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GEOTECHNICAL ASSESSMENT OF SLOPE STABILITY, TANTI CREEK, MORNINGTON

For

Mornington Peninsula Shire Council

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GEOTECHNICAL ASSESSMENT OF SLOPE STABILITY, TANTI CREEK, MORNINGTON

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LIST OF ABBREVIATIONS AND UNITS

Technical Terms

1H:2V	Slope Ratio of 1 Horizontal to 2 Vertical
AHD	Australian Height Datum
AADT	Average Annual Daily Traffic
AC	Asphalt Cement
AMG	Australian Map Grid
Base Course	Upper Layer of the pavement
CBR	California Bearing Ratio (%)
CTCR	Cement Treated Crushed Rock
DCP	Dynamic Cone Penetrometer
DTL	Daily Traffic Loading
EC	Electrical Conductivity
ESA	Equivalent Standard Axles
FoS	Factor of Safety
GPS	Global Positioning System
HDPE	High Density Polyethylene
HV	Heavy Vehicles (Usually a %)
LF	Loading Factor
NATA	National Association of Testing Authorities
PMB	Polymer Modified Binder
Prime	Application of a primer to a prepared base
Sub-Base Course	Lower layer of the pavement
Subgrade	Foundation material for the pavement
TDS	Total Dissolved Solids (salinity of water)

GEOTECHNICAL ASSESSMENT OF SLOPE STABILITY, TANTI CREEK, MORNINGTON

1 INTRODUCTION

At the request of Mr. John Annear of the Mornington Peninsula Shire Council (MPSC) on the 8 November 2002, a limited geotechnical investigation was undertaken to determine the stability of the properties between Tanti Creek and Herbert Street in Mornington.

The removal of the native vegetation during the early settlement of the Peninsula for farming has resulted in an increased rate of runoff. This, combined with the increased runoff over recent years from development of the area, has resulted in further pressurisation of the sand aquifer. The development over many years has increased the flow volume and velocity within the creek, resulting in erosion of the creek bed and banks and further exacerbated instability of the surrounding creek banks. The surface movement has resulted in distress to a number of out-buildings and potentially threatens a number of dwellings.

This initial investigation was to assess the stability of the eastern creek banks in the area designated as the erosion management overlay in the Draft Tanti Creek Management Plan (August 2002) extending from Strattons Lane to 85 Herbert Street.

At least four slope failures in the creek banks on the eastern side of the creek were identified during an initial site inspection. This area was known as the 'Designated Detailed Study Area' for this assessment, as indicated on Figure Nos. 4 & 5, Appendix A.

The geotechnical investigation was conducted in accordance with our fee proposal Q2314 dated 19th August 2002. On the 8th November 2002, an email from Gordon Fry of the MPSC extended the inspection of the creek to the length of Tanti Creek from the Nepean Highway to the Esplanade adjacent to Port Phillip Bay and to determine the extent of a proposed Erosion Management Overlay.

The report was issued to Council as 23019-1.6 dated 15 February 2006. A subsequent investigation of Tanti Creek conducted for Alluvium Pty Ltd, *207177Report01.1 Geotechnical Assessment of Creek Banks, Tanti Creek, Mornington* dated 31 October 2007 identified additional areas of concern with regard to slope stability along the banks of Tanti Creek and the nearby vicinity. This required modification to the boundaries for the proposed Erosion Management Overlay from those established in 23019-1.6. The modified boundaries were provided to Council in draft form in an email dated 15th August 2008.

This issue of the report includes the additional observations made as part of the investigation for 207177Report01.1 and provides the finalised boundary locations for the proposed Erosion Management Overlay.

2 SCOPE OF THE INVESTIGATION

The proposed investigation is to determine the following:

1. Establish the presence and extent of slope failures along the area of interest
2. Map the occurrence of these slope failures and other relevant features along the creek banks between the Nepean Highway and the Esplanade
3. Determine the geotechnical design parameters for the modelling of the slope failures in a particular area of interest between 57 and 85 Herbert Street

4. Back – analyse the existing slope failures along the creek banks, in the area south of Strattons Lane to 85 Herbert Street
5. Assess the extent of any Erosion Management Overlay
6. Suggest limited rectification works that could be done by the property owners
7. Limited modelling of the proposed rectification works

The geotechnical assessment was extended to the following:

- Plotting of the geotechnical features along the creek banks
- Examination of the aerial photographs over the last 50 years
- Geotechnical examination of the sub - surface ground along the creek banks including water levels
- Drilling of boreholes with associated sampling, insitu testing and installation of piezometers
- Geotechnical laboratory testing
- Analysis of slope stability of the creek banks
- Computer modelling of the proposed rectification works that could be completed by the home owners

3 LIMITATIONS OF THE REPORT

The geotechnical assessment of the creek area is limited to an overall assessment of the site area and is not a detailed slope stability assessment along the creek banks.

No detailed assessment has been undertaken along the creek banks apart from the area between 57 and 85 Herbert Street. The assessment in other areas is based on a brief visual assessment and aerial photographic examination and a detailed geotechnical investigation may reveal that the affected areas are either greater or smaller than revealed by the inspection.

The limitation of the geotechnical report is included in Appendix H.

4 SITE DESCRIPTION

Tanti Creek flows in a north-westerly direction from south of the Mornington Tourist Railway near Watt Street until discharging into Port Phillip Bay in the vicinity of Mills Beach. The section of the creek covered by the current study area is the length of the creek between the Nepean Highway and The Esplanade and is about 1.5km, as indicated in Figure 4-1 overleaf.

At the southern end of the study area, Tanti Creek is confined to a narrow and relatively shallow man-made gully in the order of 2 to 3m deep. The creek initially runs along at a relatively flat grade adjacent to Tanti Avenue and then between the properties on the east side of Tanti Avenue and west side of Herbert Street. There are scattered trees along the banks and the banks of the creek are readily accessible.

In the area of the creek between 75 Herbert Street and 107 Tanti Avenue, the creek drops by several metres. From 75 Herbert Street, the gully increases in width and deepens to the order of 4 to 5m. The vegetation along the creek in this area becomes significantly thicker and the creek banks become almost inaccessible. The grade of the creek in this area becomes steeper and especially in the vicinity of Strattons Lane where there are iron oxide cemented clays exposed in the base of the creek.

Downstream of Strattons Lane, the vegetation becomes less dense and the gully continues to widen and deepen. The gully reaches its deepest to the rear of the Morven Manor Retirement Village between Strattons Lane and Barkly Street.

The gully extends past Barkly Street at approximately 5 to 6m depth to the area in the vicinity of 17 Shelbourne Court, where the base of the gully broadens out to a wider valley. The creek is incised approximately 2-3m into the valley while the sides of the valley are approximately 5-6m high. The creek then flows along the western side of the valley with the eastern side some 30-50m distant. In this area the grade of the creek begins to flatten out as well. The creek continues in this form until The Esplanade.

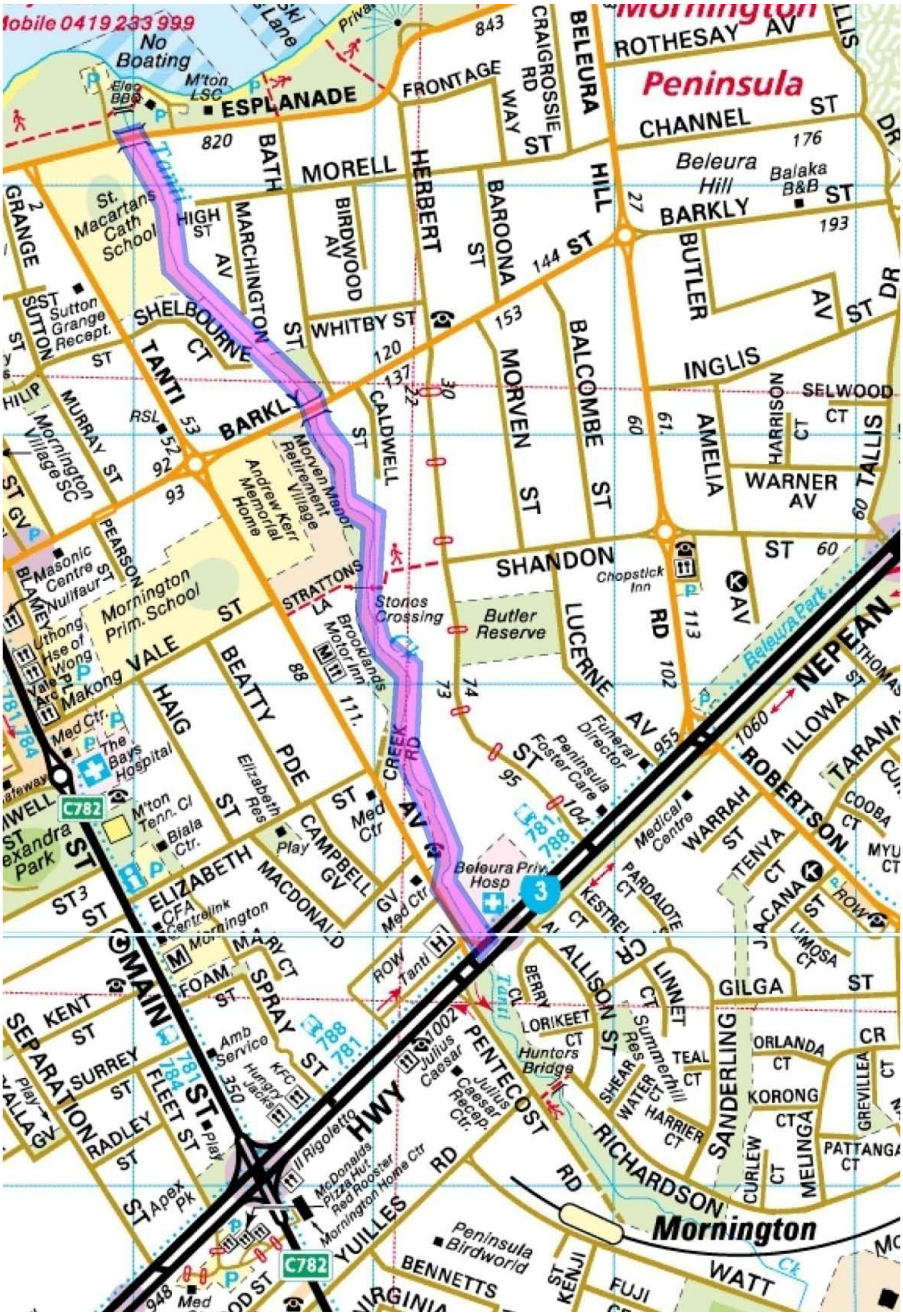

Figure 4-1: Location of Tanti Creek indicating study area

The particular area subject to the detailed investigation extended from north of Strattons Lane to 85 Herbert Street. The area was initially determined by the extent of groundwater leaching from the ground surface as identified in the Draft Tanti Creek Management Plan (August 2002). The area is generally residential with a combination of solitary houses and unit developments. The dwelling constructions are variable and yards generally comprise lawn areas with garden beds and occasional trees.

Towards the rear of these properties, in the area of the detailed investigation, the ground becomes saturated and there is evidence of land slippage. Beyond the edges of the properties there is a fairly steep drop to the creek. This bank is heavily vegetated with trees, weeds, grass and reed growth. The bank itself is obscured by fallen branches and heavy undergrowth.

The creek bed is fairly narrow in sections, particularly in the lower reaches. It is understood that in a 1988 storm event, peak flow in the creek at the Nepean Highway was 22 cubic metres per second. (Acer Wargon Chapman report Tanti Creek Catchment Hydrologic/Hydraulic Investigations –January 1993). This is a large volume of water to be contained within the narrower sections.

5 DRAINAGE STRATEGY

The ex Shire of Mornington commissioned two reports by Acer Wargon Chapman , Tanti Creek Catchment Hydrologic/Hydraulic Investigations (January 1993), Tanti Creek Catchment Flood Study (November 1996)

The reports recommended:

- Construction of a floodway within the existing reserve between Tenya Court and Richardson Drive within the Tanti Park Estate.(Works completed 1999)
- Extension of the existing Archer Drive retarding basin into Lot 83 together with raising the top of bank to RL 42.90 (Works completed 2001)
- Overflow diversion from “St Mitchell Court” basin to “Gas & Fuel” basin
- Upstream catchment diversion from Carbine Way/former Archer Drive intersection to “Gas & Fuel” basin.
- New retarding basin on the Gas & Fuel site. (Site acquired and outfall pipe to Tanti Creek completed 2004) Construction of retarding basin to be undertaken by Melbourne Water.

The implementation of the above recommendations in particular the construction of the “Gas and Fuel” retarding basin will assist in controlling the rate of discharge to the creek and reduce the peak flows. This in turn reduces the flow velocities in the creek and thus the stream bed and bank erosion.

6 CREEK-EROSION CONTROL

The ex Shire of Mornington and the Mornington Peninsula Shire have undertaken significant erosion control works within the creek which included the following:-

- Rock lining from Barkly Street to Marchington Avenue (approx.200 metres) in 1990
- Rock lining from Nepean Highway to Creek Road (approx.400metres) in 1999
- Rock lining adjacent to No 9-13 Shelbourne Court in 2005

7 SITE GEOLOGY

The geological map of the area (Cranbourne 1:63 360) indicates that the site is covered by the ferruginous sandstone, sand, sandy clay and occasional gravel of the Tertiary Baxter Sandstone unit. Generally this unit appears to overlie Devonian granite at depth with the possibility of the silty clay or marl of the Tertiary Balcombe Clay unit separating these. While the Balcombe clay occurs to the south at Fisherman’s Beach and to the north at Manyung Rocks, there is no evidence of Balcombe clays at Tanti Creek. This is consistent with the work of Gostin (1966). However, the presence of the Balcombe Clays to the west and east of Tanti Creek suggests that these clays may be present in the area, although possibly off – shore.

Without deep drilling in the area of Tanti Creek, the presence of Balcombe Clays is not known, but is at depth and does not appear to be affecting any creek bank stability.

Kitson (1900 in Keble, 1950) has indicated that palaeozoic rocks have been observed in the Tanti Creek bed near the bridge into Mornington, which may indicate that the granite intrusions are generally missing in the area.

8 FIELDWORK

The creek banks were initially inspected by an experienced geotechnical engineer on 18th December 2002 and again on 29th May 2003. Further inspections were undertaken on the 29th and 30th August 2007.

In addition, a detailed investigation was conducted across the properties from 57 to 83 Herbert Street. Fifteen boreholes were drilled across the area of detailed geotechnical investigation (Designated Detailed Study Area) to a depth of between 6.0 and 11.2m using a truck mounted drilling rig. The drilling was conducted between 16 and 20 December 2002. Undisturbed samples and Standard Penetrometer tests (SPT's) were taken in the boreholes. The location of the boreholes was very limited due to access difficulties on the private properties and the refusal of some residents to give permission for access to their land.

Upon completion of the drilling, Casagrande piezometers were installed in each of the boreholes to allow the groundwater depth to be measured. The details of the installations are shown on the borehole records.

The locations of the boreholes are shown on the Site Plan, Figure No. 1, Appendix B. The records of the boreholes are appended as Figures No. 2 – 21, Appendix B, together with the Unified Classification System as Figure No. 22.

The fieldwork was carried out under the full-time supervision of an experienced geotechnical engineer who logged the ground encountered, supervised the sampling and the insitu testing and conducted the installation of the piezometers.

9 GEOMORPHOLOGY OF THE OVERALL CREEK VALLEY

The passage from a broad and open valley to a steep-sided one with steeper gradient as the coast is approached, i.e. rejuvenation, is common with the streams entering Port Phillip Bay south of Frankston and relates to movement on the Selwyn Fault and a drop in sea-level. The valley of Tanti Creek typifies this and is essentially a natural feature pre-dating European settlement. The landscape features observed are in the uppermost Unit of loose Sands with lenses of partly cemented Clayey Sand of the widespread Sand Sheet (Tertiary or Quaternary).

The sandy terrain through which the stream flows is easily eroded and within the study area, the changes indicated above, have resulted in deepening of the valley and lateral erosion on an increased scale. The extent of valley deepening is difficult to know in the absence of old survey data. Mature pine trees about 3m above stream level suggest a maximum of 3m of valley deepening within the last half-century.

In the aerial photographs of 1949, 1957 and 1966, there is a path across the creek, just upstream from the current area of concern.

During floods the toes of the slopes are commonly eroding laterally. This leads to lack of support for the steeper slopes, inducing mass wasting down-slope and some likely shallow-seated failures suggested by scarp-like features. Active slope movements are shown in Figure Nos. 2 – 6, Appendix A.

The tops of the creek banks are also commonly the sites of slow movement and failure, as shown by some tilting fence posts and trees, and scarps. Such evidence can be complemented by failed retaining walls, cracked concrete paving and cracked masonry in some of the houses.

9.1 Aerial Photographs

Linear aerial photographs of the Tanti Creek from the Nepean Highway to Port Phillip Bay were originally provided by the MPSC for the years 1949, 1957, 1966, 1979, 1986, 1996, and 2001 for the preparation of 23019-1.6. An additional set of high resolution aerial photographs taken in 2007 was provided to Lane Piper by Council as part of a larger scale investigation of the Mornington Peninsula. These photographs have also been examined as part of this report.

The aerial photographs show that in 1949, the area was primarily grazing land south of Barkly Street and east of Tanti Creek. There was scattered housing west of Tanti Creek. The photograph shows evidence of bull-rushes and swamp areas close to the creek and a number of dams in near vicinity of the creek. There would appear to be a wet area that approximates to the elevated softened ground between Herbert Street and the creek.

Over the 52 years between the 1949 and 2001 photographs, there has been significant construction on the properties on both sides of the creek. In addition, erosion protection consisting of a large boulder lining has been installed along the banks in several areas. The observations in the 2001 photograph are given below.

- The creek banks have recently been covered in erosion protection from Nepean Highway to 81 Herbert Street. The area on either side of these banks is relatively clear of vegetation within the creek reserve.
- From 81 Herbert Street to Strattons Lane, the creek appears to be confined to a narrow channel at the base of a well-vegetated gully. There appears to be large areas of reeds and grasses, especially in the southern part of this section. In the northern part of this section, there are numerous medium sized trees. A significant scarp can be seen along the rear boundaries of 71 to 77 Herbert Street and a recent landslip can be seen at the rear of 79 Herbert Street.
- From Strattons Lane to Barkly Street there is a wide grassed and treed reserve between the western bank of the creek and the retirement village. On the eastern side of the creek, the properties appear much closer to the creek. Along the creek in this section there are small pockets of reeds and grasses.
- Between Barkly Street and the primary school, the residential properties run to the edge of the creek. The rear of these properties is mostly well treed.
- From the primary school to The Esplanade on the eastern side of the bank, there are several larger residential properties with large back yards. The yards have numerous tall trees and cleared areas.
- On the west side of the creek between the schoolyard and the creek, there are areas with reeds and long grass.

Between 2001 and 2007 there were only minor changes to the residential properties beside the creek with the exception that some of the previous dwellings were demolished and replaced with unit developments. However there were some noticeable changes to the creek itself.

- The rock lining between Nepean Highway and 81 Herbert Street is now much less noticeable and there appears to be significant siltation within the creek. In addition, the creek bed is covered by reeds and grasses.
- From 79 Herbert Street to Strattons Lane, there has been minimal change with the exception that the landslip behind 79 Herbert Street is now less noticeable but the scarp of the landslip behind 65 Herbert Street is now more pronounced.
- There is no noticeable change between Strattons Lane and Barkly Street.

In the 2007 photographs the banks have been rock lined along the creek from Barkly Street to 13 Marchington Avenue.

In addition, in the 2007 photograph a new estate can be seen between The Esplanade and the primary school, although there has been little change to the creek.

The aerial photographs for the 1949, 2001 & 2007 years are appended as Appendix F.

9.2 Field Observations

The following observations are identified on the site plans Figures No. 2 – 6, Appendix A with Figure 1, Appendix A showing the overall study area.

Although in general, along a large portion of the creek, the creek banks are stable, the creek banks have undergone mass wastage and bank slippage at numerous locations. This has resulted in loss of usable space for the properties adjacent to the affected areas and in some locations resulted in damage to structures.

Details of the observations made during the field investigation are discussed below, particularly in reference to the creek banks and the areas immediately above the banks. A detailed discussion of the damage to the structures in the neighbouring properties is outside the scope of this report. However, where minor structures such as fences and retaining walls in the vicinity of the creek have been affected, they are discussed.

Immediately to the north of Nepean Highway, the creek is confined to a narrow channel of approximately 8 to 10m wide. The channel runs for approximately 140m along Tanti Avenue between the street and the hospital before turning at a right angle to run behind 131 Tanti Avenue. Although the erosion protection was visible in the 2001 aerial photographs, the protection was not visible at the time of the investigation. There has been a significant build up of silt along the banks of this section and in areas there are noticeable slumps of this silt material into the base of the creek. Although these slumps have occurred the creek bank itself appears stable.

From 131 Tanti Avenue to approximately 81 Herbert Street and 109 Tanti Avenue the banks of the creek are rock lined. At the time of the previous investigation it was observed that active erosion was occurring along this section and several landslips were observed at the northern end on the eastern side of the creek. The rock lining appears to have stopped the erosion from occurring. However, there is evidence that some minor movement is still occurring, especially along the western bank.

From 79 Herbert Street to Strattons Lane, the creek gully widens out to the adjacent property boundaries. Along this section there are numerous landslips and slump zones. There is water at the surface and within the scarps at many locations, especially along the eastern bank. It appears that this section is subject to a series of slowly receding landslips that are causing the creek banks to gradually encroach on the adjacent properties. Several of the properties have lost their rear fences due to the slips. Figure 9-1 to Figure 9-4 on the following pages show the effect of the landslips on the neighbouring properties.

The west side of the bank between the Retirement Village and the creek, between Strattons Lane and Barkly Street, is steep and heavily grassed, about 5m high with evidence of scarps and some stabilising trees. On the opposite side of the bank from 12 Caldwell Street there was a heavily grassed area with abundant reeds and bamboo. There was also a 4-5m scarp in this area. Water was observed to be exiting to the surface at a significant rate in one location within this area, but it could not be determined whether the water was from a pipe or a spring.

Between the creek and 45 Herbert Street there is a shallow bowl shaped feature that is possibly an old scarp that has been weathered over time. The ground within the bowl is soft and there is a significant grass cover. There is also a scarp in the north-west part of this property on the creek bank that is being stabilised by two pine trees.

The slopes to the rear of the properties between Caldwell Street and the creek are quite steep. The area is heavily vegetated and hummocky. Where retaining walls have been constructed they are often leaning or showing signs of movement.

The length of creek between Barkly Street and 13 Marchington Avenue has been rock lined since our previous investigation, although the southernmost 40m is predominantly covered in silt. There have been some slumps within this silt material. In general the rock lining appears to have helped stabilise the banks. However, in the section of creek between 13 Marchington Avenue and 11 Shelbourne Court where scarps were previously noted the rock lining has collapsed, indicating movement is still occurring. Figure 9-5 and Figure 9-6 show the collapsed rock lining.

From the rear of 3 to 11 Marchington Avenue, the creek is unlined. The bank is undercut along large sections of this part of the creek and some sections of the bank have collapsed.

Figure 9-7 shows a section of the undercut bank. The east side of the creek above the bank is relatively flat for approximately 50m from the creek and then becomes steeper at what is most probably a prior creek bank. This area is slightly hummocky but appears relatively stable.

The west side of the creek bank between the school and the creek is significantly steeper and heavily vegetated. There again appears to be a secondary creek bank in the vicinity of the school fence approximately 15m from the creek, although this may be a fill batter associated with the levelling of the schoolyard. There are several scarps and flatter areas associated with this secondary bank.

The western bank from opposite 3 Marchington Avenue to The Esplanade has recently been cleared and a significant portion of the creek banks has been rock lined as can be seen in Figure 9-8. The 2007 aerial photograph indicated this area to be heavily vegetated. Therefore the works have occurred since the time of the photograph. There are some leaning trees along this section of the bank but it is likely that movement occurred prior to the construction of the creek works.

The upper bank continues on the western side of the creek at approximately 20m from the creek. There was minimal evidence of movement along this section of the bank. A rock gabion wall has recently been constructed to the rear of the new subdivision along the line of the upper western bank. The gabion wall is retaining this upper bank. A brief discussion with contractors who were drilling piers behind the gabion wall indicated that they were intersecting fill material including chicken wire and other debris. As such it is likely that the creek bank is fill.

On the eastern side of the creek the area is relatively flat for approximately 40m before rising at the upper eastern bank. The relatively flat area is very boggy in zones.

Immediately south of The Esplanade, there is a fill embankment on the eastern side of the creek associated with a property on The Esplanade. The fill embankment has had some minor slumps.

In general, a large portion of the creek has now been rock lined. The rock lining works are concentrated in the northern and southern parts of the creek with approximately 60% of the creek being lined. Where the creek has been lined the banks appear stable, although there are some sections where movement is apparently still occurring.

Of the unlined section of the creek, the area between 77 Herbert Street and Strattons Lane appears to be the most affected by erosion and landslip. This area has numerous landslips and areas of seepage.

There are other areas in the unlined section that show evidence of scarps. However, in these sections there is little surficial water present.

Figure 9-1: Rear of 65 Herbert Street

Figure 9-2: Rear of 69 Herbert Street

Figure 9-3: Rear of 77 Herbert Street

Figure 9-4: Rear of 101 Tanti Avenue

Figure 9-5: Creek between Marchington Avenue and Shelbourne Court

Figure 9-6: Collapsed rock lining behind 11 Shelbourne Court

Figure 9-7: Undercut banks to rear of 9-11 Marchington Avenue

Figure 9-8: Recently rock lined creek to south of The Esplanade

10 LABORATORY TESTING

A limited laboratory testing program was undertaken in our NATA registered soils laboratory and consisted of the following:

- Atterberg Limits
- Particle Size Distributions
- UU triaxial compression tests after back-saturation
- CU triaxial compression tests with pore water pressure measurements

The test records are included in Appendix C.

11 RESULTS OF THE GEOTECHNICAL INVESTIGATION

11.1 Ground Stratigraphy

The boreholes revealed strata varying with depth and distance along the creek alignment:

Table 1: Ground Stratigraphy

Lithology	Depth Range (m)
Concrete / Rootmatter	0 – 0.07
FILL most commonly clayey SILT (ML) Medium plasticity, moderately fissured, grey-brown, brown, stiff, slightly moist.	0 – 0.7
Silty **CLAY** (CI-CH) Medium - high plasticity, moderately fissured, dark grey, grey, brown, firm – very stiff, moist – very moist, some reeds, trace sands, hydrogen sulphide odour	1.0 – at least 6.45
Sandy **CLAY** (CI) Medium plasticity, moderately fissured, fine – medium grained, pale grey, pale orange-brown, very stiff – hard, moist	2.8 – at least 6.45
Clayey **SAND** (SC) Low plasticity, fine – medium grained, medium dense – dense, pale grey and orange-brown, very moist, some gravels with depth	4.3 – 8.3
Slightly clayey **SAND** (SC) Fine – medium grained, pale grey and orange-brown, dense – very dense, very moist, trace gravels	Extending to the full depth explored of at least 6.0 – 11.2 when encountered

Stratigraphic cross –sections through the creek banks in the area of the originally proposed geotechnical investigations are shown overleaf in Figure No. A.

