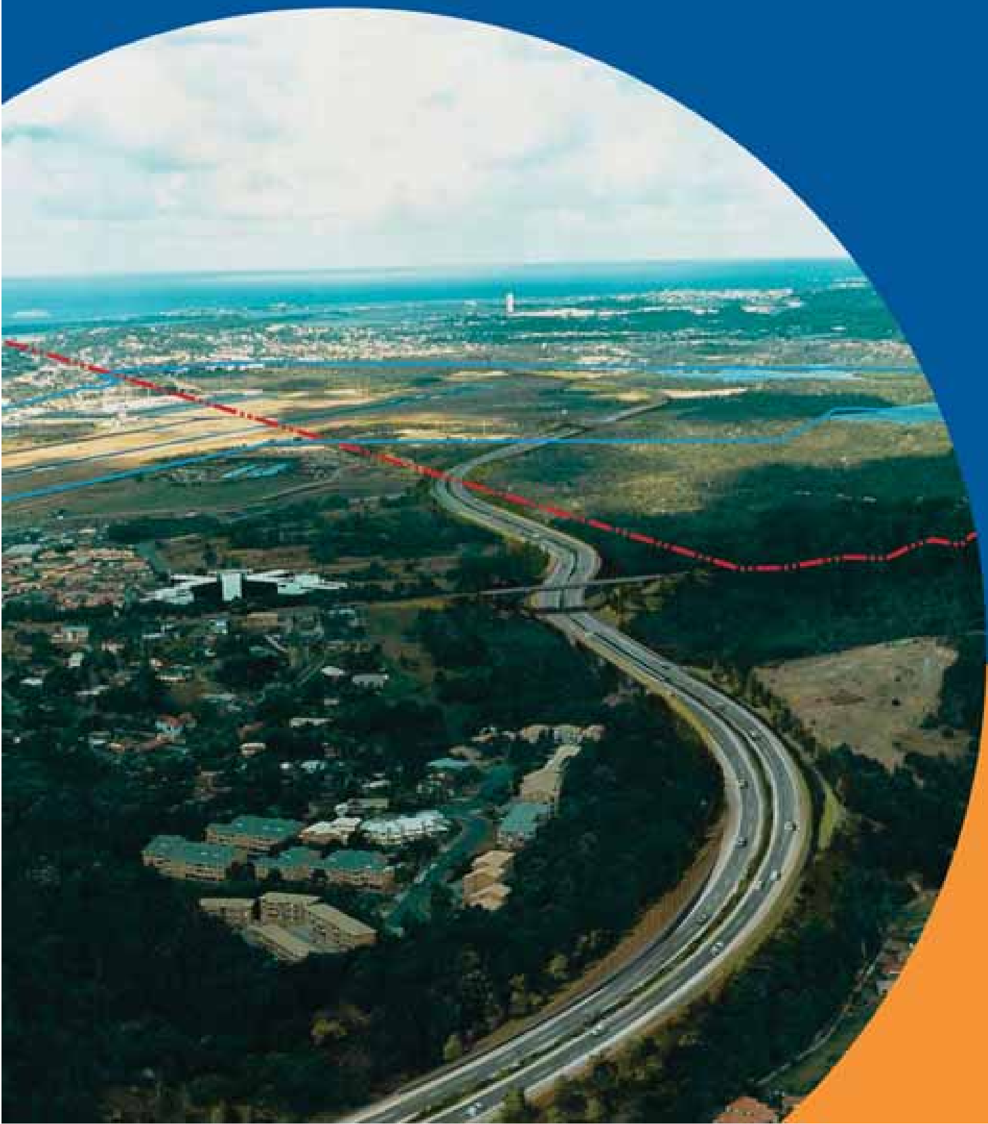
TUGUN BYPASS

stewart road to kennedy drive



Technical Papers

December 2004

Tugun Bypass Environmental Impact Statement

Technical Paper Number 4
Geotechnical Assessment

Tugun Bypass Alliance

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Glossary

Term	Meaning
Acid sulphate soils (ASS)	Acid sulphate soil (ASS) is the common name given to soils containing iron sulphides (principally iron pyrite) or products of the oxidation of sulphides.
ASSMAC	Acid Sulphate Soil Management Advisory Committee.
Alluvium	Any stream laid sediment deposit found in a stream channel and in low parts of a stream valley subject to flooding.
Aquifer	A layer of rock or soil able to hold or transmit much water.
Argillite	A compact rock, derived from mudstone or shale, more highly indurated than either of those rocks. It lacks the fissility of shale or the cleavage of slate. It is regarded as a product of weak metamorphism.
Atterberg limits	A set of arbitrarily defined boundary conditions in soils related to water (moisture) content. The limits are as follows: ▪ Shrinkage Limit (SL) – The moisture content from which a soil will continue to dry out without further change in volume (rarely determined). ▪ Liquid Limit (LL) – The moisture content at which the soil will flow under a specified small disturbing force (defined by the conditions of the test). ▪ Plastic Limit (PL) – The moisture content at which the soil can be deformed plastically. It is defined as the minimum water content at which the soil can be rolled into a 3 mm thick thread. ▪ Plasticity Index (PI) – The range of moisture content over which the soil is in the plastic condition. PI = LL – PL.
Australian Height Datum (AHD)	A level datum, uniform throughout Australia, based on an origin determined from observations of mean sea level at tide gauge stations, located at more than 30 points along the Australian coastline.
Backfill	Fill placed in an excavation.
Batter (rake)	▪ The uniform side slope of walls, banks, cuttings. ▪ The degree of such slope, usually expressed as a ratio of *x* horizontal to 1 vertical in distinction from grade.
Bearing capacity	The load per unit area which a supporting medium can carry without failure or unacceptably large settlements.
Bedrock	A general term meaning rock of considerable thickness and extent underlying relatively soft or variable surface strata.
Berm	A ledge formed at the top or bottom of an earth slope or at some intermediate level.
Borehole	A hole produced in the ground by drilling or driving.
Bored pile	A pile formed by casting concrete into a hole bored in the ground.
Borehole packer testing	Test carried out in unlined boreholes in rock formations using expanding packers to isolate a section of the borehole. High water pressure is then pumped into the section, permeability of the rock mass is evaluated based on the volume of water and pressure.
California bearing ratio (CBR)	A measure of the bearing capacity of a soil obtained from a standard soil penetration resistance test.
Carboniferous age	Geological period of time ranging from 280 to 345 million years ago.
Cast-in situ	Refers to concrete which is cast directly into its final position.
Clay	A natural earthy material possessing plastic properties and consisting of fine particles of complex hydrous silicate smaller than 2 µm.
Cobble	A water-worn rounded stone usually between 60 mm and 200 mm in size.
Coffee rock	Is a locally used term for a weakly to strongly cemented brown to black coloured fine to medium sand or silty sand. It is formed by the downward leaching of organic matter which is flocculated at the level of an often fluctuating groundwater table. It occurs as discontinuous horizons of variable strength and thickness.

Term	Meaning
Cohesion	The ability of a material to resist, by means of internal forces of attraction, the separation of its constituent particles.
Compaction (compact)	Reduction in volume of a material by inducing closer packing of its particles by rolling, tamping, vibrating or other processes to reduce the air voids content.
Compaction factor/ratio	Ratio of the final volume of the soil to the initial volume, after subjection to compaction of standard form or according to field practice.
Cone penetration test	A test in which the effort to push or drive a standard steel cone into soil at a controlled rate is used as a measure of certain soil properties.
Consolidation	The process by which soil reduces in volume under load over a period of time due to drainage of water from the voids.
Core	▪ A piece inserted into a mould for concrete before casting, to form a hole for a bolt, prestressing cable, or other use. ▪ A cylinder drilled out of soil, concrete, rock or other material for testing or other purposes.
Cut	▪ The depth from natural surface of the ground to the subgrade level. ▪ The material excavated from a cutting.
Cut and cover	A method of constructing culverts and tunnels where the structure is built in an open excavation and subsequently covered with backfill.
Cut-off wall	A watertight wall for preventing seepage or movement of water under or past a structure, or for preventing scour from undermining a structure.
Devonian age	Geological period of time ranging from 345 to 395 million years ago.
Dyke	▪ Igneous (volcanic) intrusion often near vertical or with a steep dip, occupying a widened fracture in the country rock, and typically cutting across older rock planes. ▪ A low embankment of earth, precast concrete blocks or asphalt near the edge of the formation to control water movement.
Emerson Crumb Dispersion Test	This test determines dispersion characteristics of a soil in distilled water. Soils are divided into seven classes on the basis of their coherence in water with one further class being distinguished by the presence of calcium-rich minerals. The test is carried out as per the relevant Australia Standard.
Emerson Number	Result as determined from the Emerson Crumb Dispersion Test.
Factor of safety	▪ The ratio of load or stress causing failure to the design load or stress. ▪ The ratio of the load causing failure to the actual load.
Feldspar	A group of abundant rock forming minerals of the general formula $MAl(Al,Si)_3O_8$, where M can be potassium (K), sodium (Na), calcium (Ca), barium (Ba), rubidium (Rb), steradian (Sr) or iron (Fe).
Fill (filling)	▪ The depth from the subgrade level to the natural surface. ▪ That portion of a road where the formation is above the natural surface. ▪ The material placed in an embankment.
Fissility	The property of splitting easily along closely spaced parallel planes, such as bedding in shale or cleavage in schist.
Footing	The widening at the base of a structure to spread the load to the foundation material.
Formation	The surface of the finished earthworks, excluding cut or fill batters.
Foundation	The soil or rock upon which a structure rests.
Friction pile	A pile which carries an axial load by the friction developed between the pile and surrounding ground.
Geotextile (filter fabric, geofabric)	A synthetic cloth used for various purposes including embankment reinforcing and stabilisation, as a filter layer between dissimilar materials and as a strain absorbing membrane between paving layers.
Gravel	A mixture of mineral particles occurring in natural deposits, usually passing a 75 mm sieve and with a substantial portion retained on a 4.75 mm sieve.

Term	Meaning
Greywacke	Indurated coarse grained sandstone that consists of poorly sorted angular to subangular grains of quartz and feldspar, with a variety of dark rock and mineral fragments, embedded in a compact clayey matrix having the general composition of slate. Greywacke commonly exhibits graded bedding and is believed to have been deposited by submarine turbidity currents.
Groundwater	The water below the water table.
Hydrous silicate	Formed from silicon and oxygen combined with various elements, classified by their crystalline structures and containing or combined chemically with water molecules.
Indurated	Said of a soil or rock hardened or consolidated by pressure, cementation or heat.
Interbedded	Said of beds lying between or alternating with others of different character, especially said of rock material laid down in sequence between other beds.
Internal friction angle	Shear strength parameter of a soil representing the angle of shearing resistance.
Iron pyrite	Or fool's gold, mineral composed of iron sulphide, FeS_2, the most common sulphide mineral.
kPa	Kilopascal. A unit of pressure equal to 1,000 pascals.
Leachate	Water that has dissolved soluble substances from rock or soil.
Linear shrinkage	The percentage decrease in length of a soil sample in a mould when oven dried from the liquid limit state.
Liquid limit	See Atterberg limit(s).
Lugeon	The unit expressing the water acceptance of one litre per N.m per minute obtained during a packer test at a pressure of 1,000 kN/m^2 flow.
Matrix	The soil or rock in which something such as a fossil, crystal, or mineral is embedded.
Maximum dry density (MDD)	The greatest dry density of a soil obtained when a soil is compacted in a specified manner over a full range of moisture content. The moisture content at which this density is reached is called the optimum moisture content. Two amounts of compactive effort are commonly specified, referred to as standard and modified.
Metagraywacke	Metamorphosed (altered) version of graywacke due to the influence of pressure and/or heating.
Metamorphism	The mineralogical, chemical and structural adjustment of solid rocks to physical and chemical conditions imposed at depth below the surface zones of weathering and cementation, which differ from the conditions under which the rocks originated.
Moisture content (water content)	The quantity of water which can be removed from a material by heating to 105° C till no further significant change in a mass occurs, usually expressed as a percentage of the dry mass.
NATA	National Association of Testing Authorities Australia.
Neranleigh-Fernvale Group	Geological name given to a 'group' of rocks of Devonian to Carboniferous geologic age. This 'group' is located in south-east Queensland and far northern NSW.
NMLC Triple Tube Core Barrel	Rock coring barrel with a diameter of 49.6 mm that holds the recovered core in.
NMLC	Two inch core.
Optimum moisture content	That moisture content of a soil at which a specified amount of compaction will produce the maximum dry density under specified test conditions.
Outcrop	The exposure, at the surface, of a material (usually rock) differing from its surroundings.
Overburden	The soil or other mineral matter which has to be removed to gain access to the underlying material.
Particle size distribution (grading)	The quantities of the various particle sizes present in a soil or other material, expressed as a percentage of the whole.
Passive pressure	The upper limit of the lateral resistance of soil on a face of a wall which is attained when the wall compresses the soil in front of it in a horizontal direction.
Penetration test	A test carried out with a standard instrument to determine the load bearing capacity of soil.

Term	Meaning
Permeability	The property of a material by virtue of which a fluid such as water can pass through it.
Phreatic surface	Upper zone of saturation in the water table.
Piezocones	A method of determining the in situ materials and their strength properties by measuring the penetration resistance of an electronically instrumented cone and sleeve apparatus.
Piezometer	A method of measuring groundwater levels; and a dedicated bore for measuring groundwater levels.
Pile	A slender member driven, jetted, screwed or formed in the ground to resist loads or thrust.
Pile cap	The structural member connecting and distributing load to a group of piles.
Plasticity index	The numerical difference between the value of the liquid limit and the value of the plastic limit of a soil.
POCAS	Peroxide Oxidation Combined Acidity and Sulphate.
Point Load Index Strength ($I_s(50)$) Test	Method of determining rock strength by crushing core samples in a specialised test apparatus. $I_s(50)$ can be correlated with UCS.
Pore water pressure	The pressure of the water in the voids of a soil.
RQD	A measure of fracture density in rock.
Quartz	Crystalline silica (SiO_2), an important rock forming mineral. It is the commonest gangue (waste) mineral of ore deposits, forms the major proportion of most sands, and has a wide distribution in igneous (especially granitic), metamorphic and sedimentary rocks.
Quaternary sediments	Sediments deposited during the geological period of time from the present to two million years ago.
Retaining wall	A wall constructed to resist lateral pressure from the adjoining ground, or to maintain in position a mass of earth.
Sand	Natural mineral particles which will pass through a defined sieve (normally 4.75 mm or 2.36 mm) and which are free of appreciable quantities of clay and silt.
Schmertmann's Method	This is a method of estimating settlement based on a simplified distribution of vertical strain under the centre of a shallow footing.
Scour	The erosion of a material by the action of flowing water.
Seismic Refraction Survey	Method of determining general soil and rock types based on the refraction of seismic waves as they cross the boundaries between different materials.
Seismic velocity	The rate of propagation of an elastic wave through a rock mass, usually measured in km/second. The wave velocity depends on the type of wave, as well as the material through which it travels.
Settlement	A downward movement of the soil or of the structure it supports. (See also differential settlement).
Shear/crushed zones	Zone along which movement has occurred within the rock mass which has resulted in shearing or crushing of the surrounding rock.
Silt	All alluvial material intermediate in particle size between sand and clay. It is usually non-plastic.
Site investigation	The examination of all those characteristics of a site which might affect the planning, design, construction and operation or performance of any engineering works on the site. Site investigation is not limited to determining subsurface condition but includes consideration of other aspects such as access, drainage, liability to flooding, availability of public utility services and construction materials.
Skin friction	The resistance of the ground surrounding a pile or caisson to its longitudinal movement.
Slope	▪ The inclination of a surface with respect to the horizontal, expressed as rise or fall in a certain longitudinal distance. ▪ An inclined surface.

Term	Meaning
Soil (earth)	▪ That part of the upper weathered layer of the earth's crust which can support plant growth. ▪ Any naturally occurring loose or soft deposit forming part of the earth's crust and resulting from weathering or breakdown of rock formation or from the decay of vegetation.
Soil profile	The profile of soil encountered below the natural ground surface.
SPOS	Peroxide Oxidisable Sulphur.
Standard Penetration Test (SPT)	A standard split spoon sampler, about 50 mm in diameter, is driven into the ground by blows from a drop hammer weighing 64 kgs and falling 0.76 m. The sampler is driven 0.15 m into the soil at the bottom of a borehole, and the number of blows (*N*) required to drive it a further 0.3 m is then recorded. Although the test is entirely empirical, considerable experience with its use has enabled a reasonably reliable correlation to be established between the *N* value and certain soil properties.
Static Cone Penetration Testing	See Peizocone.
Subgrade	The trimmed or prepared portion of the formation on which the pavement is constructed.
Subgrade Reaction Modulus (k)	Modulus used for simulating elastic soil properties for use in structural design.
Subsurface profile	The profile of soil and rock encountered below the natural ground surface.
Test pit (test rolling)	An excavation for examination of subsurface conditions.
Toe	▪ The part of the base of a retaining wall which is on the side remote from the retained material. ▪ The tip of a pile. ▪ The base of an earthen slope.
Topsoil	The top layer of soil that supports vegetation.
Triaxial test	A test to determine the stress-strain properties of a pavement material in which a cylindrical specimen of the material is subjected to a three dimensional stress system, and the axial strain is related to the applied stress.
Unconfined Compressive Strength (UCS)	The strength of a material determined in a triaxial test apparatus when the confining pressure is zero, i.e. unconfined.
Voids	The spaces within the bulk of material not occupied by solid matter.
Voids content	The ratio of the volume of voids to the total volume of the material, expressed as a percentage.
Voids ratio	The ratio of the volume of voids to the solid volume of a material.
Water table	The natural level at which water stands in a borehole, well, or other depression, under conditions of equilibrium.
Wick drains	Vertical drains inserted into a soil to aid in groundwater removal.

1. Introduction

1.1 Summary of the Technical Paper

This technical paper presents the results of the geotechnical investigation undertaken for the proposed Tugun transport corridor, between Stewart Road, Currumbin and Kennedy Drive, Tweed Heads.

The geotechnical work undertaken to date has been limited to those areas where access was reasonable with minimum clearing and/or construction of access tracks. The investigation work undertaken prior to publication of the Tugun Bypass Environmental Impact Statement (EIS) has been sufficient to allow development of a geotechnical model, identification of geotechnical issues and provide geotechnical information for concept design.

The transport corridor has been divided into two areas based on topography and geology. Northern section – Stewart Road to about chainage 2,500, where weathered greywacke and argillite rock dominantly underlay the corridor and southern section – with a deep alluvial soil profile from about chainage 2,500 onwards, where the ground surface is relatively flat and deep alluvial sands with occasional clay bands/layers at depth are encountered. The proposed transport corridor is shown in Figure 1.1.

The following works have been carried out during this investigation:

- obtaining and reviewing existing information from previous geotechnical investigations carried out in the vicinity of the transport corridor;
- field mapping within the area of the corridor;
- drilling boreholes to obtain disturbed and undisturbed samples of the soil profile for laboratory testing and to obtain core samples of the bedrock;
- excavation of test pits along the road sections at grade to examine the subgrade conditions and to obtain bulk samples for laboratory testing;
- performing acid sulphate soil field and laboratory testing on soils encountered along low lying areas of the proposed corridor to assess the likely presence of potential or actual acid sulphate soils (refer to Technical Paper Number 5 for more details);
- installation of groundwater piezometers for subsequent groundwater sampling and piezometric level monitoring; and
- preparation of a geotechnical report presenting the factual data, together with discussion and findings covering:
  - road cuttings;
  - cut and cover tunnelling;
  - rail tunnelling;
  - bridge foundation;.
  - fill embankments;
  - general earthworks;
  - acid sulphate soils; and
  - soil erosion potential.

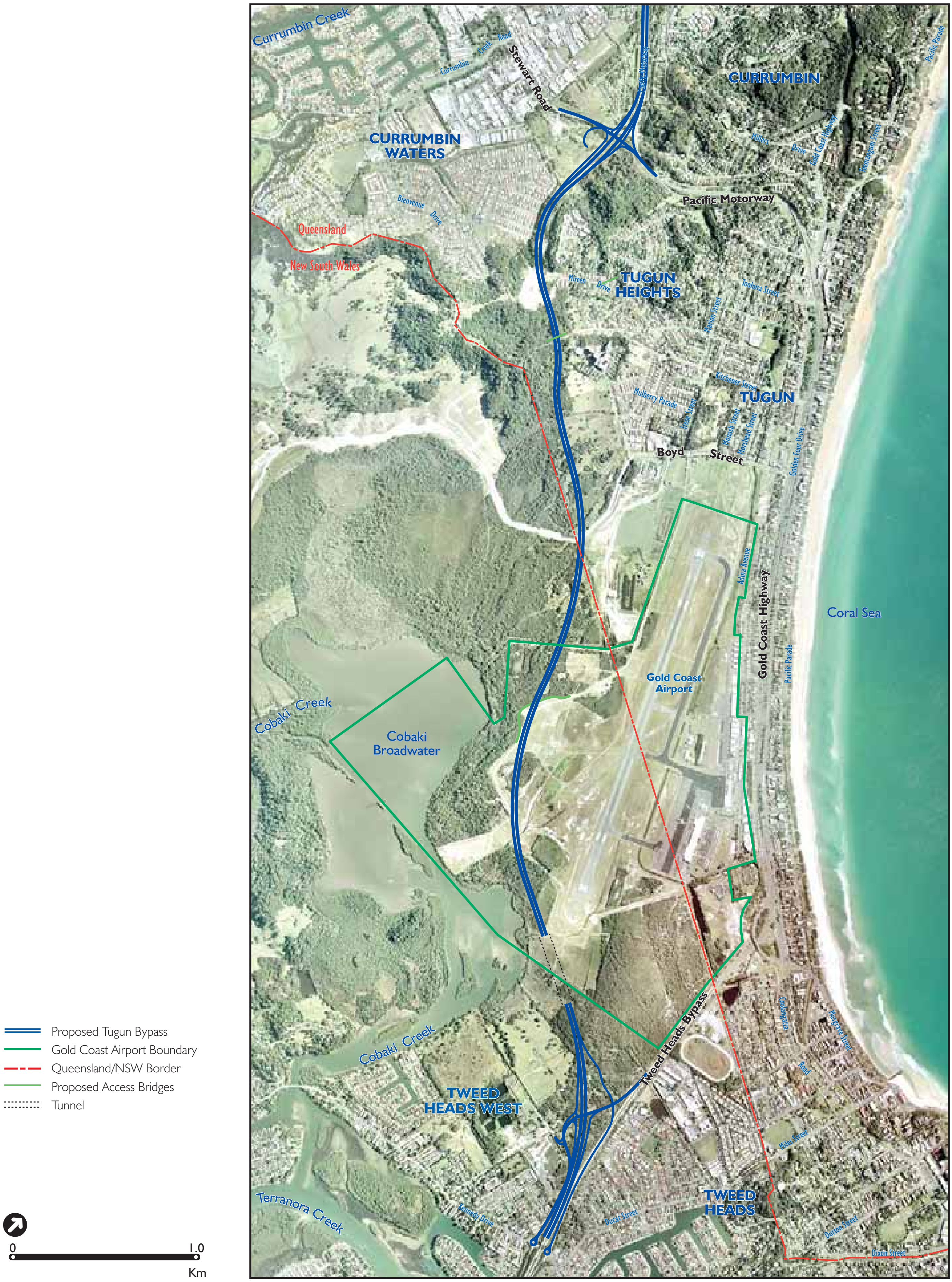


Figure 1.1 **Proposed Tugun Bypass**

1.2 Reporting of Study Findings in the EIS

The studies for the Tugun Bypass environmental impact assessment commenced in 2000. In the subsequent four years the results of the various studies have been used to refine the concept design of the proposal. Further studies were also commissioned to ensure that all aspects of the various environmental issues were fully understood.

The long time period of the assessment has meant that the content of some of the earlier reports has been superseded by newer work. Changes to the design of the bypass have also been introduced to take account of these studies.

In the event that there is a contradiction between the technical papers and the text of the EIS, the EIS takes precedence as it reports the current understanding of issues, impacts and the concept design.

The investigation indicates that there are four key areas along the corridor. The geotechnical constraints associated with the proposed road and rail developments are:

- deep cut through the ridge immediately to the north of the John Flynn Hospital and Medical Centre;
- the high water table and sandy alluvium in the low lying area which would require specific construction methods to support the excavation faces and consideration of potential groundwater drawdown during cut and cover tunnel construction;
- the occurrence of acid sulphate soils at the southern section of the corridor requiring management procedures during the excavation of the proposed cut and cover road and rail tunnels; and
- the construction of the road and later rail structures through the Tugun Landfill site near Boyd Street.

These aspects together with geotechnical issues relating to concept design of the tunnels and road works including excavation conditions, retaining wall design parameters, footing design, founding levels and allowable bearing pressures are discussed in the paper. Further geotechnical investigations will be required to gather detailed depths and thicknesses of the subsurface profile prior to the detailed design. It is envisaged that most of this work will relate to:

- low lying areas where access is difficult;
- ramp, bridge and interchange locations;
- deep rock cut at the ridge immediately to the north of the John Flynn Hospital and Medical Centre; and
- cut and cover tunnels.

The investigation has also confirmed that an acid sulphate soils management strategy will need to be developed. This is discussed in detail in Technical Paper Number 5.

2. Site Description and Proposed Route

The proposed Tugun Bypass transport corridor is located at the southernmost tip of the Gold Coast, Queensland. The proposed alignment crosses the Queensland and NSW border about 150 metres south of the existing Boyd Street alignment and passes through Commonwealth land associated with the Gold Coast Airport as shown in Figure 1.1.

The site has been divided into two areas for the purpose of this report, based on topography and geology as defined below:

- Northern section — Tugun Heights extending from Stewart Road to just north of Boyd Street; and
- Southern section — consisting of a deep alluvial, beach and estuarine soil profile extending from Boyd Street to the proposed Tweed Heads Bypass interchange.

The investigation was planned based on the alignment of the proposed Option C4 route (Queensland Department of Main Roads (Main Roads) 1999b). This route is generally designed to be at grade or on fill embankments up to 7 m in height together with the construction of:

- two road interchanges with access ramps and roundabouts (Stewart Road) at chainage 750 and Tweed Heads Bypass at chainage 6,550);
- an approximately 160 m long bridge over the area known locally as Hidden Valley at chainage 1,950 which was identified after completion of this geotechnical investigation;
- a cut of approximately 20 m deep (centre line) at the ridge immediately to the north of the John Flynn Hospital and Medical Centre (chainage 2,040 to 2,200);
- a cut and cover tunnel under the current obstacle limitation surface at Gold Coast Airport;
- a rail corridor as part of a possible extension from Robina to Gold Coast Airport. This corridor also contains a 1.15 km long rail tunnel (approximately chainage 1,350 to chainage 2,500) together with a railway station structure at chainage 2,800 to 3,000 and a cut and cover tunnel under the obstacle limitation surface at Gold Coast Airport at chainage 5,400 to 5,900; and
- fill road embankment approximately 2.8 km long with maximum height of 7 m, averaging 3 to 4 m.

2.1 Topography

The topography and geomorphology of the area has been predominately controlled by the climate and underlying geology.

The northern section of the site extending from Stewart Road to just north of Boyd Street, passing over Tugun Heights, comprises steep sloping hills and ridges rising about 40 m above the surrounding area. Most slopes are vegetated with native trees and dense undergrowth.

The southern section of the proposed corridor, between Boyd Street and Kennedy Drive, is relatively flat, with ground levels varying to less than 4 m AHD over the 4.5 km of road alignment through this section.

Anthropogenic activities, such as sand mining, land filling, reclamation and airport construction, have changed the natural topography over most of the southern section. A portion of this section crosses Gold Coast Airport land, which has been extensively cleared of vegetation except for grass cover.

2.2 Regional Hydrology

The dominant surface features within the investigation area affecting surface runoff and drainage are the Cobaki Broadwater, situated to the west of the Gold Coast Airport, Cobaki Creek, which drains the Broadwater to the south, and Coolangatta Creek. Other major drainage features, the Currumbin and Terranora Creeks, are located to the north and south of the proposed transport corridor respectively.

Coolangatta Creek rises in the elevated Tugun Heights section and flows through urban areas where it is channelled. It crosses the airport land in an unlined channel adjacent to the main runway, and then, enters an urban environment. On the airport land the creek has a very low gradient, and water levels are maintained at approximately 1.2 m AHD by a weir at the Gold Coast Highway.

Cobaki Broadwater is to the west of the investigation area. Wetland areas fringe the Broadwater, and those in NSW are designated under State Environmental Planning Policy Number 14 — Coastal Wetlands. Small, anastomosing, drainage lines (creeks) are evident, conveying run-off from the hills in the northern section toward the wetland areas on the north-western side of the investigation area.

In the southern part of the investigation area, run-off from the airport runways is channelled to the Cobaki Broadwater via two unlined channel systems. Natural creeks are not evident in this area, having either been modified by development or not been prominent due to the flat topography and sandy nature of the soil.

Low-lying lands south of the investigation area have been drained to the Cobaki Broadwater by closely-spaced, parallel drains.

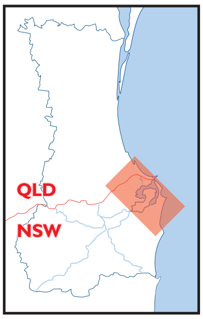
2.3 Regional Geology

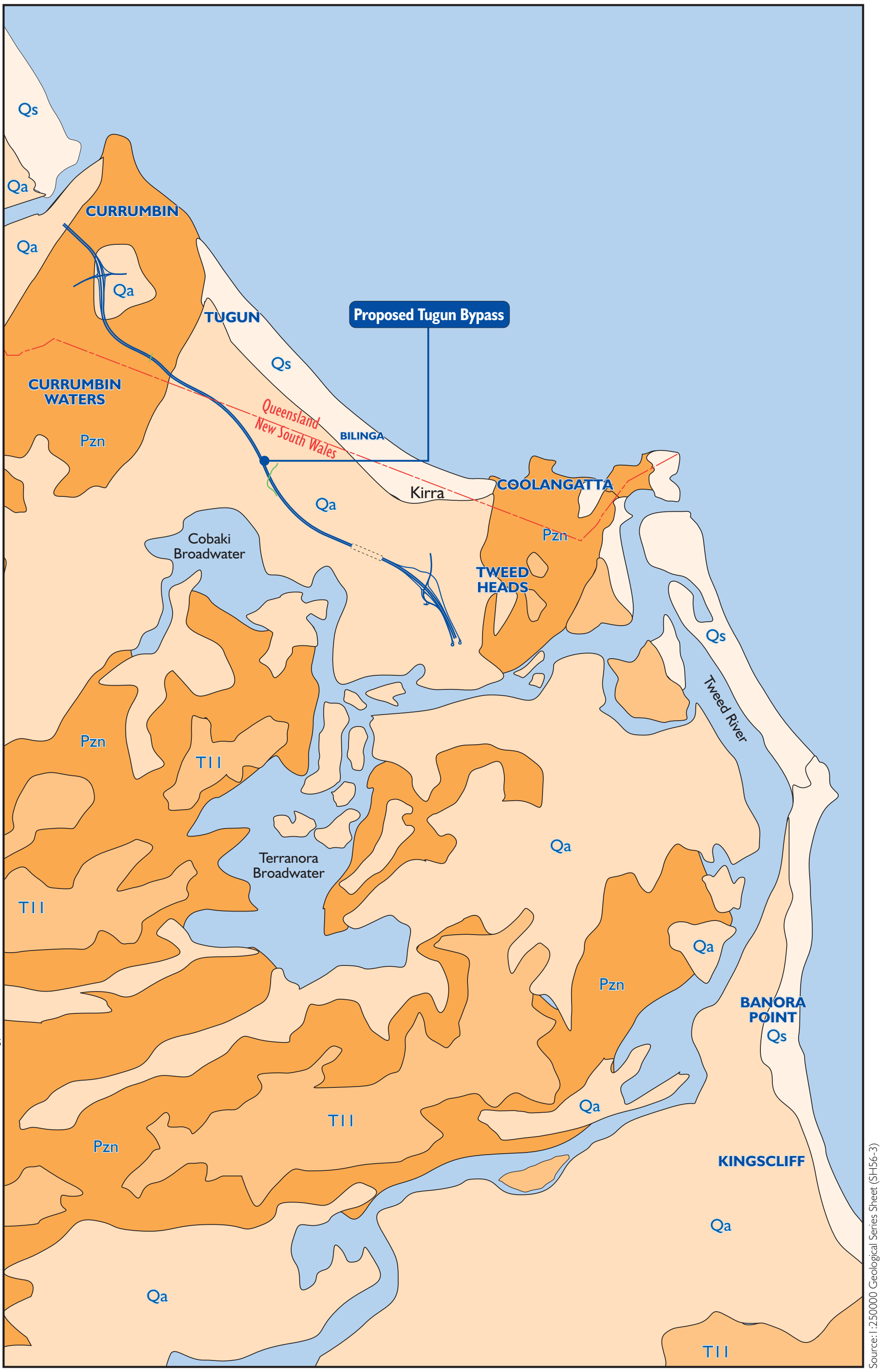
Reference to the Queensland geological survey's 1:250,000 Geological Series Sheet (SH56-3) for Tweed Heads indicates that the Neranleigh–Fernvale Group of Devonian to Carboniferous age rocks underlie the northern section of the site. This unit primarily comprises interbedded fine to medium grained, grey-brown greywacke and very fine-grained, grey argillite (Queensland Department of Mines, 1972).

This supplementary investigation focuses on the southern area, where a deep soil profile consisting of a complex sequence of Quaternary alluvial, beach and estuarine sediments, including river gravels, sand and clay, overlies the Neranleigh–Fernvale beds. The river sediments are local in origin, while the beach and estuarine sediments have a longer history, being derived from sandstones and granites in northern NSW (Willmott 1986).

The result of this natural and also anthropogenic sediment deposition and reworking is a low-lying, flat, sand-covered plain, which is bordered by the Cobaki Broadwater in the west. The Gold Coast Airport has been developed on this coastal plain and the proposed bypass route also traverses this area. East of the airport, a beach ridge and frontal dune, undisturbed by mining but covered by urban development, gives way eastwards to beaches and the ocean.

Figure 2.1 indicates the regional geology affecting the corridor.





Proposed Tugun Bypass
Queensland/NSW Border
Proposed Access Bridges
Tunnel
Qs Beach and dune sand
Qa River gravels, sand, clay - Quaternary alluvial/estaurine sediments
T l l Basalt with members of rhyolite, trachyte, tuff agglomerate, conglomerate – Lamington Volcanics
Pzn Greywacke, slate phyllite quartzite – Neranleigh Fernvale Group

NOT TO SCALE

Figure 2.1 **Regional Geology**

2.4 Regional Hydrogeology

The two geological environments described in Section 2.3 have distinct influences on the hydrogeology. The northern section from Stewart Road to just north of Boyd Street passes over impervious rocks. From Boyd Street to Kennedy Drive, these rocks are covered by younger sediments. These younger sediments, consisting predominantly of sand, form an aquifer, and the older impervious rocks under the sand form a base to this aquifer and are therefore referred to as the 'bedrock'.

2.4.1 Bedrock Aquifer

The rocks of the Neranleigh–Fernvale Group are predominantly fine-grained sediments of marine origin, and weather to clayey soils. Due to their texture, they have very low primary permeability, and any groundwater in these rocks flows in fracture zones. Bores installed to a depth of 30 m in the ridge behind the John Flynn Hospital and Medical Centre were dry on drilling and were still dry three months after installation (Main Roads 1999a), indicating very low groundwater yields.

2.4.2 Alluvial Aquifer

The recent sediments consist of coastal alluvium (principally dune and beach deposits), and are predominantly fine to medium grained sands, with variable often post depositional organic cementation processes forming what is locally known as 'coffee rock'. They comprise an unconfined aquifer, with relatively high permeability. The water table is shallow, varying from 0.11 m to 3.7 m below ground level (Main Roads 2003c).

Permeability testing has been carried out within this unit (Main Roads 2003c) and the results for hydraulic conductivity are lower than those typically encountered in coastal sand dune systems, probably as a result of the widespread but variable organic cementation throughout the upper sequence (coffee rock) and the increasing occurrence of estuarine sediments (finer grained silts and clays) with increasing depth, in this unit.

2.5 Local Hydrogeology

Previous investigation for Technical Paper Number 9, Groundwater (Queensland Department of Main Roads, 2003c), has shown that groundwater occurs as unconfined groundwater held in a sandy aquifer of limited extent. The aquifer is contained in a valley defined by outcropping bedrock in the north (Tugun Hill) and south, the ocean on the east and the Cobaki Broadwater on the west.

The bedrock, onto which the recent sediment has been deposited, are the Neranleigh–Fernvale Group, this is a metasedimentary sequence of low hydraulic conductivity. The depth to bedrock varies, depending on position in the valley, but was expected to be deeper than 30 metres.

The recent sediments consist of coastal alluvium (dune and beach deposits), and are predominantly fine to medium grained sands, with some coffee rock. They comprise an unconfined aquifer, with relatively high permeability. Some estuarine sediments (muds and silts) and minor fluvial deposition (sand and gravel) associated with past sedimentary environments similar to the present Cobaki Broadwater and Coolangatta Creek are also present at depth.

Previous interpretation of the aquifer defined it as an unconfined, layered system (Main Roads, 2003c). The layers were made up of sediments of varying hydraulic conductivity, depending on the type of deposition or degree of cementation that followed deposition. The layers were not seen to be laterally continuous, so that although they introduced an aquifer whose properties were variable both laterally and vertically, they were not considered to be continuous enough to introduce confining conditions. Some of this interpretation was based on an understanding of the sedimentary environment rather than on evidence measured in the field. Obtaining additional evidence has been the aim of this investigation.

Groundwater flow directions in the aquifer were interpreted to be south-westward to Cobaki Broadwater, and north-eastward towards Coolangatta Creek.

Rainfall recharges the aquifer directly via the flat grass-covered areas, and discharges to the Coolangatta Creek, Cobaki Broadwater and wetland areas associated with Cobaki Broadwater.

The water table is shallow, varying from 0.11 m to 3.7 m below ground level (0.3 to 4.5 m AHD). Previous groundwater measurements are listed in Table 2.1. Many were made in geotechnical holes or test pits that were not constructed for groundwater measurement. Some holes and test pits have been damaged or removed so no additional measurement is possible. An update of recent water table movements is provided in Table 5.4.

Table 2.1 Measured Groundwater Levels (mAHD)

Bore	PMG Easting (m)	PMG Northing (m)	Prior measurement[1]	31-Aug-00[2]	20-Sep-00[3]	29-Sep-00[3]	6/7-Feb-01[3]	12-Oct-01
BH-A	49806	87281		1.28	1.28	1.23	2.54	1.235
BH-B	50297	87435		1.65	1.65	1.58	2.17	1.53
BH-C	50487	87161		0.84	0.79	0.79		0.77
BH-D1	51066	87319		3.2	0.98	0.97		0.9
BH-D2	51063	87317		3.37	1.02	1.01		0.84
BH-D3	51060	87316		3.28	3.205	3.23		2.61
BH-E1	50831	86803		0.25	0.09	0.14		0.275
BH-E2	50830	86805		0.28	0.09	0.14		0.08
BH-E3	50828	86808		0.24	0.125	0.17		0.27
BH-9	50569	87056		0.49	0.49	0.50	1.01	0.51
BH-10	50770	86921		0.22	0.205	0.26		0.25
FT1	49809	88090				3.10		
FT2	49819	88071				3.14		
FT3	49857	88109				3.09		
FT4	49841	88085				2.89		
FT5	49901	88159				3.57		3.545
TGW1	48977	89060	2.9500				3.60	2.51
TGW2	49332	88411	3.4500				4.07	
TGW3	49219	88977	4.8290				5.99	5.155
TGW4	49507	88728	2.7400					2.46
TGW5	49747	88366	2.9900					2.75
TGW6	49665	88220	3.1300					2.61

Notes

1 Prior measurements are from Gold Coast City Council (2000), Queensland Dept of Main Roads (1999a) and Egis (2000).

2 Measurements reported as on August 31, 2000 were made during geotechnical investigation as reported in Queensland Dept of Main Roads (2003a).

3 Measurements were made as part of PB's investigation for Technical Paper Number 9, Groundwater, Queensland Department of Main Roads (2003c).

3. Previous Investigations

3.1 Preliminary Geotechnical Assessment, Pacific Highway

The *Preliminary Geotechnical Assessment Pacific Highway 'C' Options at Tugun* (Main Roads 1999a Report) provided an assessment of the route C options. The limited geotechnical field investigation was conducted by the Road System and Engineering/Transport Division of the Queensland Department of Main Roads on 16 August 1999, in an area west of Gold Coast Airport. The field investigation comprised six piezocone test holes and four boreholes. Most piezocones refused at depths of between 3 and 4 m. The boreholes were extended to depths of 7 to 10 m below existing grade, using wash boring techniques. Standard penetration tests (SPT) were carried out in two boreholes and samples obtained for logging. Logging in other boreholes was carried out by observation of the recovered wash bore cuttings. Two boreholes were drilled at piezocone locations for correlation purposes.