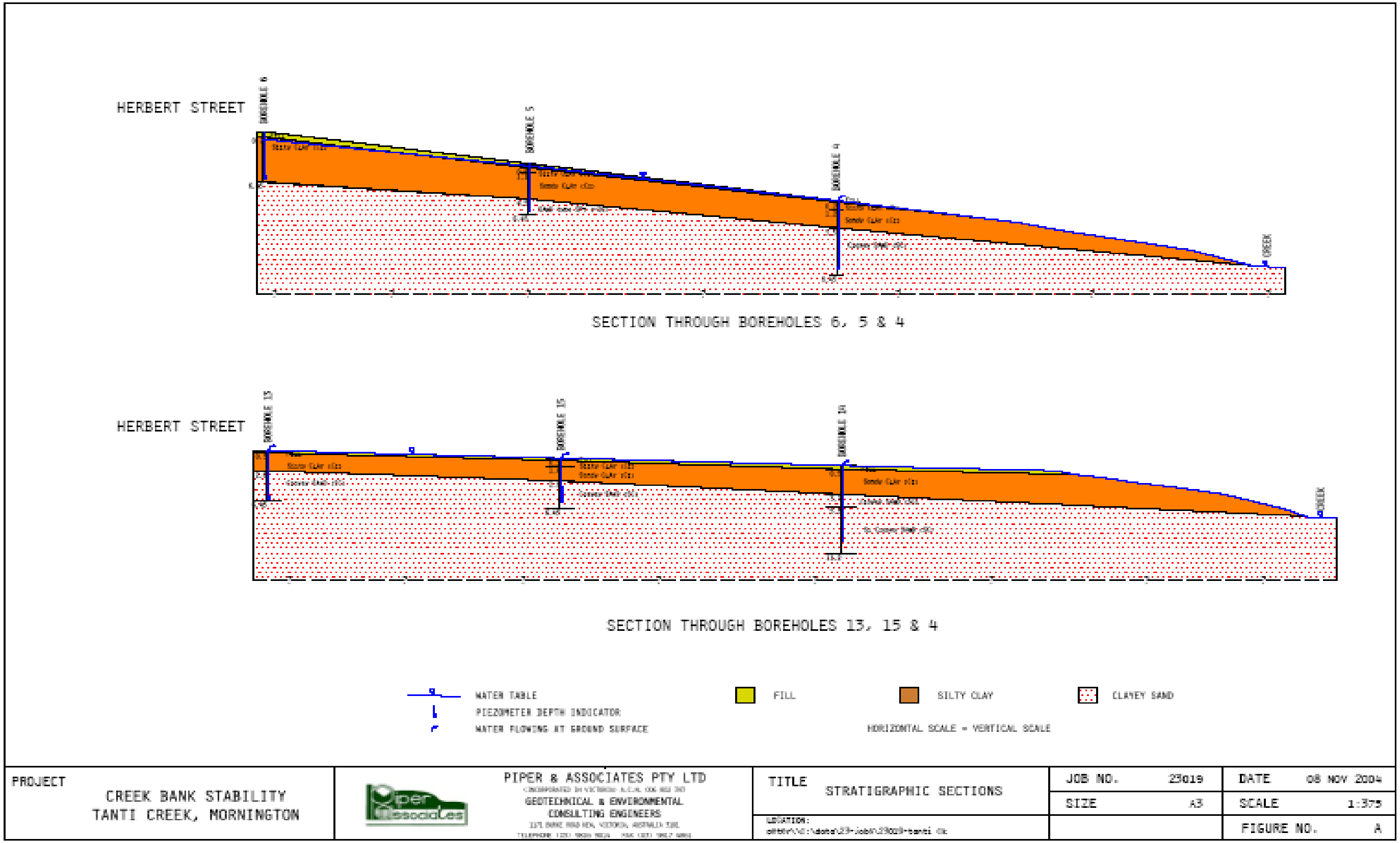

Figure 11-1: Stratigraphic Profiles through the Eastern Side of Tanti Creek bank

11.2 Groundwater

The piezometers revealed the following water depths in the boreholes as follows:

Table 2: Groundwater depths below ground level on 21 January 2003

Borehole No.	Water Depth (m) below ground level
1	0.0[1]
2	0.0
3	0.24
4	0.0
5	0.32
6	1.00
7	0.00
8	0.00
9	1.30
10	0.00
11	0.82
12	2.31
13	0.00
14	0.00
15	0.00

1. Some of the bores with groundwater at the surface had water flowing out of the top of the piezometer pipe.

The overall direction of the groundwater flow in the area of the detailed geotechnical investigation on the eastern side of the creek is apparently towards the south – west. Overall, it is considered that the regional direction of groundwater flow is most likely towards the north – west, but this is subject to confirmation.

The locations of the boreholes are shown on the Site Plan, Figure No. 1, Appendix A.

The piezometers indicate that the aquifer is artesian within the housing blocks at a distance from Herbert Street towards the creek. i.e. the lower lying area areas close to the creek. The results indicate that there is an upward and lateral flow from the underlying sand aquifer saturating the upper clays.

11.3 Laboratory Testing

The results of the laboratory testing are summarised in the following tables:

Table 3: Results of the Atterberg Limit and Particle Size Distributions

Borehole No	Depth (m)	Soil Description	Liquid Limit (%)	Plasticity Index (%)	Symbol	% passing 75 microns
2	1.5	Silty CLAY grey, sandy	37	24	CI	
4	7.5	Gravelly SAND brown, slightly clayey				13
7	9.8	Gravelly SAND yellow-brown, slightly clayey				4
10	1.5	Silty CLAY grey, slightly sandy	47	31	CI	
14	1.5	Clayey SAND light grey				49
14	6.0	SAND light brown, slightly clayey, slightly fine gravelly				12
15	1.5	Silty CLAY grey, slightly sandy	41	27	CI	
15	6.0	Clayey SAND brown, slightly gravelly				14

The sand contains a relatively high percentage of fines consisting of clays and silts. This would result in a low hydraulic conductivity for the sands.

The UU triaxial compression tests are summarised in the following tables:

Table 4: Results of the UU Triaxial Compression tests

Borehole No	Depth (m)	Soil Description	Moisture Content (%)	Dry Density (t/m^3)	Apparent Cohesion (kPa)	Shear Angle (deg.)
1	1.0	Clayey SILT black (organic)	88.7	0.73	8	2
4	1.5	Sandy CLAY grey	22.2	1.58	33	9
8	1.5	Sandy CLAY grey	15.9	1.79	50	5

Table 5: Results of the CU Triaxial Compression Tests

Borehole No	Depth (m)	Soil Description	Moisture Content (%)	Dry Density (t/m^3)	Effective Cohesion (kPa)	Effective Shear Angle (deg.)
7	1.5	Sandy CLAY grey, dark grey & orange-brown	19.5	1.60	18	21
14	1.5	Clayey SAND light grey	21.9	1.68	25	24

12 FAILURE MECHANISMS

The failure modes along the creek banks can be differentiated into two types:

1. Deep seated arcuate failures
2. Shallow, translational and arcuate failures

In the area south of Strattons Lane between Herbert Street and the creek and possibly between Caldwell Street and the creek at the Morven Manor Retirement Village, the pressurisation of the groundwater, the soil type and layer thicknesses have had a significant effect on the stability, resulting in deep seated failures. A similar mode of failure is possible on the opposite side of the creek.

The failure modes observed have occurred in the slopes on or behind the crest of the creek banks. The failures are often translational and/or deep-seated arcuate failures. In some cases the latter have extended up to 10-15m into the adjacent properties, significantly affecting the amenity of these properties. These failures are producing scarps that are generally less than 2m in height. The areas where deep-seated failures have been observed are characterised by nearby groundwater seepage indicating the existence of a pressurised aquifer at depth. The overlying silty and sandy clay layers are confining the aquifer. Where the ground surface is low and the clay layers are either thinner, or slightly more permeable due to the sand component, the groundwater is able to seep to the surface. As this occurs, shallow landslips increase in likelihood.

The fieldwork showed that the lower layers of the sub-surface profile are moderately permeable with artesian pore water pressures in the sands. The clay layer above this permeable layer acts as an aquitard, generally inhibiting the groundwater rising to the surface. It is considered likely that the thickness of this clay layer is one of the primary factors determining the location of surface seepage. The pore water pressures generated in the clay layer from the underlying sand aquifer can result in instability of the creek banks.

In other areas the steep slopes, combined with notch or vertical deepening of the creek bed, have initiated shallow slope failures on the creek banks. These failures are of small soil volume and can be readily stabilised.

13 COMPUTER ANALYSES

A numerical analysis of the steady state seepage through the existing ground profile was undertaken. The numerical analysis was calibrated against the pore water pressures measured in the piezometer installations. Limited dewatering options were then imposed to determine the pore water pressures and the ultimate groundwater level within the profile.

The change in pore water pressures was then used to model the slope stability of a section through Boreholes 4, 5 and 6.

13.1 Seepage Condition

The seepage analysis was carried out using the computer program FESEEP, developed by the University of Sydney. This program uses a 2 – dimensional, finite element, steady state seepage model to determine the pore water pressures.

13.2 Calibration of the Computer Seepage Model

The fieldwork indicated that the confined aquifer at depth was slightly artesian with a head of groundwater above the ground level a short distance from the road. The boundary conditions for the steady state, 2 - dimensional seepage were altered together with the hydraulic conductivities to calibrate the pore water pressures within the aquifer to the field results.

The boundary conditions assume that the groundwater flow from the north and east is pressurising the confined aquifer in the region under investigation. This was indicated by the piezometers showing artesian pressures.

The following hydraulic conductivities were used in the steady state, 2 – dimensional model.

Table 6: Adopted Calibrated Hydraulic Conductivities

Layer No	Description	Horizontal Hydraulic Conductivity K_h (m/s)	Vertical Hydraulic Conductivity, K_v (m/s)
1	Upper Silty Clay / Clayey Sand Layer	1.0×10^{-9}	1.0×10^{-9}
2	Lower Sand / Clayey Sand Layer	1.0×10^{-4}	1.0×10^{-4}

The finite element mesh, the pore pressures and material properties from the calibration are included in Appendix D.

13.3 Seepage Analysis Limitations

There is a degree of uncertainty within the seepage model. The model to date is a 2 – dimensional steady state model and does not include the 3-dimensional effects such as seepage from a direction lateral to the section. This would require more complex, 3-dimensional modelling. This is beyond the scope of the brief.

The computer analysis must be understood as a starting point for the design of a dewatering system, however factors beyond the scope of the model may result in any initial dewatering works having less of an influence than that indicated by the model. Monitoring of the existing piezometers is recommended.

14 SLOPE STABILITY

A slope stability model was created using the computer program XSLOPE, developed by the University of Sydney. This program uses the simplified Bishop Method of Slices to analyse the slope stability.

The slope stability model was developed by incorporation of the borehole logs, the laboratory test results and the correlation with the onsite observations. The water depth readings were used to calibrate a finite element seepage analysis, using the computer program FESEEP, to develop a pore water pressure grid that could be used in the slope stability analysis.

14.1 Selection of the Shear Strength Parameters

Based on the results of the CU triaxial compression tests, and correlations developed by Stark and Eid (1997) for the soils based on the Atterberg Limits and clay fraction, the following design parameters were adopted:

Table 7: Selected Design Parameters

Material Description	Adopted Effective Shear Strength (c' kPa)	Adopted Effective Friction Angle (φ' deg)
Sandy / silty CLAY	2	23
Clayey SAND / SAND	0	30
Sandy / silty CLAY	2	23

14.2 Calibration with the Onsite Observations

These results were calibrated against the known failures, particularly the shallow slips at the rear of 67-69 Herbert Street. The results of the calibrations are included in Appendix E.

A reasonable correlation was developed between the onsite observations and the computer model.

The thickness of the clay layer, the steepness of the slope of the land and the resultant difference between ground level and pore water pressures have a significant effect on the stability of the banks. This can result in the observed seepage and subsequent slope failures on seemingly gentle slopes.

14.3 Slope Stability of Creek Bank, 57 Herbert Street.

Typical outputs from the computer slope stability are included in Appendix D, and the results are tabulated below.

The computer slope stability analysis indicates that the rear of the properties between Herbert Street and the creek are at risk and an analysis was undertaken for 57 Herbert Street. The analysis had a number of issues affecting its reliability and its problems. These issues are noted on the following page:

- The section was adopted because of the measurable groundwater head in two out of the three piezometers. In all the other possible sections, the groundwater was either flowing out the top of the piezometers in at least two of the three piezometers, making calibration of the model impossible, or the slopes were located beyond the management overlay and considered gentle.

- There was no contour data available from about half way along the blocks and extending to the road.
- The contour data of the creek bank has been averaged between the rear of the properties and the water level of the creek. The steepness of the actual creek bank has been inferred from observations and interpretation of the contour plan. Additionally, more accurate contour surveying is required between the rear half of the blocks and the creek itself.

In our opinion, a minimum factor of safety of at least 1.5 is desirable for the properties above the crest of the creek slopes. The computer analysis is very sensitive to the angle of slope, groundwater pressures and also to the thickness and permeability of the overlying clay layers. With naturally occurring slopes, the desired factor of safety is often difficult to achieve and a lesser factor of safety is tolerated, although the risk of failure is greater than would be accepted for say, a civil engineering structure.

Table 8: Slope Stability of the Existing Condition from Slope Stability and Groundwater Analyses

Description	Factor of Safety[1]
Existing slope behind the edge of the creek bank	1.78
Existing Creek Bank	1.03[2]

[1] = Factor of safety against a circular slope failure.
[2] = Conservative artesian water pressures in the underlying aquifer were adopted. Higher artesian water pressures resulted in negative effective stresses, reducing the factor of safety to a very low value i.e. failure.

More accurate survey is required to correctly model the stability in this area, but the model does indicate that the creek banks are marginally stable to unstable, and prone to slope failures if the pore water pressures increase slightly, the ground surface is steeper, surcharged or the creek bed has deepened. The ground surface at a distance more than some 20m away from the creek is relatively stable.

Note that the existing slope failures at the end of 65 and 69 Herbert Street will have a factor of safety of less than 1.

15 GENERAL OBSERVATIONS AND GEOLOGY ALONG ENTIRE CREEK BANKS

The creek bank was briefly inspected for evidence of landslips and other features between the Nepean Highway and the Esplanade. The dense vegetation limited the observations of the creek bed and banks. The discussion of the observations is included above and located on the site plans in Appendix B. Pertinent summary comments are included here.

The conclusions are as follows:

1. A flat-lying, mainly sandy sheet of Tertiary or Quaternary materials, with the uppermost consisting of silty and/or sandy clay overlying medium dense to very dense sand and clayey sand, covers the study area between Herbert Street and Tanti Creek.
2. There is no evidence of any other Tertiary formation between the sand sheet and the level of the base of the creek. A deep-seated global failure affecting the central section of the lots is considered to be unlikely due to the lack of a highly fissured deep clay profile.

3. Within the last century, Tanti Creek has deepened its valley in the study area. It is still actively eroding, both vertically and laterally.
4. The failures of concern in this study are in the rear of residential properties and are generally relatively shallow in nature and located within the rear section of the lots.
5. The area of the detailed investigation within the designated study zone is underlain by an artesian sand aquifer with an overlying clay aquitard. The groundwater level in the aquifer is either below or just above the ground surface depending on the thickness and permeability of the overlying clay aquitard layer.
6. A number of masonry homes have already suffered damage[1] (Nos. 10 & 12 Caldwell Street, and 39 & 77 Herbert Street) and others are endangered (Nos 69, 71 and 77 Herbert Street).

The areas that are recommended to be included in the management overlays with regards to slope stability are shown on Figure Nos. 2 – 6, Appendix A with an overall layout plan as Figure No. 1. It should be noted that these management overlays are not to prohibit construction in these areas, but to highlight the need for a detailed geotechnical investigation in these areas with appropriate evaluation of the underlying pressurised aquifers and/or steep creek banks.

The reader also needs to be aware that the creek slopes have formed as a result of slope failures from the creek erosion and houses and properties close to such creek banks will be potentially at risk.

The presence of trees and other deep-rooted vegetation can inhibit the erosion of the near surface clays on the creek banks. Hubble and Hull (1996) indicates that the cohesion and shear angle of the clay banks will decrease for the defoliated slope. The tree roots are assisting in the stabilisation the existing creek bank.

The computer analysis tends to confirm that the failures are mostly limited to shallow circular slope failures in the upper 3 – 4m. Therefore, only the dwellings or structures close to the edge of the creek banks are considered to be at risk.

16 CONCLUSIONS

The geotechnical investigation has found that instability of the creek banks in the area of the Designated Detailed Study Area is occurring as a result of increased pore water pressure below the surface, and within the sand aquifer, and as a result of steepening of the creek banks by creek erosion. The development of properties along the creek banks over the last 50 years has required the clearing of vegetation closer to the creek, which has resulted in increased paving and runoff intensity during storm events. The upstream section of the creek has been rock lined both in the floor and the walls, including rock drop structures to flatten creek bed gradients. Generally the erosion in this area has been limited, although a section of the creek bank opposite 81 – 83 Herbert Street showed instability during construction of the lining of the creek banks. The instability of the creek banks was clearly demonstrated with a slope failure at the rear of 69 Herbert Street when the site was being developed and failures at the rear of 65 and 71 Herbert Street, Mornington.

The previous investigations and inspections have found that instability of the creek banks, especially between Herbert Street and Tanti Creek and possibly on both sides of the creek

[1] It is beyond the scope of the investigation to complete a dilapidation study of the dwellings and other features on the properties. There may be other buildings showing damage that were not obvious.

near Caldwell Street, is occurring as a result of increased pore water pressure below the surface.

The area between 79 Herbert Street and Strattons Lane is silted up with considerable vegetation. This results in accumulation of silt at the sides of the wide creek bed in this area.

Throttling of the creek in this area can result in partial damming of the creek with subsequent removal of this ‘dammed’ silt during times of flooding. Immediately following the removal of the silt, creek bank failures can be re-initiated, resulting in creek floor heave until equilibrium is again achieved.

Consequently, any removal of this ‘dammed’ material in the creek bed needs to be accompanied by appropriate geotechnical measures to avoid slope instability further affecting the adjacent properties. Possible measures include rock boulder walls and floor, gabions and mattresses or piping of the creek with appropriate engineering design.

In other areas, a different mode of failure is a result of steepening of the creek banks by either deepening or notch erosion.

The deepening of the creek due to increased runoff has also contributed to slope failures, particularly in the lower reaches. In these areas, the steepness and height of the creek banks can be contributing to instability of the creek banks.

It is concluded that the slope stability of the area adjacent to the creek is affected by one or more of the following: a) steeper slopes, b) notch erosion, c) soft alluvial creek deposits and d) artesian water pressures. The artesian groundwater can result in softened ground and rising damp as well as other issues around the dwellings above the creek banks.

The analyses and observations along the edges of Tanti Creek indicate that a management overlay is required along the creek banks to ensure that there is adequate geotechnical investigation and assessment of any structures in the properties in near proximity of Tanti Creek.

There are a number of possible options to improve the stability of the slope. It is unlikely that a solution involving structural stabilisation of the creek banks will be environmentally acceptable and is not to be considered further. Therefore measures such as piping the creek, deep dewatering systems, gabion or sheet pile retaining walls, sub-horizontal bored drainage systems or other similar options will not be considered further.

17 MANAGEMENT OVERLAYS & FUTURE DEVELOPMENTS

In Figure Nos. 1 – 6, Appendix A, the areas of potential management overlays are shown on the Site Plan. The affected areas are shown in these figures. The accuracy of these management overlays is approximate only.

It is recommended that the Council restrict future developments in the area, unless justified with a comprehensive geotechnical investigation. It is recommended that no future dwellings, extensions or outbuildings be constructed on the properties covered by the Management Overlays, (this includes any part of a building or structure) without a complete geotechnical investigation being undertaken by a suitably experienced geotechnical engineer, registered as a Building Practitioner. The geotechnical investigation needs to be of sufficient depth and quality to allow an assessment of the slope stability, should consider the effects of an artesian or high groundwater table below the dwellings and the impact of steep slopes.

Regardless, no new dwellings or buildings should be constructed within close proximity to the creek banks i.e. with 10m of the edge of the creek banks.

Access along the creek banks for construction of any drainage layers and to allow any maintenance of the creek banks is recommended by the creation of an easement or Creek Reserve.

18 SCOPE OF FUTURE GEOTECHNICAL INVESTIGATIONS

Future geotechnical investigation should consist of at a minimum, the following:

- Appropriate site inspection and consideration of the slope stability issues by an experienced geotechnical engineer or engineering geologist. Depending on the results of this site inspection, further geotechnical investigation may be warranted and could comprise the following
- At least 3 geotechnical investigation boreholes to a minimum depth of 9m but may need to be varied depending on the particular site.
- Appropriate undisturbed sampling, Standard Penetrometer testing and possibly coring of the rock, if encountered
- Installation of piezometers to measure groundwater levels and/or pressures
- Appropriate geotechnical testing in a NATA accredited soil laboratory to confirm the geotechnical shear strength design parameters, or at least by established correlations.
- Computer modelling of the slope with the imposed structures, fill and cuts to determine that there is an adequate factor of safety for the proposed development.
- Reporting and discussion of any site restrictions or on-going site management as required by the geotechnical engineer. The analysis and reporting to be conducted by an experienced geotechnical engineer or engineering geologist who is experienced in slope stability assessments and who has inspected the site.
- Depending on the structures proposed to be constructed, independent geotechnical peer review may be required to review the results of the geotechnical investigation.
- A risk assessment may be appropriate for some of the sites in accordance with AGS 2007 guidelines.

A typical checklist for a slope stability assessment is included in Appendix G.

19 SITE MANAGEMENT REQUIREMENTS

In the areas of higher risk, development controls may be required to ensure that the stability of the site does not deteriorate. The development controls should consist of (at a minimum):

Professional geotechnical engineering advice on the site prior to starting work

- Limitations on the fills and cut slopes within the site and any retaining walls. Any fills or cuts in proximity to the creek should be carefully considered.
- Any retaining walls or structures should be drained and designed by a professional engineer
- Limitations on changes to the natural site profile
- Minimise any vegetation removal
- Control surface water and sub-surface groundwater

- Avoid structures that are sensitive to movement, such as pools and basements
- Consider carefully the location of the soakage pits and any absorption trenches
- Control stormwater runoff to ensure that the water is piped to the creek and does not over-flow the creek banks.
- Introduction of monitoring by inclinometers or extensometers as deemed appropriate for the particular site

The risk of landslip is higher on some sections of the lots than others. The development of these lots should avoid the higher risk areas.

20 FURTHER WORK

It is recommended that survey work be carried out in close vicinity of the creek to improve the accuracy of the model and the piezometers be located and levelled.

It is recommended that data loggers be installed in the piezometers to measure the porewater pressures over a period, including a winter – spring period to assist in the modelling of the creek bank stability.

Further geotechnical work is required to evaluate the potential settlement and increased shrink-swell ground movement if the water table is lowered.

21 REFERENCES

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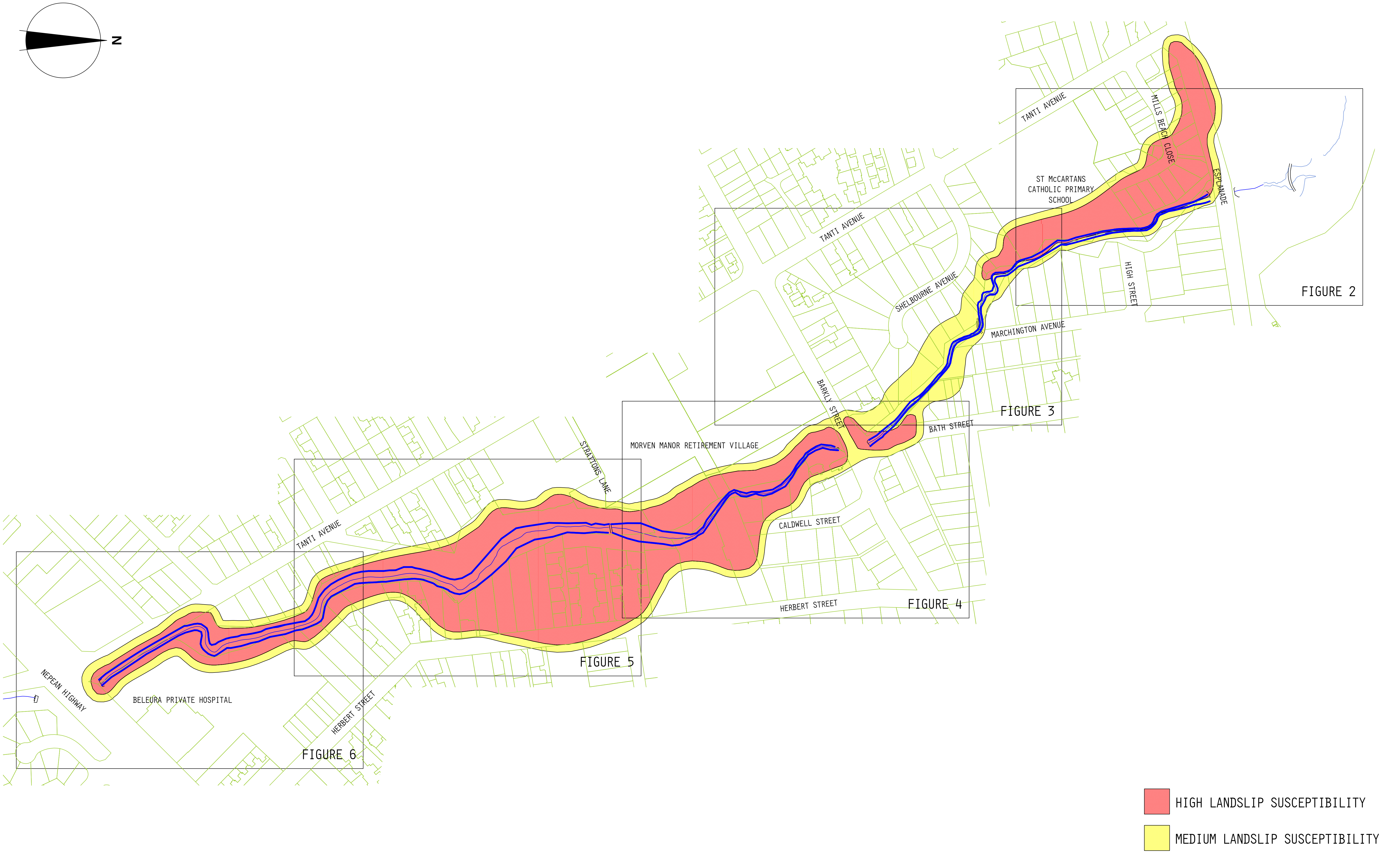
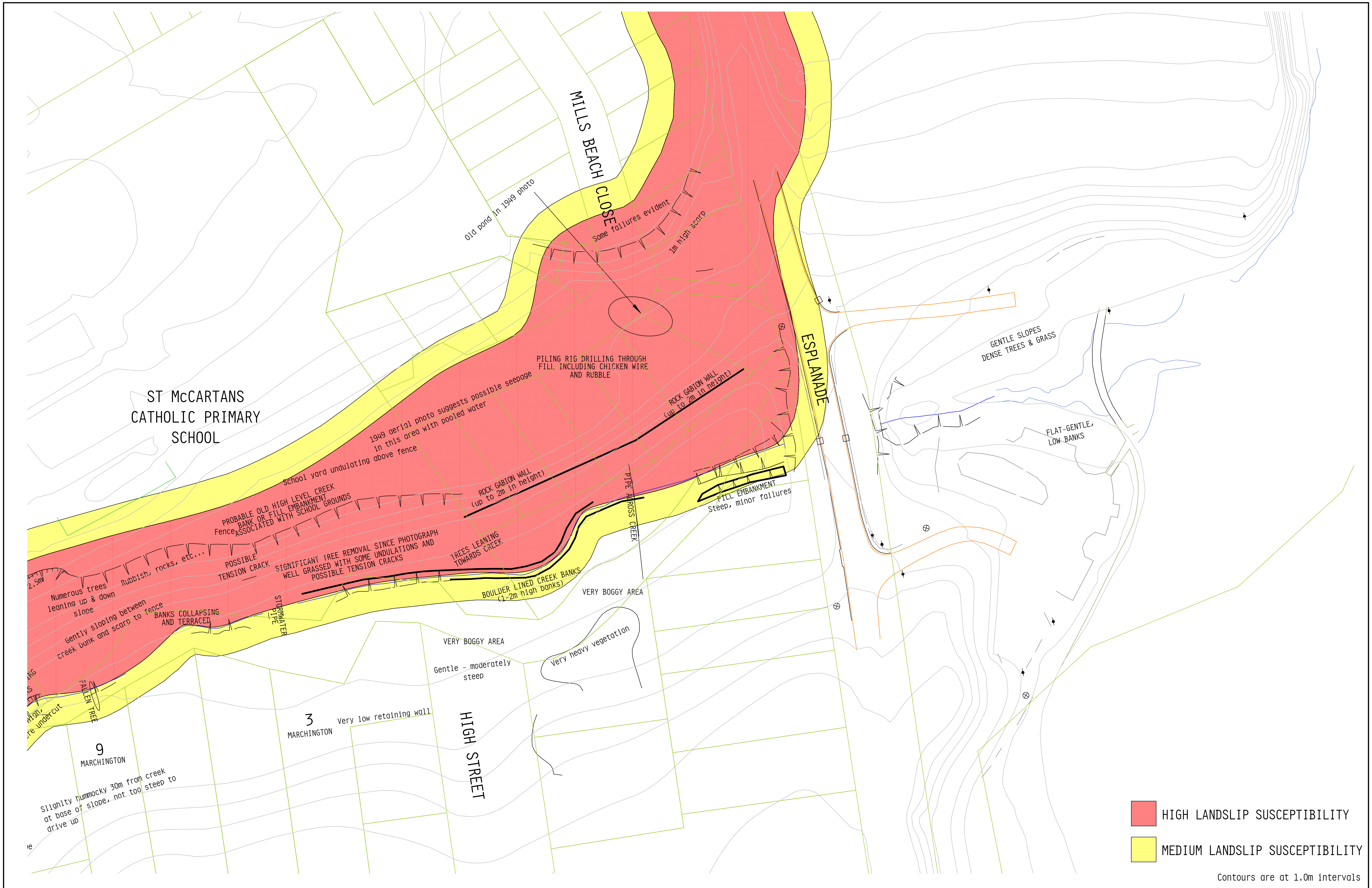
Appendix A