3.2 Geotechnical Investigation for Tugun Bypass, Tunnel/Cut Options, Tugun Hill

The Geotechnical Investigation for Tugun Bypass, Tunnel/Cut Options Tugun Hill, chainage 2,070 to chainage 2,360 (C4 Option report) was released in December 1999 by Main Roads (Main Roads 1999c). The aim of this report was to assess tunnel/cut options at the ridge immediately to the north of the John Flynn Hospital and Medical Centre (referred to in the investigation as Tugun Hill) between chainage 2,070 and 2,360. The investigation followed a request from the District Director, South Coast Hinterland, for a geotechnical investigation to advise on the engineering issues involved in road tunnel and cutting options on a section with steep topography located west of the John Flynn Hospital and Medical Centre. The investigation consisted of field mapping, a seismic refraction survey (five seismic lines), borehole drilling (three boreholes), borehole packer testing (two Lugeon Tests) and the installation of piezometers and limited groundwater monitoring. Laboratory testing on rock core was carried out to assess the characteristics of the in situ rock mass.

3.3 Tugun Bypass Environmental Impact Statement

The *Tugun Bypass Environmental Impact Statement* was released in 2004 by PB. This report consisted of 16 technical papers prepared as part of the environmental impact assessment for the Tugun section of the transport corridor. They addressed the possible impacts of the proposal and identified management strategies and mitigation measures, as required to meet the environmental assessment requirements of the Commonwealth, Queensland and NSW governments.

Detailed summaries of these papers are presented below.

3.3.1 Technical Paper Number 4

This technical paper, entitled *Geotechnical Assessment*, presents the results of the geotechnical investigation undertaken for the proposed Tugun transport corridor between Stewart Road, Currumbin and Kennedy Drive, Tweed Heads. The purpose of this study was to develop a geotechnical model, identify geotechnical issues and provide geotechnical information for the development of the concept design.

The investigation involved a desktop study, field mapping, borehole drilling and the excavation of test pits and subsequent laboratory testing of acquired samples. The field program also involved the testing of samples for acid sulphate soil analysis (Technical Paper Number 5) and the installation of groundwater piezometers for subsequent sampling and piezometric surface monitoring (Technical Paper Number 9).

Standard penetration tests (SPT) and, where bedrock was encountered, NMLC coring techniques, were used to obtain the required samples. Bulk samples were also obtained from the test-pit excavations.

In situ geotechnical field-testing included standard penetration testing in the drilled boreholes while dynamic cone penetrometer tests were conducted to assess the strength of sub-grade material strength and to provide correlation with the laboratory California bearing ratio (CBR) tests.

Laboratory analysis of the samples also involved testing for Atterberg limits, moisture content, Emerson number, particle size distribution, triaxial tests and point load index strength tests.

The investigation indicates that there are four key areas along the corridor. The geotechnical constraints associated with the proposed road and rail developments are:

- deep cut through the ridge immediately to the north of the John Flynn Hospital and Medical Centre;
- the high water table and sandy alluvium in the low lying area which would require specific construction methods to support the excavation faces and consideration of potential groundwater drawdown during cut and cover tunnel construction for the underpass beneath the current obstacle limitation surface at Gold Coast Airport allowing for any future runway extension;
- the occurrence of acid sulphate soils in the southern section of the corridor requiring management procedures during the excavation of the proposed cut and cover road and rail tunnels; and
- the construction of the road and later rail structures through the Tugun Landfill site near Boyd Street.

These aspects together with geotechnical issues relating to concept design of the tunnels and road works including excavation conditions, retaining wall design parameters, footing design, founding levels and allowable bearing pressures are construction and operational phases. Mitigation measures are defined following an assessment of alternatives.

These initial investigations were limited to areas where access was reasonable, with minimum clearing and/or construction of access tracks. Due to these limitations, the paper specifies that further geotechnical investigations and laboratory testing would be required prior to any detailed design. Parts of these further investigations in particular some of those for the cut and cover tunnel have been conducted and their results are presented in this supplementary report.

3.3.2 Technical Paper Number 5

This technical paper, entitled *Acid Sulphate Soil Management*, examines the potential impacts of disturbing actual acid sulphate soils (AASS) during the construction phase, including road and rail tunnels. An acid sulphate soils management strategy is proposed for use during construction and operation of the Tugun Bypass.

The investigation involved testing soil samples acquired during the geotechnical studies (Technical Paper Number 5) and the analysis of groundwater samples acquired as a part of the contamination study (Technical Paper Number 7).

The soil samples were tested for potential and actual acid sulphate soils (PASS and AASS), while the groundwater samples were analysed to determine whether acid sulphate soil impacts could be identified from the groundwater chemistry.

The results of these investigations were prepared to satisfy the requirements of all three jurisdictions (Commonwealth, Queensland and NSW governments) and were evaluated using guidelines provided in the *Acid Sulphate Soil Manual* (Stone *et al*. 1998), guidelines prepared by the Queensland Acid Sulphate Soils Investigation Team (QASSIT) (Ahern *et al*. 1998), and technical papers from the *Acid Sulphate Soils, Environmental Issues, Assessment and Management Conference,* June 2000.

3.3.3 Technical Paper Number 9

This technical paper, entitled *Groundwater*, examines the potential impacts on groundwater from the proposed Tugun Bypass. The purpose of this study was to investigate the impacts that may ensue from the construction and operation of the Boyd Street to Kennedy Drive section of the proposed Tugun transport corridor.

The proposal has the potential to affect groundwater during both construction and operation of the proposed bypass. While the impervious surface created by the bypass has the potential to alter groundwater recharge, the greatest potential impact to groundwater would be from the construction and operation of the proposed tunnel and approach ramps. Thus a significant portion of this report focuses on impacts and mitigation measures relating to the construction and operation of the proposed tunnel.

A groundwater model was developed to investigate the extent of these impacts and the potential success of mitigation measures that have been incorporated into the design.

The study shows that with mitigation, impacts on sensitive environmental groundwater receptors are minimised during construction, so that groundwater quantity and quality would not be compromised. During the operational phase, cross-tunnel drains would equilibrate groundwater on each side of the tunnel and allow unhindered groundwater movement past the tunnel, thus successfully reinstating existing groundwater conditions.

3.4 Other relevant documentation

Information relating to the legislative context of these investigations and the scope of all 16 technical papers that comprise the *Tugun Bypass Environmental Impact Statement* are provided as a component of this report.

4. Current Investigation Methods

The field components of the geotechnical investigation for the Tugun Bypass were undertaken between August to September 2000 and between January to February 2002 and May 2003 for the Supplementary Geotechnical Investigation Reoprt. A summary of the investigation methods is given in the following sections. Reference should be made to *Limitations of Geotechnical Site Investigations* in Appendix A for a discussion of the limitations of the investigation procedures.

The overview for the two studies to date are as follows:

4.1 Overview of Investigations to 2000

The scope of investigations undertaken for this study comprised:

- obtaining and reviewing existing information from previous geotechnical investigations carried out in the vicinity of the proposed transport corridor;
- field mapping in the vicinity of the centre line of the proposed bypass alignment;
- drilling boreholes to obtain disturbed and undisturbed soil samples for laboratory testing and to obtain core samples of the bedrock;
- excavation of test pits along the road sections at grade to examine the subgrade conditions and to obtain bulk samples for laboratory testing;
- performing a survey along low lying areas of the proposed corridor to assess the likely presence of potential or actual acid sulphate soils;
- installation of groundwater piezometers for subsequent groundwater sampling and piezometric level monitoring; and
- preparation of a geotechnical report presenting the factual data, together with discussion and recommendations covering:
  - road cuttings;
  - cut and cover tunnelling;
  - rail tunnelling;
  - bridge foundations;
  - subgrade construction;
  - fill embankments;
  - general earthworks;
  - subgrade preparation;
  - placement of fill materials;
  - compaction control; and
  - construction materials.

4.2 Desktop Study

A desktop study was undertaken to review all available geological and geotechnical data for the site in order to identify and describe the geology and soils within the area of the proposed development. The study involved the review of the following maps/ documents:

- Main Roads 1999a - Report 591500CB Rev 0, *Preliminary Geotechnical Assessment Pacific Highway C Options at Tugun*, prepared by Connell Wagner and dated 6 September 1999;
- Main Roads 1999c - Report R3188, *Geotechnical Investigation for Tugun Bypass: Tunnel/Cut Options Tugun Hill chainage 2,070 – chainage 2,360 (C4 Option)* dated 2 December 1999;
- Reference to the 1:250,000 Geological Series Sheet for Tweed Heads (SH56-3); and
- NSW Department of Land and Water Conservation, 1:25,000 Acid Sulphate Soils Risk Maps for Tweed Heads (9641 S4) and for Bilambil (9541 S1).

4.3 Surface Mapping

The field mapping was undertaken to confirm desktop study information and comprised a walk-over survey of the proposed corridor by an experienced geologist. Field mapping incorporated logging of rock outcrops and exposures/cuttings, measurement of surface grades and noting other relevant surface features including topographic and drainage characteristics, erosion and indications of slope instability (if any), which may have an impact on the design or construction of the proposed transport corridor.

4.4 Borehole Drilling in 2000

Drilling in 2000 for this report has been undertaken in two stages in 2000 and in 2002–2003. These are discussed s follows.

4.4.1 Borehole Locations

Twelve boreholes (BH-s) labelled BH-1 to BH-11 were drilled along the proposed corridor using a Daly 1,000 truck mounted drilling rig at locations shown on Figure 4.1. An additional nine boreholes labelled BH-A to BH-E3 were drilled outside the corridor for the purpose of groundwater assessment. Details of the 21 boreholes are set out in Table 4.1. The locations shown in Table 4.1 and on the borelogs use the grid references with the MGA datum these have been converted to the Pacific Motorway Grid (PMG) datum.

Northern Section – Tugun Heights

Boreholes BH-1, BH-1a, BH-2, BH-3 and BH-4 were positioned at critical locations along the northern section of the proposed corridor (refer to Figure 4.1) to investigate near surface rock at Tugun Heights. The sitings of these boreholes were as follows:

- borehole BH-1 was positioned near the proposed Stewart Road interchange (chainage 860) and drilled to a depth of 10.73 m below existing grade;

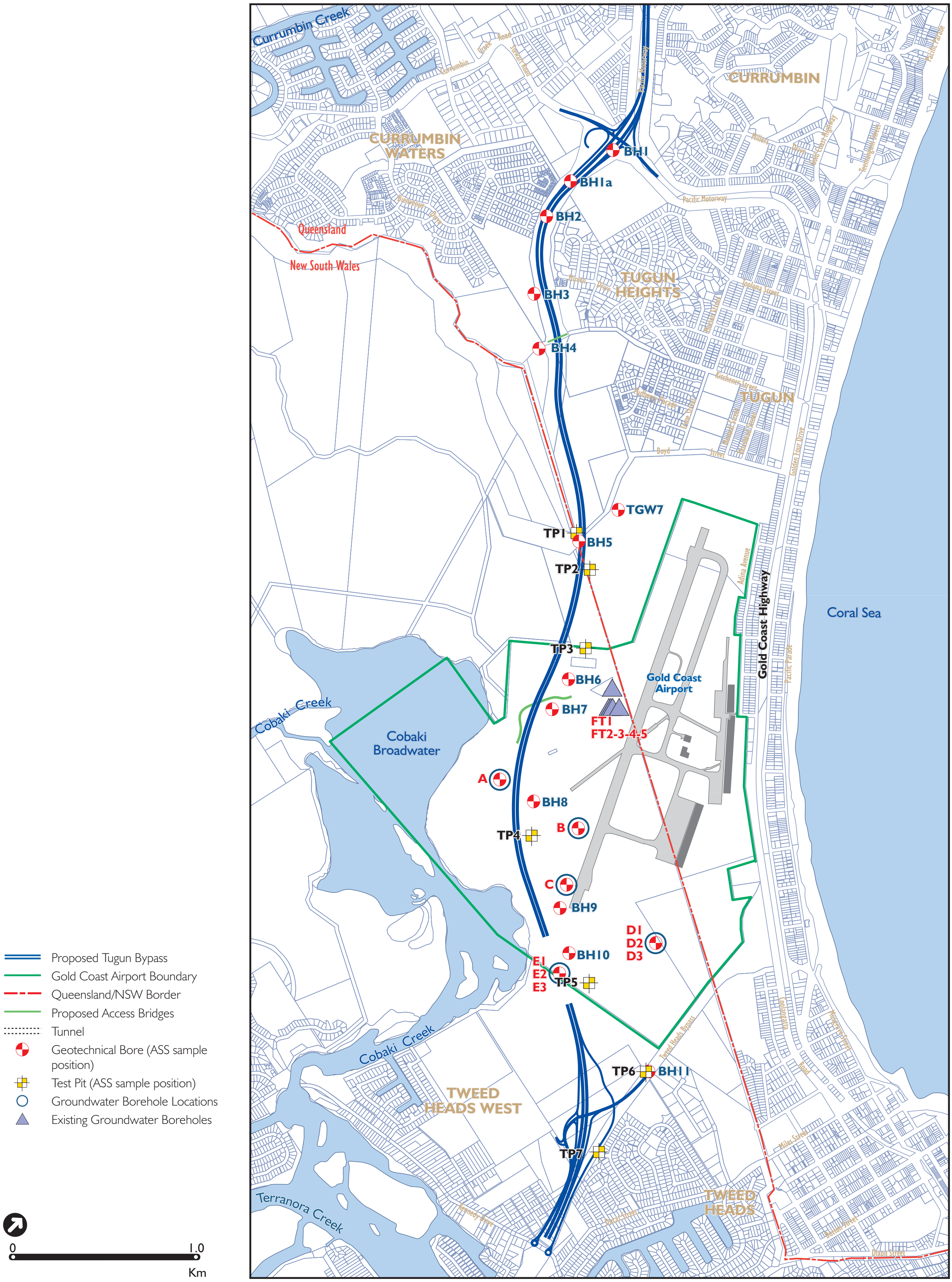


Figure 4.1 **Geotechnical Investigation Plan**

- borehole BH-1a was located at chainage 1,120 to investigate the proposed deep filling area between chainages 1,100 to 1,300, drilled to a depth of 5.95 m;
- borehole BH-2 was positioned at chainage 1,380 and drilled to a depth of 23.25 m below existing grade to investigate the subsurface conditions for the proposed bored rail tunnel;
- borehole BH-3 was drilled at chainage 1,840 to a depth of 11.6 m to investigate an area of cut; and
- borehole BH-4 was located within the deep rock cut between chainages 2,090 and 2,200. This borehole was drilled at an angle of 60° from horizontal to a depth of 39.75 m below existing grade. This borehole aimed to intersect subvertical joints identified in previous reports.

Table 4.1: Borehole Details

Borehole	Locations		Surface Reduced Level (m)	Borehole Termination Depth Below Existing Surface Level (m)
	PMG Easting	PMGNorthing		
BH-1a	47573	89848	6.00	5.95
BH-1	47600	90135	6.76	10.73
BH-2	47630	89605	20.17	23.25
BH-3	47908	89272	34.08	11.60
BH-4	48149	89075	62.40	39.75
BH-5	49103	88514	3.68	29.95
BH-6	49642	87951	4.12	5.00
BH-7	49698	87765	4.15	6.00
BH-8	50017	87343	2.24	14.95
BH-9	50569	87056	1.29	16.95
BH-10	50770	86921	1.22	5.95
BH-11	51570	86797	0.65	14.95
BH-A	49806	87281	2.78	6.00
BH-B	50297	87435	2.45	6.00
BH-C	50487	87161	2.34	6.00
BH-D1	51066	87319	4.60	12.00
BH-D2	51063	87317	4.72	9.00
BH-D3	51060	87316	4.73	6.00
BH-E1	50831	86803	1.00	5.50
BH-E2	50830	86805	0.98	8.50
BH-E3	50828	86808	1.04	11.50

Note 1: BH 4 inclined borehole at 60° to horizontal.

Southern Section – Deep Alluvial Sediment Profile

The remaining seven boreholes targeted critical areas within the deep alluvial soil profile found along the southern section of the proposed corridor. The positioning of some of these boreholes was limited on the airport site due to height restrictions associated with aircraft operations.

Borehole drilling was employed instead of Static Cone Penetration testing due to the possible presence of the cement sand layers as indicated in the previous investigations. These layers may cause premature refusal of the cone testing and prohibit further exploration of the subsurface conditions. The sitings of these boreholes are shown in Figure 4.1 and detailed as follows:

- borehole BH-5 was positioned at chainage 3,200 and drilled to a depth of 29.95 m below existing grade;
- boreholes BH-6 and BH-7 were located at chainages 4,100 and 4,250 respectively to maintain a regular test interval and were drilled to depths of 5 m and 6 m below grade respectively;
- borehole BH-8 was drilled to a depth of 5.95 m below existing grade at chainage 4,800 to investigate subsurface conditions near a proposed concrete box culvert at chainage 4,800;
- boreholes BH-9 and BH-10 were positioned at chainages 5,420 and 5,650 respectively to investigate the proposed cut and cover tunnels, south-east of the main Gold Coast Airport runway. Both boreholes were drilled to a depth of 14.95 m below existing grade; and
- borehole BH-11 was positioned near the proposed Tweed Heads Bypass interchange (chainage 6,550) and drilled to a depth of 16.45 m below existing grade.

4.5 Test Pits

Seven test pits were excavated using a medium size excavator along the proposed corridor within the deep alluvial soils west and south of the airport runway where construction to or close to existing grade is anticipated. The test pits (refer to Figure 4.1) were excavated to a maximum depth of 3 m to examine the subgrade conditions including groundwater inflow. Bulk samples were obtained for laboratory analysis and Dynamic Cone Penetrometer Tests (Standards Australia 1289.6.3.3 – 1997) were carried out to assess the strength of the material and to provide correlation with the in situ California Bearing Ratio (CBR).

The surface level of each test pit as indicated on the respective log was determined by instrument survey techniques. The details of each test pit are shown in Table 4.2.

Table 4.2: Test Pit Details

Test Pit	Location		Surface Reduced Level (m)	Test Pit Termination Depth Below Existing Surface Level (m)
	AMG Easting	AMG Northing		
TP-1	49067	88534	3.03	3.00
TP-2	49273	88452	3.68	2.60
TP-3	49584	88135	4.19	2.55
TP-4	50160	87209	2.32	2.50
TP-5	50987	86889	0.67	3.00
TP-6	51574	86803	0.71	3.00
TP-7	51729	86287	1.05	3.00

4.6 Acid Sulphate Soils

An acid sulphate soil field and laboratory testing program was carried out on soils encountered along low lying areas of the proposed corridor, to assess the likely presence of potential or actual acid sulphate soils. The following was undertaken during the field investigation:

- sampling of all test pits from near surface level and at approximately 0.5, 1.0, 1.5, 2.0 and 3.0 m below existing grade;
- boreholes BH-6 to BH-11 (excluding BH-9 and BH-10) were sampled to a depth of 3 m using the SPT sampler in intervals of 0.5 m below existing grade; and
- boreholes BH-9 and BH-10, within the proposed cut and cover road and rail tunnel area were sampled at 0.5 m intervals using the SPT sampler to a depth of 3 m then at 1 m intervals to a depth of 12 m below existing grade.

The acid sulphate soil sampling was conducted in accordance with the *Guidelines for Sampling and Analysis Procedure for Lowland Acid Sulphate Soils (ASS) in Queensland* (Queensland Department of Natural Resources 1998) and *Acid Sulphate Soil Manual* by Acid Sulphate Soil Management Advisory Committee (ASSMAC) (Stone *et al*. 1998).

A total of 109 soil samples recovered from test locations along the corridor were subjected to field indicator testing for actual and potential acid sulphate soils. These samples were frozen for the duration of the geotechnical investigation. At the conclusion of the field testing 45 samples identified as comprising actual or potential acid sulphate soils were chosen for further laboratory Peroxide Oxidation Combined Acidity and Sulphate (POCAS) testing. Refer to Technical Paper Number 5 for more details.

4.7 Laboratory Testing

A summary of the completed laboratory testing is provided in the following section. Contamination testing of site soils was outside the scope of this geotechnical investigation. An assessment of contaminated land is provided in Technical Paper Number 6.

4.7.1 Geotechnical

Selected representative samples of the alluvial soils and bedrock from the boreholes were tested in a NATA registered laboratory. A summary is provided in Table 4.3.

Table 4.3: Geotechnical Laboratory Testing

Test Description	Method	Number of Tests
CBR (four day soaked)	AS 1289 5.5.1, 6.1.1	5
Atterberg Limits	AS 1289 3.1.1, 3.2.1, 3.3.1, 3.4.1	3
Moisture Content	AS 1289 2.1.1	3
Emerson Number	AS 1289 3.8.1	3
Particle Size Distribution to 60 µm	AS 1289 3.6.1	5
Triaxal tests on weak rock	AS 1289 6.4.2	4
Point Load Index Strength Tests (undertaken by site staff using NATA certified test equipment)	AS 4133 4.1	99

Note: Refer to glossary for definitions of tests undertaken.

4.7.2 Acid Sulphate Soils Testing

Forty-five soil samples were chosen for laboratory POCAS testing. The samples were chosen so that at least one sample came from each test location along the corridor. In addition, all samples producing a positive field result obtained from within the cut and cover tunnel area between chainages 5,300 and 5,800 were tested. The test results are included in Technical Paper Number 5.

5. Results of Investigation

5.1 Desktop Study

5.1.1 Previous Investigations

The findings and the recommendations of two previous investigations are summarised in the following section and have been considered in the analysis and findings in Chapter 7.

Tugun Bypass Geotechnical Invesigation cut at chainage 35,750 m (Main Roads 1992).

The results of this investigation were unavailable at the time of writing this report and will be considered in future studies.

Geotechnical Investigation for Tugun Bypass: Tunnel/Cut Options Tugun Hill (Main Roads 1999c).

The results of the geotechnical investigation for tunnel and cut options at the ridge immediately to the north of the John Flynn Hospital and Medical Centre were presented in a report prepared by Main Roads (Main Roads 1999c). The investigation consisted of field mapping, seismic refraction survey (five seismic lines), borehole drilling (three boreholes), borehole packer testing (two Lugeon Tests) and installation of piezometer and limited groundwater monitoring. Laboratory testing on rock core was carried out to assess the characteristics of the in situ rock mass. Table 5.1 summarises the point load/unconfined compressive strength (UCS) test results from the report.

Table 5.1: Point Load/Unconfined Compressive Strength Test Results

Rock Type and Weathering	Point Load Index $I_{s(50)}$ (Diametral)			UCS (MPa)	Seismic Velocity (m/s)
	Average	Range	No. Test		
HW[1] Argillite	0.03	0.02-0.04	4		<800
HW[1] Argillite	0.22	0.11-0.47	6	1.84	950-1,550
HW Metagreywacke	0.08	0.08-0.08	3		<800
MW Metagreywacke	0.35	0.19-0.72	20	2.73,5.34, 9.36[1]	950-1,550
MW Interbedded Argillite/ Metagreywacke	0.25	0.24-0.28	3	2.02	950-1,550

Notes 1: This specimen was dry and not tested at in situ moisture content like other specimens.
Source: Main Roads 1999c.

Conclusions and recommendations for road cutting based on the results were as follows:

- it is unlikely that the bored tunnel option could be justified in view of the extreme cost differential. It was recommended that it would be more economical to use open cut construction with an extensive revegetation program for the batters;
- the rock mass at the site has undergone extensive weathering to a significant depth. A profile of 6 to 22 m of extremely weathered to highly weathered metagreywacke and argillite overlies mainly moderately weathered rock;

- it is reported that defects (bedding, partings and joints) are common with the rock having a close to medium defect spacing;
- the most significant defect orientation is parallel to bedding/foliation, which dips at 45° to 70° with a dip direction of 240° to 250°. Other major defects intersected during drilling dipped at low, medium and high angles. Most defect planes had iron-stained coatings indicating water seepage is present;
- at this cut location, the bedding strike is at 20° to the proposed road direction with the bedding dipping out of the proposed north-east cut face and into the south-west cut face;
- the groundwater table is expected to be below grade;
- the weathered rocks were generally classified as being 'fully rippable' in terms of the Main Roads Specification MRS 11.04 Clause 8.2 with the assessment being based on the Weaver's Rippability Classification (Main Roads 1999d);
- batter slopes no steeper than one horizontal to one vertical gradient with 5 m wide berms at 7 m vertical spacing were recommended;
- treatment of the batters (vegetation) to prevent erosion and lessen visual impact was recommended;
- appropriate drainage to minimise the effects of surface water on the cut were recommended;
- a bulking/compaction factor of 0 to 5 percent has been estimated for the cut; and
- an overpass structure could be required to access Lot 7 RP214065 if the cut option is implemented. Preliminary data indicates that bored piles would probably be required at the abutments and piers on the cut slope. Spread footings (with an allowable bearing capacity of 900 kPa on highly weathered rock with factor of safety = 3) are considered suitable foundation types for the piers at the base of the cut.

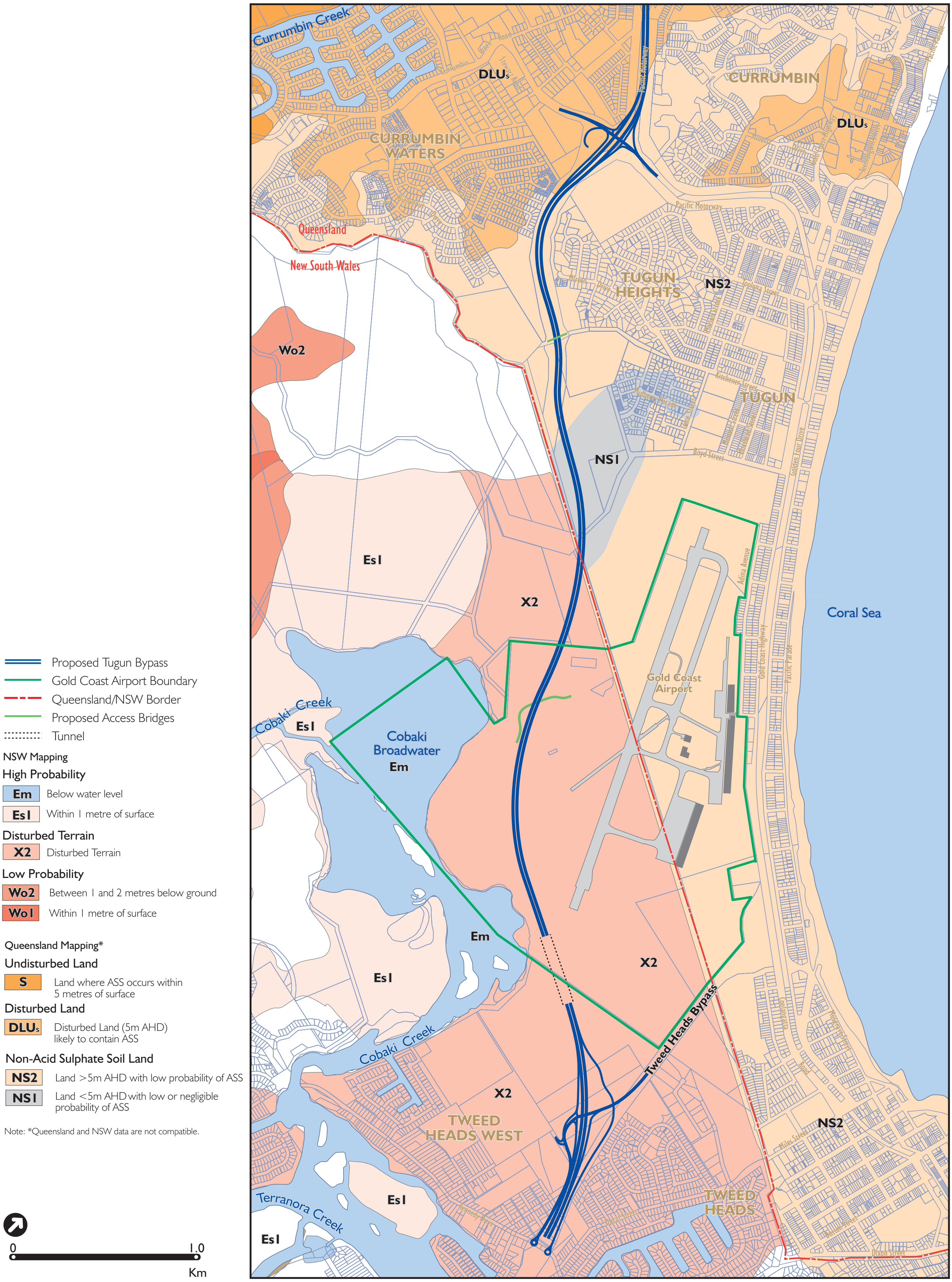
Preliminary Geotechnical Assessment Pacific Highway C Options at Tugun

The results of the preliminary geotechnical assessment, (Main Roads 1999a) indicated that the southern section of the site is underlain by unconsolidated coastal alluvium of Quaternary Age. The boreholes and piezocones indicated that the alluvium is predominantly loose to medium dense quartz sand which extend to depths of at least 10 m. The report indicated that the sand has been lightly cemented by organics in some areas forming layers of indurated cemented sand locally referenced as 'coffee rock'. Soft clay was reported to be intersected at only one piezocone location at the existing Kennedy Drive interchange. Groundwater was reported at approximately 0.5 m below existing grade.

5.1.2 Acid Sulphate Soils

Reference to the NSW Department of Land and Water Conservation Acid Sulphate Soil Risk Maps for Tweed Heads (9641 S4) and (9541 S1) for Bilambil reveals a high probability for the occurrence of acid sulphate soils within the soil profile in the vicinity of the proposed corridor. Soils at or near the ground surface together with those below the water level pose a severe environmental risk if disturbed by activities such as dredging, shallow drainage, excavation or clearing.

Figure 5.1 indicates the extent and risk of acid sulphate soils within the study area. Refer to Technical Paper Number 5 for more details.



Source: Acid Sulphate Soil Risk Maps (9641 S4)/Acid Sulphate Soils - Tweed Heads to Redcliffe (DNR-SEA-1-A0 3234)
* Queensland Acid Sulphate Soil Risk Map has been compiled using different assessment criteria from that in NSW.

Figure 5.1 **Areas with Potential Acid Sulphate Soils**

5.1.3 Existing Groundwater Bores

Five groundwater bores have previously been installed to monitor the fire training area within Gold Coast Airport. No previous data was available prior to the fieldwork of this investigation.

5.1.4 Soil Contamination

The assessment of soil contamination is not part of the scope of this geotechnical investigation. However, during the desktop study, two areas were identified as potentially contaminated and they are highlighted in this section.

These are the Tugun Landfill between chainages 3,200 and 3,700 and a fire fighting training area in Gold Coast Airport which is to the east of the proposed corridor near chainages 4,000 to 4,100. Potential contamination could result from use of fuels and fire retardants.

Details of the assessment of contamination issues along the route of the proposed transport corridor are provided in Technical Paper Number 6.

5.2 Field Mapping

Field mapping was conducted within the vicinity of the proposed corridor that had been pegged during the field investigation. The mapping was to survey the major geological exposures and note any potential areas of instability that could influence design or construction of the proposed transport corridor.

The topography is generally flat along the southern section of the proposed corridor beyond chainage 2,500. Steep sloping hills and ridges in Tugun Heights dominate the northern section between Stewart Road and chainage 2,500.

Inspection and logging of rock outcrops and road cuttings along the proposed corridor in the Tugun Heights area revealed strata dipping to the south-west with dip angles of 45° to 55° from near surface. Outcrops of rock were extremely to distinctly weathered, extremely low to low strength argillite and greywacke. Exposures were generally highly fractured to fractured, with irregular joint orientations.

Mapping along the southern section of the corridor identified areas where potentially difficult ground conditions can be expected. The proposed corridor traverses the Tugun Landfill between chainages 3,200 and 3,700. Low lying swampy ground was identified to the north and west of the landfill and to the south of the airport runway. A large area of fill was identified south of the airport runway between chainages 5,500 and 5,800.

5.3 Field Investigation

The subsurface conditions encountered in the boreholes and test pits are summarised in Section 6.1. Bridge/structure sites and pavements have been discussed separately.

It should be noted that in making an assessment of the subsurface conditions across a site from a few widely spaced test locations there is a risk of undetected variations occurring between test locations. However the program undertaken is considered to have provided an adequate indication of the general subsurface conditions for concept design purposes and for the environmental impact assessment.

A summary of the more pertinent aspects of the conditions encountered is given for each section of the route below.

5.3.1 Northern Section – Tugun Heights

The subsurface profile within the northern section of the site generally comprises a thin residual soil profile overlying bedrock from the Neranleigh-Fernvale Group. However, minor fill material overlying alluvial soils was encountered in BH-1a.

Fill

In the localised depressions within the low lying, natural drainage channel between chainages 1,100 and 1,300, fill material was encountered in borehole BH-1a to a depth of 0.9 m below existing grade. The fill comprised gravely, silty sand and clay and was used to form a road embankment.

Alluvial Sediments

Alluvial sediments were encountered beneath fill material at borehole BH-1a. The sediments were likely to occur between chainages 1,100 and 1,300 and are inferred to overlie bedrock. The alluvial soils comprise clayey silt of medium plasticity, very stiff sandy clays and fine grained sand.

Residual Soils

Residual soils generally comprise medium plasticity, pale grey-yellow brown very stiff to hard clays, with cobbles and bands of extremely weathered argillite and greywacke (possibly isolated boulders). The thickness of the residual soil profile ranged from zero in most locations up to 6 m along particularly weathered sections of the proposed corridor.

Bedrock

Bedrock was encountered from ground level within boreholes BH-1, BH-2, BH-3 and BH-4.

Extremely and distinctly weathered argillite and greywacke was encountered for the full depth of boreholes BH-3 and BH-4.

Interbedded bands of extremely weathered, very low to low strength argillite and greywacke were encountered in boreholes BH-1 and BH-2 above depths of approximately 6.7 m and 14 m respectively. In BH-2, the rock strength increased gradually from low to medium strength below 14 m to the termination of the hole.

Beddings, partings and joints mainly orientated between 45° to 70°/230° to 264° (dip/dip direction) were identified. Other major defects measured near boreholes BH-1, BH-2, BH-3 and BH-4 are listed in Table 5.2.

Table 5.2: Structural Defects

Borehole	Bedding Defect Orientation (degrees)	Defect (Mainly Joint and Weathered Zone) Orientation (degrees)
BH-1	45/256; 55/260	81/158; 80/164 85/39 81/312; 85/300 65/112
BH-2	70/264; 21/242; 40/232	64/65; 55/62 70/32 85/122 50/204
BH-3	45/255; 45/260	80/170; 85/170; 85/172 46/30; 45/20; 45/20; 30/24
BH-4	50/252; 47/230; 60/228	85/172; 80/160 70/132; 70/108 64/88 45/27

Bedding within the recovered cores was generally measured at approximately 45°. Other major defects (joints and weathered zone), intersected during drilling, dipped at 45° to 85° as shown on the borehole logs. The dip directions of these defects appeared to be random when determined from the drill core using the bedding parting as reference.

5.3.2 Southern Section – Deep Alluvial Sediment Profile

The subsurface profile within the southern section of the site generally comprises loose to medium dense sands with cemented dense to very dense sands 'coffee rock' at varying depths within the profile. Stiff alluvial clays were only intersected within borehole BH-5. Residual clays were also encountered within borehole BH-5 at greater depth. Disturbed natural materials indicating prior sandmining operations were not encountered in any of the boreholes or test pits dug for this investigation. However it is well known that sandmining was carried out in this area in the past.

Fill

Fill was encountered in boreholes BH-9 and BH-10 to depths of 0.9 m and 0.3 m respectively. Fill material has also been spread over the ground surface around the southern end of the airport runway and generally comprises gravely sands and sand.

Alluvial Sediments

The alluvial soils from the low-lying areas of the site are predominantly brown/grey loose to medium dense sands. Dark brown slightly cemented (dense) and cemented (very dense) sands, locally referred to as 'coffee rock' were intersected within the boreholes at depths varying from 0.8 to 14.4 m. The thickness of the cemented sands varied from thin (0.1 m to 0.2 m) bands up to 1.8 m layers and were found to vary greatly over the study area.

Medium plasticity, stiff alluvial sandy clays were only intersected within borehole BH-5 in 0.5 m bands between 10 m and 15 m below existing grade.

Possible Residual Soils

Possible residual clays as determined by visual inspection were only intersected within borehole BH-5 at a depth of 25.5 m below existing grade. These clays are high plasticity, very stiff and pale grey with a brown-red mottling.

Bedrock

Bedrock was not encountered within any of the boreholes in the southern section of the proposed corridor.

5.4 Groundwater

5.4.1 During Field Investigations

Northern section – Tugun Heights

Groundwater was observed in boreholes BH-1a, BH-2 and BH-4 at 4.0, 6.7 and 8.5 m respectively. No free groundwater was observed in boreholes BH-1 or BH-3. During the field investigation period, groundwater was measured within borehole BH-2 for six days following drilling of the borehole. Groundwater measurements within borehole BH-4 were taken 24 hours after drilling. Groundwater measurements taken during drilling are presented in Table 5.3.

No long-term groundwater monitoring was performed as part of this geotechnical investigation in Tugun Heights.

Southern section

Groundwater inflow was noted during drilling of all boreholes. Generally groundwater levels in the southern section of the proposed corridor vary between 0.6 to 2.2 m below existing grade. Groundwater measurements taken during drilling are presented in Table 5.3.

Further groundwater testing/monitoring in the southern section has been completed subsequent to the geotechnical investigation. The results of this groundwater investigation are presented in Technical Paper Number 9. A brief discussion of these results is presented in Section 5.4.2.

5.4.2 After Field Investigation

Piezometers were installed in seven boreholes located in the areas surrounding the proposed corridor in the southern section. Monitoring of the water levels within these piezometers was undertaken subsequent to the geotechnical investigation. The purposes of the monitoring and testing were to examine the groundwater regime, in terms of permeability, water table gradient and variation. Groundwater levels were measured at the time of bore installation, at the commencement of aquifer testing and nine days after the completion of the testing. The pre-existing groundwater bores within the fire fighting training area of Gold Coast Airport were also measured. The location of these existing groundwater bores are shown on Figure 4.1 and denoted as FT1 to FT5. For detailed results refer to Technical Paper Number 9. A brief summary on the water table measurements are given in Table 5.4.

Table 5.3: Groundwater Levels at Time of Borehole Installation

Test Location	Groundwater Depth (m)	Reduced Level (mAHD)
BH-1	NFGWO1	NFGWO
BH-1a	4.0	2.0
BH-2	6.7	13.47
BH-3	NFGWO	NFGWO
BH-4	8.5	55.04
BH-5	0.8	2.88
BH-6	2.0	2.12
BH-7	2.2	1.95
BH-8	0.9	1.24
BH-9	0.8	0.49
BH-10	1.0	0.22
BH-11	0.6	0.05
BH-A	1.5	1.28
BH-B	0.8	1.65
BH-C	1.5	0.84
BH-D1	1.4	3.2
BH-D2	1.35	3.37
BH-D3	1.45	3.28
BH-E1	0.7	0.3
BH-E2	0.7	0.28
BH-E3	0.8	0.24
TP-1	0.9	2.13
TP-2	1.0	2.60
TP-3	1.6	2.55
TP-4	2.0	0.32
TP-5	2.1	-1.23
TP-6	1.5	-0.79
TP-7	1.1	0.04

Note 1: NFGWO – denotes No Free Groundwater Observed.

Table 5.4: Groundwater Monitoring Levels

Borehole	PMG Easting	PMG Northing	Calculated Water Level after Construction (refer to Technical Paper Number 9) (mAHD)	Water Level 20-9-2000 (mAHD)	Water Level 29-9-2000 (mAHD)
BH-A	49806	87281	1.28	1.28	1.23
BH-B	50297	87435	1.65	1.65	1.58
BH-C	50487	87161	0.84	0.79	0.79
BH-D1	51066	87319	3.20	0.98	0.97
BH-D2	51063	87317	3.-37	1.02	1.01
BH-D3	51060	87316	3.28	3.21	3.23
BH-E1	50831	86803	0.25	0.09	0.14
BH-E2	50830	86805	0.28	0.09	0.14
BH-E3	50828	86808	0.24	0.13	0.17
BH-9	50569	87056	0.49	0.49	0.5
BH-10	50770	86921	0.22	0.21	0.26
FT1	49809	88090			3.1
FT2	49819	88071			3.14
FT3	49857	88109			3.09
FT4	49841	88085			2.89
FT5	49901	88159			3.57

Note: Extracted from Technical Paper Number 9.

5.5 Acid Sulphate Soils

A total of 109 soil samples, recovered from test locations along and near the proposed corridor, were subjected to field indicator testing for actual and potential acid sulphate soils. Of these, four produced field results indicating the presence of actual acid sulphate soils at test pit TP-6 and borehole BH-C. A further 36 samples indicated uncertainty to the presence of actual acid sulphate soils at test pits TP-3, TP-4, TP-5, TP-6 and boreholes BH-1a, BH-6, BH-7, BH-8, BH-10, BH-A, BH-C and BH-D1. A further 91 soil samples indicated the presence of potential acid sulphate soils.

Details of testing and the results obtained are provided in Technical Paper Number 5. Those areas where there is a risk of finding acid sulphate soils are shown on Figure 5.1.