6 Pages

OBSERVATIONS & MANAGEMENT OVERLAYS

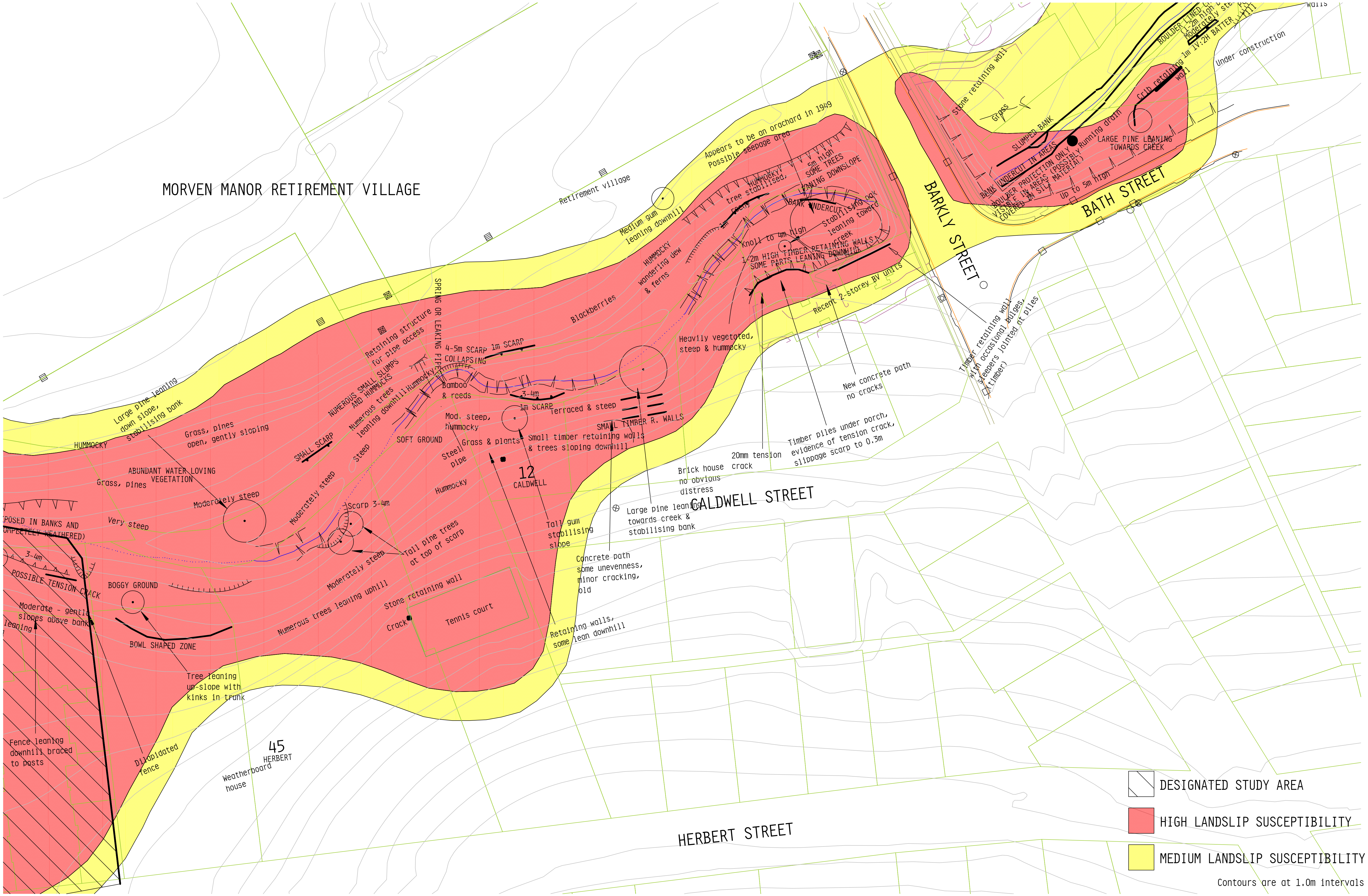


PROJECT		TITLE	SCALE (A3) 1:3750	DATE 29 MAR 2010
CREEK BANK STABILITY TANTI CREEK, MORNINGTON	LANE PIPER PTY LTD A.C.N. 120 109 935 Geotechnical and Environmental Engineers Hydrogeologists and Environmental Scientists BLDG 2, 154 HIGHBURY RD, BURWOOD, VICTORIA, 3125 TELEPHONE (03) 9888 0100 FAX (03) 9808 3511	SITE PLAN (OVERALL)	DRAWN DBS	JOB NO. 23019
		REF: 23019-Site Plan-3-10		REV. NO. 1 / FIG. NO. 1



PROJECT	LANE PIPER PTY LTD	TITLE	SCALE (A3) 1:1000	DATE 29 MAR 2010
CREEK BANK STABILITY TANTI CREEK, MORNINGTON	LanePiper A.C.N. 120 109 935 Geotechnical and Environmental Engineers Hydrogeologists and Environmental Scientists BLDG 2, 154 HIGHBURY RD, BURWOOD, VICTORIA, 3125 TELEPHONE (03) 9888 0100 FAX (03) 9808 3511	SITE PLAN	DRAWN DBS	JOB NO. 23019
		REF: 23019-Site Plan-3-10		REV. NO. 1 FIG. NO. 2

PROJECT	LANE PIPER PTY LTD	TITLE	SCALE (A3) 1:1000	DATE 29 MAR 2010
CREEK BANK STABILITY TANTI CREEK, MORNINGTON	LanePiper A.C.N. 120 109 935 Geotechnical and Environmental Engineers Hydrogeologists and Environmental Scientists BLDG 2, 154 HIGHBURY RD, BURWOOD, VICTORIA, 3125 TELEPHONE (03) 9888 0100 FAX (03) 9808 3511	SITE PLAN	DRAWN DBS	JOB NO. 23019
		REF: 23019-Site Plan-3-10		REV. NO. 1 FIG. NO. 3



PROJECT	LANE PIPER PTY LTD	TITLE	SCALE (A3) 1:1000	DATE 29 MAR 2010
CREEK BANK STABILITY TANTI CREEK, MORNINGTON	LanePiper A.C.N. 120 109 935 Geotechnical and Environmental Engineers Hydrogeologists and Environmental Scientists BLDG 2, 154 HIGHBURY RD, BURWOOD, VICTORIA, 3125 TELEPHONE (03) 9888 0100 FAX (03) 9808 3511	SITE PLAN	DRAWN DBS	JOB NO. 23019
		REF: 23019-Site Plan-3-10		REV. NO. 1 FIG. NO. 4

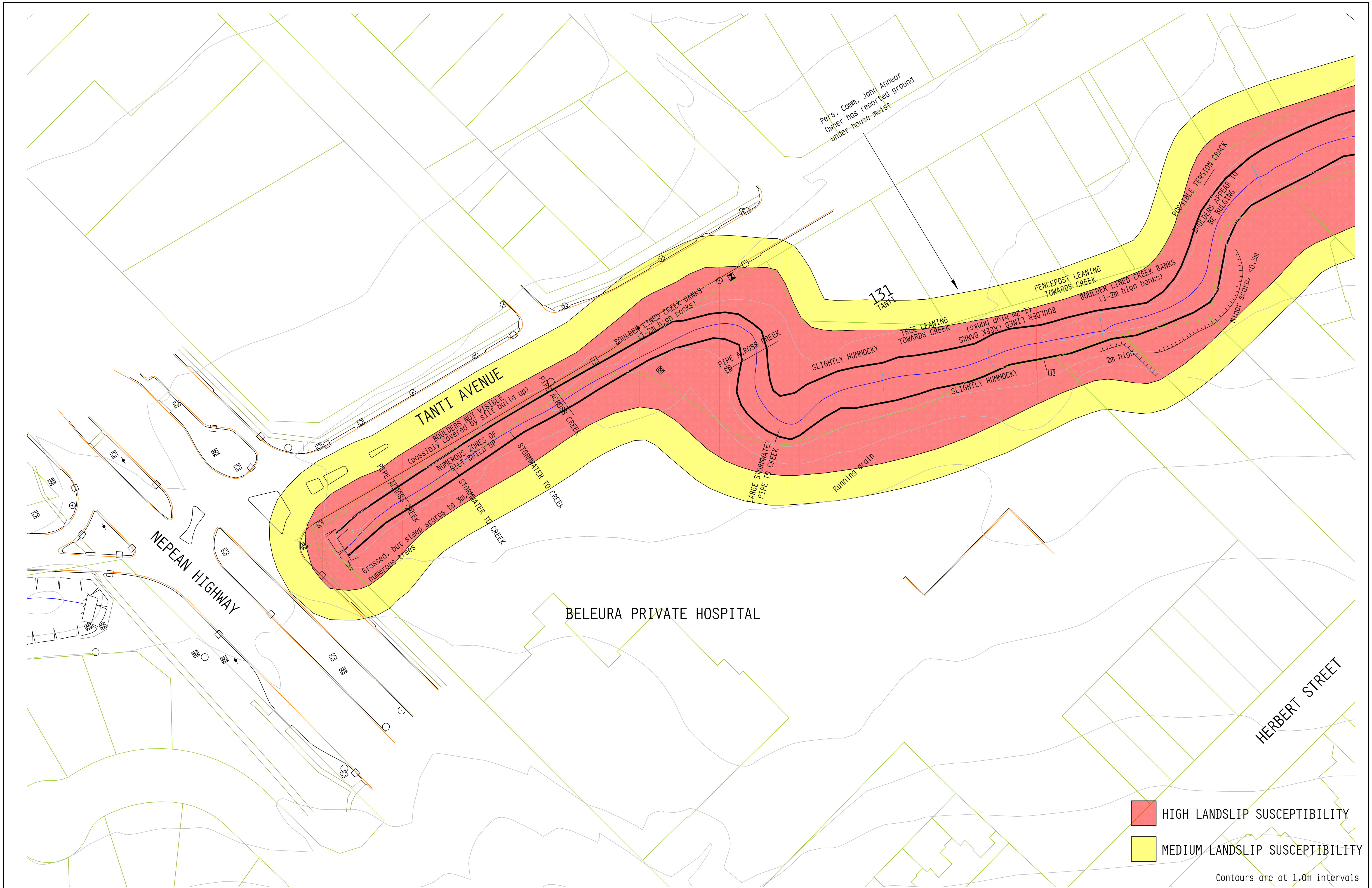
DESIGNATED STUDY AREA

HIGH LANDSLIP SUSCEPTIBILITY

MEDIUM LANDSLIP SUSCEPTIBILITY

Contours are at 1.0m intervals

PROJECT	LANE PIPER PTY LTD	TITLE	SCALE (A3) 1:1000	DATE 29 MAR 2010
CREEK BANK STABILITY TANTI CREEK, MORNINGTON	LanePiper A.C.N. 120 109 935 Geotechnical and Environmental Engineers Hydrogeologists and Environmental Scientists BLDG 2, 154 HIGHBURY RD, BURWOOD, VICTORIA, 3125 TELEPHONE (03) 9888 0100 FAX (03) 9808 3511	SITE PLAN	DRAWN DBS	JOB NO. 23019
		REF: 23019-Site Plan-3-10		REV. NO. 1 / FIG. NO. 5

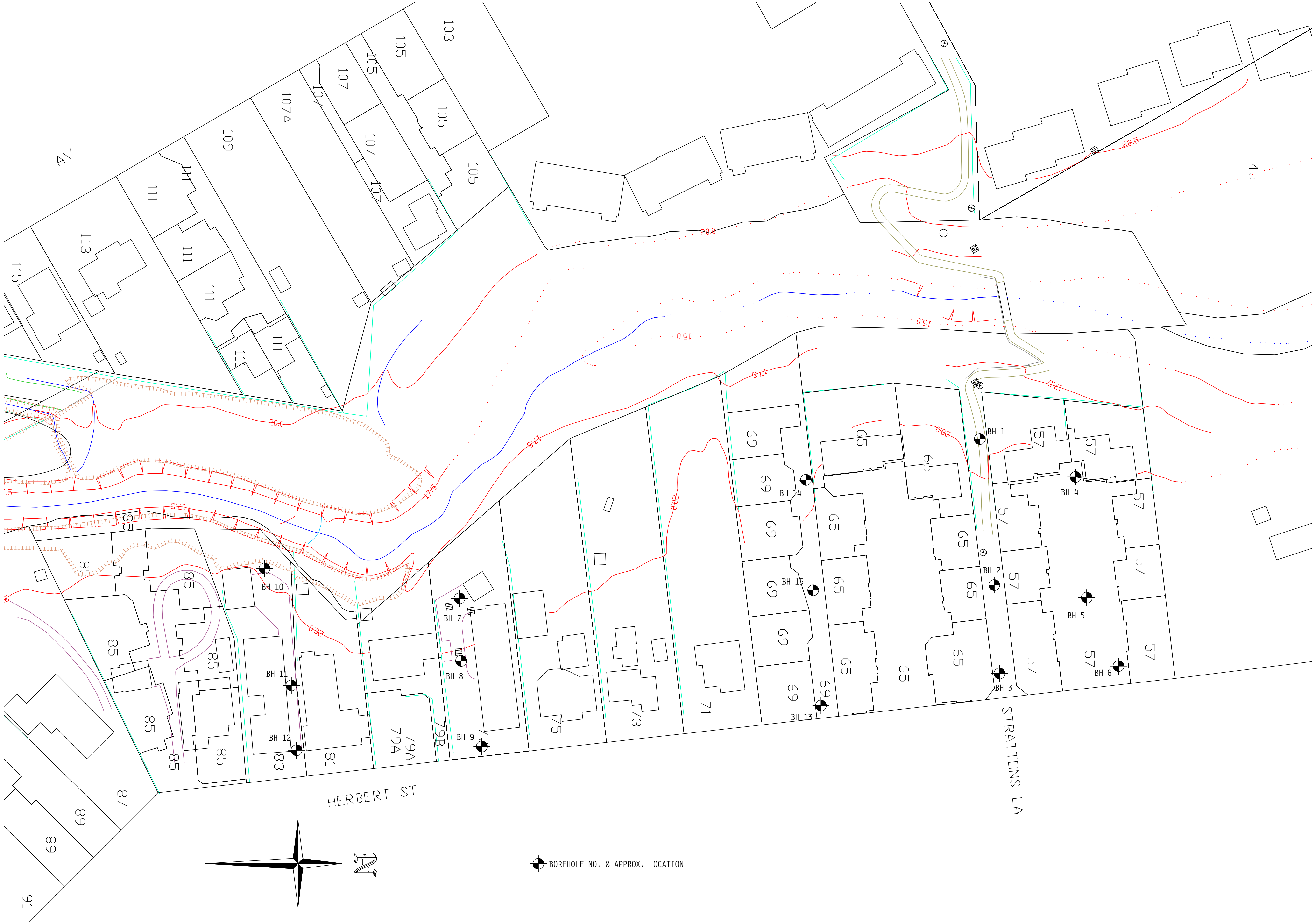


PROJECT	LANE PIPER PTY LTD	TITLE	SCALE (A3) 1:1000	DATE 29 MAR 2010
CREEK BANK STABILITY TANTI CREEK, MORNINGTON	LanePiper A.C.N. 120 109 935 Geotechnical and Environmental Engineers Hydrogeologists and Environmental Scientists BLDG 2, 154 HIGHBURY RD, BURWOOD, VICTORIA, 3125 TELEPHONE (03) 9888 0100 FAX (03) 9808 3511	SITE PLAN	DRAWN DBS	JOB NO. 23019
		REF: 23019-Site Plan-3-10		REV. NO. 1 FIG. NO. 6

Appendix B

22 Pages

SITE PLAN
RECORDS OF THE BOREHOLES
UNIFIED CLASSIFICATION SYSTEM



PROJECT	PIPER & ASSOCIATES PTY LTD	TITLE	JOB NO. 23019	DATE 9 OCT 2003
CREEK BANK STABILITY TANTI CREEK, MORNINGTON	(INCORPORATED IN VICTORIA) A.C.N. 006 802 393 GEOTECHNICAL & ENVIRONMENTAL CONSULTING ENGINEERS 1171 BURKE ROAD KEW, VICTORIA, AUSTRALIA 3101 TELEPHONE (03) 9816 9014 FAX (03) 9817 4861	Borehole Locations	SIZE A3	SCALE 1:1000
		REFERENCES OR NOTES		FIGURE NO. 1

PIPER AND ASSOCIATES PTY LTD

PROJECT:	**CREEK BANK STABILITY**	**BOREHOLE NO.:**	1
		DATE DRILLED:	16 - 20/12/02
LOCATION:	**TANTI CREEK, MORNINGTON**	**DRILL TYPE:**	SOLID AUGER
JOB NO.:	23019	**INCLINATION:**	VERTICAL
GROUND SURFACE (RL):		**LOGGED BY:** JN	**SHEET:** 1 OF 2

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
FILL clayey SILT (ML) medium plasticity, moderately fissured, stiff, grey-brown, moist, trace gravels.		0.00				A 25mm diameter PVC pipe (with a casagrande piezometer tip from 8.0m–8.3m) was inserted to the full depth of the borehole. The borehole was sand packed to 5.8m and then sealed with Bentonite to 5.4m before the borehole was backfilled with spoil. A steel borehole cover was concreted into place over the top of the borehole.
Silty CLAY (CI – CH) medium – high plasticity, moderately fissured, firm, dark grey, very moist, some reeds, trace fine gravel sands, moderate hydrogen sulphide odour.		0.2				
		1.5	U64		=70	
Sandy CLAY (CI) medium plasticity, moderately fissured, very stiff, pale grey, moist, sand, fine grained.		1.7				
		3.0	U64		=220	
Some orange-brown mottling, sand fine to medium grained at 3.5m.						
Clayey SAND (SC) low plasticity, dense, fine to medium grained, pale grey and orange-brown, very moist.		3.8				
		4.5	SPT			4 7 24 N=31
Some gravels 5.7 – 5.9m.						
Slightly clayey SAND (SC) very dense, fine grained, pale reddish-brown and pale orange-brown, very moist.		5.9				
		6.0	SPT			23 27 for 110mm N>50
Becoming pale yellow-brown and pale grey at 7.2m.						
		7.5	SPT			25 25 for 50mm Double Bouncing N>50

GROUNDWATER:
1.0m During Drilling.

PIPER AND ASSOCIATES PTY LTD

PROJECT:	**CREEK BANK STABILITY**	**BOREHOLE NO.:**	1
		DATE DRILLED:	16 - 20/12/02
LOCATION:	**TANTI CREEK, MORNINGTON**	**DRILL TYPE:**	SOLID AUGER
JOB NO.:	23019	**INCLINATION:**	VERTICAL
GROUND SURFACE (RL):		**LOGGED BY:** JN	**SHEET:** 2 OF 2

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
		8.0				
		8.5	SPT			6 25 19 for 50mm Double Bouncing N>50
		8.85				
END OF BOREHOLE 1 AT 8.85m Target Depth Reached						

GROUNDWATER:

Soil log.doc
REV: 3/00

Figure No. 3.

PIPER AND ASSOCIATES PTY LTD

PROJECT: CREEK BANK STABILITY
BOREHOLE NO.: 2
DATE DRILLED: 16 - 20/12/02
LOCATION: TANTI CREEK, MORNINGTON
DRILL TYPE: SOLID AUGER
JOB NO.: 23019
INCLINATION: VERTICAL
GROUND SURFACE (RL):
LOGGED BY: JN
SHEET: 1 OF 1

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
FILL clayey SILT (ML) medium plasticity, moderately fissured, stiff, grey-brown, moist and grass roots.		0.00				A 25mm diameter PVC pipe (with a casagrande piezometer tip from 5.7m–6.0m) was inserted to the full depth of the borehole. The borehole was sand packed to 4.1m and then sealed with Bentonite to 3.7m before the borehole was backfilled with spoil. A steel borehole cover was concreted into place over the top of the borehole.
Silty CLAY (CI) medium plasticity, moderately fissured, stiff, dark grey and grey, very moist, some fine grained sand, some reeds.		0.1				
		1.5	U64		=150	
Slightly sandy CLAY (CI) medium plasticity, moderately fissured, firm, pale grey, trace pale yellow-brown, moist, sand fine grained.		1.7				
		3.0	SPT			2 3 4 N=7
Becoming sandy clay, very dense, pale reddish-brown and pale orange-brown, very moist to saturated at 3.8m.						
		4.5	SPT			21 29 for 100mm N>50
Becoming pale grey at 5.4m.						
		6.0	SPT			5 5 13 N=18
		6.45				
END OF BOREHOLE 2 AT 6.45m Target Depth Reached						

GROUNDWATER:
4.0m During Drilling

PIPER AND ASSOCIATES PTY LTD

PROJECT:	CREEK BANK STABILITY	**BOREHOLE NO.:**	3
		DATE DRILLED:	16 - 20/12/02
LOCATION:	TANTI CREEK, MORNINGTON	**DRILL TYPE:**	SOLID AUGER
JOB NO.:	23019	**INCLINATION:**	VERTICAL
GROUND SURFACE (RL):		**LOGGED BY:** JN	**SHEET:** 1 OF 1

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
FILL silty CLAY (CI) medium plasticity, moderately fissured, very stiff, brown and grey-brown, slightly moist, some grass roots.		0.00				A 25mm diameter PVC pipe (with a casagrande piezometer tip from 5.7m–6.0m) was inserted to the full depth of the borehole. The borehole was sand packed to 4.1m and then sealed with Bentonite to 3.5m before the borehole was backfilled with spoil. A steel borehole cover was concreted into place over the top of the borehole.
Silty CLAY (CI) medium plasticity, moderately fissured, very stiff, pale grey and grey, moist to very moist, trace fine grained sand, some reeds.		0.2				
		1.5	U64		=220	
Slightly sandy CLAY (CI) medium plasticity, moderately fissured, firm, pale grey and pale orange-brown, moist, trace fine grained sand, trace ferruginised gravel.		1.8				
		3.0	SPT			3 3 3 N=6
Becoming sandy clay, very dense, pale grey, very moist at 3.7m.						
		4.5	SPT			2 23 28 N=51
Becoming clayey sand, medium dense, pale grey at 5.0m.						
		6.0	SPT			11 12 8 N=20
		6.45				
END OF BOREHOLE 3 AT 6.45m Target Depth Reached						

GROUNDWATER:
3.4m During Drilling

PIPER AND ASSOCIATES PTY LTD

PROJECT: CREEK BANK STABILITY
BOREHOLE NO.: 4
DATE DRILLED: 16 - 20/12/02
LOCATION: TANTI CREEK, MORNINGTON
DRILL TYPE: SOLID AUGER
JOB NO.: 23019
INCLINATION: VERTICAL
GROUND SURFACE (RL):
LOGGED BY: JN
SHEET: 1 OF 2

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
Reinforced concrete. Reinforcing 7mm diameter at 0.06m.		0.00				A 25mm diameter PVC pipe (with a casagrande piezometer tip from 8.5m–8.8m) was inserted to the full depth of the borehole. The borehole was sand packed to 7.3m and then sealed with Bentonite to 6.5m before the borehole was backfilled with spoil. A steel borehole cover was concreted into place over the top of the borehole.
FILL sandy GRAVEL (GP) appears well compacted, fine grained, blue-grey and dark grey, moist, trace copper wires.		0.12				
Silty CLAY (CI) medium plasticity, moderately fissured, stiff, dark grey, very moist, trace fine grained sand, some reeds, moderate hydrogen sulphide odour.		0.2				
Sandy CLAY (CI) very stiff, pale grey, very moist, sand fine grained.		1.2				
		1.5	U64		=350	
Becoming silty clay, stiff, pale grey and pale orange-brown, moist at 2.8m.		3.0	SPT			4 6 8 N=14
Clayey SAND (SC) low plasticity, medium dense, fine grained, pale grey, very moist.		3.5				
		4.5	SPT			4 5 6 N=11
Some sandy clay lenses at 6.0m.		6.0	SPT			6 6 7 N=13
Becoming very dense, fine to medium grained, pale grey and orange-brown, very moist, some ferruginised gravels at 6.8m.		7.5	SPT			7 24 8 for 20mm Double Bouncing N>50

GROUNDWATER:
1.2m During Drilling.

PIPER AND ASSOCIATES PTY LTD

PROJECT:	CREEK BANK STABILITY	BOREHOLE NO.:	4
		DATE DRILLED:	16 - 20/12/02
LOCATION:	TANTI CREEK, MORNINGTON	DRILL TYPE:	SOLID AUGER
JOB NO.:	23019	INCLINATION:	VERTICAL
GROUND SURFACE (RL):		LOGGED BY: JN	SHEET: 2 OF 2

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
		8.0				
		9.0	SPT			2 27 29 N=56
		9.45				
END OF BOREHOLE 4 AT 9.45m Target Depth Reached						

GROUNDWATER:

Soil log.doc
REV: 3/00

Figure No. 7.

PIPER AND ASSOCIATES PTY LTD

PROJECT:	CREEK BANK STABILITY	BOREHOLE NO.:	5
		DATE DRILLED:	16 - 20/12/02
LOCATION:	TANTI CREEK, MORNINGTON	DRILL TYPE:	SOLID AUGER
JOB NO.:	23019	INCLINATION:	VERTICAL
GROUND SURFACE (RL):		LOGGED BY: JN	SHEET: 1 OF 1

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
Rootmatter.		0.00				A 25mm diameter PVC pipe (with a casagrande piezometer tip from 5.8m–6.1m) was inserted to the full depth of the borehole. The borehole was sand packed to 5.0m and then sealed with Bentonite to 4.0m before the borehole was backfilled with spoil. A steel borehole cover was concreted into place over the top of the borehole.
FILL clayey SILT (ML) medium plasticity, moderately fissured, stiff, dark brown, slightly moist. Basalt gravels at 0.2m.		0.1				
Silty CLAY (CI) medium plasticity, moderately fissured, stiff, dark brown, moist.		0.5				
Sandy CLAY (CI) very stiff, pale grey, very moist, sand fine grained.		1.1				
		1.5	U64		=280	
Becoming slightly sandy silty clay, stiff, pale grey and pale orange-brown, moist at 2.7m.						
		3.0	SPT			4 5 7 N = 12
		4.5	SPT			5 8 13 N = 21
		6.0	SPT			8 14 30 N = 44
		6.45				
END OF BOREHOLE 5 AT 6.45m Target Depth Reached						

GROUNDWATER:
1.2m During Drilling

PIPER AND ASSOCIATES PTY LTD

PROJECT:	CREEK BANK STABILITY	BOREHOLE NO.:	6
		DATE DRILLED:	16 - 20/12/02
LOCATION:	TANTI CREEK, MORNINGTON	DRILL TYPE:	SOLID AUGER
JOB NO.:	23019	INCLINATION:	VERTICAL
GROUND SURFACE (RL):		LOGGED BY: JN	SHEET: 1 OF 1

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
Rootmatter.		0.00				A 25mm diameter PVC pipe (with a casagrande piezometer tip from 5.7m–6.0m) was inserted to the full depth of the borehole. The borehole was sand packed to 4.9m and then sealed with Bentonite to 3.0m before the borehole was backfilled with spoil. A steel borehole cover was concreted into place over the top of the borehole.
FILL clayey SILT (ML) medium plasticity, moderately fissured, stiff, dark brown, slightly moist, trace basalt gravels.		0.01				
Silty CLAY (CI) medium plasticity, moderately fissured, very stiff, pale grey, trace pale orange-brown mottling, moist, sand fine grained.		0.7				
		1.5	U64		=340	
Becoming pale grey and orange-brown, some ferruginised gravels at 2.1m.						
		3.0	SPT			6 8 8 N = 16
		4.5	SPT			4 5 9 N = 14
Becoming pale grey at 5.1m.						
		6.0	SPT			5 6 8 N = 14
		6.45				
END OF BOREHOLE 6 AT 6.45m Target Depth Reached						

GROUNDWATER:
1.0m During Drilling

PIPER AND ASSOCIATES PTY LTD

PROJECT:	CREEK BANK STABILITY	**BOREHOLE NO.:**	7
		DATE DRILLED:	16 - 20/12/02
LOCATION:	TANTI CREEK, MORNINGTON	**DRILL TYPE:**	SOLID AUGER
JOB NO.:	23019	**INCLINATION:**	VERTICAL
GROUND SURFACE (RL):		**LOGGED BY:** JN	**SHEET:** 1 OF 2

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
Reinforced concrete. Reinforcing 7mm diameter at 0.06m.		**0.00**				A 25mm diameter PVC pipe (slotted from 7.5m–10.5m) was inserted to the full depth of the borehole. The borehole was sand packed to 5.0m and then sealed with Bentonite to 4.5m before the borehole was backfilled with spoil. A steel borehole cover was concreted into place over the top of the borehole.
FILL / CRUSHED ROCK		**0.07**				
Silty CLAY (CI) medium plasticity, moderately fissured, stiff, dark grey and grey, very moist, some reeds, moderate hydrogen sulphide odour.		**0.12**				
Sandy CLAY (CI) medium plasticity, moderately fissured, very stiff, pale grey, moist, sand fine grained.		**1.0**				
		1.5	U64		=220	
Clayey SAND (SC) medium dense, fine grained, pale grey, very moist, some pale orange-brown mottling.		**2.8**				
		3.0	SPT			2 10 12 N=22
Slightly clayey SAND (SC) very dense, fine to medium grained, pale grey and orange-brown, very moist, trace ferruginised gravels to 5.0m.		**4.3**				
		4.5	SPT			20 30 for 100mm Double Bouncing N > 50
		6.0	SPT			15 20 for 100mm Double Bouncing N > 50
Becoming orange-brown, trace pale grey at 7.2m.						
		7.5	SPT			21 18 for 50mm Double Bouncing N > 50

GROUNDWATER:
2.0m During Drilling.