5.6 Laboratory Results

Results of the laboratory tests are summarised in Tables 5.5 to 5.8. The detailed laboratory report sheets are available from Parsons Brinckerhoff on request.

Table 5.5: Results of Moisture Content and Atterberg Limits Testing

Sample Location	Depth (m)	General Description[1]	FMC[1] (%)	LL[2] (%)	PL[3] (%)	PI[4] (%)	LS[5] (%)
BH-5	20.5-20.95	Clay (CL) (Alluvial)	30.7	33	14	19	9
BH-5	26.5-26.95	Clay (CL) (Residual)	22.2	44	17	27	12
BH-5	29.5-29.95	Clay (CL) (Residual)	26.4	39	18	21	10

Notes 1: FMC = Field Moisture Content
2: LL = Liquid Limit
3: PL = Plastic Limit
4: PI = Plasticity Index
5: LS = Linear Shrinkage

Table 5.6: Results of California Bearing Ratio Testing

Sample Location	Depth (m)	Description	California Bearing Ratio Value
TP-1	0.5-1.0	Sand; fine to medium grained, grey	9
TP-3	0.5-1.0	Sand; fine to medium grained, grey	18
TP-4	0.5-1.0	Sand; fine to medium grained, grey	22
TP-6	0.5-1.0	Sand; fine to medium grained, grey-green, with shells	9
TP-7	0.5-1.0	Sand; fine to medium grained, grey	18

Table 5.7: Results of Particle Size Distribution Testing

Sample Location	Depth (m)	Description	Percentage Passing									
			9.5 (mm)	4.75 (mm)	2.36 (mm)	1.18 (mm)	0.6 (mm)	0.425 (mm)	0.3 (mm)	0.15 (mm)	0.075 (mm)	0.38 (mm)
TP-1	0.5-1.0	Sand; fine to medium grained, grey					100	98	89	6	2	2
TP-3	0.5-1.0	Sand; fine to medium grained, grey					100	98	76	2	1	1
TP-4	0.5-1.0	Sand; fine to medium grained, grey				100	99	91	58	1	0.4	0.3
TP-6	0.5-1.0	Sand; fine to medium trained, grey-green, with shells	100	98	94	97	96	89	69	9	4	3
TP-7	0.5-1.0	Sand; fine to medium grained, grey					100	99	88	5	2	1

Table 5.8: Results of Triaxal Consolidated Undrained Test with Pore Water Measurement

Sample Location	Depth (m)	Description	Effective Internal Friction Angle (degree)	Effective Cohesion (kPa)
BH-4	3-4	Argillite, extremely weathered, extremely low strength	30	0
BH-4	7-8	Argillite, extremely weathered, extremely low strength	52	204
BH-4	27-28	Greywacke, extremely weathered, extremely low strength	42	169
BH-4	36-37	Argillite, extremely weathered, extremely low to very low strength	50	0

6. Methods of Analysis

6.1 Geotechnical Model

On the basis of the conditions encountered during the fieldwork, a geotechnical model was developed for each of the fills and cuts along the proposed Tugun Bypass alignment. The geotechnical model adopted at each location was an idealised subsurface profile with appropriate design parameters assigned on each stratum. This idealised profile was based on the site investigation information obtained from boreholes or test pits in the vicinity. The appropriate design parameters are then mostly correlated from the results of laboratory test or SPT data obtained from field testing. The models then provided a framework for undertaking geotechnical assessment of the following:

- excavation conditions;
- stability for excavated cuts through the ridge immediately to the north of the John Flynn Hospital and Medical Centre;
- cut and cover and bored/drilled rail tunnel construction;
- embankment construction; and
- settlement analyses of highway embankments over deep alluvial soils.

Figures 6.1 to 6.3 illustrate the geotechnical profile along the length of the proposed transport corridor. The longitudinal section depicted is approximate only and does not represent the final proposed road alignment. The proposed road profile can be found in Technical Paper Number 2.

6.2 Settlement Analyses

Settlement calculations were undertaken for typical fill embankments. A spreadsheet program using Schmertmann's method was used to calculate elastic settlements (Schmertmann 1970). Elastic parameters were adopted on the basis of the field test results including SPT and Static Cone Penetration Test data from and on the basis of the material descriptions.

The consolidation settlement of the structure was analysed by classical soil mechanics principles using 1–d consolidation theory. Parameters were based on empirical correlations with clay strength and moisture content. Consolidation settlement was assessed only at the vicinity of borehole BH-1a at the depression gully and for BH-5 at the proposed Boyd Street overpass where significantly thicker clay materials were encountered. No settlement analysis was carried out elsewhere along the corridor, due to the relatively limited presence of the clay materials.

Findings from these analyses are dealt with in Sections 7.3.2 and 7.3.3.

6.3 Stability Analyses

Slope stability analyses have been undertaken for all major cut slopes, and fill embankments. Analyses were performed using the commercial program SLOPE/W Version 4.

A minimum factor of safety for the cut slopes and embankments of 1.5 has been adopted for long-term stability conditions. A factor of safety of 1.3 has been adopted for short-term duration cases such as during or immediately following construction or under earthquake loading.

The slope stability analysis for the fills have included a nominal 20 kPa distributed load applied to simulate traffic loading within the carriageways and 10 kPa on the embankment crests. The stability analysis for the cuttings have included a 10 kPa distributed load on crests and berms to simulate maintenance vehicles.

All sections have been analysed for long-term conditions, earthquake conditions and immediately following construction. A horizontal acceleration coefficient of 0.06 G has been used to model earthquake loading. This value has been adopted from AS1170–4 *Minimum Design Loads on Structures, Part 4: Earthquake Loads* (1993).

LEGEND

SOIL / ROCK TYPE

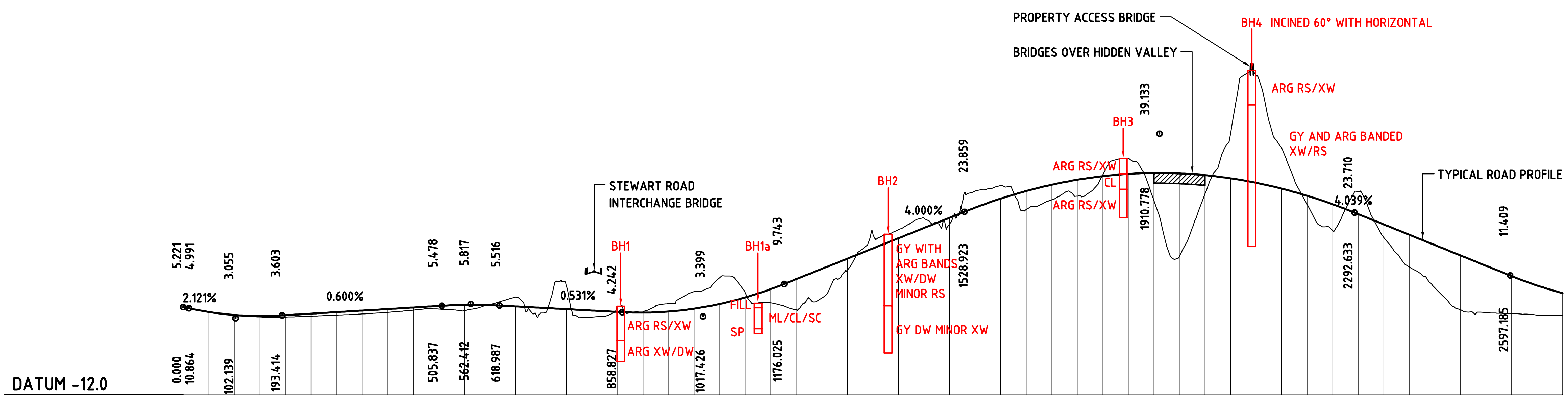
ARG	ARGILLITE	SP	POORLY GRADED SAND
GY	GREY WACKE	ML	LOW PLASTICITY SILT
RS	RESIDUAL SOIL	CL	LOW PLASTICITY CLAY
SM	SILTY SAND	CH	HIGH PLASTICITY CLAY
SC	CLAYEY SAND		

ROCK WEATHERING CONDITION

XW	EXTREMELY WEATHERED
DW	DISTINCTLY WEATHERED
SW	SLIGHTLY WEATHERED

NOTES

THE LONGITUDINAL SECTION DEPICTED IS APPROXIMATE ONLY AND DOES NOT REPRESENT THE FINAL ALIGNMENT OF THE PROPOSAL. THE PROPOSED ROAD PROFILES CAN BE FOUND IN THE TECHNICAL PAPER NO.3.



CHAINAGE	DESIGN HEIGHTS	EXISTING HEIGHTS
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100	3.692	3.606
150	3.483	3.294
200	3.642	3.271
250	3.942	3.447
300	4.242	3.652
350	4.542	3.881
400	4.842	4.204
450	5.142	4.626
500	5.442	4.833
550	5.645	4.543
600	5.599	5.724
650	5.352	7.182
700	5.086	4.083
750	4.820	9.345
800	4.554	3.256
850	4.289	5.382
900	4.144	4.286
950	4.351	5.924
1000	4.915	8.242
1050	5.836	10.966
1100	7.115	8.066
1150	8.750	6.039
1200	10.702	4.688
1250	12.702	6.989
1300	14.702	12.418
1350	16.702	19.528
1400	18.702	20.364
1450	20.702	22.599
1500	22.702	21.458
1550	24.679	27.912
1600	26.436	28.994
1650	27.930	24.327
1700	29.162	26.187
1750	30.130	26.939
1800	30.834	31.129
1850	31.276	34.302
1900	31.455	26.069
1950	31.370	15.164
2000	31.022	27.340
2050	30.411	39.216
2100	29.537	50.545
2150	28.400	36.314
2200	26.999	25.305
2250	25.336	20.181
2300	23.412	27.826
2350	21.393	18.848
2400	19.373	13.066
2450	17.353	8.985
2500	15.334	4.707
2550	13.314	4.138
2600	11.295	3.986
2650	9.460	3.658
2700	7.955	3.610

LONGITUDINAL SECTION - ROAD

0 10 20 30 40 1:1000 VERT

0 100 200 300 400 1:10000 HORIZ

Figure 6.1 Longitudinal Section
Sheet 1 of 3

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LEGEND

SOIL / ROCK TYPE

ARG	ARGILLITE	SP	POORLY GRADED SAND
GY	GREY WACKE	ML	LOW PLASTICITY SILT
RS	RESIDUAL SOIL	CL	LOW PLASTICITY CLAY
SM	SILTY SAND	CH	HIGH PLASTICITY CLAY
SC	CLAYEY SAND		

ROCK WEATHERING CONDITION

XW	EXTREMELY WEATHERED
DW	DISTINCTLY WEATHERED
SW	SLIGHTLY WEATHERED

NOTES

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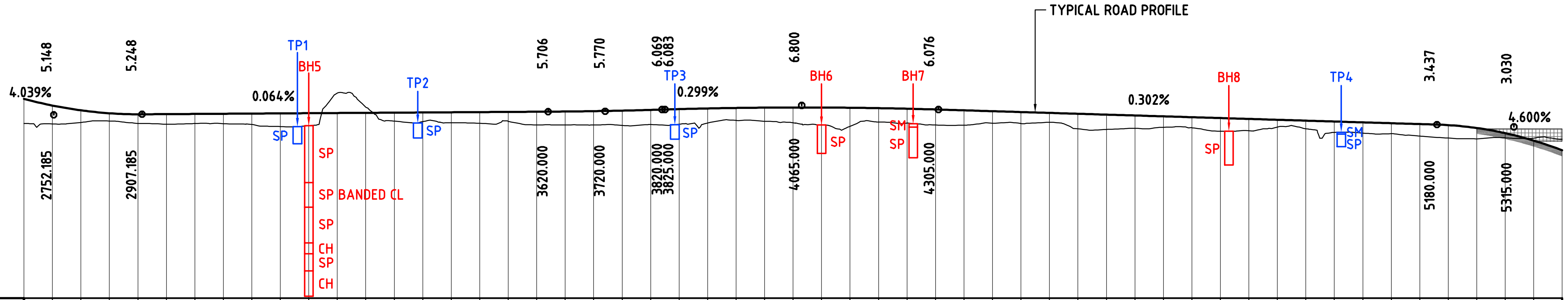

DATUM -27.0

CHAINAGE	DESIGN HEIGHTS	EXISTING HEIGHTS
2700	7.955	3.610
2750	6.782	3.557
2800	5.939	3.815
2850	5.427	4.067
2900	5.246	3.645
2950	5.275	3.510
3000	5.307	3.618
3050	5.339	3.594
3100	5.372	3.462
3150	5.404	3.091
3200	5.436	3.258
3250	5.468	8.945
3300	5.500	6.525
3350	5.532	4.054
3400	5.564	3.967
3450	5.596	3.763
3500	5.629	3.662
3550	5.661	4.021
3600	5.693	3.380
3650	5.730	3.389
3700	5.795	3.369
3750	5.888	3.298
3800	6.011	3.440
3850	6.154	3.505
3900	6.272	3.711
3950	6.359	4.172
4000	6.414	3.998
4050	6.439	3.744
4100	6.432	3.362
4150	6.393	3.134
4200	6.324	3.886
4250	6.223	3.701
4300	6.091	3.382
4350	5.940	3.336
4400	5.790	3.680
4450	5.639	3.615
4500	5.488	3.848
4550	5.337	2.680
4600	5.186	2.588
4650	5.036	2.779
4700	4.885	2.969
4750	4.734	2.885
4800	4.583	2.284
4850	4.432	2.366
4900	4.282	3.009
4950	4.131	2.717
5000	3.980	2.073
5050	3.829	1.943
5100	3.678	1.848
5150	3.528	1.424
5200	3.345	1.078
5250	2.836	0.933
5300	1.929	1.112
5350	0.624	1.055
5400	-1.079	0.887

LONGITUDINAL SECTION - ROAD

0 10 20 30 40 1:1000 VERT

0 100 200 300 400 1:10000 HORIZ

Figure 6.2 Longitudinal Section
Sheet 2 of 3

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LEGEND

SOIL / ROCK TYPE

ARG	ARGILLITE	SP	POORLY GRADED SAND
GY	GREY WACKE	ML	LOW PLASTICITY SILT
RS	RESIDUAL SOIL	CL	LOW PLASTICITY CLAY
SM	SILTY SAND	CH	HIGH PLASTICITY CLAY
SC	CLAYEY SAND		

ROCK WEATHERING CONDITION

XW	EXTREMELY WEATHERED
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NOTES

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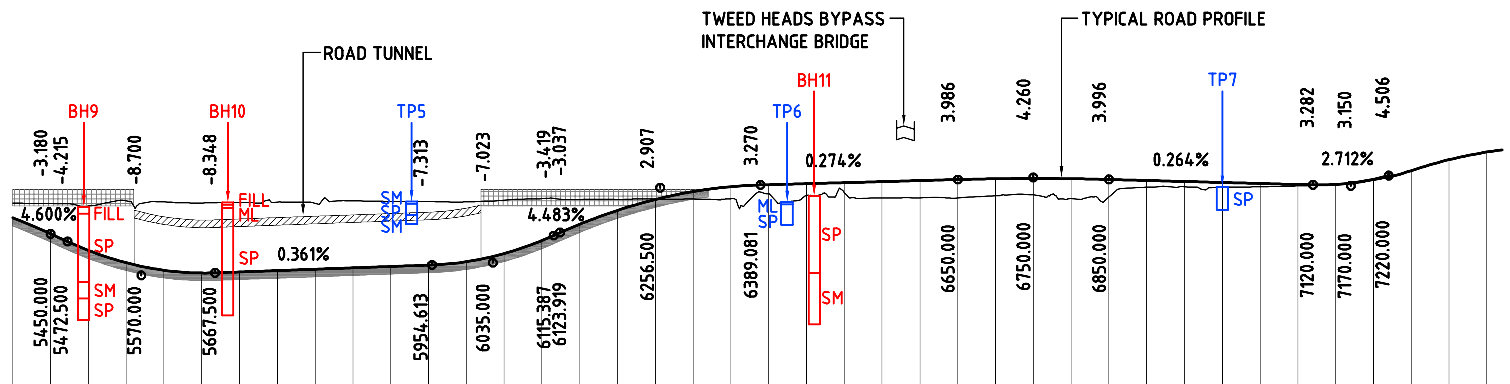

DATUM -24.0

CHAINAGE	DESIGN HEIGHTS	EXISTING HEIGHTS
5400	-1.079	0.887
5450	-3.180	0.815
5500	-5.384	0.488
5550	-7.016	1.179
5600	-8.012	1.042
5650	-8.373	0.885
5700	-8.231	0.943
5750	-8.051	1.001
5800	-7.871	0.838
5850	-7.690	1.177
5900	-7.510	1.084
5950	-7.330	1.079
6000	-6.885	1.035
6050	-5.802	1.263
6100	-4.079	1.452
6150	-1.921	1.351
6200	-0.085	1.358
6250	1.354	1.406
6300	2.396	1.406
6350	3.042	1.439
6400	3.300	2.086
6450	3.438	1.881
6500	3.575	2.283
6550	3.712	1.564
6600	3.849	1.549
6650	3.986	1.560
6700	4.089	1.836
6750	4.125	1.960
6800	4.094	1.450
6850	3.996	1.664
6900	3.864	2.977
6950	3.731	3.036
7000	3.599	3.003
7050	3.467	3.032
7100	3.335	3.145
7150	3.337	3.285
7200	4.023	3.882
7250	5.165	5.165
7300	6.620	6.620
7350	7.627	7.627
7370	7.919	7.919

LONGITUDINAL SECTION - ROAD

0 10 20 30 40 1:1000 VERT

0 100 200 300 400 1:10000 HORIZ

Figure 6.3 Longitudinal Section Sheet 3 of 3

7. Analysis and Findings

7.1 Northern Section – Tugun Heights

7.1.1 Stewart Road Interchange

This interchange is currently under construction with bridge foundations, reinforced earth abutments and embankments complete. Geotechnical investigations for the interchange comprised borehold BH-1 undertaken in the 2000 phase of the work and a further geotechnical investigation *Stewart Road Overpass the Access Ramp* by Parsons Brinckerhoff dated 30 June 2003.

7.1.2 Bridge – Hidden Valley

A bridge is to be constructed over a 16 m deep gully approximately between chainages 1,880 and 2,000. The proposal for the bridge at this location was developed after completion of the geotechnical field investigation. As a result, no field investigation testing has been directly positioned at this location. However, based on the two nearest boreholes (BH-3 and BH-4) drilled at either side of the bridge site and observation of rock outcrops nearby, it is likely that extremely weathered argillite or better is present around the level within the gully base (RL+15.1 mAHD). However, further site investigation would need to be undertaken for the detailed design of this bridge as it is also possible that the gully has been covered by minor alluvium or sediments associated with the gully base.

For concept design of the bridge foundation, there is no borehole or other geotechnical information available, except for the two boreholes (BH-3 and BH-4) nearby. Therefore, bored piers socketed into distinctly weathered, very low to low strength argillite are recommended. However, should further investigations during detailed design, confirm competent rock occurs near the surface, high level footings founded on rock may be considered. It is noted that deep foundations may be required due to the design consideration to resist lateral loadings of the bridge. The proposed bored pier footings should be designed based on an allowable end bearing pressure of 1,200 kPa. Deeper socket lengths may be required depending on the vertical and lateral loading of the piers.

In general, if shaft adhesion is to be used for tension and compression loads, the piers may be designed for an allowable socket adhesion value of 120 kPa for compression loads and 60 kPa for tension loads below the nominal socket. These values are conditional on the walls of piers being roughened, clean and free of clay smear.

For foundation design which is not governed by uplift forces, a high level spread footing embedded at least 0.5 m into extremely to highly weathered, very low to strength rock could be considered. An allowable wearing capacity of 600 kPa could be adopted.

It is anticipated that piers founded in weathered rock using the above recommendations would result in settlement less than 1 percent of the diameter of the pier.

7.1.3 Deep Cutting at Ridge Behind the John Flynn Hospital and Medical Centre

Geotechnical Model

The proposed cut at the ridge behind the John Flynn Hospital and Medical Centre (chainage 2,100) would involve excavation to approximately 20 m depth below the existing grade line at the centre of the road alignment and approximately 30 m in height at the batter. On the basis of the investigations undertaken, the expected profile on the cut face comprises a minor residual silty clay profile overlying interbedded extremely weathered argillite and greywacke bedrock. The argillite and greywacke revealed in BH-4 is extremely to distinctly weathered, extremely low to very low strength. The examination of the rock core indicated that the most of the bedding and joints are iron-stained, indicating the possible presence of localised groundwater seepage in the slope.

However, in the previous investigation (BH-1 and BH-3 of Main Roads 1999c) it was reported that slightly better quality rocks were encountered in this area. This previous investigation reported highly to moderately weathered interbedded argillite and greywacke of low strength was encountered below 6 to 22 m below ground.

Due to the discrepancy of rock strength in these two investigations and the possible evidence of localised groundwater seepage, a lower bound approach has been adopted and the rock modelled to be of extremely to distinctly weathered and extremely low to low strength in the slope analysis. Shear parameters of the rock mass based on saturated triaxial testing were adopted for the slope analysis. A conservative phreatic surface has been adopted in the geotechnical model, with reference to the encountered groundwater level in BH-4.

Slope Stability

Instability of the extremely weathered rock, in particular in the upper cut batters can occur as a result of a shear failure through the weathered rock material. This is typified by deep seated classical slip circle failure or possibly in a form of block or wedge failures by movement along preferential planes (joints, bedding and other defects). However, due to the deep weathering profile encountered, the rock is anticipated to perform predominantly with a soil-like behaviour. Therefore a shear failure model in the form of a slip circle is considered appropriate and has been adopted in the analysis. Further assessment may have to be undertaken to assess the block and wedge mode of failure, if the rock quality is found to be stronger coupled with adverse jointing from further site investigations.

Slope stability using the SLOPE/W computer program has been undertaken in this cutting adopting the geotechnical parameters set out in Table 7.1.

Table 7.1: Strength Parameters Used for Slope Stability Analysis for the Ridge Behind the John Flynn Hospital and Medical Centre Cutting

Material Description	Effective Cohesion, c' (kPa)	Effective Angle of Internal Friction, ϕ ' (Degrees)	Unit Weight (kN/m^3)
Extremely Weathered Argillite	5	37	18
Extremely to Distinctly Weathered Argillite/Greywacke	10	40	19

The factor of safety for significant shear failure is estimated to be 1.37 for the short - term with seismic load and 1.47 for long-term conditions. The critical slip surfaces of both cases occur at the upper portion of the cut within the weaker extremely weathered argillite (the output of the computer analysis is attached as Appendix B).

On the basis of this preliminary slope stability analyses undertaken for this cut, permanent batter slopes are recommended at 1.5 horizontal to 1 vertical for extremely to distinctly weathered rocks. Benches at maximum height intervals of 7 m are recommended to provide drainage and debris collection. The drains on benches should be graded longitudinally at grades of between 0.5 and 2 percent. Should longitudinal falls of greater than 2 percent be required the drains should be concrete lined to prevent scouring and erosion. The berm width of 5 m is proposed to allow free movement of maintenance vehicles or machinery.

The argillite and greywacke would tend to weather rapidly on exposure and potentially cause erosion of the batters. It is recommended that the batters are vegetated as soon as practicable after excavation to mitigate any erosion potential. Alternatively, shotcrete or flattening of the batter to minimise the erosion potential could be used. The vegetation may require synthetic matting or similar construction.

All dirty run-off water from above the cutting would be directed to a sediment control pond. An open drain would be provided behind the crest of the cutting and at the back of each bench. These drains would be concrete lined to prevent scouring. Such a drain would minimise water flow over the face of the cutting and assist in minimising erosion. Energy dissipation structures may also be required on the toe of the bench. Run-off in these drains could contain sediment and would be directed to a sediment control pond.

With additional information regarding the groundwater and the rock strength it may be possible to consider a steeper slope in the lower portion of the cut batter analysis. Analysis has suggested it may be possible to steepen these slopes to batter slopes of 1:1 with bench widths of 4 m. Also, the top batter of each side of the cutting may have to be soil nailed and shotcreted to remove the risk of failures.

Slope design must be verified after further work and it is expected that other constraints such as erosion control and maintenance would dictate adoption of a flatter batter.

Although the adopted slope design will be one with acceptably low risk there will always be some potential for small to medium scale failures. Small failures include fretting and loosening of small blocks in the face while medium scale failures would be expected to generally be in the order of 5 to 10 m in length. These failures are most likely to occur during the construction phase and can generally be controlled using appropriate stabilisation measures such as ground anchors, meshing, shotcreting and/or perhaps localised rock bolts. Therefore, it is essential that an experienced Geotechnical Engineer should regularly and progressively inspect the batters as excavation proceeds, to check for any potential unstable blocks or other adverse geological structure upon exposure.

Rippability

The investigation by Main Roads (1999c) and the core recovered from BH-4 indicate that the rock mass has a Rock Quality Designation (RQD) of less than 50 percent which results in a rock classification of very poor and poor quality with respect to excavation. The extremely to distinctly weathered bedrock is expected to be easily rippable in terms of the Main Roads Specification (MRS 11.04) Clause 14.3.2. Occasional moderately weathered rock, if encountered, would require some harder ripping.

7.2 Southern Section – Deep Alluvial Sediment Profile

7.2.1 Cut and Cover Road and Rail Tunnels

Site Conditions

The proposed cut and cover road tunnel extends from chainage 4,740 to chainage 6,940 and the actual submerged tunnel box is about 460 m long. The road tunnel would require minimum excavation to approximately 10.5 m below ground (assuming 2 m thick tunnel base), with the groundwater level at approximately RL-0.7 m, 0.5 m below existing ground level.

Boreholes BH-8, BH-9 and BH-10 drilled along the alignment of the proposed road tunnel, generally show loose sands to 1 to 6 m depth overlying lightly cemented sands (coffee rock) to the termination of the boreholes (5 m to 15 m). The SPT N value of the lightly cemented sands ranges from 20 to 40. As indicated by the SPT N value, the cemented sand layer is not uniform or consistent. A surficial minor fill material consisting of gravely silty sand of 0.9 m thickness was encountered in BH-9.

Details of the groundwater model and mass permeability are provided in Technical Paper Number 9. In that paper, the mass permeability of the area is reported as ranging from 0.68 to 0.74 m/day (7.8 E-6 to 8.5 E-6 m/second).

Construction Methods

The construction method would be governed by way of:

- providing excavation support;
- groundwater control;
- ground movement control; and
- feasibility and ease of construction.

In general, possible construction methods would be:

- open cut trench with dewatering control using slurry wall or grout curtains;
- temporary ground cut off wall/structure for groundwater control and temporary excavation support; and
- permanent wall, (diaphragm wall as groundwater cut-off and excavation support).

Further discussion is provided in the following section on the feasibility of various construction methods and their limitations. The proposed method of construction is described in Technical Paper Number 2.

Excavation/Retaining Structure

If open cut option is considered for tunnel construction, the cut slope in loose and medium dense sand would need to allow a batter of at least three horizontal to one vertical, which may hinder its feasibility as it would encroach further into the airport site.

The extent of the batter suggests a cut-off wall should be considered for the tunnel design. To overcome piping failure (safety factor of two) at the base of the excavation, a minimum 10.5 m penetration cut-off wall is required. However, this embedded length should also be governed by the volume of inflow seepage and should be further analysed based on the groundwater data. It would be preferred that the cut-off wall is installed within the dense cemented sand layer on which the permeability would be significantly less than the non-cemented sands. It should be noted that based on the limited investigation data, the occurrence of this cemented sand is not consistent along the alignment of the tunnel and the cut-off wall design should take into account this variation.

The cut–off walls could be in a form of high modulus sheet piles, secant bored pile walls or diaphragm walls if a larger structural capacity is required. As an alternative, temporary excavation adopting a contiguous secant grout injected pile wall with strutting or anchors may be considered. However, the diaphragm wall incorporating deeper slurry cut-offs for groundwater control provide an advantage that the wall could be part of the permanent structure.

Lateral support during construction of the retaining walls would be required using anchors/bracing, or the wall would be designed as a free cantilever. Use of anchor or top down construction methods with the tunnel roof as a lateral support should be considered, however construction could be potentially difficult in collapsing sand. Advice should be sought from specialist contractors on the feasibility of forming soil anchors in this area commencing prior to design. An allowable bond stress of 25 and 50 kPa may be respectively adopted for soil anchors formed in the natural medium dense and dense sands. All anchors should be proof loaded to 1.3 times the design-working load. An engineer independent of the contractor should inspect the anchor testing.

If it is not feasible to install anchors due to site restrictions, consideration could be given to design of a top down construction method using the tunnel roof and slab as part of the bracing strut with intermediate raked strutting. This method is considered highly attractive to control ground movement in relation to the nearby sensitive airport structures.

Alternatively, the support wall could be designed as a free cantilever. It is recommended that the temporary retaining walls at the site should be designed for a coefficient of active earth pressure (K_a) of 0.3 and a bulk unit weight of 19 kN/m^3. It is assumed that permanent lateral restraint would be provided by the tunnel roof for the permanent retaining walls. Therefore, an 'at rest' earth pressure coefficient (K_o) of 0.55 may be adopted for the soil profile for the free cantilever wall design.

It is recommended that a rectangular earth pressure distribution using a value of 6H(kPa) where H is the height of excavation in metres should be used for design of the retaining walls that would be restrained from movement by structural elements (such as anchors, bracing and/or the building floors).

All applicable surcharge loadings (for example, adjacent structures, compaction stresses, construction traffic and hydrostatic pressures) should be taken into account in the retaining wall design.

It is recommended that an allowable coefficient of Passive Pressure, Kp, of 2.8 and 3.3 (based on NAVFAC 1982 and Caquot and Kerisel 1948) for medium dense and dense sand respectively is adopted for determination of passive restraint for walls embedded below excavation level. These values have been determined by applying a reduction factor to the ultimate value of Kp, to take into account strain incompatibility between active and passive pressure conditions. A bulk unit weight of 19 kN/m^3 should be adopted for the sand profile. No allowance for passive restraint should be made for soil above the base level of these isolated excavations in the base of the tunnels where footing or service trench excavations exists adjacent to the wall.

It is understood that subgrade reaction modulus (k) values are required for input into the structural model for flexible wall design. Empirical correlation with SPT "N" gives the indicative values of k, set out in Table 7.2. It should be noted that the subgrade reaction modulus (k) is related to the loading conditions and the movement of the wall and the indicative k values are applicable for uniform loading conditions and not for concentrated loading.

Table 7.2: Indicative Subgrade Reaction Modulus Correlation

SPT "N" (blows/300 mm)	Density Conditions	Indicative subgrade Reaction Modulus (k) (kPa/mm)
5 –15	Loose	20 to 50
15-30	Medium dense	20 to 70
30-50	Dense	70 to 120

Note: The correlation is based on the Figures 5.2 and 5.4 of *A Guide to the Structural Design of Road Pavements*, (AUSTROADS 1992).

Assessment of the possible ground movement caused by retaining wall installation, lateral movement of the retaining walls during excavation and groundwater drawdown would be required to meet the appropriate criteria by the airport authorities for the retaining wall design.

Foundation

The road and rail tunnels would be constructed within the saturated alluvial sand. Due to the saturated sand the tunnels would be subject to large buoyancy forces (uplift). If a pile foundation is to be used, it is probable that friction piles or barrettes (using the diaphragm wall technique) deriving their capacity from the medium dense and dense sand would be adopted because of the unknown depth of the rock.

The allowable bearing capacity of 600 kPa could be adopted for concept design for tunnel footings founded on medium dense to dense sand. This bearing pressure would limit the elastic settlement to within 25 mm. Long-term settlement performance would need to be further assessed prior to the detailed design.

7.2.2 Tweed Heads Bypass Interchange

It is anticipated that the footings of the interchange would have to be founded on medium dense to dense sand below RL-3.5 m based on the limited data from borehole BH-11 and test pit TP-6. The design parameters shown in Table 7.3 would be adopted for this interchange.

To provide an adequate bearing stratum and resistance to lateral and uplift forces, a piled foundation is required to transfer the loads. Piles should be founded and embedded into medium dense to dense sand.

In this sandy alluvial area, continuous auger piles, (CFA), screw piles, Franki expanded base piles, G-piles, or non displacement steel H-piles are feasible. However, if noise and vibration control is governing, CFA, screw piles and G-piles may be preferred. The allowable pile capacity for each type is related to the installation method and ground conditions. Recommended design parameters for CFA piles founding in the sand profile are shown in Table 7.3. The allowable end bearing pressures for medium sand and dense sand are devised by using a Nq of 60 and 100 for CFA pile (bored pile) and with a factor of safety of three against the ultimate loading capacity.

Table 7.3: Recommended Design Parameters for CFA Piles

Design Parameters	Medium Dense Sand	Dense/Very Dense Sand
Limiting Depth Ratio (pile length/diameter)	8	15
Limiting depth (m)	4.8	9
Bulk Density (kN/m^3)	18	19
Allowable average skin friction above limiting depth (kPa)	10	20
Allowable skin friction below limiting depth (kPa)	20	45
Allowable end bearing pressure above limiting depth (kPa) with minimum embedded length shown in brackets	750 (4 m)	2,000 (6 m)
Allowable end bearing pressure below limiting depth (kPa)	1,000	3,000

Note: Values are based on a 600 mm-diameter cast-in situ CFA pile.

The preliminary geotechnical investigation (Main Roads 1999a) indicated that soft clay was encountered at the existing Kennedy Drive located approximately 1,500 m south of the proposed Tweed Heads Bypass interchange. It is therefore possible that soft clay may be present in the vicinity of the proposed interchange and further geotechnical investigation should be undertaken prior to the final design of the interchange footing system.

7.3 Fill Embankments

Embankments with a total length of 2.8 km would be constructed along the proposed bypass alignment. The maximum fill height is approximately 7 m occurring around chainages 1,100 to 1,320 in the northern section and up to 12 m at chainages 2,500 to 2,800 towards Boyd Street. The average fill embankment height is between 3 and 4 m.

7.3.1 Geotechnical Model

The embankments are expected to be constructed from compacted fill obtained from excavations in the northern area, excavated materials from the cut and cover road tunnel in the southern section or imported from other borrow areas in the region. The excavated material from the tunnel may comprise acid sulphate soils which may require treatment prior to placement as embankment fill. The treatment of acid sulphate soils is discussed in detail in Section 7.5 of this technical paper and in Technical Paper Number 5.

It is anticipated that the fill would comprise sand, clay and weathered argillite/greywacke. The fill embankment is expected to be founded on extremely weathered rocks in the northern section (chainage 0 to approximately chainage 2,500), and predominantly alluvial sands and silty clays overlying extremely weathered rocks at depth in the southern section.

At chainage 3,200, the proposed embankment would be located over the Tugun Landfill site. Special treatment in terms of material replacement or reinforcement of the exposed landfill material may be necessary. The methods of treatment could only be assessed after the extent and condition of the material is determined.

7.3.2 Southern Section

Fill Slope Stability

The alluvial subgrade generally comprises sand and occasionally silty clay. The alluvium in the vicinity of the southern section tends to be sandy as revealed in most of the boreholes. A significantly thick alluvial clay layer (2 m) was encountered at depths of 20.5 m and 25.5 m in BH-5.

Slope stability has been undertaken for the fill embankment (maximum height of 12 m) by adopting the subsurface profile at BH-5. The geotechnical parameters set out in Table 7.4 have been adopted.

Table 7.4: Strength Parameters Used for Slope Stability Analyses for Fill Embankments – Southern Section

Material Description	Undrained Cohesion, c_u (kPa)	Drained Cohesion,c' (kPa)	Angle of Internal Friction,ϕ' (Degrees)	Unit Weight (kN/m^3)
Embankment Fill	100	5	32	19
Alluvium – Sand (medium Dense – Dense)	C	0	30 - 35	18
Alluvial – Clay (stiff)	50	C	C	18
Extremely Weathered to Distinctly Weathered argillite/greywacke	200	10	40	19

The outputs of the slope stability analysis using SLOPE/W are contained in Appendix B. The factor of safety for significant deep-seated shear failure is estimated to be 1.21 for short-term with seismic load and 1.39 for the long-term condition.

On the basis of the slope stability analyses undertaken for the embankment, permanent batter slopes of two horizontal to one vertical are recommended. For the fill embankment between chainages 2,380 and 2,680, where the embankment height is up to 12 m, intermediate benches should be considered. The purpose of these benches is for ease of maintenance, provision of drainage and debris collection. A bench width of 5 m is proposed to allow free movement of maintenance vehicles or machinery. The bare fill material would tend to lose strength on exposure and become eroded. It is recommended that the batters are vegetated as soon as practicable after construction to mitigate any erosion potential.

All dirty run-off water from above the embankment would be directed to a sediment control pond. An open drain would be provided behind the crest of the fill slope connecting with a concrete lined batter drain to minimise the flow of water over the face of the embankment. Energy dissipation structures may also be required on the toe of the slope. Concentrated flows over the embankment crests should be avoided. The crests should be chamfered to assist in the minimisation of scouring.

Embankment Settlement

Elastic moduli were selected for the foundation materials on the basis of the results of the fieldwork and laboratory testing. Table 7.5 indicates the parameters adopted for determination of the initial elastic compression.

Table 7.5: Parameters Used in the Assessment of Embankment Settlement

Material Description	Elastic Modulus, E_u (MPa)	Poisson's Ratio, ν
Alluvial – Sand (medium to dense sand)	18 (2 N)[1]	0.35
Alluvial Clay (Stiff)	8 (100 Cu)[2]	0.5
Extremely Weathered Argillite/greywacke	60 (1 N)[1]	0.35

Notes 1: N is SPT value
2: Cu is undrained shear strength

Based on a fill embankment height of 12 m proposed at BH-5 where clay was encountered, the estimated elastic compression determined from the analyses was in the order of 250 mm at the existing ground surface. However, for a predominantly sandy profile along the southern section, the estimated compression would occur as the embankment is being constructed. At maximum embankment height, the compression would have been compensated for by the placement of additional fill.

A significant layer of alluvial firm to stiff clay layer was encountered in borehole BH-5 (20.5 to 22.5 m and 25.5 to at least 30 m). Preliminary estimates of long-term settlements were determined for the fill embankment in this area. These were based on consolidation data derived from empirical correlation with clay strength and moisture content. Long-term consolidation settlements in addition to elastic compression, were calculated to be in the order of 280 mm under a fill embankment height of 12 m. It is estimated that it would take approximately 1.5 years to achieve 90 percent consolidation based on a C_v value of 6 m^2/year. To improve the time of consolidation, ground treatment by surcharge may be required. However, further testing on the underlaying clay for its consolidation parameters is required to reassess the need of such ground treatment.

7.3.3 Northern Section

Fill Slope Stability

As the subgrade below the fill embankment in this section would mainly be founded on extremely weathered rocks except at chainage 1,100 to chainage 1,320, embankment stability is not considered a major issue. The subgrade would be prepared in compliance with the procedure discussed in Section 7.6.1. The exposed rock would be scarified and subgrade drainage may be required prior to filling.

Fill and alluvial stiff to very stiff clay (1.5 to 5 m) was encountered in BH-1a. Slope stability has been undertaken for a fill embankment (maximum height of 7 m) by adopting the subsurface profile at BH-1a. The geotechnical parameters adopted are set out in Table 7.6.

Table 7.6: Strength Parameters Used for Slope Stability Analyses for Fill Embankments – Northern Section

Material Description	Undrained Cohesion (kPa)	Drained Cohesion (kPa)	Angle of Internal Friction (Degrees)	Unit Weight (kN/m^3)
Embankment Fill	100	5	32	19
Alluvium – Sand (medium Dense – Dense)		0	30 – 35	18
Alluvial – Clay (stiff- very stiff)	50 - 75	C	C	18
Extremely Weathered to Distinctly Weathered argillite/greywacke	200	10	40	19

The outputs of the slope stability analysis using SLOPE/W are contained in Appendix B. The factor of safety for significant deep-seated shear failure is estimated to be 1.67 for short-term with seismic load and 1.93 for long-term condition.

Recommendations for the fill embankment are similar to those for the southern section. Permanent batter slopes of (2:1) two horizontal to one vertical are recommended.

If the fill batter is higher than 7 m, intermediate benches should be considered to ease maintenance, provide drainage and allow the collection of debris to be undertaken. The drains on the benches would be graded longitudinally at grades of between 0.5 and 2 percent. Should longitudinal falls greater than 2 percent be required, the drains should be concrete lined to prevent scouring and erosion. The bench width of 5 m is proposed to allow free movement of maintenance vehicles or machinery.

It is also recommend that the batters are vegetated as soon as practicable after excavation to mitigate any erosion potential. Run-off from the pavements would be directed to sediment control ponds via chutes or energy dissipaters to avoid batter erosion. Concentrated flows over the embankment crests should be avoided. The crests should be chamfered to assist in the minimisation of scouring.

Embankment Settlement

The estimated elastic compression determined from the analyses is in the order of 80 mm at the existing ground surface based on a maximum fill embankment height of 7 m. This would occur as the embankment is being constructed. At maximum embankment height, the compression would have been compensated for by the placement of additional fill.