PIPER AND ASSOCIATES PTY LTD

PROJECT:	**CREEK BANK STABILITY**	**BOREHOLE NO.:**	7
		DATE DRILLED:	16 - 20/12/02
LOCATION:	**TANTI CREEK, MORNINGTON**	**DRILL TYPE:**	SOLID AUGER
JOB NO.:	23019	**INCLINATION:**	VERTICAL
GROUND SURFACE (RL):		**LOGGED BY:** JN	**SHEET:** 2 OF 2

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
		8.0				
		9.0				
Some ferruginised gravels 9.6 – 9.65m.						
		9.8	SPT			32 18 for 50mm Double Bouncing N > 50
Trace ferruginised gravels at 10.2m.						
		10.5 **10.65**	SPT			40 for 150mm Double Bouncing N > 50
END OF BOREHOLE 7 AT 10.65m Target Depth Reached						

GROUNDWATER:

PIPER AND ASSOCIATES PTY LTD

PROJECT:	CREEK BANK STABILITY	BOREHOLE NO.:	8
		DATE DRILLED:	16 - 20/12/02
LOCATION:	TANTI CREEK, MORNINGTON	DRILL TYPE:	SOLID AUGER
JOB NO.:	23019	INCLINATION:	VERTICAL
GROUND SURFACE (RL):		LOGGED BY: JN	SHEET: 1 OF 1

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
Rootmatter.		0.00				A 25mm diameter PVC pipe (with a casagrande piezometer tip from 5.7m–6.0m) was inserted to the full depth of the borehole. The borehole was sand packed to 4.9m and then sealed with Bentonite to 3.9m before the borehole was backfilled with spoil. A steel borehole cover was concreted into place over the top of the borehole.
FILL clayey SILT (ML) medium plasticity, moderately fissured, stiff, brown and grey-brown, moist to very moist, trace gravels.		0.05				
Silty CLAY (CI) medium plasticity, moderately fissured, stiff, dark grey, very moist, moderate hydrogen sulphide odour.		0.2				
Slightly sandy CLAY (CI) medium plasticity, moderately fissured, very stiff, pale grey, moist, sand fine grained.		1.1				
		1.5	U64		=250	
		3.0	SPT			4 4 8 N = 12
Slightly clayey SAND (SC) dense, fine grained, pale grey, trace orange-brown, wet.		3.4				
		4.5	SPT			13 19 16 N = 35
Some ferruginised gravels to 5.3 - 5.5m, becoming orange-brown, some pale grey.						
		6.0	SPT			11 17 17 N = 34
		6.45				
END OF BOREHOLE 8 AT 6.45m Target Depth Reached						

GROUNDWATER:
1.2m During Drilling

PIPER AND ASSOCIATES PTY LTD

PROJECT:	CREEK BANK STABILITY	BOREHOLE NO.:	9
		DATE DRILLED:	16 - 20/12/02
LOCATION:	TANTI CREEK, MORNINGTON	DRILL TYPE:	SOLID AUGER
JOB NO.:	23019	INCLINATION:	VERTICAL
GROUND SURFACE (RL):		LOGGED BY: JN	SHEET: 1 OF 1

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
Rootmatter.		0.00				A 25mm diameter PVC pipe (with a casagrande piezometer tip from 5.9m–6.2m) was inserted to the full depth of the borehole. The borehole was sand packed to 4.3m and then sealed with Bentonite to 3.6m before the borehole was backfilled with spoil. A steel borehole cover was concreted into place over the top of the borehole.
Clayey SILT (ML) medium plasticity, moderately fissured, stiff, grey-brown and brown, slightly moist.		0.05				
Silty CLAY (CI) medium plasticity, moderately fissured, pale grey-brown, moist. Becoming pale grey, trace pale orange-brown mottling, trace fine grained sand at 1.0m.		0.6				
		1.5	U64		=210	
Becoming slightly sandy clay, increase in orange-brown mottling at 2.5m.						
		3.0	SPT			5 8 11 N = 19
Slightly clayey SAND (SC) medium dense to dense, fine grained, pale grey and pale orange-brown, wet.		4.0				
		4.5	SPT			10 19 16 N = 35
Trace ferruginised gravels 5.5 – 5.6m.						
Becoming sandy clay at 6.35m.		6.0	SPT			6 10 11 N = 21
		6.45				
END OF BOREHOLE 9 AT 6.45m Target Depth Reached						

GROUNDWATER:
4.0m During Drilling

PIPER AND ASSOCIATES PTY LTD

PROJECT:	CREEK BANK STABILITY	BOREHOLE NO.:	10
		DATE DRILLED:	16 - 20/12/02
LOCATION:	TANTI CREEK, MORNINGTON	DRILL TYPE:	SOLID AUGER/WASHBORE
JOB NO.:	23019	INCLINATION:	VERTICAL
GROUND SURFACE (RL):		LOGGED BY: JN	SHEET: 1 OF 2

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
Rootmatter.		0.00				A 25mm diameter PVC pipe (with a casagrande piezometer tip from 8.4m–8.7m) was inserted to the full depth of the borehole. The borehole was sand packed to 4.8m and then sealed with Bentonite to 4.3m before the borehole was backfilled with spoil. A steel borehole cover was concreted into place over the top of the borehole.
FILL clayey SILT (ML) medium plasticity, moderately fissured, stiff, dark brown and brown, moist.		0.05				
Silty CLAY (CI) medium plasticity, moderately fissured, stiff, brown grading to pale grey, moist. Becoming sandy clay at 0.7m.		0.3				
		1.5	U64		=170	
		3.0	SPT			4 7 10 N = 17
Clayey SAND (SC) medium dense, fine grained, pale grey and orange-brown, very moist.		3.25				
		4.5	SPT			4 8 20 N = 28
Trace ferruginised gravels 6.0 to 6.05m, trace coarse grained quartz sand.		6.0	SPT			20 16 13 N = 29
Silty CLAY (CI) medium plasticity, moderately fissured, stiff, pale grey and pale orange-brown, moist, trace ferruginised gravels.		7.1				
		7.5	SPT			3 4 7 N = 11

GROUNDWATER:
2.9m During Drilling.

PIPER AND ASSOCIATES PTY LTD

PROJECT: CREEK BANK STABILITY
BOREHOLE NO.: 10
DATE DRILLED: 16 - 20/12/02
LOCATION: TANTI CREEK, MORNINGTON
DRILL TYPE: SOLID AUGER/WASHBORE
JOB NO.: 23019
INCLINATION: VERTICAL
GROUND SURFACE (RL):
LOGGED BY: JN
SHEET: 2 OF 2

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
		8.0				
Slightly clayey SAND (SC) dense, fine grained, pale yellow-grey and pale orange-brown.		8.3 9.0 9.45	SPT			4 9 31 N = 40
END OF BOREHOLE 10 AT 9.45m Target Depth Reached						

GROUNDWATER:

Soil log.doc
REV: 3/00

Figure No. 15.

PIPER AND ASSOCIATES PTY LTD

PROJECT:	CREEK BANK STABILITY	BOREHOLE NO.:	11
		DATE DRILLED:	16 - 20/12/02
LOCATION:	TANTI CREEK, MORNINGTON	DRILL TYPE:	SOLID AUGER
JOB NO.:	23019	INCLINATION:	VERTICAL
GROUND SURFACE (RL):		LOGGED BY: JN	SHEET: 1 OF 1

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
FILL sandy clayey SILT (ML) medium plasticity, moderately fissured, stiff, brown, moist, some gravels.		0.00				A 25mm diameter PVC pipe (with a casagrande piezometer tip from 5.8m–6.1m) was inserted to the full depth of the borehole. The borehole was sand packed to 4.5m and then sealed with Bentonite to 3.3m before the borehole was backfilled with spoil. A steel borehole cover was concreted into place over the top of the borehole.
Silty CLAY (CI) medium plasticity, moderately fissured, very stiff, pale grey, moist.		0.5				
Some fine grained sand at 1.7m.		1.5	U64		=290	
		3.0	SPT			4 6 8 N = 14
Slightly clayey SAND (SC) dense, fine grained, pale grey, very moist.		3.7				
		4.5	SPT			10 24 13 N = 37
Slightly clayey SAND (SC) medium dense, fine grained, orange-brown, trace ferruginised gravels, very moist.		4.8				
Becoming sandy clay at 5.9m.						
		6.0	SPT			7 11 13 N = 24
		6.45				
END OF BOREHOLE 11 AT 6.45m Target Depth Reached						

GROUNDWATER:
3.5m During Drilling

PIPER AND ASSOCIATES PTY LTD

PROJECT:	**CREEK BANK STABILITY**	**BOREHOLE NO.:**	12
		DATE DRILLED:	16 - 20/12/02
LOCATION:	**TANTI CREEK, MORNINGTON**	**DRILL TYPE:**	SOLID AUGER
JOB NO.:	23019	**INCLINATION:**	VERTICAL
GROUND SURFACE (RL):		**LOGGED BY:** JN	**SHEET:** 1 OF 1

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
FILL sandy clayey SILT (ML) stiff, brown, moist, some gravels.		0.00				A 25mm diameter PVC pipe (with a casagrande piezometer tip from 5.7m–6.0m) was inserted to the full depth of the borehole. The borehole was sand packed to 4.8m and then sealed with Bentonite to 4.3m before the borehole was backfilled with spoil. A steel borehole cover was concreted into place over the top of the borehole.
Sandy CLAY (CI) medium plasticity, moderately fissured, hard, pale grey, moist, sand fine grained.		0.6				
		1.5	U64		>600	
Becoming very stiff at 2.2m.						
		3.0	SPT			4 8 9 N = 17
Slightly clayey SAND (SC) dense, fine grained, pale grey, very moist – wet.		3.7				
		4.5	SPT			4 7 8 N = 15
		6.0	SPT			6 13 19 N = 32
		6.45				
END OF BOREHOLE 12 AT 6.45m Target Depth Reached						

GROUNDWATER:
3.7m During Drilling

PIPER AND ASSOCIATES PTY LTD

PROJECT:	**CREEK BANK STABILITY**	**BOREHOLE NO.:**	13
		DATE DRILLED:	16 - 20/12/02
LOCATION:	**TANTI CREEK, MORNINGTON**	**DRILL TYPE:**	SOLID AUGER
JOB NO.:	23019	**INCLINATION:**	VERTICAL
GROUND SURFACE (RL):		**LOGGED BY:** JN	**SHEET:** 1 OF 1

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
FILL sandy GRAVEL (GP) medium dense, fine grained, blue-grey, slightly moist.		0.00				A 25mm diameter PVC pipe (with a casagrande piezometer tip from 5.6m–5.9m) was inserted to the full depth of the borehole. The borehole was sand packed to 4.8m and then sealed with Bentonite to 4.3m before the borehole was backfilled with spoil. A steel borehole cover was concreted into place over the top of the borehole.
Silty CLAY (CI) medium plasticity, moderately fissured, very stiff, grey-brown and brown grading to pale grey and pale orange-brown, moist.		0.3				
		1.5	U64		=310	
Slightly clayey SAND (SC) dense, fine grained, pale grey and pale orange-brown, very moist.		2.6				
		3.0	SPT			11 16 15 N = 31
Trace pale green-grey mottling at 4.5m.		4.5	SPT			17 19 18 N=37
Becoming medium grained at 5.7m.						
		6.0	SPT			6 6 8 N = 14
		6.45				No recovery.
END OF BOREHOLE 13 AT 6.45m. Target Depth Reached						

GROUNDWATER:
2.6m During Drilling

Soil log.doc
REV: 3/00

Figure No. 18.

PIPER AND ASSOCIATES PTY LTD

PROJECT:	CREEK BANK STABILITY	BOREHOLE NO.:	14
		DATE DRILLED:	16 - 20/12/02
LOCATION:	TANTI CREEK, MORNINGTON	DRILL TYPE:	SOLID AUGER
JOB NO.:	23019	INCLINATION:	VERTICAL
GROUND SURFACE (RL):		LOGGED BY: JN	SHEET: 1 OF 2

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
FILL gravelly silty CLAY (CI) stiff, grey-brown, moist to very moist.		0.00				A 25mm diameter PVC pipe (with a casagrande piezometer tip from 9.4m–9.7m) was inserted to the full depth of the borehole. The borehole was sand packed to 7.6m and then sealed with Bentonite to 6.1m before the borehole was backfilled with spoil. A steel borehole cover was concreted into place over the top of the borehole.
Sandy CLAY (CI) medium plasticity, moderately fissured, very stiff, pale grey, moist, sand fine to medium grained.		0.5				
		1.5	U64		=320	
Becoming stiff, sand fine grained at 2.7m		2.7				
		3.0	SPT			3 4 6 N = 10
Clayey SAND (SC) medium dense, fine grained, pale grey, very moist.		3.7				
		4.5	SPT			5 5 10 N = 15
Becoming slightly clayey sand, pale orange-brown, trace ferruginised gravel at 5.3m		5.3				
		6.0	SPT			11 16 13 N = 29
		7.0				
Becoming very dense, orange-brown, fine to medium grained sand at 7.0m		7.5	SPT			22 15 for 20mm Double Bouncing N > 50

GROUNDWATER:
2.5m During Drilling

PIPER AND ASSOCIATES PTY LTD

PROJECT:	**CREEK BANK STABILITY**	**BOREHOLE NO.:**	14
		DATE DRILLED:	16 - 20/12/02
LOCATION:	**TANTI CREEK, MORNINGTON**	**DRILL TYPE:**	SOLID AUGER
JOB NO.:	23019	**INCLINATION:**	VERTICAL
GROUND SURFACE (RL):		**LOGGED BY:** JN	**SHEET:** 2 OF 2

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
		8.0				
		9.0				
		9.5	SPT			32 18 for 50mm Double Bouncing N > 50
		11.2				
END OF BOREHOLE 14 AT 11.2m Target Depth Reached						

GROUNDWATER:

PIPER AND ASSOCIATES PTY LTD

PROJECT:	CREEK BANK STABILITY	**BOREHOLE NO.:**	15
		DATE DRILLED:	16 - 20/12/02
LOCATION:	TANTI CREEK, MORNINGTON	**DRILL TYPE:**	SOLID AUGER
JOB NO.:	23019	**INCLINATION:**	VERTICAL
GROUND SURFACE (RL):		**LOGGED BY:** JN	**SHEET:** 1 OF 1

STRATIGRAPHY	GRAPHIC LOG	DEPTH (m)	SAMPLE	SAMPLE PUSHED & RECOVERED	pp Value	BORE COMPLETION DETAILS
FILL sandy GRAVEL (GP) medium dense, fine grained, blue-grey and pale brown, slightly moist.		**0.00**				A 25mm diameter PVC pipe (slotted from 3.6m–5.6m) was inserted to the full depth of the borehole. The borehole was sand packed to 3.1m and then sealed with Bentonite to 2.7m before the borehole was backfilled with spoil. A steel borehole cover was concreted into place over the top of the borehole.
Very silty CLAY (CI) medium plasticity, moderately fissured, firm, grey-brown, moist.		**0.3**				
Becoming silty clay at 0.7m		**0.7**				
Becoming pale grey at 0.9m		**0.9**				
Sandy **CLAY (CI)** medium plasticity, moderately fissured, firm, pale grey, very moist, sand, fine grained.		**1.0**				
		1.5	SPT			2 2 3 N = 5
Clayey SAND (SC) medium dense, fine grained, pale grey, very moist.		**2.9**				
		3.0	SPT			5 8 6 N = 14
		4.5	SPT			6 9 15 N = 24
Becoming orange-brown, trace ferruginised at 5.2m.		**5.2**				
		6.0	SPT			19 33 for 100mm N > 50
END OF BOREHOLE 15 AT 6.0m. Target Depth Reached						

GROUNDWATER:
1.1m During Drilling

Piper & Associates Pty Ltd

UNIFIED CLASSIFICATION SYSTEM

(in accordance with AS1726 – 1993)

PARTICLE SIZES

TERM	SIZE (mm)
BOULDER	>200
COBBLE	60 to 200
GRAVEL	
Coarse	20 to 60
Medium	6 to 20
Fine	2 to 6
SAND	
Coarse	0.6 to 2
Medium	0.2 to 0.6
Fine	0.06 to 0.2
SILT	0.002 to 0.06
CLAY	< 0.002

COHESIVE SOILS

TERM	UNDRAINED SHEAR STRENGTH (kPa)
Very Soft	0 to 12.5
Soft	12.5 to 25
Firm	25 to 50
Stiff	50 to 100
Very Stiff	100 to 200
Hard	≥ 200

COHESIONLESS SOILS

TERM	'N' (SPT) VALUE (blows / 300mm)	RELATIVE DENSITY (%)	ANGLE SHEAR RESISTANCE (degrees)
Very Loose	0 to 4	< 15	25 to 30
Loose	4 to 10	15 to 35	27 to 32
Medium Dense	10 to 30	35 to 65	30 to 35
Dense	30 to 50	65 to 85	35 to 40
Very Dense	> 50	≥ 85	38 to 43

STRUCTURE

TERM	SIZE OF BLOCKS (mm)
Blocky	> 60
Cloddy	20 to 60
Nutty	6 to 20
Granular	0.6 to 6
Prismatic	Stated
Shattered	< 10

SAMPLES

U = undisturbed tube sample
D = disturbed sample
SPT(*9*) = standard penetrometer test (*blows per 300 mm*) (63.5 kg hammer dropped 760mm)
BS = bulk sample
C = contamination jar sample
V = head space vial sample

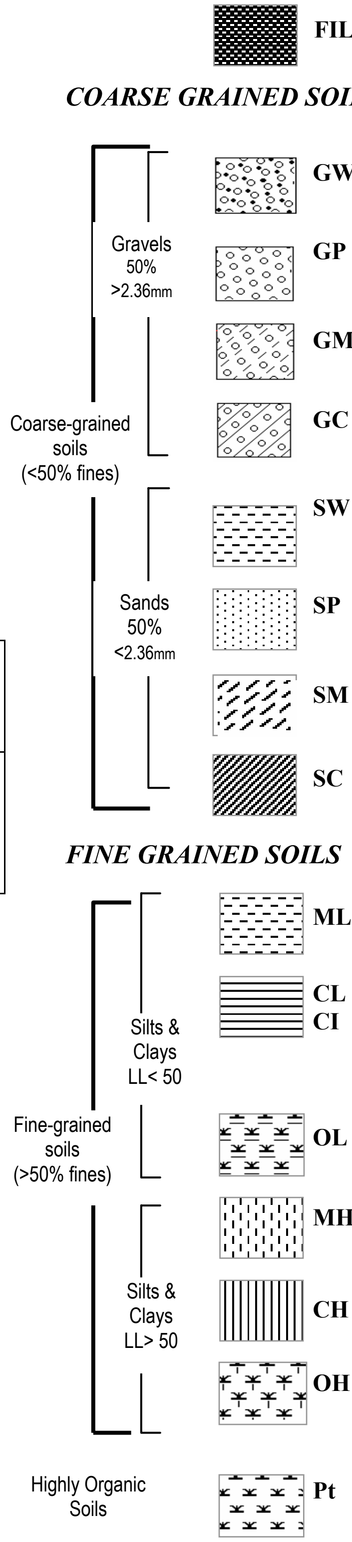
INDEX PROPERTIES

ρ = bulk density (t/m^3)
DD = dry density (t/m^3)
mc = natural moisture content (%)
LL = Liquid Limit (%)
PL = Plastic Limit (%)
PI = Plastic Index (%)
LS = Linear Shrinkage (%)

GROUNDWATER

GW = Groundwater depth (m) or level (RL)

IDENTIFICATION OF SOILS



GW	Well graded gravels and gravel-sand mixtures, little or no fines
GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
GM	Silty gravels and gravel-sand-clay mixtures
GC	Clayey gravels, gravel-sand-clay mixture
SW	Well graded sands and gravelly sands, little or no fines
SP	Poorly graded sands and gravelly sands, little or no fines
SM	Silty sand, sandy silt mixture
SC	Clayey sands, sandy clay mixtures
ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
CL CI	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
OL	Organic silts and organic silty clays of low plasticity
MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
CH	Inorganic clays of high plasticity
OH	Organic clays of medium to high plasticity
Pt	Peat muck and other highly organic soils

FIELD TESTS

W = Field permeability
P = Pressuremeter test
R = Refusal
pp = Pocket penetrometer (kPa)
ID = Insitu density test

Appendix C

19 Pages

LABORATORY TEST RESULTS

PIPER & ASSOCIATES PTY. LTD.

1171 Burke Road, Kew 3101 NATA Accreditation No. 3145

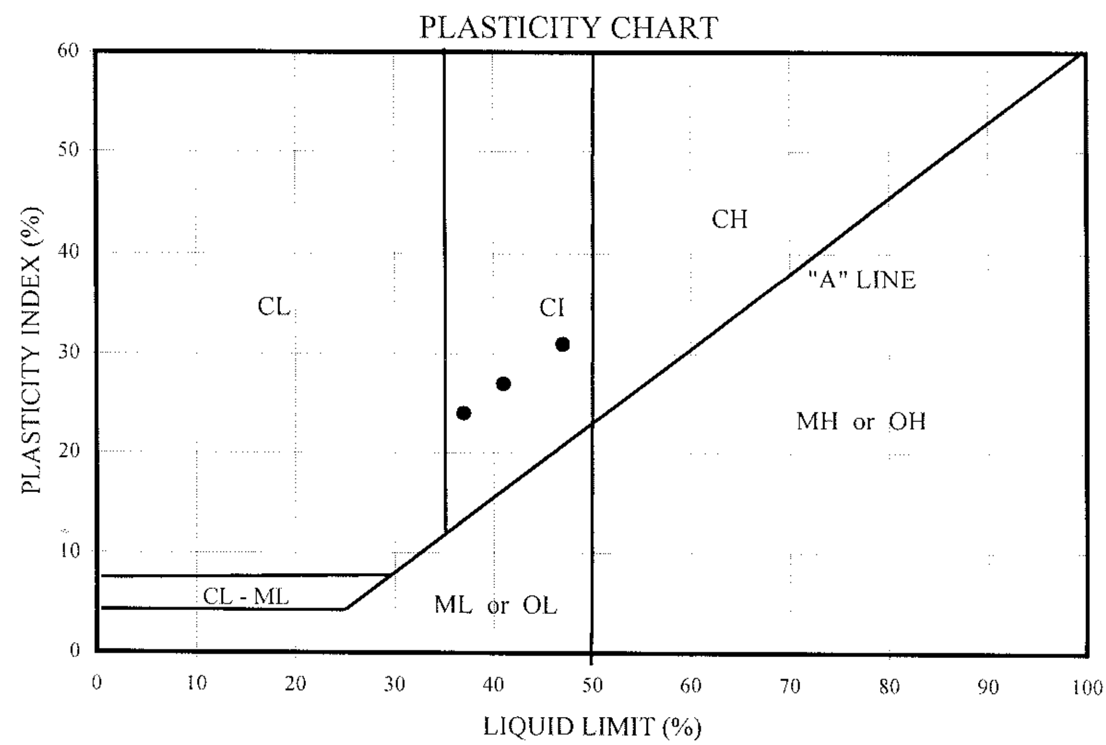
ATTERBERG LIMITS & UNIFIED CLASSIFICATION

These tests were carried out in accordance with Australian Standard 1289, 3.1.2 - 1995 (One-Point Method), 3.2.1 - 1995, 3.3.1 - 1995, and 2.1.1 - 1992.
The test was carried out only on the part of the soil finer than 0.425mm

PROJECT: TESTING OF CREEK BANK
JOB/DOC No.: 23019/1
OPERATOR: GDH
CLIENT: MORNINGTON PENINSULA S. C.
DATE: 17/02/03
LOCATION: TANTI CREEK, MORNINGTON
ISSUE DATE: 3/03/03

SAMPLE No.			
TESTPIT/BOREHOLE	BH2	BH10	BH15
DEPTH (M)	1.5	1.5	1.5
PREPARATION METHOD	AIR DRIED DRY SIEVED	AIR DRIED DRY SIEVED	AIR DRIED DRY SIEVED
SAMPLE DESCRIPTION	silty CLAY grey, sandy	silty CLAY grey, slightly sandy	silty CLAY grey, slightly sandy
GROUP SYMBOL	CI	CI	CI
PLASTIC LIMIT (%)	13	16	14
LIQUID LIMIT (%)	37	47	41
PLASTICITY INDEX (%)	24	31	27

Plasticity Chart for classification of fine grained soil - Table A1 AS1726 - 1993



COMMENTS:

Test authorized by: Graham Hodgson
Laboratory Manager

National Association of Testing Authorities, Australia
NATA ENDORSED TEST REPORT
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3/3/03

ATTERBERG.XLS LABSHEET
REV: 6/02

Figure No. 1.

PIPER & ASSOCIATES PTY. LTD.

1171 Burke Rd. Kew 3101 NATA Accreditation No. 3145

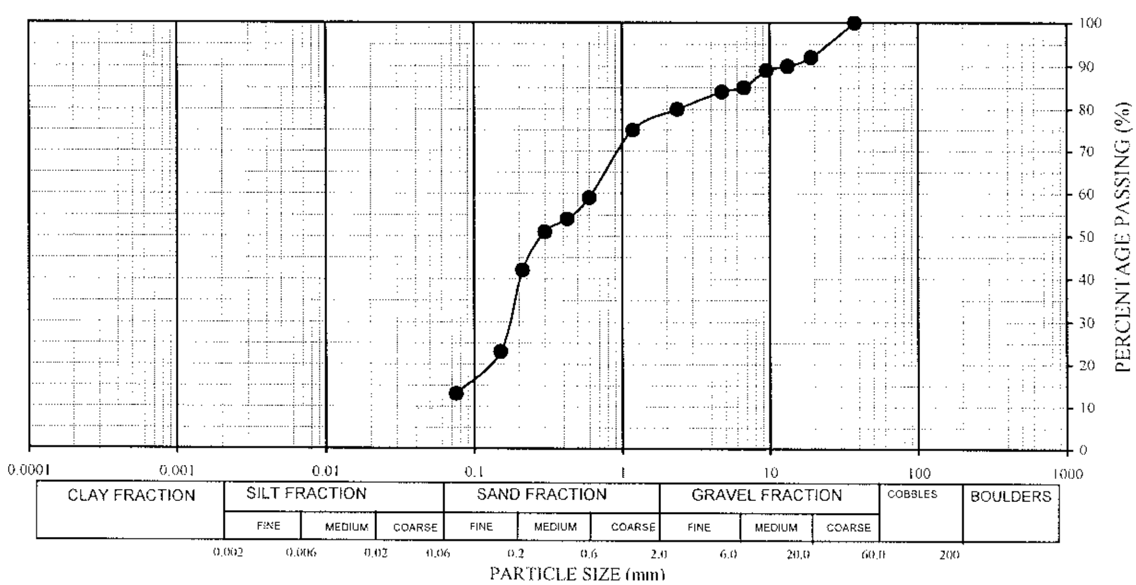
DETERMINATION OF PARTICLE SIZE DISTRIBUTION OF A SOIL
STANDARD METHOD OF ANALYSIS BY SIEVING

This test was carried out in accordance with Australian Standard 1289.3.6.1 - 1995

PROJECT: TESTING OF CREEK BANK **JOB/DOC. No.:** 23019/2
CLIENT: MORNINGTON PENINSULA S. C. **DATE:** 13/02/03
LOCATION: TANTI CREEK, MORNINGTON **ISSUE DATE:** 3/03/03
SAMPLE SOURCE: BH4 7.5m **OPERATOR:** GDH
SAMPLE DESCRIPT.: gravelly SAND brown, slightly clayey

TOTAL DRY MASS OF SAMPLE: 495.5 grams

SIEVE APERTURE (MM)	PERCENTAGE PASSING OF TOTAL	SIEVE APERTURE (Micron)	PERCENTAGE PASSING OF TOTAL
37.5	100	600	59
19	92	425	54
13.2	90	300	51
9.5	89	212	42
6.7	85	150	23
4.75	84	75	13
2.36	80		
1.18	75		



REMARKS:

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Test authorized by: Graham Hodgson
Laboratory Manager

SIEVE.XLS LABSHEET
REV: 6/02

Figure No. 2.

PIPER & ASSOCIATES PTY. LTD.