Preliminary estimates of long-term settlements were calculated to be in the order of 120 mm under a fill embankment height of 7 m. It is estimated that it would take approximately a year to achieve 90 percent consolidation based on a correlated C_v value of 18 m^2/year. Ground treatment by surcharge or by installation of wick drains may be required to reduce the time of consolidation. However, further testing of the consolidation parameters of the underlaying clay is required prior to detailed design to confirm the need for such ground treatments.

7.4 Soil Erosion Potential

The predominant soil under the alignment in the northern section is extremely weathered greywacke/argillite rock. Available field sampling of these materials indicated an intact rock structure and testing of the dispersiveness of this rock sample is precluded. However, observations during recent site visits indicated the erosion gullies have formed on the extremely weathered rock when it is excavated. These observations provide a good indication that the weathered rock is highly erodable under concentrated stormwater flow.

Medium to fine alluvial sand covers most of the southern section. Alluvial clay was only encountered in BH-5 at a depth of 20 m below ground. It is possible that this alluvial clay may occur at a shallow depth and within the excavation level. An Emerson Crumb Dispersion test was then carried out on the alluvial clay soils and indicated an Emerson Number of 5, indicating that the remoulded soil would not disperse in water and a one to five soil/water suspension would remain dispersed after five minutes.

The erodibility potential of sand soils has to be considered in terms of the grain size of the sand, land form and the discharge flow velocity on to the sand. In our experience, medium to fine sand is highly erodable under concentrated flow.

The erosion potential of sand is related to the flow velocity. For example, in designing waterways on bare soil, the maximum design velocity must be limited to 0.3 to 0.7 m for soil of high to low erodibility respectively (NSW Department of Housing 1998). If these velocities cannot be limited, some surface protection or even energy dissipation structures have to be devised to minimise the erosion.

7.5 Handling and Re-use of Excavated Acid Sulphate Soils

The potential acid sulphate soils occur mainly along the southern section of the proposed corridor. This has been confirmed by laboratory POCAS testing and the additional acid sulphate soil work carried out during 2002–2003. This has indicated that the construction works for the proposed bypass would encounter acid sulphate soils, particularly during excavation along the southern alignment and at the proposed cut and cover road and rail tunnel section. There is a range of procedures for the treatment of acid sulphate soils disturbed during construction activities. General procedures include:

- avoiding areas where acid sulphate soils have been identified;
- preventing oxidisation by controlling the water table, constructing a capping layer and/or burying the acid sulphate soil below the water table;
- neutralising the acid produced by oxidation via the addition of alkaline agents such as agricultural lime;
- collecting and treating the leachate following deliberate oxidisation of the acid sulphate soils; and
- removing and disposing of the acid sulphate soils in an appropriate landfill.

Adoption of one, or a combination of the above strategies, would allow the affected soils to be re-used as general fill material on the site.

Acid neutralisation is considered to be the most appropriate treatment for the excavated acid sulphate soil if it is to be re-used as a general fill for the road embankment. If this method is adopted a suitably sized and located area should be assigned for the treatment of excavated acid sulphate soils prior to the commencement of site activities. The assigned area should have an impermeable base and bunds to contain soil and associated leachate, with run-off directed to lined collection ponds. It is recommended that the treatment of the acid sulphate soil is undertaken in stages for effective neutralisation management. The treatment area should therefore be divided into at least two separately bunded areas, one for liming and mixing soils and a second for containment and monitoring.

Acid sulphate soil material excavated from the site should be placed in the treatment area within one day of disturbance. Neutralisation of the acid sulphate soils may be most appropriately carried out by the addition of granulated or powdered agricultural lime to the stockpiles excavated from the proposed site. Based on the laboratory test results from the recent site investigation (TP-A of up to 141 and Spos up to 0.89 percent) and the reference to ASSMAC Management Guidelines (Stone *et al*. 1998), it is suggested that an application rate of approximately 1 percent lime by weight of soil is required to be initially applied for neutralisation.

It is also essential that mixing of the lime with the acid sulphate soil is thorough and implemented under good quality control. Most efficient mixing would be achieved with specialist equipment such as a pug mill. If a pug mill is not available, the soil should be spread out in a maximum 300 mm thick layer and covered with the required amount of lime. Soils should be dried out to allow trafficking and mixing with a rotary hoe or equivalent. Thorough mixing and aeration is essential and trials should be conducted to ensure effective treatment.

This should be immediately followed by pH field testing with additional TAA, TP-A and POCAS laboratory testing after several days of curing. Dependent upon the results of this testing, additional remedial lime dosage may be required. Treated and tested soils should be placed in containment areas prior to use as general backfill on the site.

The acid sulphate soil management plan should address the following issues:

- a description of the site conditions prior to construction, including a plan showing the location and classification of acid sulphate soils;
- a schedule of the construction activities which involves the excavation or disturbance of acid sulphate soils;
- a description of the measures or procedures to be undertaken in areas of acid sulphate soils which, when implemented, would prevent, control or minimise the escape of acid leachate into the surrounding environment;
- a focussed program monitoring surface water and groundwater quality, and soil acidity and salinity;
- a description of the contingency procedures to be implemented in the case of failure of management procedures; and
- a record of consultation with the coordinating organisations and relevant government agencies.

An acid sulphate soils management strategy is outlined in Technical Paper Number5. Work done during the 2003 investigation suggests the southern section of the site falls into the 'very high' level of treatment category according to the Queensland State Planning Policy (Planning and Management Development Involving Acid Sulphate Soils).

7.6 General Earthworks

7.6.1 Foundation Preparation

Prior to placement of any embankment materials, the area for the formation including cut and fill batters would require clearing of all vegetation. All trees and stumps unable to be removed by clearing operations would need to be removed by grubbing. Any low lying areas with ponded water would require the construction of bunding and the water removed prior to placement of fill. Following dewatering of ponds, soft and saturated materials would be removed from the inundated area.

Soft soils occurring in low lying areas near existing watercourses or ponds within the southern section, may require placement of a bridging layer and geofabrics in order to improve trafficability. Due to the presence of acid sulphate soils in some of these areas, disturbance of these soils should be minimised. Any disturbance in these areas would need to be undertaken in accordance with the acid sulphate soils management strategy.

In areas unaffected by acid sulphate soils, unsuitable materials such as topsoil and other soft compressible materials should be removed prior to placement of embankment materials. Alternatively, a bridging/drainage layer may be placed on top of the alluvium to provide trafficability and a working platform for construction. The bridging layer shall consist of free-draining granular material, which should be end-dumped and spread in a single layer and in sufficient depth to allow the passage of earthmoving equipment with minimal surface heaving. Suitably designed geotextiles would need to be laid prior to construction of the layer. Materials replacing unsuitable soils should be placed and compacted to achieve a characteristic relative compaction in excess of 98 percent of standard maximum dry density (SMDD) at a moisture content within the range of 0 to + 2 percent of the optimum moisture content (OMC).

The floors of cuttings would be ripped or loosened to a depth of 300 mm and recompacted to achieve a characteristic relative compaction in excess of 98 percent of SMDD at moisture content within the range of 0 to + 2 percent of OMC.

The foundations for embankments should be ripped or loosened to a depth of 200 mm compacted to achieve a characteristic relative compaction in excess of 98 percent of SMDD at a moisture content within the range of 0 to + 2 percent of OMC.

7.6.2 Placement of Fill Materials

General fill in embankments beneath the select pavement material zone would be placed in layers to achieve a compacted layer thickness not exceeding 300 mm. The fill would be compacted to achieve a characteristic relative compaction in excess of 98 percent of SMDD at a moisture content within the range of 0 to + 2 percent of OMC.

7.6.3 Compaction Control

Compaction control would comprise full time quality testing by a NATA registered laboratory. It is expected that Main Road's Quality System requirements would be applied to this project.

7.6.4 Subgrade California Bearing Ratio

It is envisaged that the subgrade of the road would comprise general fill for the embankments and the exposed weathered rock. Soaked California Bearing Ratio tests undertaken on the sand and weathered rock indicate values of between 9 and 22 percent.

8. Further Investigation and Geotechnical Advice

The geotechnical work undertaken to date has been limited to those areas to which access was reasonable with minimum clearing and/or construction of access tracks. Work undertaken in 2000 was further extended for the supplementary geotechnical investigation in 2003. The work undertaken prior to publication of the environmental impact assessment has been sufficient to allow development of a geotechnical model and identification of geotechnical issues. These issues have been taken into account during development of the concept design.

Following the receipt of approvals for the proposal, further geotechnical investigations and laboratory testing would be required prior to the detailed design. These are categorised as follows:

- investigation of the areas where access is difficult, in particular low lying and waterlogged areas;
- investigation for the possible presence of soft compressible soil at the southern end of the bypass alignment near Kennedy Drive and at the deep fill area between chainages 1,100 to 1,380;
- at the proposed bridge piers, ramps and roundabout locations at proposed interchanges and bridges locations including the area nominated as 'Hidden Valley';
- at deep rock cut at the ridge behind the John Flynn Hospital and Medical Centre;
- along the cut and cover road and rail tunnels;
- along the rail tunnel; and
- further testing on clay soils, subgrade materials and rock strength, (consolidation tests, CBR, density and moisture relationship, index testings, and unconfined compressive strength of the weathered rocks).

It is possible that the subsurface soil and rock conditions encountered during construction may vary from those identified in this report. It is also noted that groundwater conditions are transient and may vary, particularly due to variations in weather. The limitations of the geotechnical investigation are outlined in Appendix A.

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Appendix A

Limitations of Geotechnical Site Investigations

Appendix A – Limitations of Geotechnical Site Investigations

Scope of Services

This geotechnical site assessment report ('the report') has been prepared in accordance with the scope of services set out in the contract, or as otherwise agreed, between the Client and PB (formerly PPK) ('scope of services'). In some circumstances the scope of services may have been limited by a range of factors such as time, budget, access and/or site disturbance constraints.

Reliance on Data

In preparing the report, PB has relied upon data, surveys, analyses, designs, plans and other information provided by the Client and other individuals and organisations, most of which are referred to in the report ('the data'). Except as otherwise stated in the report, PB has not verified the accuracy or completeness of the data. To the extent that the statements, opinions, facts, information, conclusions and/or recommendations in the report ('conclusions') are based in whole or part on the data, those conclusions are contingent upon the accuracy and completeness of the data. PB will not be liable in relation to incorrect conclusions should any data, information or condition be incorrect or have been concealed, withheld, misrepresented or otherwise not fully disclosed to PB.

Geotechnical Investigation

Geotechnical engineering is based extensively on judgment and opinion. It is far less exact than other engineering disciplines. Geotechnical engineering reports are prepared to meet the specific needs of individuals. A report prepared for a consulting civil engineer may not be adequate for a construction contractor or even some other consulting civil engineer. This report was prepared expressly for the Client and expressly for purposes indicated by the Client or his representative. Use by any other persons for any purpose, or by the Client for a different purpose, might result in problems. The Client should not use this report for other than its intended purpose without seeking additional geotechnical advice.

This Geotechnical Report is Based on Project-specific Factors

This geotechnical engineering report is based on a subsurface investigation which was designed for project-specification factors, including the nature of any development, its size and configuration, the location of any development on the site and its orientation, and the location of access roads and parking areas. Unless further geotechnical advice is obtained this geotechnical engineering report cannot be used:

> when the nature of any proposed development is changed; or
>
> when the size, configuration location or orientation of any proposed development is modified.

This geotechnical engineering report cannot be applied to an adjacent site.

The Limitations of Site Investigation

In making an assessment of a site from a limited number of boreholes or test pits there is the possibility that variations may occur between test locations. Site exploration identifies specific subsurface conditions only at those points from which samples have been taken. The risk that variations will not be detected can be reduced by increasing the frequency of test locations;

however this often does not result in any overall cost savings for the project. The investigation programme undertaken is a professional estimate of the scope of investigation required to provide a general profile of the subsurface conditions. The data derived from the site investigation programme and subsequent laboratory testing are extrapolated across the site to form an inferred geological model and an engineering opinion is rendered about overall subsurface conditions and their likely behaviour with regard to the proposed development. Despite investigation the actual conditions at the site might differ from those inferred to exist, since no subsurface exploration programme, no matter how comprehensive, can reveal all subsurface details and anomalies.

The borehole logs are the subjective interpretation of subsurface conditions at a particular location, made by trained personnel. The interpretation may be limited by the method of investigation, and can not always be definitive. For example, inspection of an excavation or test pit allows a greater area of the subsurface profile to be inspected than borehole investigation, however, such methods are limited by depth and site disturbance restrictions. In borehole investigation, the actual interface between materials may be more gradual or abrupt than a report indicates.

Subsurface Conditions are Time Dependent

Subsurface conditions may be modified by changing natural forces or man-made influences. A geotechnical engineering report is based on conditions which existed at the time of subsurface exploration.

Construction operations at or adjacent to the site, and natural events such as floods, or groundwater fluctuations, may also affect subsurface conditions, and thus the continuing adequacy of a geotechnical report. The geotechnical engineer should be kept appraised of any such events, and should be consulted to determine if additional tests are necessary.

Avoid Misinterpretation

A geotechnical engineer should be retained to work with other appropriate design professionals explaining relevant geotechnical findings and in reviewing the adequacy of their plans and specifications relative to geotechnical issues.

Bore/Profile Logs Should Not Be Separated from the Engineering Report

Final bore/profile logs are developed by geotechnical engineers based upon their interpretation of field logs and laboratory evaluation of field samples. Customarily, only the final bore/profile logs are included in geotechnical engineering reports. These logs should not under any circumstances be redrawn for inclusion in architectural or other design drawings. To minimise the likelihood of bore/profile log misinterpretation, contractors should be given access to the complete geotechnical engineering report prepared or authorised for their use. Providing the best available information to contractors helps prevent costly construction problems. For further information on this matter reference should be made to *Guidelines for the Provision of Geotechnical Information in Construction Contracts* published by the Institution of Engineers Australia, National Headquarters. Canberra 1987.

Geotechnical Involvement During Construction

During construction, excavation is frequently undertaken which exposes the actual subsurface conditions. For this reason geotechnical consultants should be retained through the construction stage, to identify variations if they are exposed and to conduct additional tests which may be required and to deal quickly with geotechnical problems if they arise.

Report for Benefit of Client

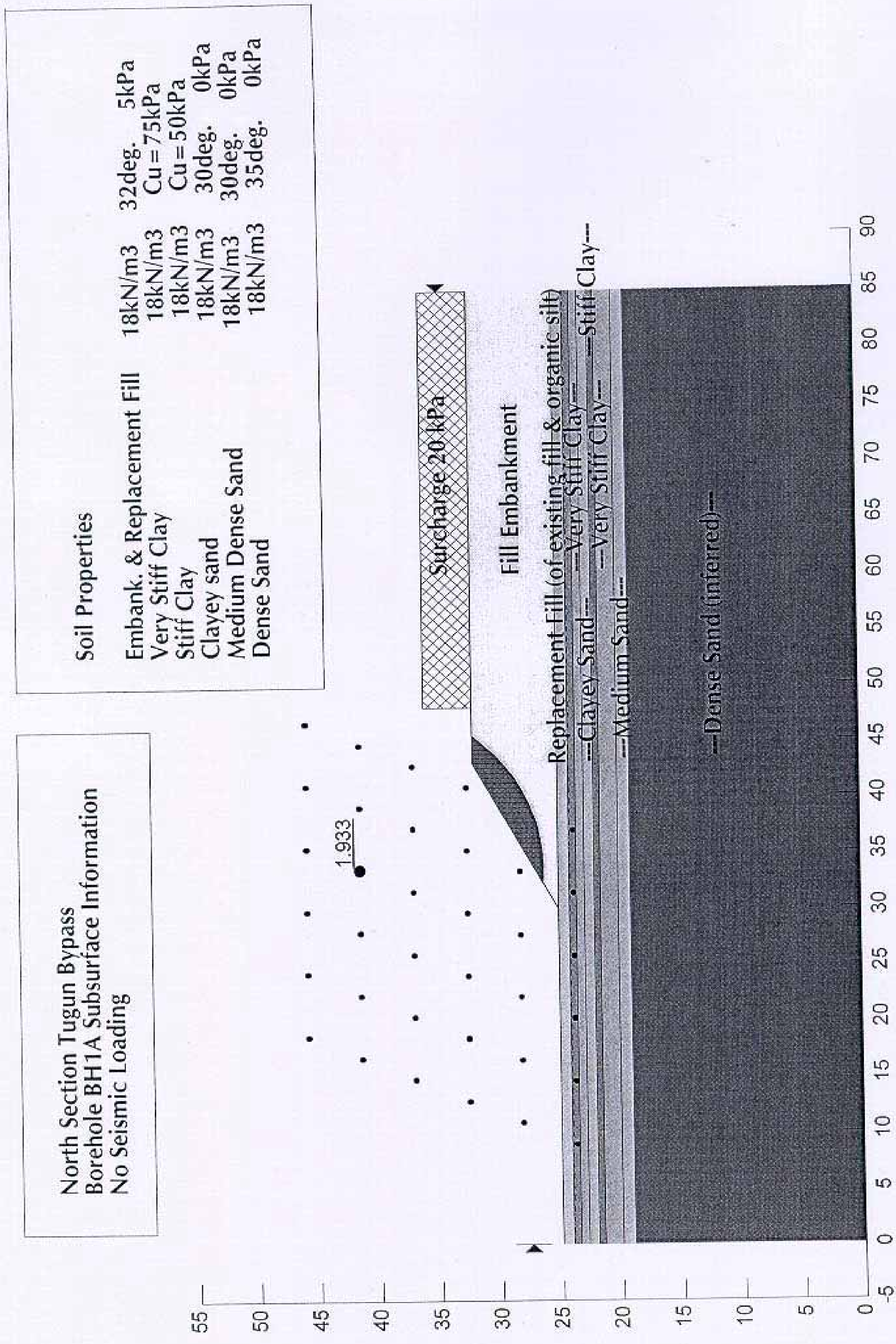
The report has been prepared for the benefit of the Client and no other party. PB assumes no responsibility and will not be liable to any other person or organisation for or in relation to any matter dealt with or conclusions expressed in the report, or for any loss or damage suffered by any other person or organisation arising from matters dealt with or conclusions expressed in the report (including without limitation matters arising from any negligent act or omission of PPK or for any loss or damage suffered by any other party relying upon the matters dealt with or conclusions expressed in the report). Other parties should not rely upon the report or the accuracy or completeness of any conclusions and should make their own enquiries and obtain independent advice in relation to such matters.