1171 Burke Rd, Kew 3101 NATA Accreditation No. 3145

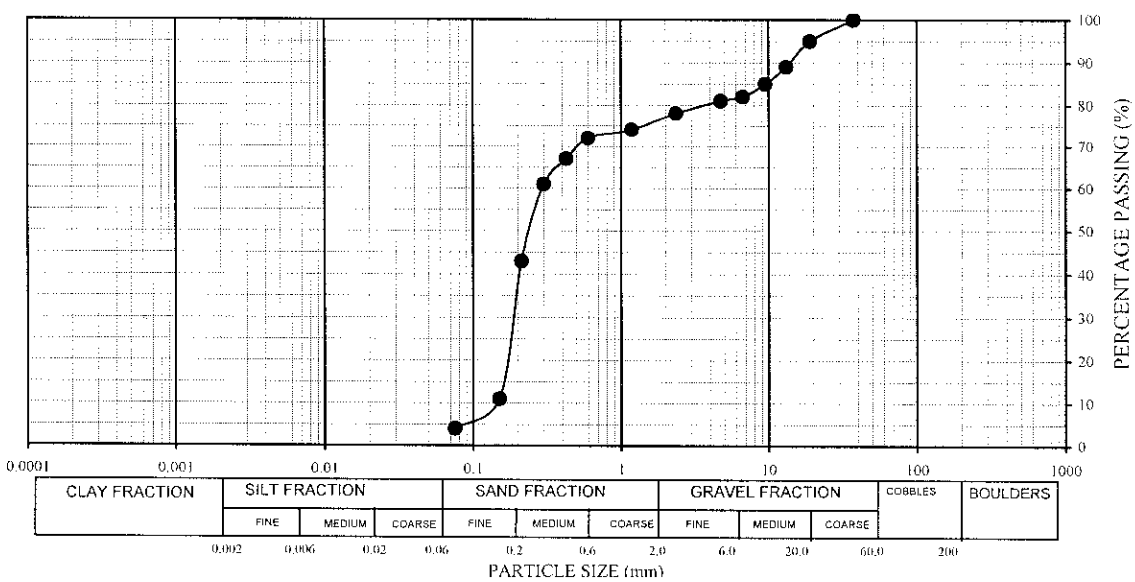
DETERMINATION OF PARTICLE SIZE DISTRIBUTION OF A SOIL
STANDARD METHOD OF ANALYSIS BY SIEVING

This test was carried out in accordance with Australian Standard 1289.3.6.1 - 1995

PROJECT: TESTING OF CREEK BANK **JOB/DOC. No.:** 23019/3
CLIENT: MORNINGTON PENINSULA S. C. **DATE:** 13/02/03
LOCATION: TANTI CREEK, MORNINGTON **ISSUE DATE:** 3/03/03
SAMPLE SOURCE: BH7 9.8m **OPERATOR:** GDH
SAMPLE DESCRIPT.: gravelly SAND yellow-brown, slightly clayey

TOTAL DRY MASS OF SAMPLE: 247.3 grams

SIEVE APERTURE (MM)	PERCENTAGE PASSING OF TOTAL	SIEVE APERTURE (Micron)	PERCENTAGE PASSING OF TOTAL
37.5	100	600	72
19	95	425	67
13.2	89	300	61
9.5	85	212	43
6.7	82	150	11
4.75	81	75	4
2.36	78		
1.18	74		



REMARKS:

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Test authorized by: Graham Hodgson
Laboratory Manager

3/3/03

SIEVE.XLS LABSHEET
REV: 6/02

Figure No. 3

PIPER & ASSOCIATES PTY. LTD.

1171 Burke Rd. Kew 3101 NATA Accreditation No. 3145

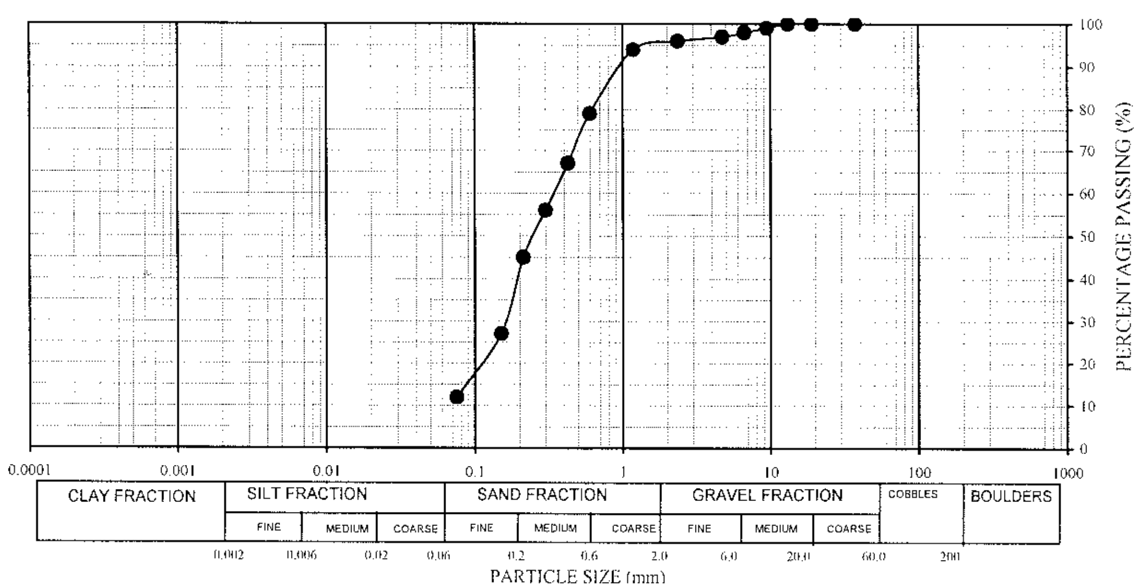
DETERMINATION OF PARTICLE SIZE DISTRIBUTION OF A SOIL
STANDARD METHOD OF ANALYSIS BY SIEVING

This test was carried out in accordance with Australian Standard 1289.3.6.1 - 1995

PROJECT: TESTING OF CREEK BANK **JOB/DOC. No.:** 23019/4
CLIENT: MORNINGTON PENINSULA S. C. **DATE:** 13/02/03
LOCATION: TANTI CREEK, MORNINGTON **ISSUE DATE:** 3/03/03
SAMPLE SOURCE: BH14 6.0m **OPERATOR:** GDH
SAMPLE DESCRIPT.: SAND light brown, slightly clayey, slightly fine gravelly

TOTAL DRY MASS OF SAMPLE: 400.2 grams

SIEVE APERTURE (MM)	PERCENTAGE PASSING OF TOTAL	SIEVE APERTURE (Micron)	PERCENTAGE PASSING OF TOTAL
37.5	100	600	79
19	100	425	67
13.2	100	300	56
9.5	99	212	45
6.7	98	150	27
4.75	97	75	12
2.36	96		
1.18	94		



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Laboratory Manager

3/3/03

SIEVE.XLS LABSHEET
REV: 6/02

Figure No. 4.

PIPER & ASSOCIATES PTY. LTD.

1171 Burke Rd. Kew 3101 NATA Accreditation No: 3145

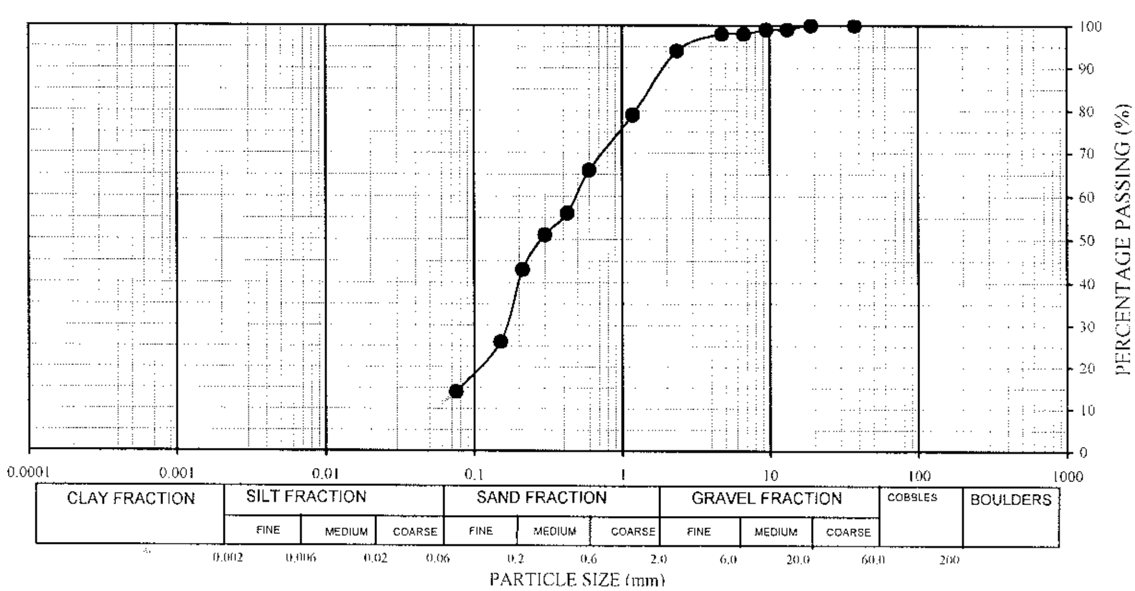
DETERMINATION OF PARTICLE SIZE DISTRIBUTION OF A SOIL
STANDARD METHOD OF ANALYSIS BY SIEVING

This test was carried out in accordance with Australian Standard 1289.3.6.1 - 1995

PROJECT: TESTING OF CREEK BANK **JOB/DOC. No.:** 23019/5
CLIENT: MORNINGTON PENINSULA S. C. **DATE:** 17/02/03
LOCATION: TANTI CREEK, MORNINGTON **ISSUE DATE:** 3/03/03
SAMPLE SOURCE: BH15 6.0m **OPERATOR:** GDH
SAMPLE DESCRIPT.: clayey SAND brown, slightly gravelly

TOTAL DRY MASS OF SAMPLE: 555.9 grams

SIEVE APERTURE (MM)	PERCENTAGE PASSING OF TOTAL	SIEVE APERTURE (Micron)	PERCENTAGE PASSING OF TOTAL
37.5	100	600	66
19	100	425	56
13.2	99	300	51
9.5	99	212	43
6.7	98	150	26
4.75	98	75	14
2.36	94		
1.18	79		



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Laboratory Manager

3/3/03

SIEVE.XLS LABSHEET
REV: 6/02

Figure No. 5.

Piper & Associates Pty Ltd

1171 Burke Road, Kew 3101

Determination of the Percent Passing a 75μm Sieve

PROJECT:	TESTING OF CREEK BANK	JOB/DOC. No.:	23019/6
CLIENT:	MORNINGTON PENINSULA S.C.	DATE:	7/3/03
LOCATION:	TANTI CREEK, MORNINGTON	ISSUE DATE:	7/5/03
		OPERATOR:	GDH

SAMPLE DESCRIPTIONS

Test No.	Sample Location	Depth (M)	Sample Description
1	BH14	1.5	clayey SAND light grey

PERCENTAGE PASSING

TEST No.	TOTAL DRY MASS (gm)	% PASSING 75μm SIEVE
1	416.2	49

REMARKS:

Test authorized by: Graham Hodgson
Laboratory Manager

PIPER & ASSOCIATES PTY LTD

1171 Burke Road, Kew 3101 NATA Accreditation No. 3145

DETERMINATION OF THE MULTI STAGE UNDRAINED TRIAXIAL COMPRESSIVE STRENGTH OF A SOIL WITHOUT MEASUREMENT OF A PORE WATER PRESSURE

This test was carried out in accordance with AS 1289 6.4.1-1998, but without the stress-strain plot[1]
Moisture Content Determination carried out in accordance with AS 1289 Method 2.1.1 - 1992.

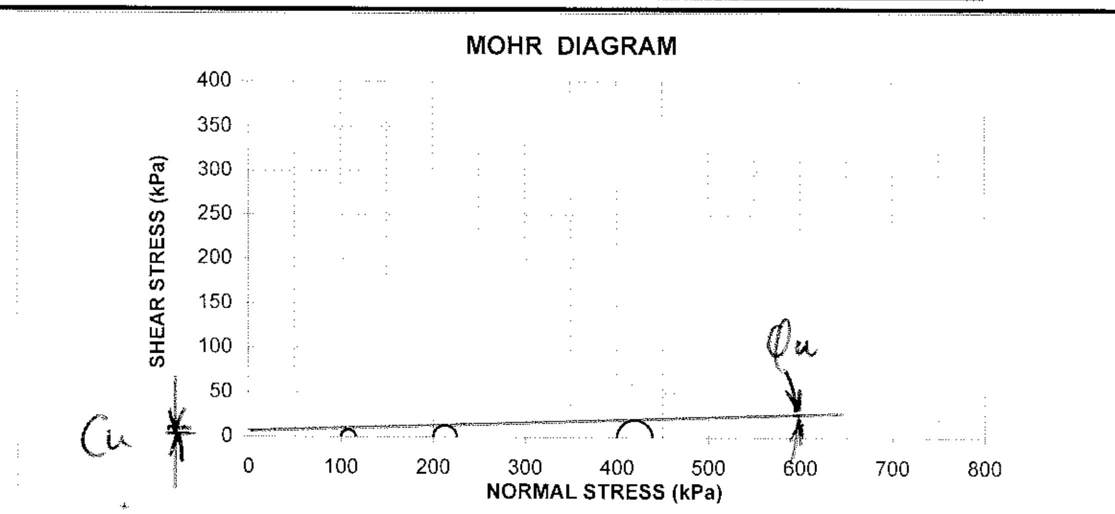
PROJECT: TESTING OF CREEK BANK
JOB\ DOC. NO.: 23019/7
DATE SAMPLED: 23/12/02
CLIENT: MORNINGTON PENINSULA S.C.
DATE TESTED: 7/03/03
LOCATION: TANTI CREEK, MORNINGTON
ISSUE DATE: 7/05/03
BOREHOLE/ TESTPIT No. : BH1
OPERATOR: GDH
DEPTH (m): 1
SAMPLE METHOD: HYD PUSH
SAMPLER TYPE: TUBE
END CONDITIONS: FRICTION
RATE OF FEED: 1.3 mm/min
SPECIMEN CONDITION: UNDISTURBED
COMMENTS: 0
SAMPLE DESCRIPTION: clayey SILT black (organic)

SAMPLE DETAILS

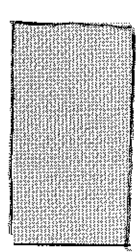
INITIAL DIAMETER OF SAMPLE:	63.5 mm
INITIAL LENGTH OF SAMPLE:	127 mm
INITIAL MASS OF SAMPLE:	551.88 grams
MOISTURE CONTENT:	88.7 %
DRY DENSITY:	0.73 t/m^3

TEST DETAILS

	STAGE 1	STAGE 2	STAGE 3
CELL PRESSURE (kPa):	100	200	400
DEVIATOR STRESS (kPa):	16	26	39
STRAIN TO FAILURE (%)	1.8	3.9	6.7



MODE OF FAILURE



APPARENT COHESION (Cu) = 8 kPa
ANGLE OF SHEARING RESISTANCE (ϕu) = 2 Degrees

Test authorized by: Graham Hodgson
Laboratory Manager

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[1] Stress- strain plot available upon request.

UUTRIAX-w'out stress-strain plot.XLS LABSHEET
REV: 6/02

PIPER & ASSOCIATES PTY LTD

1171 Burke Road, Kew 3101   NATA Accreditation No. 3145

DETERMINATION OF THE MULTI STAGE UNDRAINED TRIAXIAL COMPRESSIVE STRENGTH OF A SOIL WITHOUT MEASUREMENT OF A PORE WATER PRESSURE

This test was carried out in accordance with AS 1289 6.4.1-1998, but without the stress-strain plot[1]
Moisture Content Determination carried out in accordance with AS 1289 Method 2.1.1 - 1992.

PROJECT: TESTING OF CREEK BANK
CLIENT: MORNINGTON PENINSULA S.C.
LOCATION: TANTI CREEK, MORNINGTON
BOREHOLE/ TESTPIT No. : BH4
DEPTH (m): 1.5
SAMPLER TYPE: TUBE
RATE OF FEED: 1.3 mm/min
SPECIMEN CONDITION: UNDISTURBED
COMMENTS: 0
SAMPLE DESCRIPTION: sandy CLAY grey

JOB\ DOC. NO.: 23019/8
DATE SAMPLED: 23/12/02
DATE TESTED: 7/03/03
ISSUE DATE: 7/05/03
OPERATOR: GDH
SAMPLE METHOD: HYD PUSH
END CONDITIONS: FRICTION

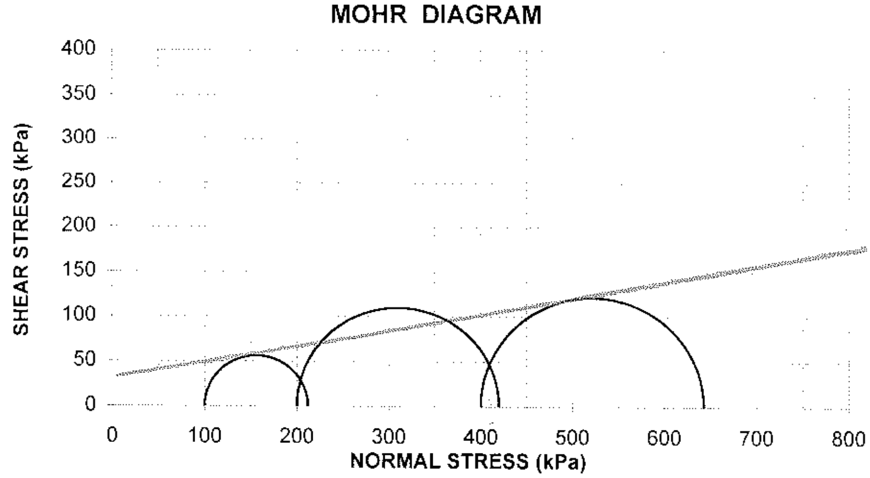
SAMPLE DETAILS

INITIAL DIAMETER OF SAMPLE:	63.5 mm
INITIAL LENGTH OF SAMPLE:	127 mm
INITIAL MASS OF SAMPLE:	776.64 grams
MOISTURE CONTENT:	22.2 %
DRY DENSITY:	1.58 t/m^3

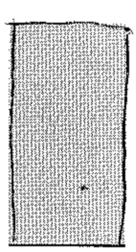
TEST DETAILS

	STAGE 1	STAGE 2	STAGE 3
CELL PRESSURE (kPa):	100	200	400
DEVIATOR STRESS (kPa):	112	220	243
STRAIN TO FAILURE (%)	3.7	6.9	9.8

MOHR DIAGRAM



MODE OF FAILURE



APPARENT COHESION (Cu) = 33 kPa
ANGLE OF SHEARING RESISTANCE (ϕu) = 9 Degrees

Test authorized by: Graham Hodgson
Laboratory Manager

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[1] Stress- strain plot available upon request.

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Figure No. 8.

PIPER & ASSOCIATES PTY LTD

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DETERMINATION OF THE MULTI STAGE UNDRAINED TRIAXIAL COMPRESSIVE STRENGTH OF A SOIL WITHOUT MEASUREMENT OF A PORE WATER PRESSURE

This test was carried out in accordance with AS 1289 6.4.1-1998, but without the stress-strain plot[1]
Moisture Content Determination carried out in accordance with AS 1289 Method 2.1.1 - 1992.

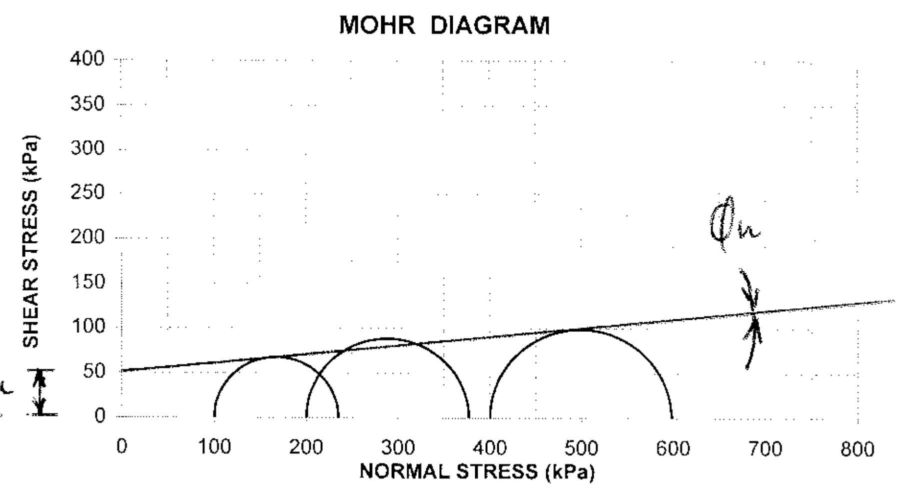
PROJECT: TESTING OF CREEK BANK
JOB\ DOC. NO.: 23019/9
DATE SAMPLED: 23/12/02
CLIENT: MORNINGTON PENINSULA S.C.
DATE TESTED: 7/03/03
LOCATION: TANTI CREEK, MORNINGTON
ISSUE DATE: 7/05/03
BOREHOLE/ TESTPIT No. : BH8
OPERATOR: GDH
DEPTH (m): 1.5
SAMPLE METHOD: HYD PUSH
SAMPLER TYPE: TUBE
END CONDITIONS: FRICTION
RATE OF FEED: 1.3 mm/min
SPECIMEN CONDITION: UNDISTURBED
COMMENTS: 0
SAMPLE DESCRIPTION: sandy CLAY grey

SAMPLE DETAILS

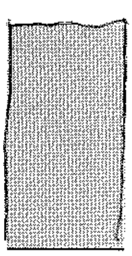
INITIAL DIAMETER OF SAMPLE:	63.5 mm
INITIAL LENGTH OF SAMPLE:	127 mm
INITIAL MASS OF SAMPLE:	833.16 grams
MOISTURE CONTENT:	15.9 %
DRY DENSITY:	1.79 t/m^3

TEST DETAILS

	STAGE 1	STAGE 2	STAGE 3
CELL PRESSURE (kPa):	100	200	400
DEVIATOR STRESS (kPa):	135	177	198
STRAIN TO FAILURE (%)	7.3	11.3	14.4



MODE OF FAILURE



APPARENT COHESION (Cu) = 50 kPa
ANGLE OF SHEARING RESISTANCE (ϕu) = 5 Degrees

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Laboratory Manager

[1] Stress- strain plot available upon request.

6/8/03

UUTRIAX-w'out stress-strain plot.XLS LABSHEET
REV: 6/02

Figure No. 9.

PIPER & ASSOCIATES PTY. LTD.

MULTI-STAGE CONSOLIDATED UNDRAINED TRIAXIAL TEST

with pore water pressure measurements

This test was carried out in accordance with the procedure outlined in the Manual of Soil Laboratory Testing Vol. 3 - *Effective Stress Tests* by K. H. Head, 1986. Moisture Content Determination carried out in accordance with Australian Standard AS 1289 Method 2.1.1 - 1992.

PROJECT: **GI FOR CREEK BANK**
PAGE: **1 OF 5**
JOB/DOC. No.: **23019**
LOCATION: **TANTI CK MORNINGTON**
DATE SAMPLED: **23/12/02**
SAMPLE NO.: **2**
DATE TESTED: **21/03/03**
SAMPLE TYPE: **TUBE**
OPERATOR: **GDH**
SAMPLE SOURCE: **BH7**
DEPTH (M): **1.5**
SAMPLE DESCRIPTION: **sandy CLAY grey dark grey & orange-brown**

SAMPLING METHOD: **HYD PUSH**
UNDISTURBED/REMOULDED: **UNDISTURBED**
COMMENT:

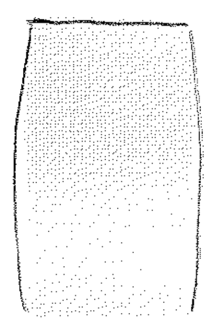
SAMPLE DETAILS

		INITIAL	FINAL
DIAMETER OF SAMPLE:	mm	63.50	
LENGTH OF SAMPLE:	mm	127.00	
DRY DENSITY	g/cm^3	1.60	
MOISTURE CONTENT	%	19.5	19.8

TEST DETAILS

STAGE NO.		1	2	3
CELL PRESSURE	kPa	575	650	800
BACK PRESSURE	kPa	500	500	500
DEVIATOR STRESS AT FAILURE	kPa	89	152	239
EFFECTIVE PORE PRESSURE AT FAILURE	kPa	41	62	128
STRAIN AT FAILURE	%	2.8	5.4	8.1
PORE PRESSURE PARAMETER B		0.97	0.97	0.97
RATE OF STRAIN	mm/min	0.007	0.007	0.007

FAILURE SHAPE



DRAINAGE CONDITIONS: SIDE & ENDS

FAILURE CRITERIA: PRINCIPAL EFFECTIVE STRESS

PIPER & ASSOCIATES PTY. LTD.

MULTI-STAGE CONSOLIDATED UNDRAINED TRIAXIAL TEST
with pore water pressure measurements

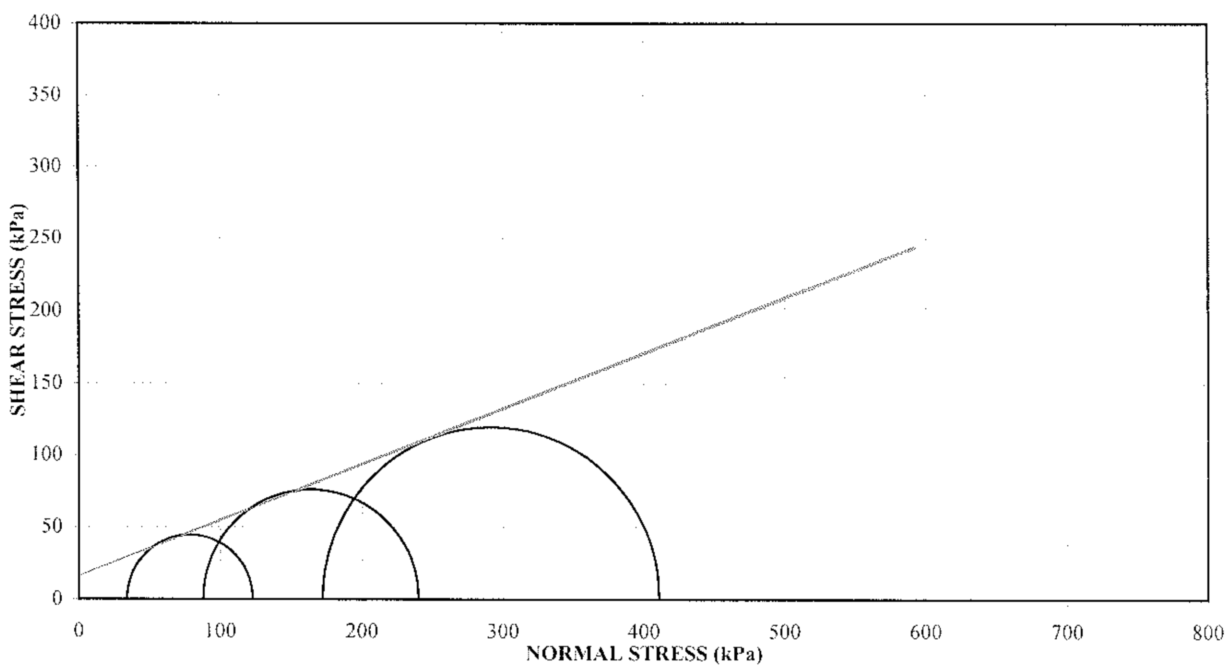
This test was carried out in accordance with the procedure outlined in the Manual of Soil Laboratory Testing Vol. 3 - *Effective Stress Tests* by K. H. Head, 1986. Moisture Content Determination carried out in accordance with Australian Standard AS 1289 Method 2.1.1 - 1992.

PROJECT:	**GI FOR CREEK BANK**	PAGE:	**2 OF 5**
LOCATION:	**TANTI CK MORNINGTON**	JOB/DOC No.:	**23019**
SAMPLE NO.:	**2**	DEPTH (M):	**1.5**

TEST RESULTS:

STAGE #1	PRINCIPAL MAJOR STRESS (kPa)	123
	PRINCIPAL MINOR STRESS (kPa)	34
	AVERAGE OF PRINCIPAL STRESSES (kPa)	78.5
STAGE #2	PRINCIPAL MAJOR STRESS (kPa)	240
	PRINCIPAL MINOR STRESS (kPa)	88
	AVERAGE OF PRINCIPAL STRESSES (kPa)	164
STAGE #3	PRINCIPAL MAJOR STRESS (kPa)	411
	PRINCIPAL MINOR STRESS (kPa)	172
	AVERAGE OF PRINCIPAL STRESSES (kPa)	291.5

SHEAR STRESS VS. NORMAL STRESS PLOT



EFFECTIVE COHESION	18	kPa
EFFECTIVE ANGLE OF FRICTION	21	Degrees

PIPER & ASSOCIATES PTY. LTD.

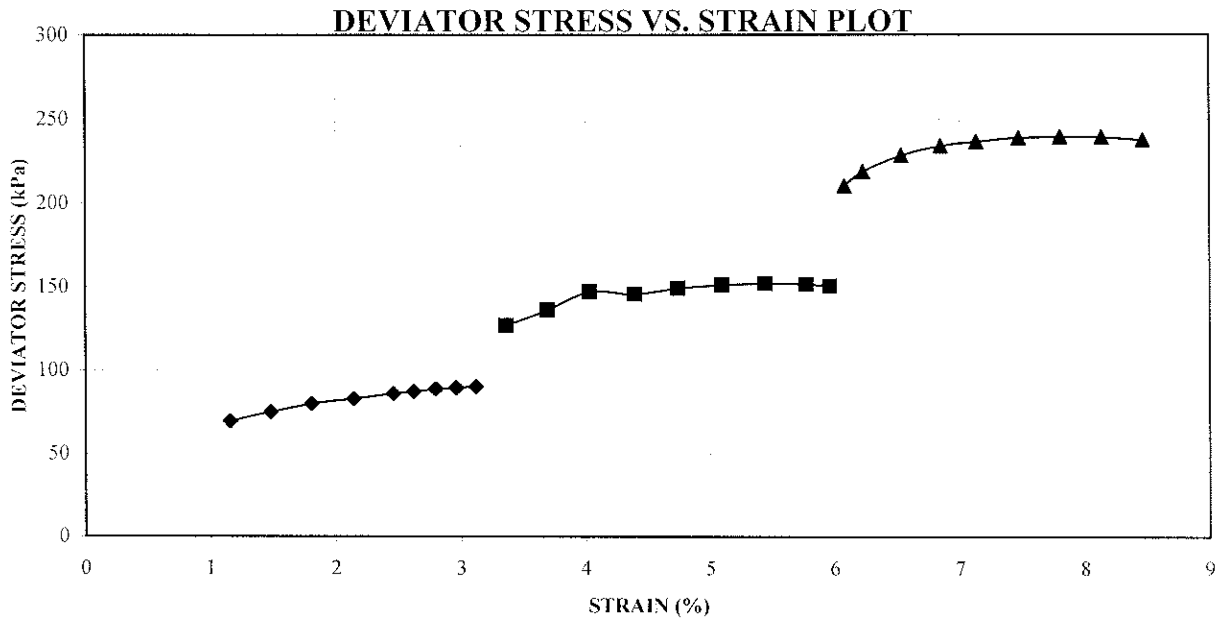
MULTI-STAGE CONSOLIDATED UNDRAINED TRIAXIAL TEST

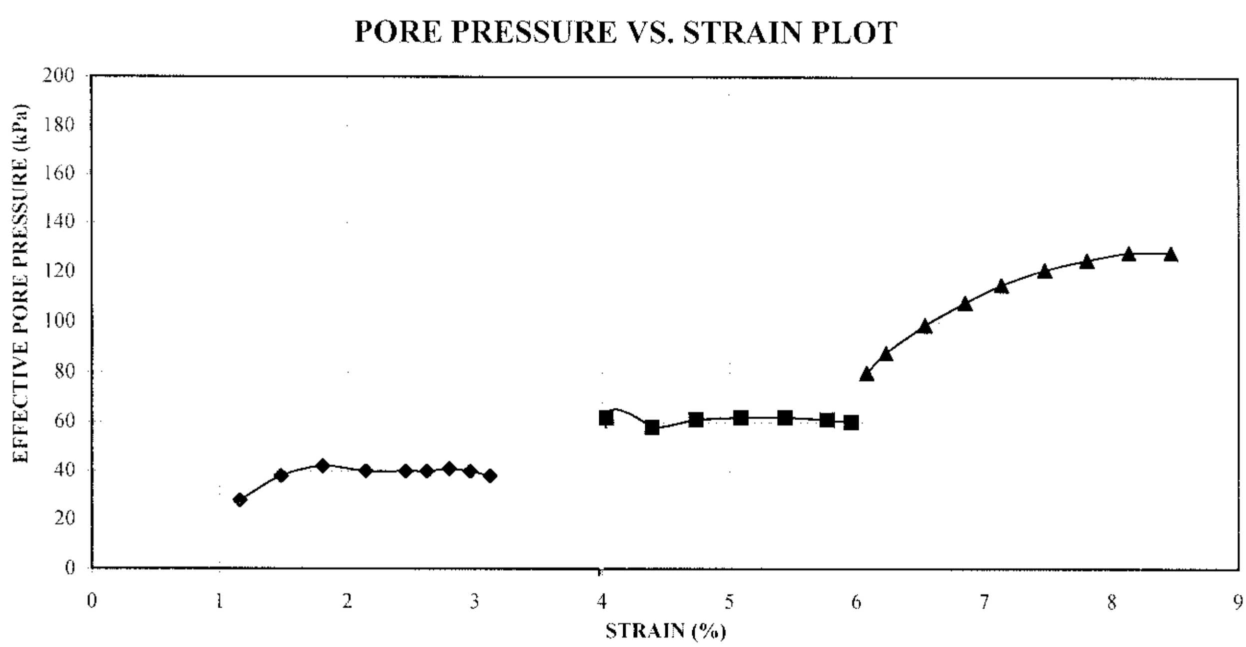
with pore water pressure measurements

This test was carried out in accordance with the procedure outlined in the Manual of Soil Laboratory Testing Vol. 3 - *Effective Stress Tests* by K. H. Head, 1986. Moisture Content Determination carried out in accordance with Australian Standard AS 1289 Method 2.1.1 - 1992.

PROJECT:	**GI FOR CREEK BANK**	PAGE:	**3 OF 5**
LOCATION:	**TANTI CK MORNINGTON**	JOB/DOC No.:	**23019**
SAMPLE NO.:	**2**	DEPTH (M):	**1.5**

TEST RESULTS:





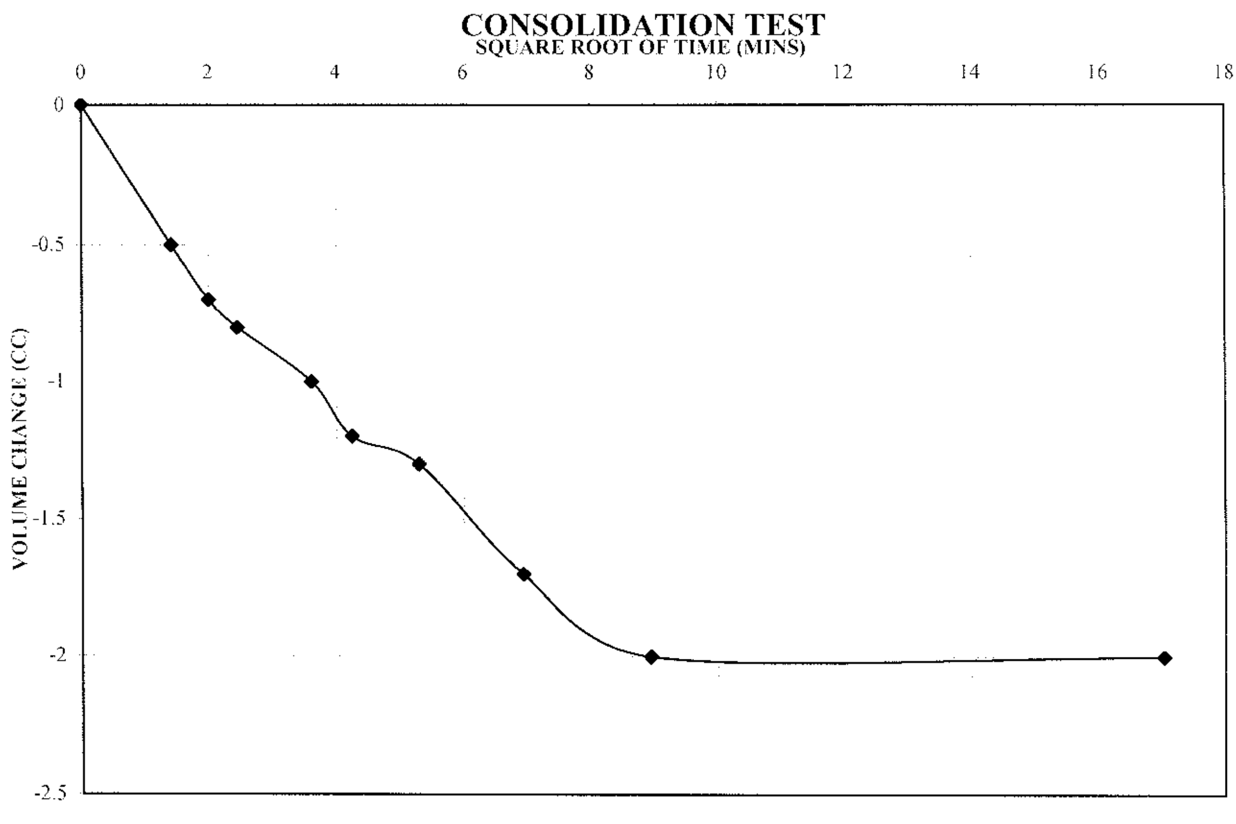
PIPER & ASSOCIATES PTY. LTD.

MULTI-STAGE CONSOLIDATED UNDRAINED TRIAXIAL TEST
with pore water pressure measurements

This test was carried out in accordance with the procedure outlined in the <u>Manual of Soil Laboratory Testing</u> Vol. 3 - *Effective Stress Tests* by K. H. Head, 1986. Moisture Content Determination carried out in accordance with Australian Standard AS 1289 Method 2.1.1 - 1992.

PROJECT:	**GI FOR CREEK BANK**	PAGE:	**4 OF 5**
LOCATION:	**TANTI CK MORNINGTON**	JOB/DOC No.:	**23019**
SAMPLE NO.:	**2**	DEPTH (M):	**1.5**

TEST RESULTS:



CONSOLIDATION RESULTS

STAGE 1

EFFECTIVE CELL PRESSURE	75	kPa
CONSOLIDATION 100% (t_{100})	77	mins
COEFFICIENT OF CONSOLIDATION, C_v	0.865	m^2/year

PIPER & ASSOCIATES PTY. LTD.

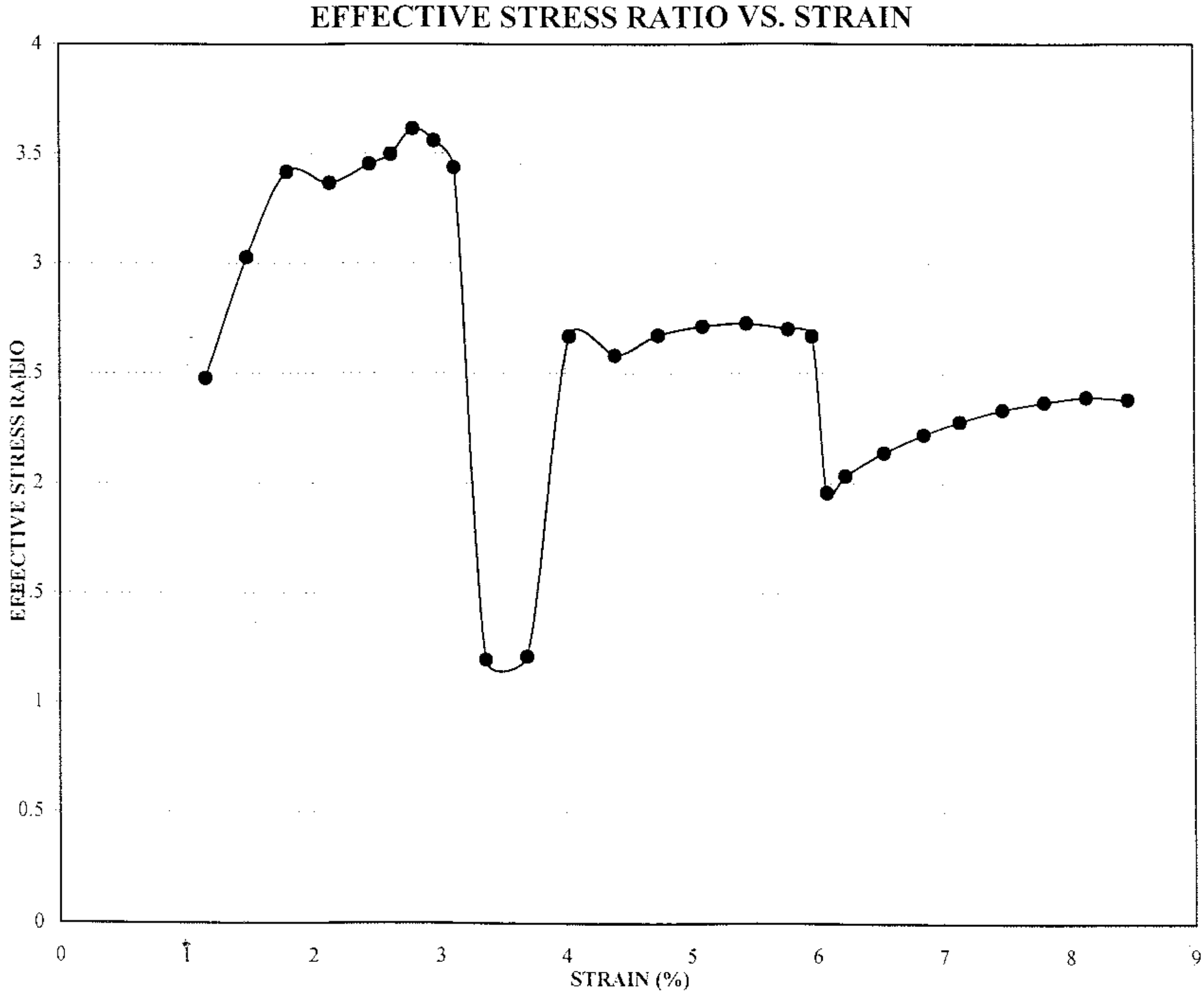
MULTI-STAGE CONSOLIDATED UNDRAINED TRIAXIAL TEST
with pore water pressure measurements

This test was carried out in accordance with the procedure outlined in the Manual of Soil Laboratory Testing Vol. 3 - *Effective Stress Tests* by K. H. Head, 1986. Moisture Content Determination carried out in accordance with Australian Standard AS 1289 Method 2.1.1 - 1992.

PROJECT:	**GI FOR CREEK BANK**	PAGE:	**5 OF 5**
LOCATION:	**TANTI CK MORNINGTON**	JOB/DOC No.:	**23019**
SAMPLE NO.:	**2**	DEPTH (M):	**1.5**

TEST RESULTS:

EFFECTIVE STRESS RATIO VS. STRAIN



CUTRIAX1.XLS
REV: 4/00

Figure No. 14.

PIPER & ASSOCIATES PTY. LTD.

MULTI-STAGE CONSOLIDATED UNDRAINED TRIAXIAL TEST

with pore water pressure measurements

This test was carried out in accordance with the procedure outlined in the Manual of Soil Laboratory Testing Vol. 3 - *Effective Stress Tests* by K. H. Head, 1986. Moisture Content Determination carried out in accordance with Australian Standard AS 1289 Method 2.1.1 - 1992.

PROJECT: **TESTING OF CREEK BANK**

PAGE: **1 OF 5**
JOB/DOC. No.: **23019**

LOCATION: **TANTI CREEK MORNINGTON**

DATE SAMPLED: **28/12/02**

SAMPLE NO.: **1**

DATE TESTED: **19/02/03**

SAMPLE TYPE: **TUBE**

OPERATOR: **GDH**

SAMPLE SOURCE: **BH 14**

DEPTH (M): **1.5**

SAMPLE DESCRIPTION: **clayey SAND light grey**

SAMPLING METHOD: **HYD PUSH**
UNDISTURBED/REMOULDED: **UNDISTURBED**
COMMENT: **0**

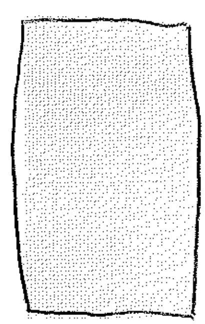
SAMPLE DETAILS

		INITIAL	FINAL
DIAMETER OF SAMPLE:	mm	63.50	
LENGTH OF SAMPLE:	mm	127.00	
DRY DENSITY	g/cm^3	1.68	
MOISTURE CONTENT	%	21.9	17.8

TEST DETAILS

STAGE NO.		1	2	3
CELL PRESSURE	kPa	575	650	800
BACK PRESSURE	kPa	500	500	500
DEVIATOR STRESS AT FAILURE	kPa	159	232	347
EFFECTIVE PORE PRESSURE AT FAILURE	kPa	17	42	108
STRAIN AT FAILURE	%	2.8	5.2	8.3
PORE PRESSURE PARAMETER B		0.99	0.99	0.99
RATE OF STRAIN	mm/min	0.007	0.007	0.007

FAILURE SHAPE



DRAINAGE CONDITIONS: SIDE & ENDS

FAILURE CRITERIA: PRINCIPAL EFFECTIVE STRESS

PIPER & ASSOCIATES PTY. LTD.

MULTI-STAGE CONSOLIDATED UNDRAINED TRIAXIAL TEST

with pore water pressure measurements

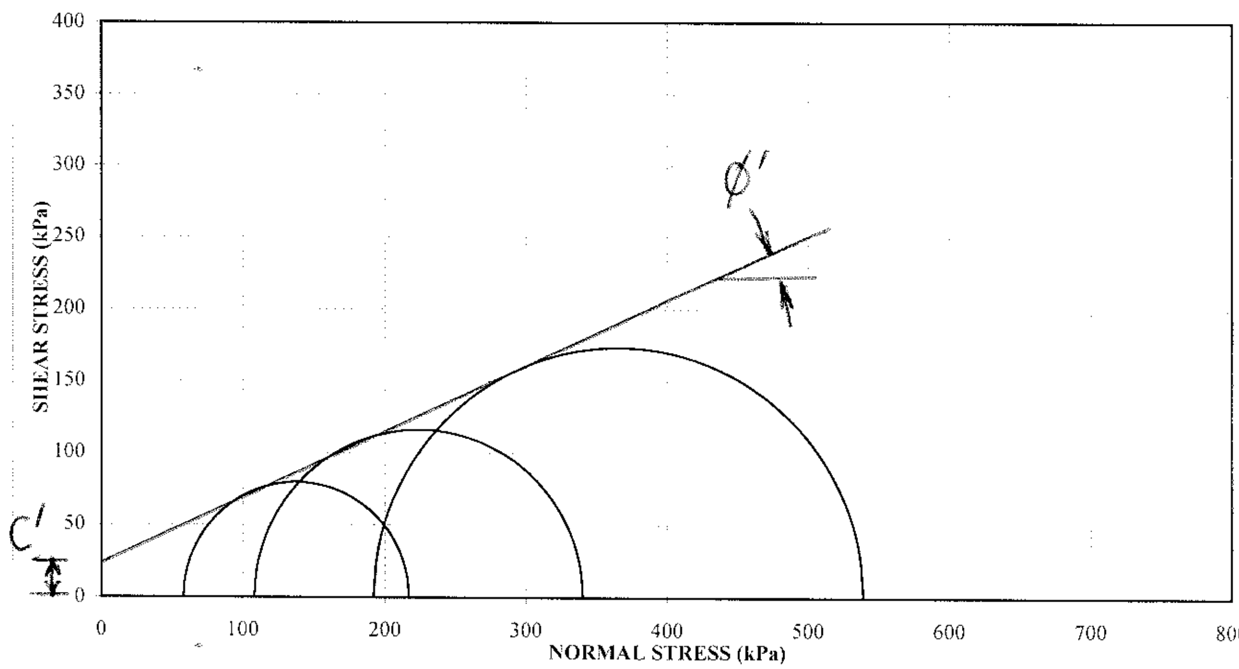
This test was carried out in accordance with the procedure outlined in the Manual of Soil Laboratory Testing Vol. 3 - *Effective Stress Tests* by K. H. Head, 1986. Moisture Content Determination carried out in accordance with Australian Standard AS 1289 Method 2.1.1 - 1992.

PROJECT:	**TREATMENT PLANT & STORAGE**	PAGE:	**2 OF 5**
LOCATION:	**TANTI CREEK MORNINGTON**	JOB/DOC No.:	**23019**
SAMPLE NO.:	**1**	DEPTH (M):	**1.5**

TEST RESULTS:

STAGE #1	PRINCIPAL MAJOR STRESS (kPa)	217
	PRINCIPAL MINOR STRESS (kPa)	58
	AVERAGE OF PRINCIPAL STRESSES (kPa)	137.5
STAGE #2	PRINCIPAL MAJOR STRESS (kPa)	340
	PRINCIPAL MINOR STRESS (kPa)	108
	AVERAGE OF PRINCIPAL STRESSES (kPa)	224
STAGE #3	PRINCIPAL MAJOR STRESS (kPa)	539
	PRINCIPAL MINOR STRESS (kPa)	192
	AVERAGE OF PRINCIPAL STRESSES (kPa)	365.5

SHEAR STRESS VS. NORMAL STRESS PLOT



EFFECTIVE COHESION	25	kPa
EFFECTIVE ANGLE OF FRICTION	24	Degrees

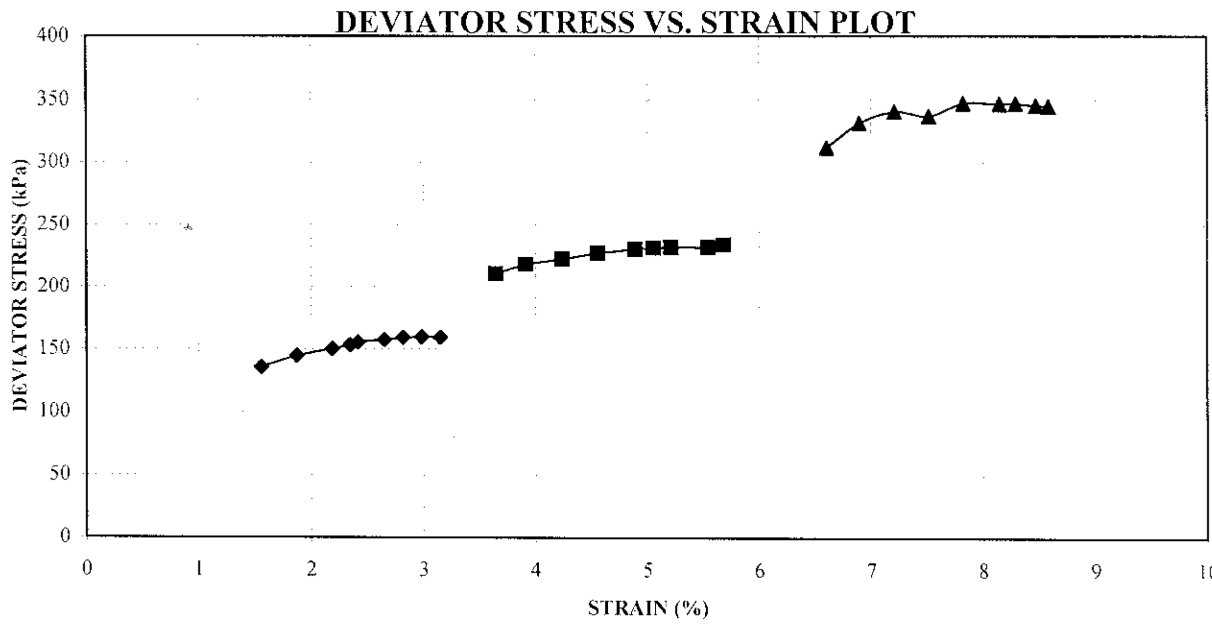
PIPER & ASSOCIATES PTY. LTD.

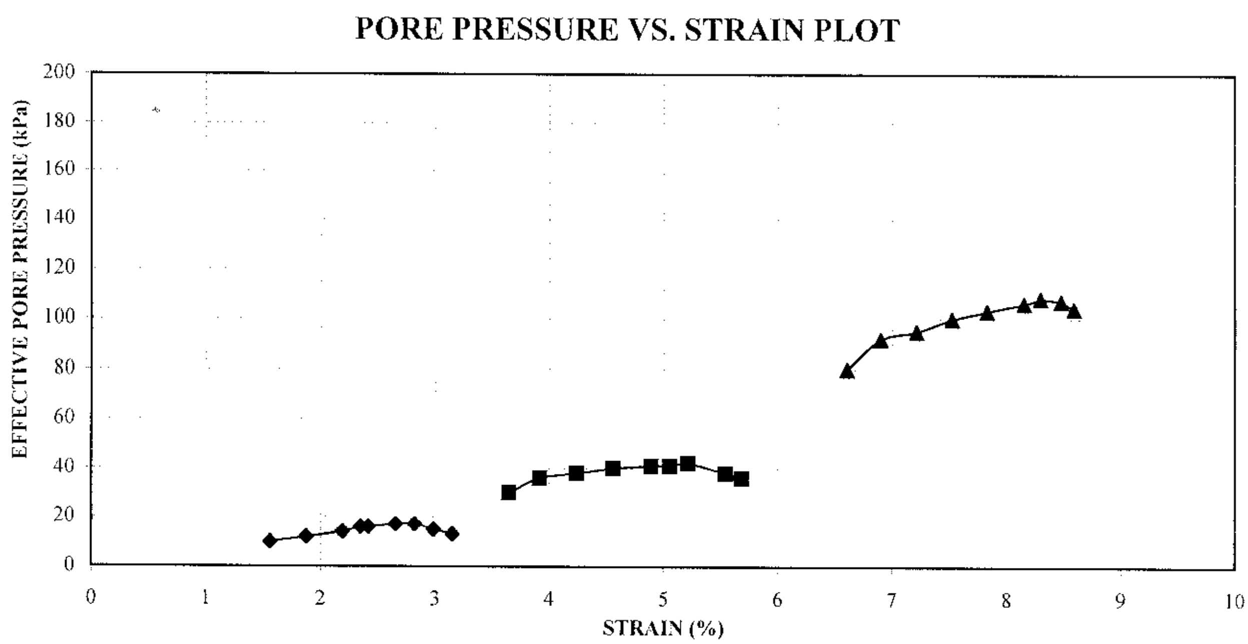
MULTI-STAGE CONSOLIDATED UNDRAINED TRIAXIAL TEST
with pore water pressure measurements

This test was carried out in accordance with the procedure outlined in the Manual of Soil Laboratory Testing Vol. 3 - *Effective Stress Tests* by K. H. Head, 1986. Moisture Content Determination carried out in accordance with Australian Standard AS 1289 Method 2.1.1 - 1992.

PROJECT:	**TESTING OF CREEK BANK**	PAGE:	**3 OF 5**
LOCATION:	**TANTI CREEK MORNINGTON**	JOB/DOC No.:	**23019**
SAMPLE NO.:	**1**	DEPTH (M):	**1.5**

TEST RESULTS:





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REV: 4/00

Figure No. 17.

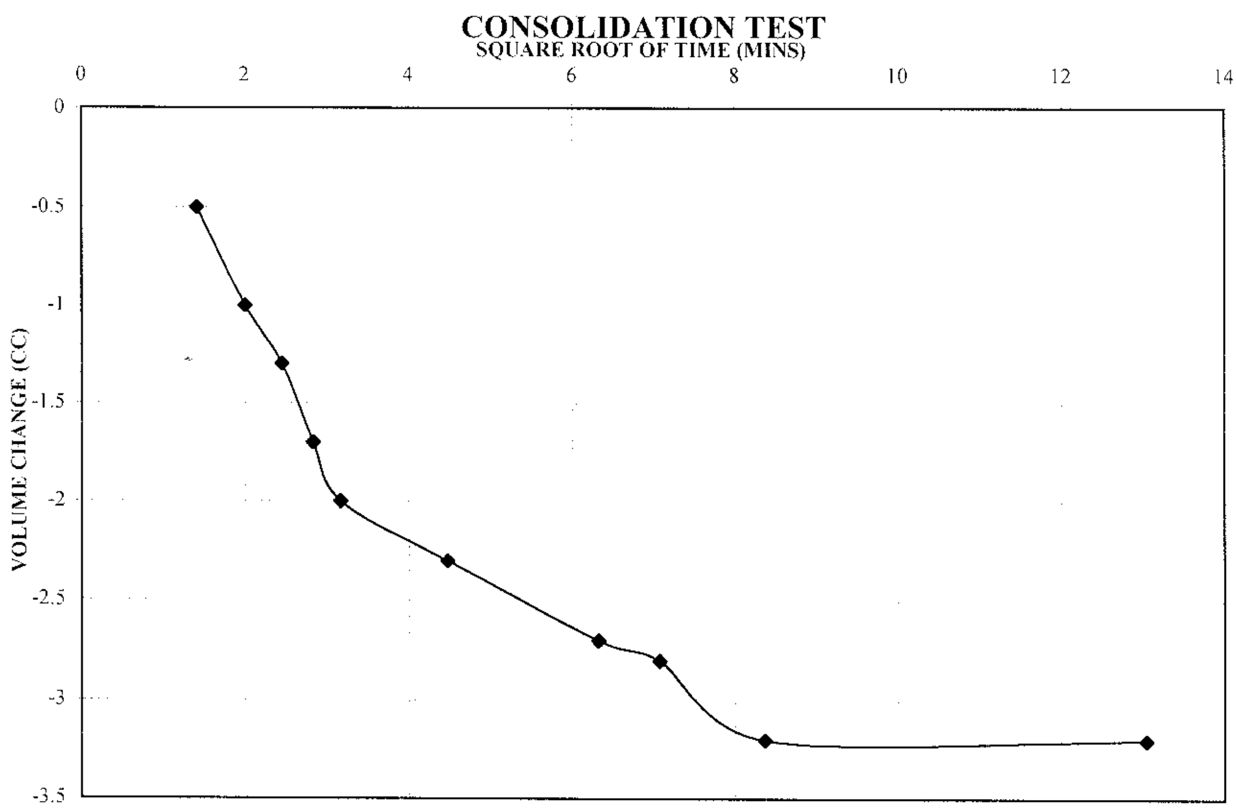
PIPER & ASSOCIATES PTY. LTD.

MULTI-STAGE CONSOLIDATED UNDRAINED TRIAXIAL TEST
with pore water pressure measurements

This test was carried out in accordance with the procedure outlined in the Manual of Soil Laboratory Testing Vol. 3 - *Effective Stress Tests* by K. H. Head, 1986. Moisture Content Determination carried out in accordance with Australian Standard AS 1289 Method 2.1.1 - 1992.

PROJECT:	**TESTING OF CREEK BANK**	PAGE:	**4 OF 5**
LOCATION:	**TANTI CREEK MORNINGTON**	JOB/DOC No.:	**23019**
SAMPLE NO.:	**1**	DEPTH (M):	**1.5**

TEST RESULTS:



CONSOLIDATION RESULTS

STAGE 1

EFFECTIVE CELL PRESSURE	75	kPa
CONSOLIDATION 100% (t_{100})	81	mins
COEFFICIENT OF CONSOLIDATION, C_v	0.823	m^2/year

CUTRIAX1.XLS
REV: 4/00

Figure No. 18.

PIPER & ASSOCIATES PTY. LTD.