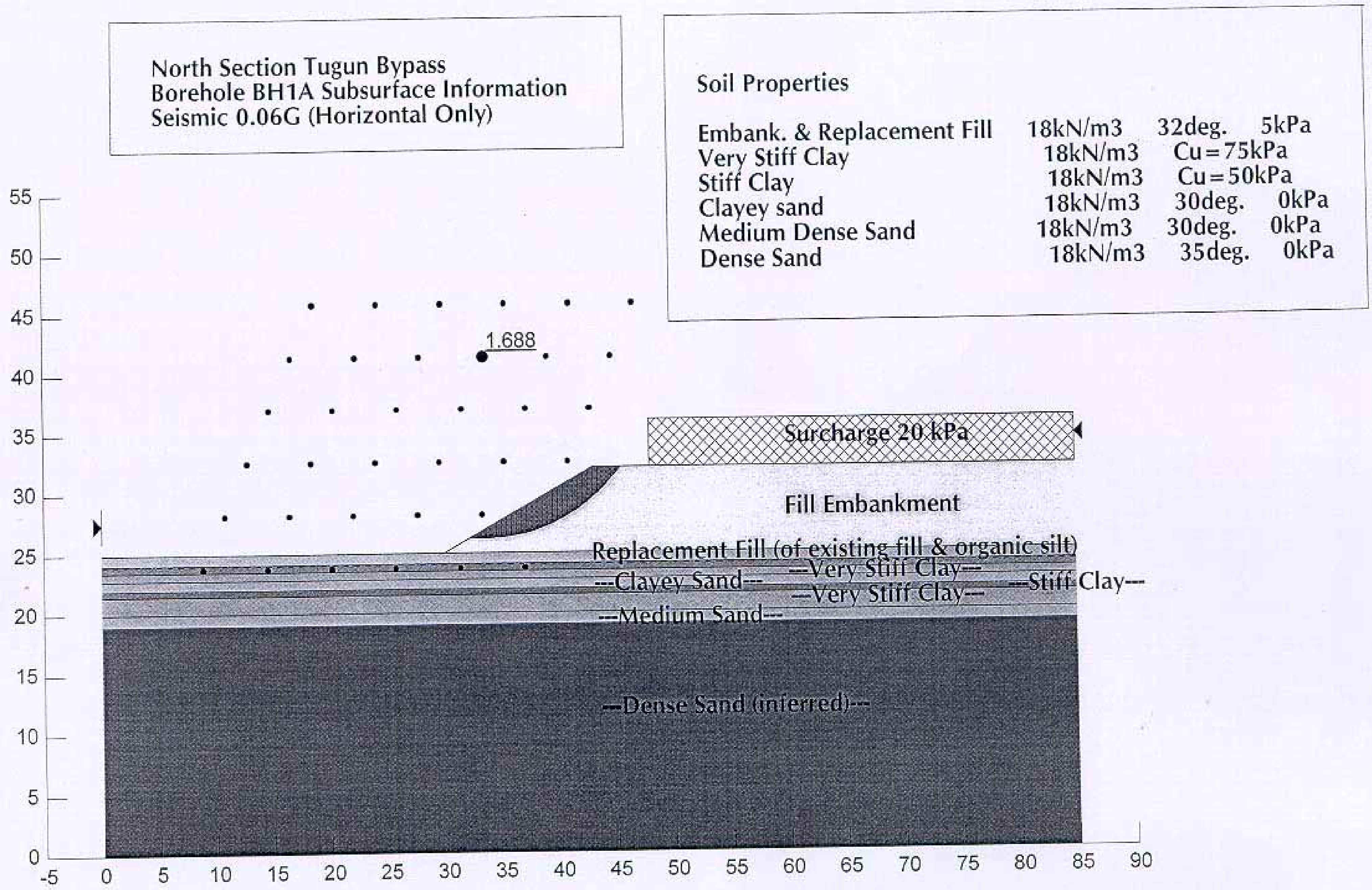
Other Limitations

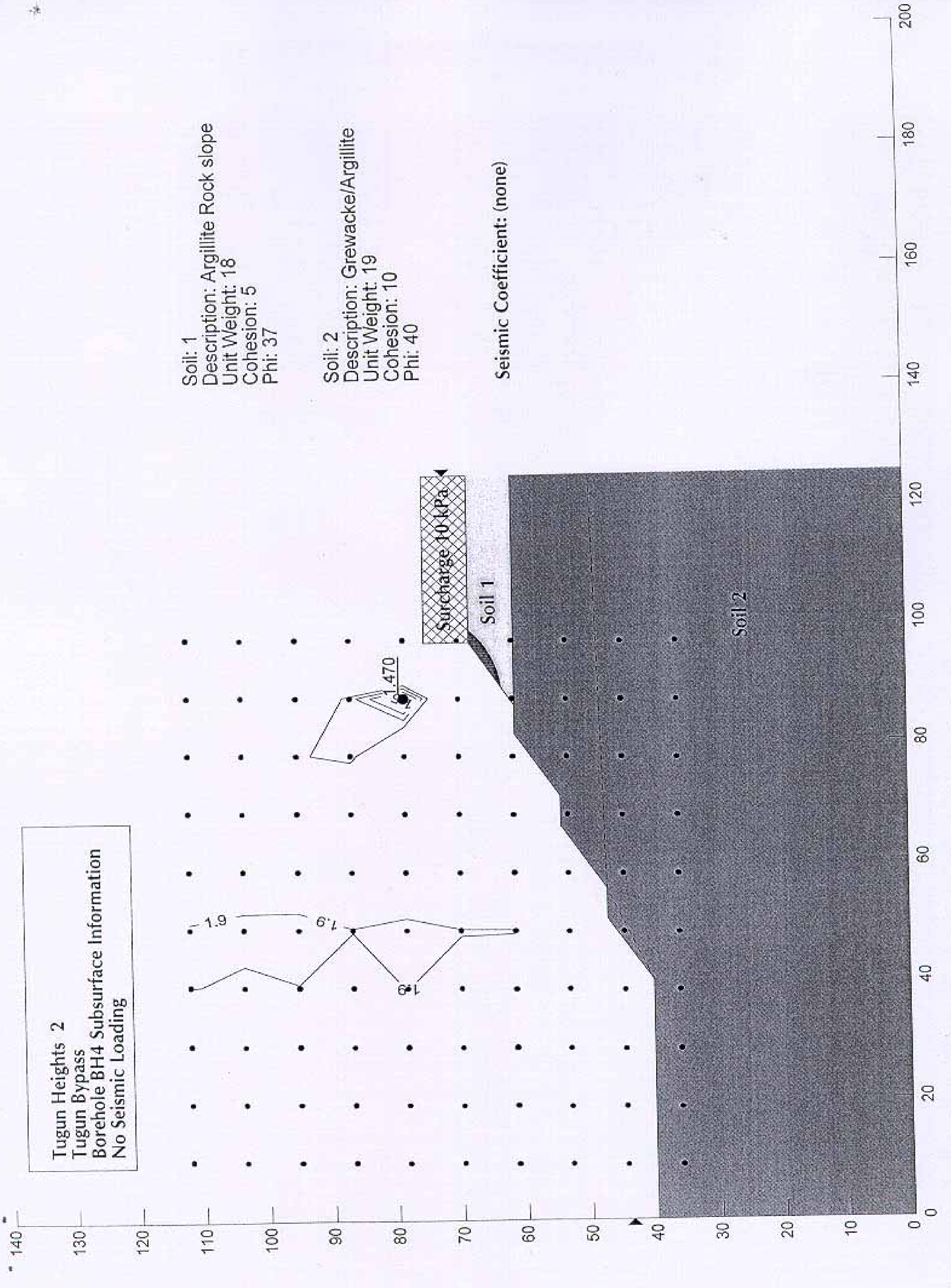
PB will not be liable to update or revise the report to take into account any events or emergent circumstances or facts occurring or becoming apparent after the date of the report.

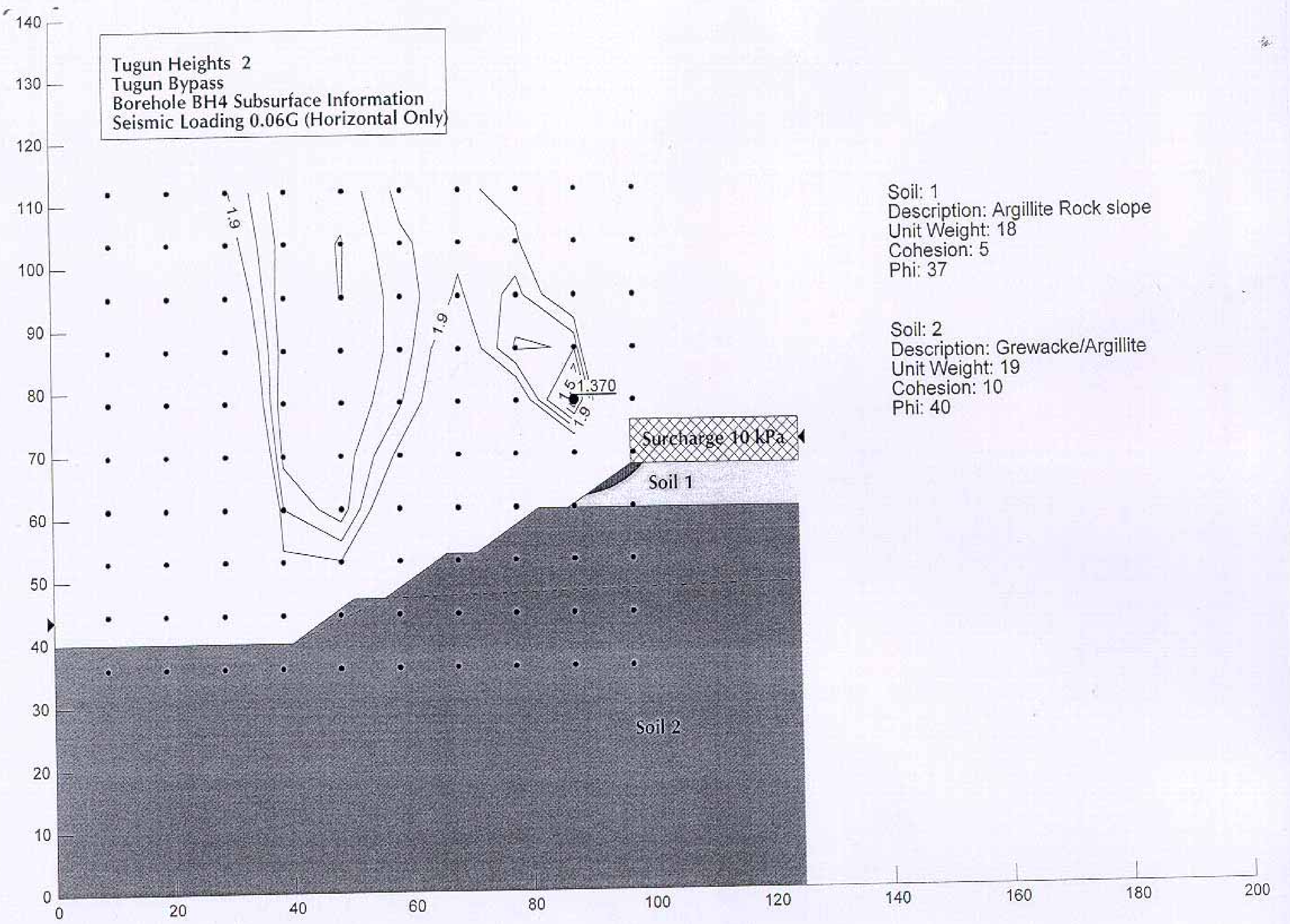
Appendix B

Outputs of Slope/W Stability Analysis


North Section Tugun Bypass
Borehole BH1A Subsurface Information
No Seismic Loading
Soil Properties
Embank. & Replacement Fill 18kN/m3 32deg. 5kPa
Very Stiff Clay 18kN/m3 Cu=75kPa
Stiff Clay 18kN/m3 Cu=50kPa
Clayey sand 18kN/m3 30deg. 0kPa
Medium Dense Sand 18kN/m3 30deg. 0kPa
Dense Sand 18kN/m3 35deg. 0kPa
1.933
Surcharge 20 kPa
Fill Embankment
Replacement Fill (of existing fill & organic silt)
--Very Stiff Clay--
--Clayey Sand--
--Stiff Clay--
--Very Stiff Clay--
--Medium Sand--
--Dense Sand (inferred)--
55
50
45
40
35
30
25
20
15
10
5
0
-5
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90


North Section Tugun Bypass
Borehole BH1A Subsurface Information
Seismic 0.06G (Horizontal Only)
Soil Properties
Embank. & Replacement Fill 18kN/m3 32deg. 5kPa
Very Stiff Clay 18kN/m3 Cu=75kPa
Stiff Clay 18kN/m3 Cu=50kPa
Clayey sand 18kN/m3 30deg. 0kPa
Medium Dense Sand 18kN/m3 30deg. 0kPa
Dense Sand 18kN/m3 35deg. 0kPa
1.688
Surcharge 20 kPa
Fill Embankment
Replacement Fill (of existing fill & organic silt)
---Very Stiff Clay---
---Clayey Sand---
---Stiff Clay---
---Very Stiff Clay---
---Medium Sand---
---Dense Sand (inferred)---
55
50
45
40
35
30
25
20
15
10
5
0
-5
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90


Tugun Heights 2
Tugun Bypass
Borehole BH4 Subsurface Information
No Seismic Loading
Soil: 1
Description: Argillite Rock slope
Unit Weight: 18
Cohesion: 5
Phi: 37
Soil: 2
Description: Grewacke/Argillite
Unit Weight: 19
Cohesion: 10
Phi: 40
Seismic Coefficient: (none)
Surcharge 10 kPa
Soil 1
Soil 2
1.470
1.9
1.9
1.9
0
20
40
60
80
100
120
140
160
180
200
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140


Tugun Heights 2
Tugun Bypass
Borehole BH4 Subsurface Information
Seismic Loading 0.06G (Horizontal Only)
Soil: 1
Description: Argillite Rock slope
Unit Weight: 18
Cohesion: 5
Phi: 37
Soil: 2
Description: Grewacke/Argillite
Unit Weight: 19
Cohesion: 10
Phi: 40
Surcharge 10 kPa
Soil 1
Soil 2
1.370
1.9
1.5
0
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50
60
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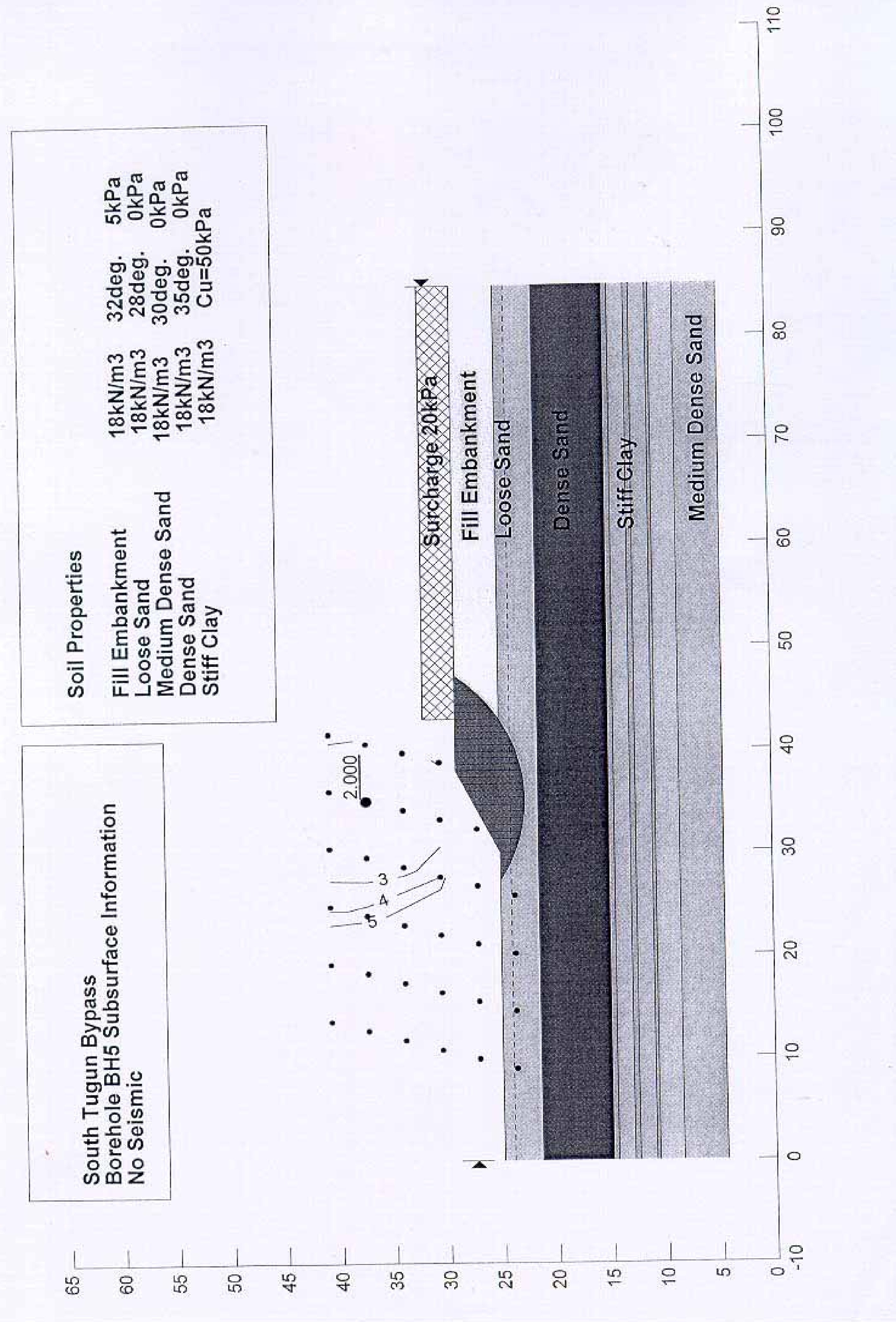
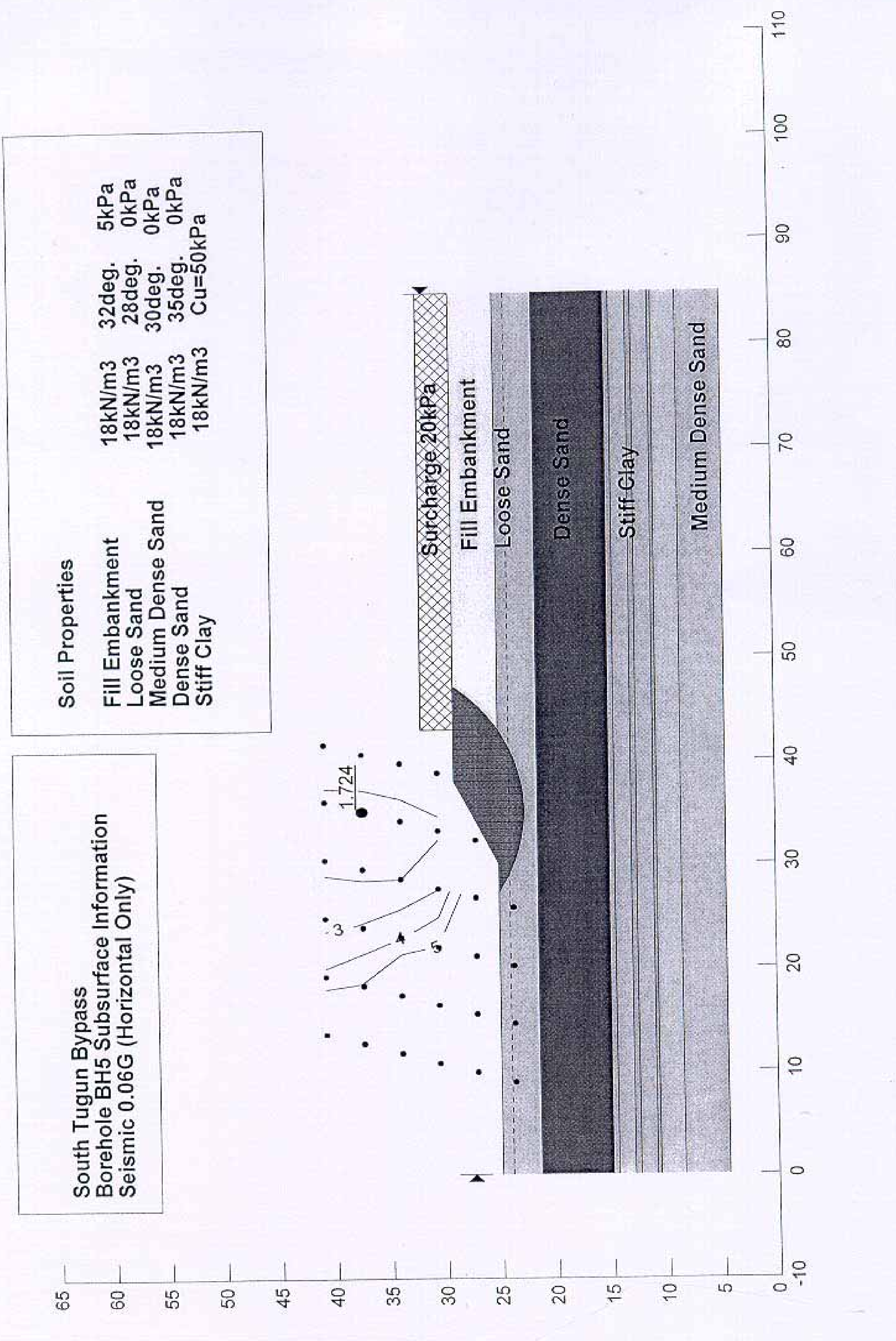
90
100
110
120
130
140
160
180
200


South Tugun Bypass
Borehole BH5 Subsurface Information
No Seismic
Soil Properties
Fill Embankment 18kN/m3 32deg. 5kPa
Loose Sand 18kN/m3 28deg. 0kPa
Medium Dense Sand 18kN/m3 30deg. 0kPa
Dense Sand 18kN/m3 35deg. 0kPa
Stiff Clay 18kN/m3 Cu=50kPa
Surcharge 20kPa
Fill Embankment
Loose Sand
Dense Sand
Stiff Clay
Medium Dense Sand
2.000
3
4
5
0
5
10
15
20
25
30
35
40
45
50
55
60
65
-10
0
10
20
30
40
50
60
70
80
90
100
110


South Tugun Bypass
Borehole BH5 Subsurface Information
Seismic 0.06G (Horizontal Only)
Soil Properties
Fill Embankment 18kN/m3 32deg. 5kPa
Loose Sand 18kN/m3 28deg. 0kPa
Medium Dense Sand 18kN/m3 30deg. 0kPa
Dense Sand 18kN/m3 35deg. 0kPa
Stiff Clay 18kN/m3 Cu=50kPa
1.724
3
4
5
Surcharge 20kPa
Fill Embankment
Loose Sand
Dense Sand
Stiff Clay
Medium Dense Sand
-10
0
10
20
30
40
50
60
70
80
90
100
110
0
5
10
15
20
25
30
35
40
45
50
55
60
65