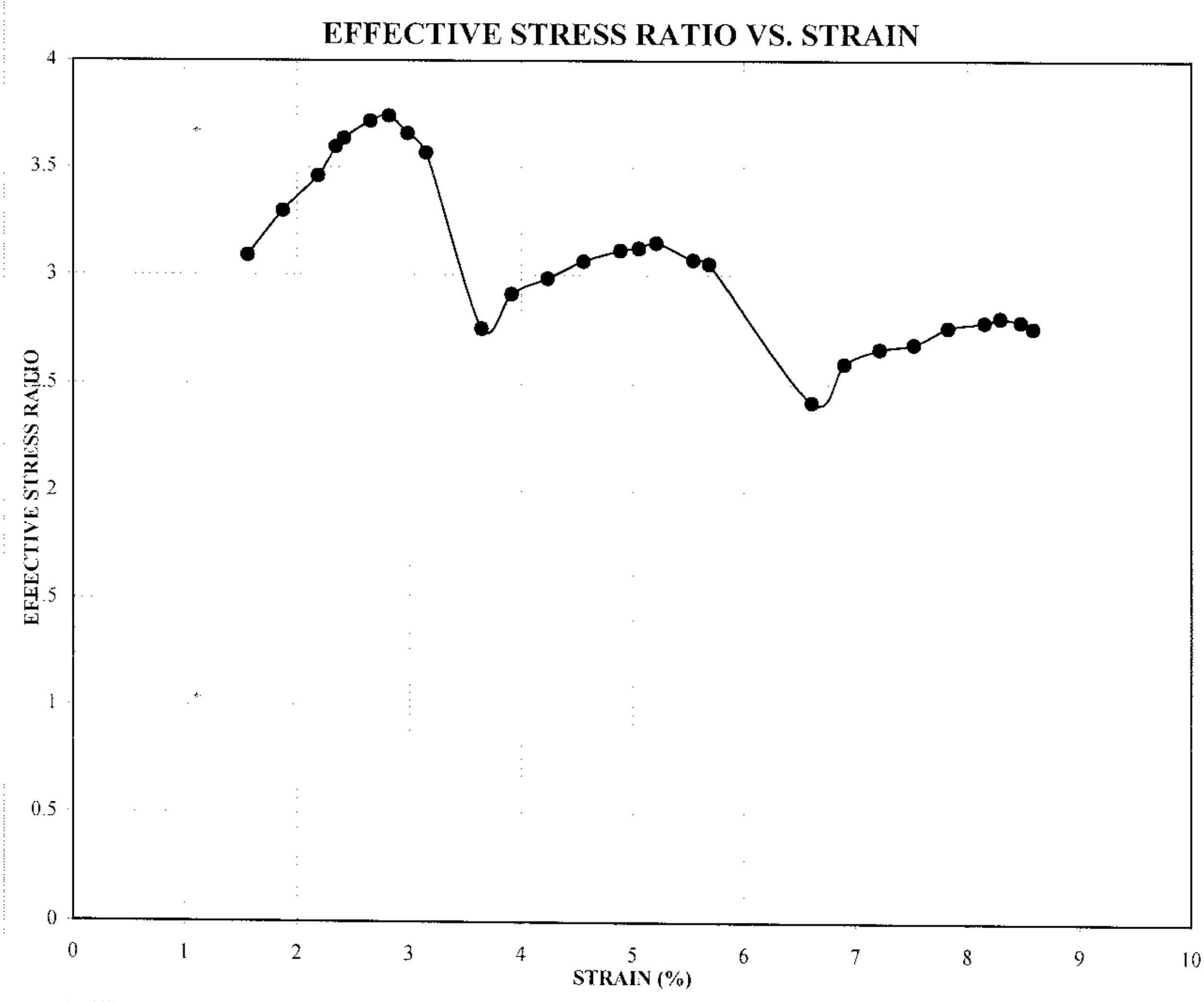
MULTI-STAGE CONSOLIDATED UNDRAINED TRIAXIAL TEST
with pore water pressure measurements

This test was carried out in accordance with the procedure outlined in the Manual of Soil Laboratory Testing Vol. 3 - *Effective Stress Tests* by K. H. Head, 1986. Moisture Content Determination carried out in accordance with Australian Standard AS 1289 Method 2.1.1 - 1992.

PROJECT: **TESTING OF CREEK BANK**
LOCATION: **TANTI CREEK MORNINGTON**
SAMPLE NO.: **1**

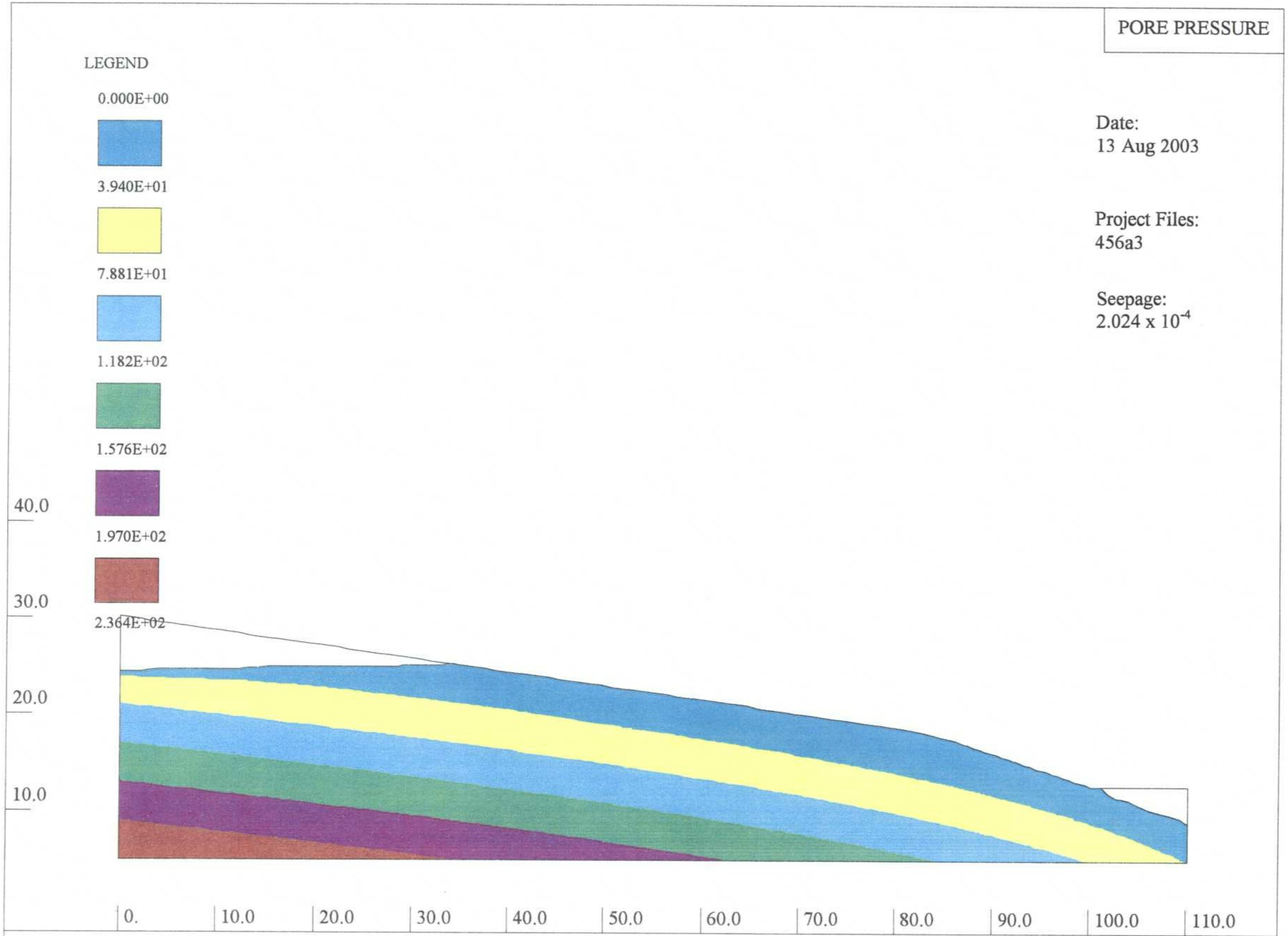
PAGE: **5 OF 5**
JOB/DOC No.: **23019**
DEPTH (M): **1.5**

TEST RESULTS:

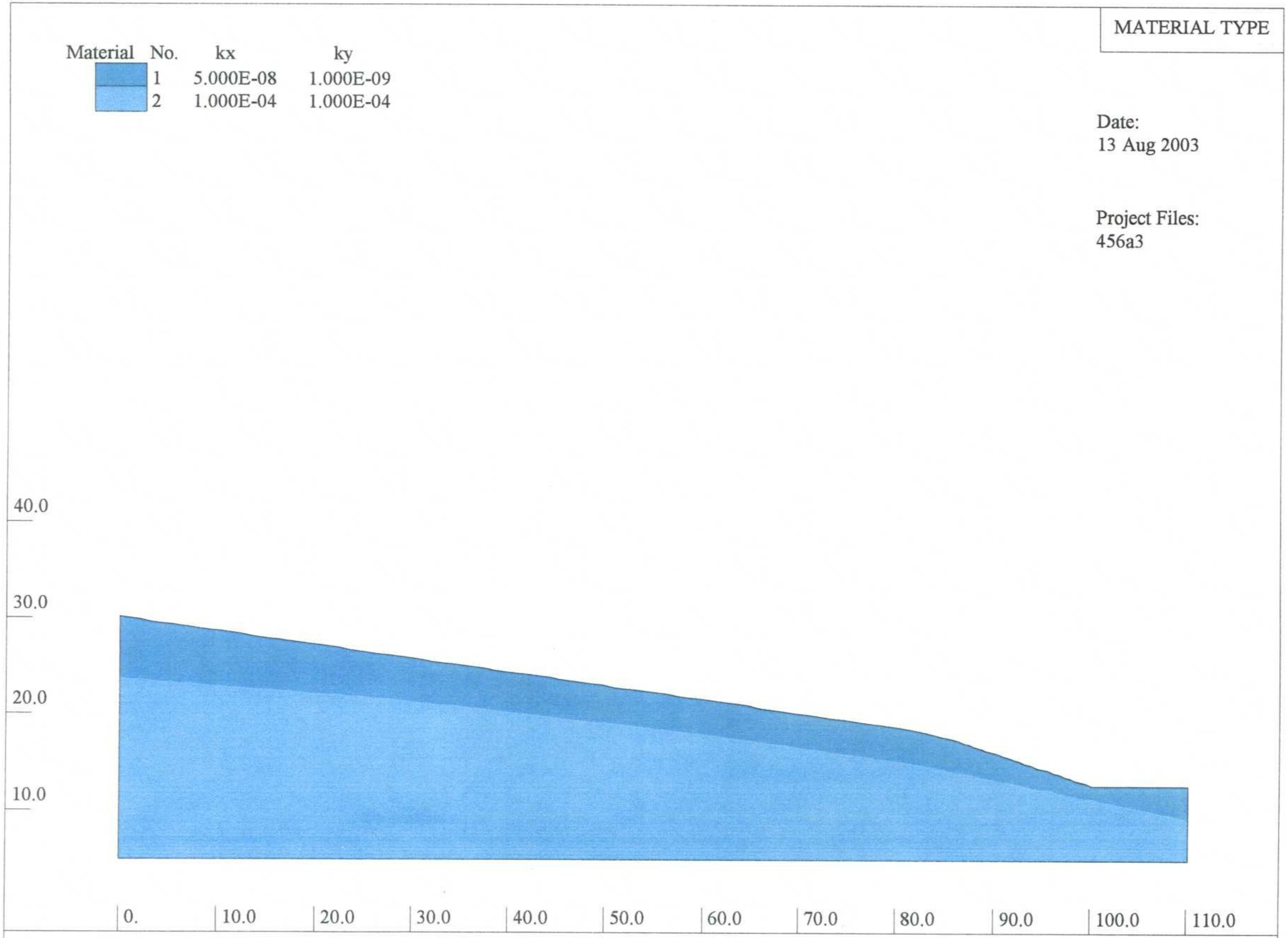


Appendix D
3 Pages

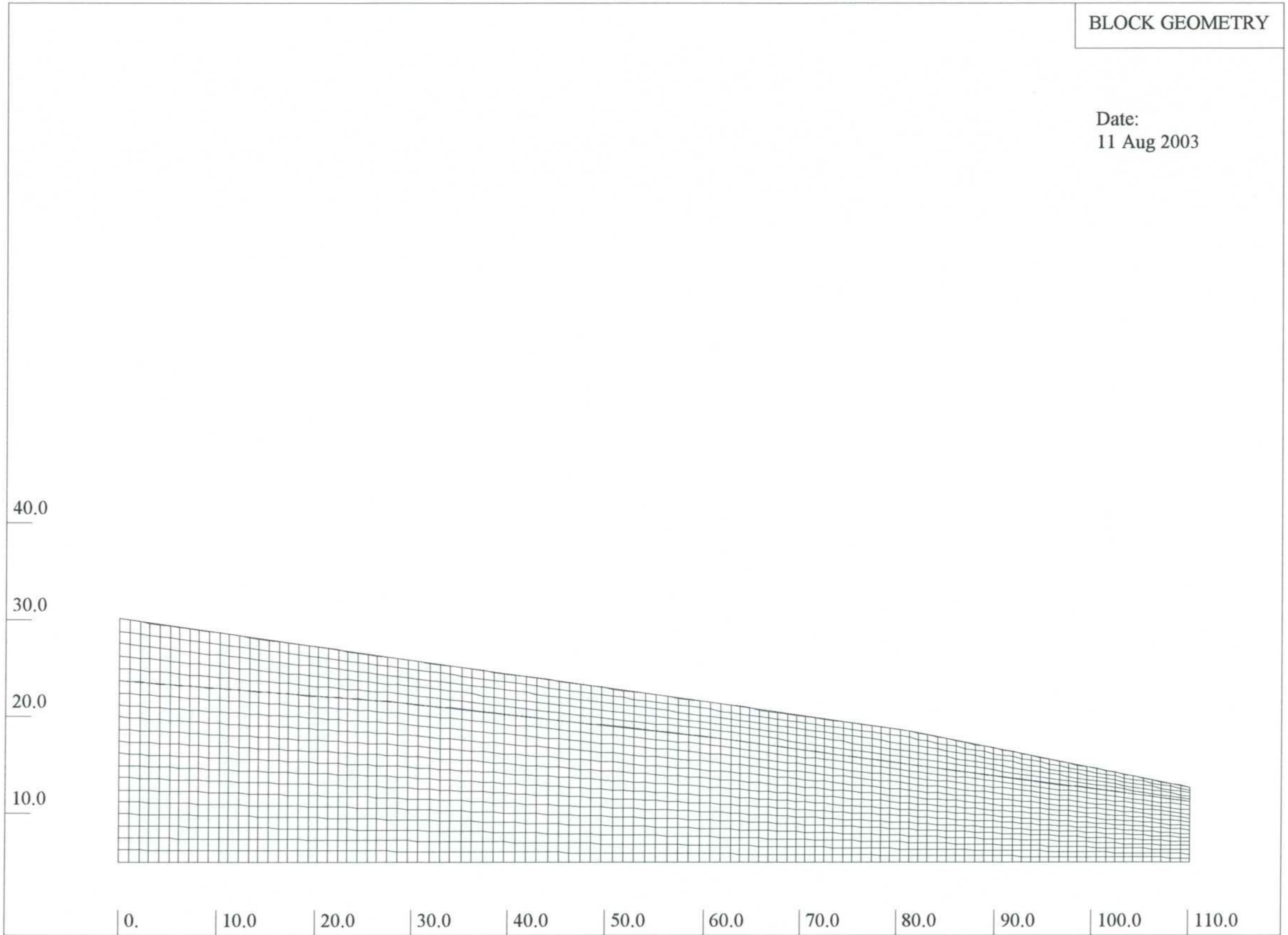
TYPICAL SEEPAGE ANALYSIS OUTPUTS



23019 - Section 456



23019 - Section 456



23019 - Section 456

Appendix E
4 Pages

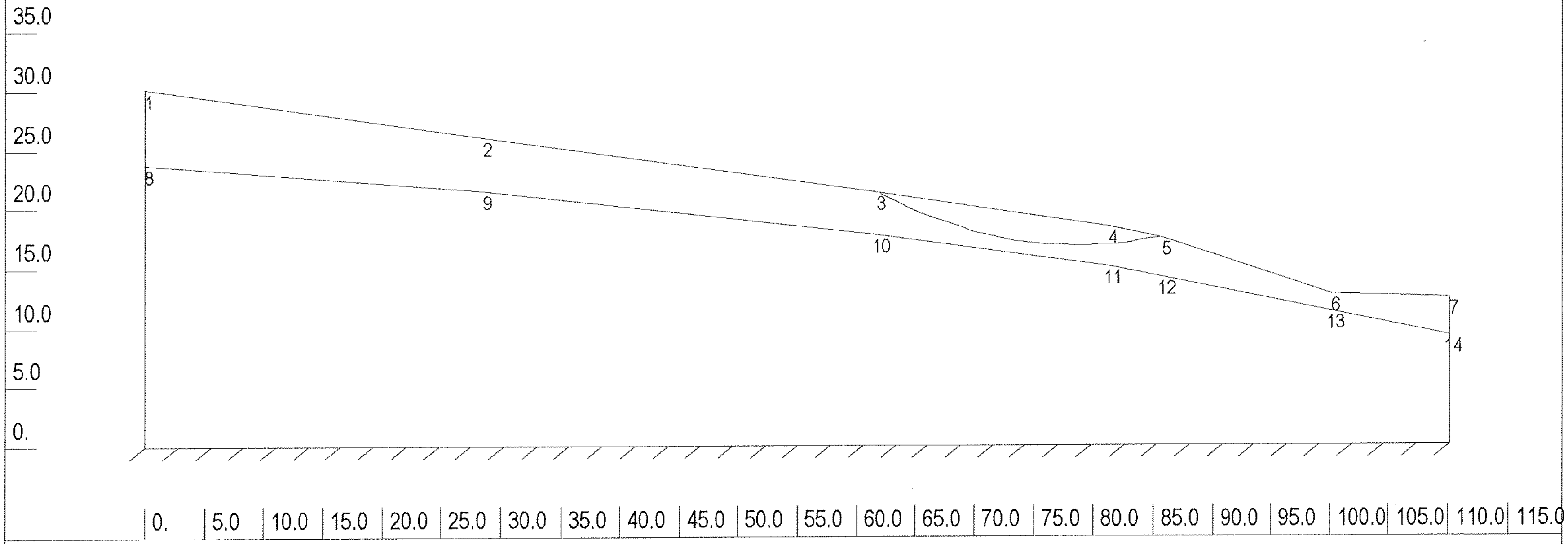
TYPICAL SLOPE STABILITY OUTPUTS

x0	y0	Radius	FoS
78.9	51.4	34.6	1.78

INTERACTIVE ANALYSIS

Date:
13 Aug 2003

Project Files:
456st03b



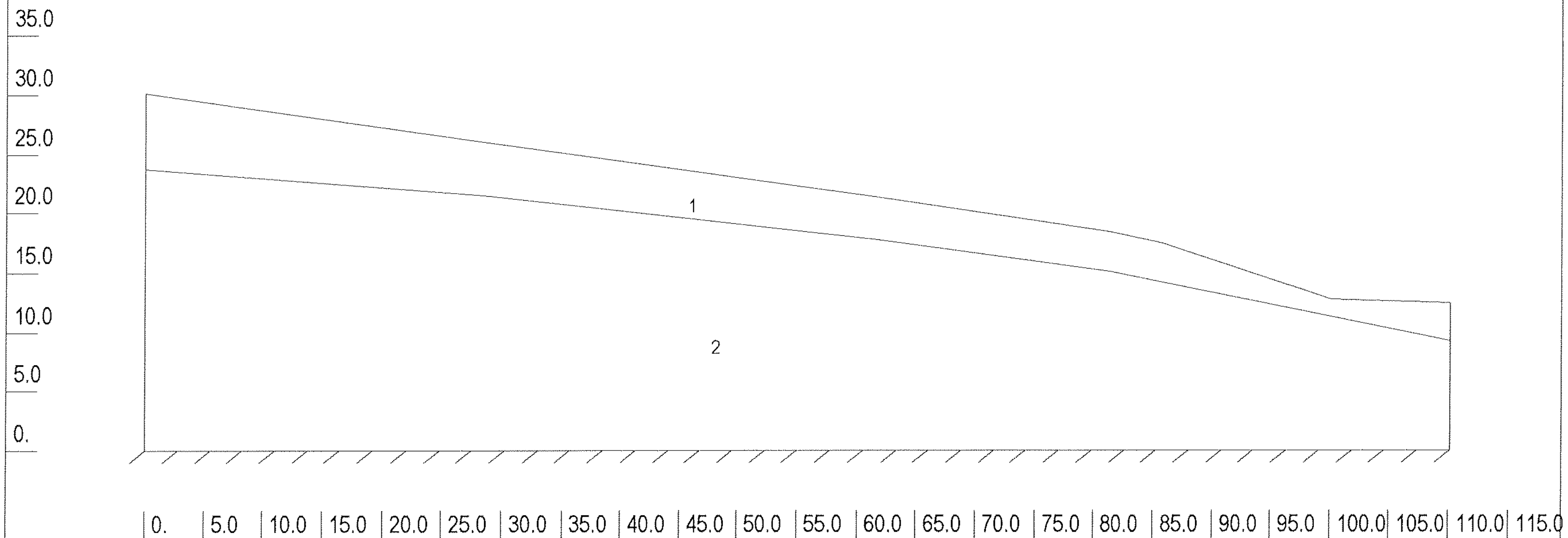
23019 - Section 456 Steady State

SOIL PROPERTIES

Layer	Cohesion	Phi	Gamma
1	2.0	23.0	19.0
2	.0	30.0	20.0

Date:
13 Aug 2003

Project Files:
456st03



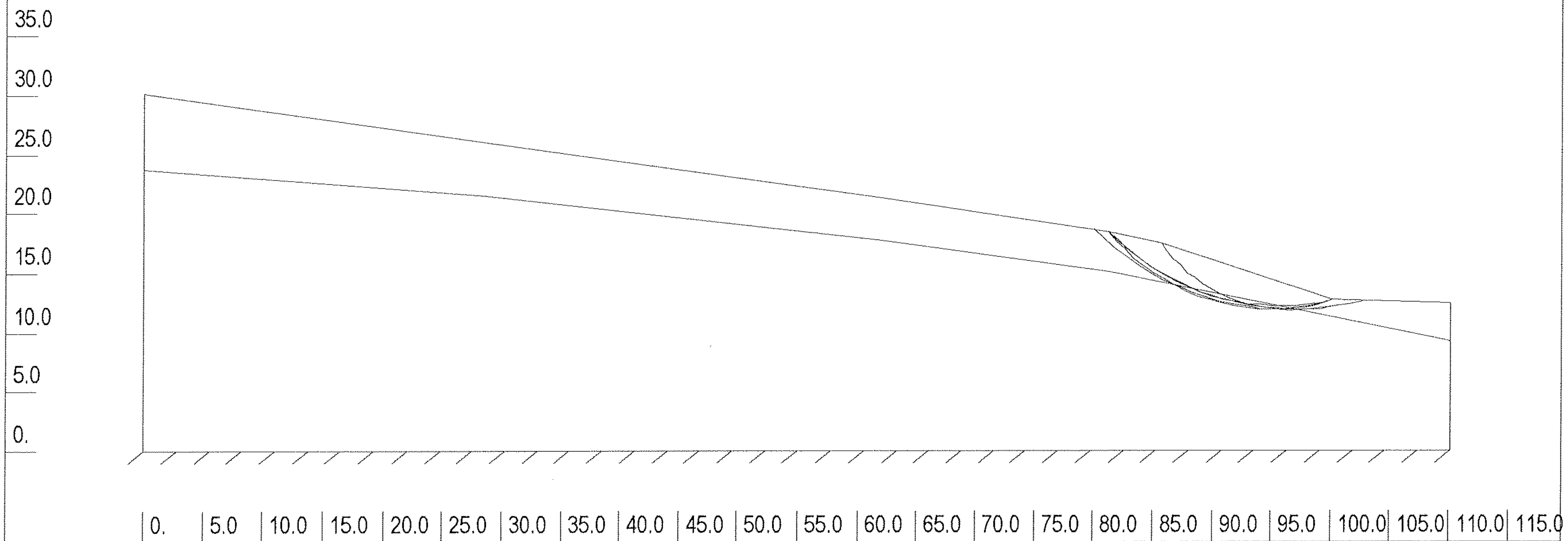
23019 - Section 456 Steady State

CRITICAL CIRCLE(S)

No.	x0	y0	Radius	FoS
1	97.05	34.09	22.16	1.03
2	95.05	29.61	17.68	1.05
3	95.77	31.97	19.77	1.05
4	96.07	24.32	12.33	1.05
5	96.60	35.23	23.38	1.05

Date:
13 Aug 2003

Project Files:
456st03b



23019 - Section 456 Steady State

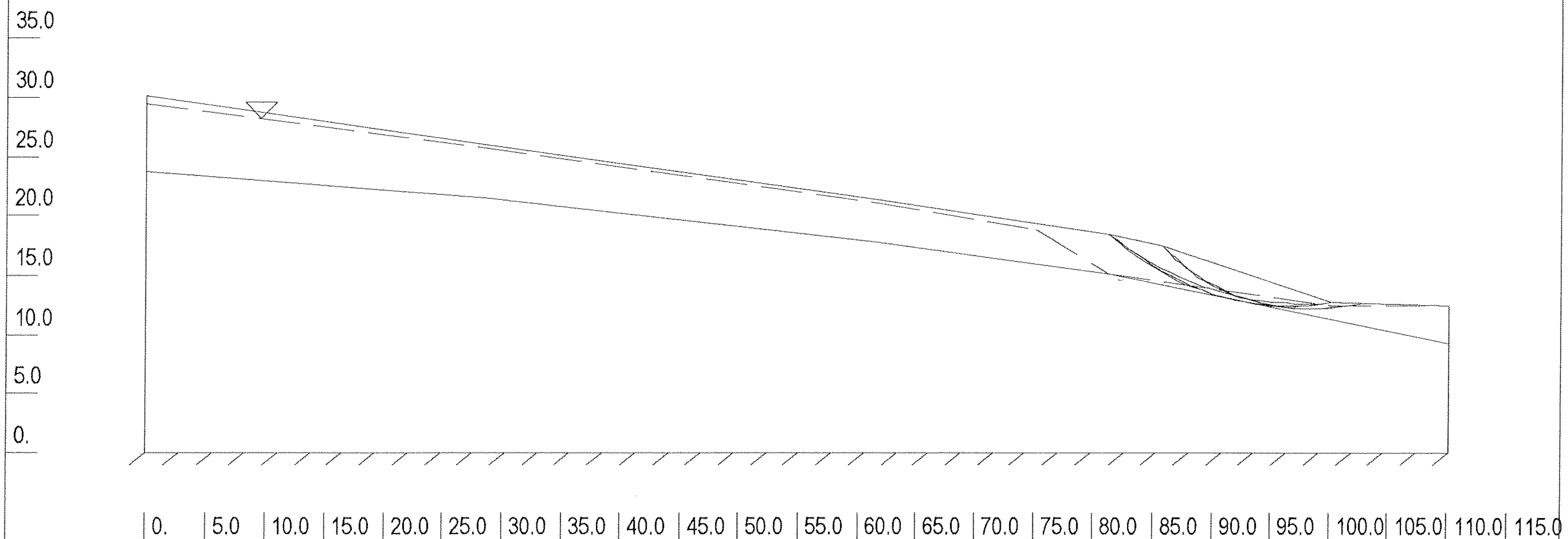
3.

CRITICAL CIRCLE(S)

No.	x0	y0	Radius	FoS
1	97.16	27.57	15.18	1.56
2	98.44	29.69	17.56	1.57
3	96.70	34.98	22.55	1.60
4	97.93	38.99	26.39	1.61
5	97.98	37.49	25.28	1.61

Date:
7 Oct 2003

Project Files:
456st04



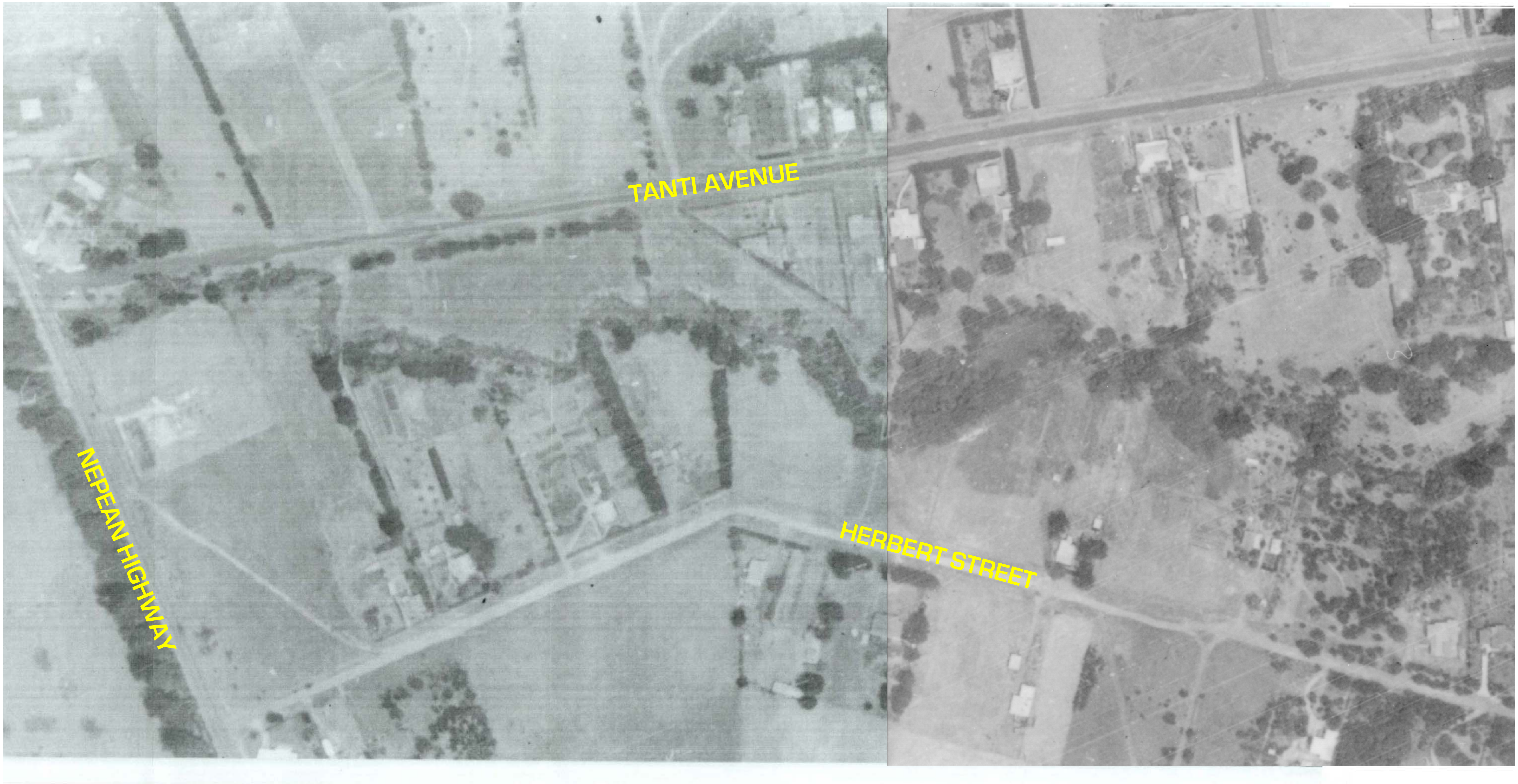
23019 - Section 456 Steady State

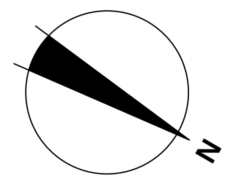
Appendix F

6 Pages

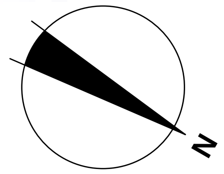
AERIAL PHOTOGRAPHS FROM 1949
AERIAL PHOTOGRAPHS FROM 2001
AERIAL PHOTOGRAPHS FROM 2007

PROJECT	LANE PIPER PTY LTD	TITLE	SCALE (A3) N.T.S.	DATE 29 MAR 2010
CREEK BANK STABILITY TANTI CREEK, MORNINGTON	LanePiper A.C.N. 120 109 935 Geotechnical and Environmental Engineers Hydrogeologists and Environmental Scientists BLDG 2, 154 HIGHBURY RD, BURWOOD, VICTORIA, 3125 TELEPHONE (03) 9888 0100 FAX (03) 9808 3511	1949 AERIAL PHOTOGRAPH (NORTH SECTION)	DRAWN DBS	JOB NO. 23019
		REF: 23019 - 1949		REV. NO. 0 FIG. NO. 1



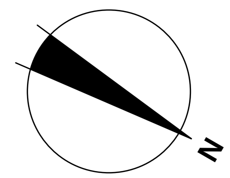


PROJECT CREEK BANK STABILITY TANTI CREEK, MORNINGTON	LanePiper **LANE PIPER PTY LTD** A.C.N. 120 109 935 **Geotechnical and Environmental Engineers** **Hydrogeologists and Environmental Scientists** BLDG 2, 154 HIGHBURY RD, BURWOOD, VICTORIA, 3125 TELEPHONE (03) 9888 0100 FAX (03) 9808 3511	TITLE 1949 AERIAL PHOTOGRAPH (SOUTH SECTION) REF: 23019 - 1949	SCALE (A3) N.T.S. DRAWN DBS	DATE 29 MAR 2010 JOB NO. 23019 REV. NO. 0 FIG. NO. 2

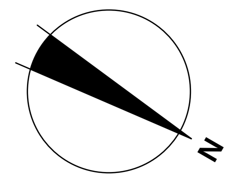


PROJECT	LANE PIPER PTY LTD	TITLE	SCALE (A3) N.T.S.	DATE 29 MAR 2010
CREEK BANK STABILITY TANTI CREEK, MORNINGTON	LanePiper A.C.N. 120 109 935 Geotechnical and Environmental Engineers Hydrogeologists and Environmental Scientists BLDG 2, 154 HIGHBURY RD, BURWOOD, VICTORIA, 3125 TELEPHONE (03) 9888 0100 FAX (03) 9808 3511	2001 AERIAL PHOTOGRAPH (NORTH SECTION)	DRAWN DBS	JOB NO. 23019
		REF: 23019 - 2001		REV. NO. 0 FIG. NO. 3

PROJECT		TITLE	SCALE (A3) N.T.S.	DATE 29 MAR 2010
CREEK BANK STABILITY TANTI CREEK, MORNINGTON	LanePiper **LANE PIPER PTY LTD** A.C.N. 120 109 935 **Geotechnical and Environmental Engineers** **Hydrogeologists and Environmental Scientists** BLDG 2, 154 HIGHBURY RD, BURWOOD, VICTORIA, 3125 TELEPHONE (03) 9888 0100 FAX (03) 9808 3511	2001 AERIAL PHOTOGRAPH (SOUTH SECTION)	DRAWN DBS	JOB NO. 23019
		REF: 23019 - 2001		REV. NO. 0 FIG. NO. 4



PROJECT		TITLE	SCALE (A3) N.T.S.	DATE 29 MAR 2010
CREEK BANK STABILITY TANTI CREEK, MORNINGTON	LanePiper **LANE PIPER PTY LTD** A.C.N. 120 109 935 **Geotechnical and Environmental Engineers** **Hydrogeologists and Environmental Scientists** BLDG 2, 154 HIGHBURY RD, BURWOOD, VICTORIA, 3125 TELEPHONE (03) 9888 0100 FAX (03) 9808 3511	2007 AERIAL PHOTOGRAPH (NORTH SECTION)	DRAWN DBS	JOB NO. 23019
		REF: 23019 - 2007		REV. NO. 0 FIG. NO. 5



PROJECT		TITLE	SCALE (A3) N.T.S.	DATE 29 MARCH 2010
CREEK BANK STABILITY TANTI CREEK, MORNINGTON	LanePiper **LANE PIPER PTY LTD** A.C.N. 120 109 935 **Geotechnical and Environmental Engineers** **Hydrogeologists and Environmental Scientists** BLDG 2, 154 HIGHBURY RD, BURWOOD, VICTORIA, 3125 TELEPHONE (03) 9888 0100 FAX (03) 9808 3511	2007 AERIAL PHOTOGRAPH (SOUTH SECTION)	DRAWN DBS	JOB NO. 23019
		REF: 23019 - 2007		REV. NO. 0 FIG. NO. 6

Appendix G

5 Pages

TYPICAL CHECKLIST FOR SLOPE STABILITY ASSESSMENTS
SOME GUIDELINES FOR HILLSIDE CONSTRUCTION & PRACTICE

CHECKLIST FOR STABILITY ASSESSMENTS

Factual Information

1. INFORMATION
   - Report prepared for who?
   - Site Location
   - Outline of proposed development([b])
   - Comment on need for earthquake assessment

2. TOPOGRAPHY
   - Outline current landform (slope, shape, height gradient, irregularities, erosion, soil creep/terracettes)
   - Outline surface drainage patterns([b])
   - Review aerial photos
   - Comment on any previous earthworks
   - Comment on any existing instability([c])
   - Additional site features (eg. Vegetation/trees structures([b]) retaining walls, roads/driveways, services)

3. SITE HISTORY
   - Outline current/previous landuse
   - Comment on previous siteworks([b])
   - Reference "District Hazard Map"/GIS
   - Comment on previous instability([c])
   - Performance of existing structures
   - Review aerial photos
   - Comment on previous contamination([c])

4. GEOLOGY
   - Describe geological setting
   - Refer to relevant maps
   - Geological influences on stability (eg. bedding, weak materials, faults)
   - Describe seismic setting

5. INVESTIGATIONS
   - ♦ FIELD
   - Inspections by geotechnical specialist
   - Descriptions of soils/rock in borelogs (Ref. 1)
   - Outcrop/cutting descriptions([c])
   - Record Extent of any cracking([c])
   - Other field tests (eg. EPT, etc)
   - Monitoring of ground movements([c])
   - Groundwater measurements and observations (seepage, subsurface erosion) ([c])
   - ♦ LABORATORY
   - Outline tests undertaken
   - Summarise results
   - Previous testing in local area

6. SUBSURFACE CONDITIONS
   - Geological interpretation([c])
   - Summarise subsoil conditions, eg. extent of fill([c]) topsoil, nature and distribution of soils/rock
   - Describe soil strengths/density, likely behaviour – refer to tests and logs
   - Highlight weak/sensitive/loose soils or rock defects
   - Describe groundwater conditions, subsurface drainage, expected seasonal fluctuation

Interpretation/Discussion

7. SLOPE STABILITY (Ref. 2,3,4,5)
   - ♦ ENGINEERING GEOLOGICAL ASSESSMENT
   - Discuss site features
   - Discuss geological setting/influences([e])
   - Influence of rainfall/groundwater
   - Reasons for landform (local, regional)
   - Likely slope failure mechanisms
   - Potential for instability
   - Effects of the development on slopes([f])
   - Consequence of instability
   - Empirical assessment (qualitative)
   - Risk rating applied([g])
   - State whether stability analyses are required
   - ♦ GEOLOGICAL ENGINEERING ANALYSES
   - Geotechnical slope model correct?
   - Analytical method stated
   - Determination of critical section of slope
   - Assessment of strength parameters
   - Assessment of groundwater profile/rainfall
   - Back analysis of any existing failures
   - External loads due to the development
   - State need for seismic analysis
   - Normal FOS requirements:
     - Static (Design gwt) FOS ≥ 1.5
     - Static (Extreme gwt) FOS ≥ 1.2
     - Seismic (150 year EQ) FOS ≥ 1.2
   - Sensitive analyses for parameters required?
   - Results and comments.

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8. GEOTECHNICAL EFFECTS OF DEVELOPMENT

- Slope stability risk increased or reduced?
- Is the development feasible?
- Need to drain slopes (surface/subsurface)?
- Subsurface drainage beneath fills?
- Need to retain slopes/secure rock faces?
- Foundation conditions/requirements
- Effect of stormwater/effluent disposal
- Effect of service lines rupture (eg. SW, sewer)
- Effect of river/coastal erosion
- Seismic effects on development and slope
- Maintenance requirements for life of the development

9. CONCLUSIONS AND RECOMMENDATIONS

10. STATEMENT

Statement by geotechnical assessor as to their ability & qualifications to prepare this geotechnical assessment

---

APPENDICES

- Borelogs, Testpit Logs, Logs of Exposures (Ref.1)
- Laboratory Results
- Specifications for Remedial Works/Fills
- Site Photos

DRAWINGS/FIGURES

- Site Plan([d]): O Borehole/Testpit Locations
  O Outline of Proposed Development

Site Engineering Geological Maps([d])

- Site Contours Maps([d])) Cuts and fills
- Cross Sections) Indicated
- Geotechnical Model
- Stability Analyses Results

REFERENCES

1. Australian Standard 1726 – 1993, Geotechnical Site Investigations
2. Assessment of Slope Stability at Building Sites, BRANZ Study SR4, (1997)
3. Slope Stability in Urban Development, DSIR Series 122 (1981)
4. Stability of House Sites & Foundations, Earthquake & War Damages Commission, NZ Geomechanics Society (1980)
5. Landslide Risk Management Concept & Guidelines AGS, Sub-committee on Landslide Risk Management Australian Geomechanics Vol. 35, Vol. 35 March 2000
   Land Assessment for Development Suitability, Burns & Farquhar, NZ Geotechnical Symposium (1996)

NOTES

(a) This checklist is intended as a guide for typical stability investigation & assessments for residential developments. There may be additional requirements for specifically difficult sites, large scale developments and regional hazards
(b) Indicate on site plan
(c) Indicate on site engineering geological map
(d) These plans/maps are best combined if possible
(e) Ref.3 provides a valuable outline of stability problems peculiar to selected areas of NZ
(f) Refer BRANZ document Fig 3 (ref 2 above), Stability House Sites and Foundations (ref. 4 above)
(g) See Table 1. Design of Permanent Slopes for Residential Development, Crawford & Millar (1999)

EXCERPT

from Crawford SA & Millar P.J.
The Design of Permanent Slopes for Residential Development, 8th Aust & NZ Conference on Geomechanics, Vol. 1999

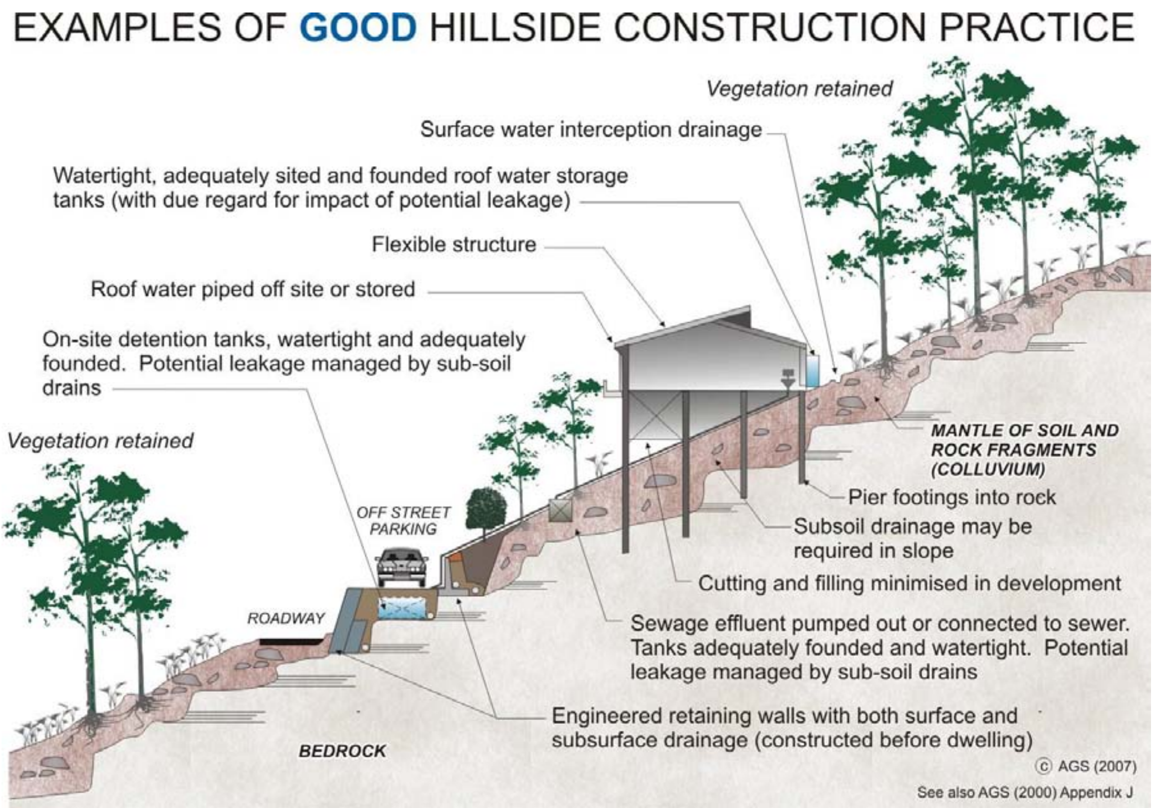
APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

	GOOD ENGINEERING PRACTICE	*POOR ENGINEERING PRACTICE*
ADVICE		
GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical practitioner at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
PLANNING		
SITE PLANNING	Having obtained geotechnical advice, plan the development with the risk arising from the identified hazards and consequences in mind.	Plan development without regard for the Risk.
DESIGN AND CONSTRUCTION		
HOUSE DESIGN	Use flexible structures which incorporate properly designed brickwork, timber or steel frames, timber or panel cladding. Consider use of split levels. Use decks for recreational areas where appropriate.	Floor plans which require extensive cutting and filling. Movement intolerant structures.
SITE CLEARING	Retain natural vegetation wherever practicable.	Indiscriminately clear the site.
ACCESS & DRIVEWAYS	Satisfy requirements below for cuts, fills, retaining walls and drainage. Council specifications for grades may need to be modified. Driveways and parking areas may need to be fully supported on piers.	Excavate and fill for site access before geotechnical advice.
EARTHWORKS	Retain natural contours wherever possible.	Indiscriminatory bulk earthworks.
CUTS	Minimise depth. Support with engineered retaining walls or batter to appropriate slope. Provide drainage measures and erosion control.	Large scale cuts and benching. Unsupported cuts. Ignore drainage requirements
FILLS	Minimise height. Strip vegetation and topsoil and key into natural slopes prior to filling. Use clean fill materials and compact to engineering standards. Batter to appropriate slope or support with engineered retaining wall. Provide surface drainage and appropriate subsurface drainage.	Loose or poorly compacted fill, which if it fails, may flow a considerable distance including onto property below. Block natural drainage lines. Fill over existing vegetation and topsoil. Include stumps, trees, vegetation, topsoil, boulders, building rubble etc in fill.
ROCK OUTCROPS & BOULDERS	Remove or stabilise boulders which may have unacceptable risk. Support rock faces where necessary.	Disturb or undercut detached blocks or boulders.
RETAINING WALLS	Engineer design to resist applied soil and water forces. Found on rock where practicable. Provide subsurface drainage within wall backfill and surface drainage on slope above. Construct wall as soon as possible after cut/fill operation.	Construct a structurally inadequate wall such as sandstone flagging, brick or unreinforced blockwork. Lack of subsurface drains and weepholes.
FOOTINGS	Found within rock where practicable. Use rows of piers or strip footings oriented up and down slope. Design for lateral creep pressures if necessary. Backfill footing excavations to exclude ingress of surface water.	Found on topsoil, loose fill, detached boulders or undercut cliffs.
SWIMMING POOLS	Engineer designed. Support on piers to rock where practicable. Provide with under-drainage and gravity drain outlet where practicable. Design for high soil pressures which may develop on uphill side whilst there may be little or no lateral support on downhill side.	
DRAINAGE		
SURFACE	Provide at tops of cut and fill slopes. Discharge to street drainage or natural water courses. Provide general falls to prevent blockage by siltation and incorporate silt traps. Line to minimise infiltration and make flexible where possible. Special structures to dissipate energy at changes of slope and/or direction.	Discharge at top of fills and cuts. Allow water to pond on bench areas.
SUBSURFACE	Provide filter around subsurface drain. Provide drain behind retaining walls. Use flexible pipelines with access for maintenance. Prevent inflow of surface water.	Discharge roof runoff into absorption trenches.
SEPTIC & SULLAGE	Usually requires pump-out or mains sewer systems; absorption trenches may be possible in some areas if risk is acceptable. Storage tanks should be water-tight and adequately founded.	Discharge sullage directly onto and into slopes. Use absorption trenches without consideration of landslide risk.
EROSION CONTROL & LANDSCAPING	Control erosion as this may lead to instability. Revegetate cleared area.	Failure to observe earthworks and drainage recommendations when landscaping.
DRAWINGS AND SITE VISITS DURING CONSTRUCTION		
DRAWINGS	Building Application drawings should be viewed by geotechnical consultant	
SITE VISITS	Site Visits by consultant may be appropriate during construction/	
INSPECTION AND MAINTENANCE BY OWNER		
OWNER'S RESPONSIBILITY	Clean drainage systems; repair broken joints in drains and leaks in supply pipes. Where structural distress is evident see advice. If seepage observed, determine causes or seek advice on consequences.	

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

HILLSIDE CONSTRUCTION PRACTICE

Sensible development practices are required when building on hillsides, particularly if the hillside has more than a low risk of instability (GeoGuide LR7). Only building techniques intended to maintain, or reduce, the overall level of landslide risk should be considered. Examples of good hillside construction practice are illustrated below.



WHY ARE THESE PRACTICES GOOD?

Roadways and parking areas - are paved and incorporate kerbs which prevent water discharging straight into the hillside (GeoGuide LR5).

Cuttings - are supported by retaining walls (GeoGuide LR6).

Retaining walls - are engineer designed to withstand the lateral earth pressures and surcharges expected, and include drains to prevent water pressures developing in the backfill. Where the ground slopes steeply down towards the high side of a retaining wall, the disturbing force (see GeoGuide LR6) can be two or more times that in level ground. Retaining walls must be designed taking these forces into account.

Sewage - whether treated or not is either taken away in pipes or contained in properly founded tanks so it cannot soak into the ground.

Surface water - from roofs and other hard surfaces is piped away to a suitable discharge point rather than being allowed to infiltrate into the ground. Preferably, the discharge point will be in a natural creek where ground water exits, rather than enters, the ground. Shallow, lined, drains on the surface can fulfil the same purpose (GeoGuide LR5).

Surface loads - are minimised. No fill embankments have been built. The house is a lightweight structure. Foundation loads have been taken down below the level at which a landslide is likely to occur and, preferably, to rock. This sort of construction is probably not applicable to soil slopes (GeoGuide LR3). If you are uncertain whether your site has rock near the surface, or is essentially a soil slope, you should engage a geotechnical practitioner to find out.

Flexible structures - have been used because they can tolerate a certain amount of movement with minimal signs of distress and maintain their functionality.

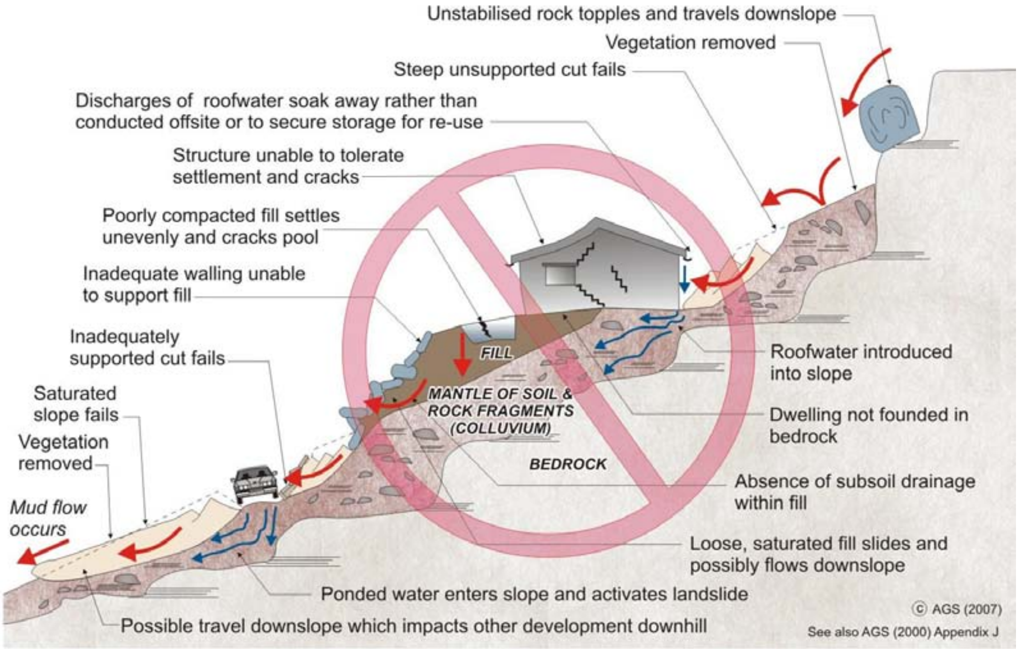
Vegetation clearance - on soil slopes has been kept to a reasonable minimum. Trees, and to a lesser extent smaller vegetation, take large quantities of water out of the ground every day. This lowers the ground water table, which in turn helps to maintain the stability of the slope. Large scale clearing can result in a rise in water table with a consequent increase in the likelihood of a landslide (GeoGuide LR5). An exception may have to be made to this rule on steep rock slopes where trees have little effect on the water table, but their roots pose a landslide hazard by dislodging boulders.

Possible effects of ignoring good construction practices are illustrated on page 2. Unfortunately, these poor construction practices are not as unusual as you might think and are often chosen because, on the face of it, they will save the developer, or owner, money. You should not lose sight of the fact that the cost and anguish associated with any one of the disasters illustrated, is likely to more than wipe out any apparent savings at the outset.

ADOPT GOOD PRACTICE ON HILLSIDE SITES

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

EXAMPLES OF POOR HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES POOR?

Roadways and parking areas - are unsurfaced and lack proper table drains (gutters) causing surface water to pond and soak into the ground.

Cut and fill - has been used to balance earthworks quantities and level the site leaving unstable cut faces and added large surface loads to the ground. Failure to compact the fill properly has led to settlement, which will probably continue for several years after completion. The house and pool have been built on the fill and have settled with it and cracked. Leakage from the cracked pool and the applied surface loads from the fill have combined to cause landslides.

Retaining walls - have been avoided, to minimise cost, and hand placed rock walls used instead. Without applying engineering design principles, the walls have failed to provide the required support to the ground and have failed, creating a very dangerous situation.

A heavy, rigid, house - has been built on shallow, conventional, footings. Not only has the brickwork cracked because of the resulting ground movements, but it has also become involved in a man-made landslide.

Soak-away drainage - has been used for sewage and surface water run-off from roofs and pavements. This water soaks into the ground and raises the water table (GeoGuide LR5). Subsoil drains that run along the contours should be avoided for the same reason. If felt necessary, subsoil drains should run steeply downhill in a chevron, or herring bone, pattern. This may conflict with the requirements for effluent and surface water disposal (GeoGuide LR9) and if so, you will need to seek professional advice.

Rock debris - from landslides higher up on the slope seems likely to pass through the site. Such locations are often referred to by geotechnical practitioners as "debris flow paths". Rock is normally even denser than ordinary fill, so even quite modest boulders are likely to weigh many tonnes and do a lot of damage once they start to roll. Boulders have been known to travel hundreds of metres downhill leaving behind a trail of destruction.

Vegetation - has been completely cleared, leading to a possible rise in the water table and increased landslide risk (GeoGuide LR5).

DON'T CUT CORNERS ON HILLSIDE SITES - OBTAIN ADVICE FROM A GEOTECHNICAL PRACTITIONER

More information relevant to your particular situation may be found in other Australian GeoGuides:

- GeoGuide LR1 - Introduction
- GeoGuide LR2 - Landslides
- GeoGuide LR3 - Landslides in Soil
- GeoGuide LR4 - Landslides in Rock
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR7 - Landslide Risk
- GeoGuide LR9 - Effluent & Surface Water Disposal
- GeoGuide LR10 - Coastal Landslides
- GeoGuide LR11 - Record Keeping

The Australian GeoGuides (LR series) are a set of publications intended for property owners; local councils; planning authorities; developers; insurers; lawyers and, in fact, anyone who lives with, or has an interest in, a natural or engineered slope, a cutting, or an excavation. They are intended to help you understand why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local council approval (if required) to remove, reduce, or minimise the risk they represent. The GeoGuides have been prepared by the Australian Geomechanics Society, a specialist technical society within Engineers Australia, the national peak body for all engineering disciplines in Australia, whose members are professional geotechnical engineers and engineering geologists with a particular interest in ground engineering. The GeoGuides have been funded under the Australian governments' National Disaster Mitigation Program.

Appendix H
1 Page

LIMITATIONS OF GEOTECHNICAL REPORTS

LIMITATIONS OF GEOTECHNICAL REPORTS

The purpose of this report is to provide a geotechnical assessment of the sites examined. The information provided herein will reduce the exposure to risks, but no geotechnical assessment can eliminate them. Nonetheless, even a rigorous assessment may fail to detect all of the geotechnical conditions on a site. Site variations may have occurred in areas not investigated or sampled.

This geotechnical report should not be used when the nature of the proposed site usage changes, when the size, layout, or location of the development is modified, when the site ownership changes nor should it be applied to a nearby area. No environmental assessment has been undertaken nor is implied.

This site geotechnical assessment identifies actual subsurface conditions where the samples were taken and at the time they were taken. Any soil tests completed, were carried out in Lane Pipers NATA accredited soil laboratory. Geotechnical engineers then interpreted the laboratory results and field data and rendered an opinion about the overall subsurface conditions, including the soil type, extent of the soil layers, and their likely impact on the proposed development, with a discussion of the implications considered likely. The actual conditions may differ from the inferred conditions, as no person (no matter how qualified) or even the most detailed subsurface investigation can predict with confidence what may be hidden by soil or water or may have altered with time. Often the interface between different geotechnical areas may be more abrupt or gradual than anticipated. The actual conditions in an area may differ from those predicted.

Site assessments are limited by time, and natural processes such as erosion, or mankind altering the ground conditions, including the site levels or filled areas, may affect a site assessment. This geotechnical assessment is prepared in response to a client's specific requirements. No person other than the client should apply the report without first conferring with Lane Piper Pty Ltd.

Costly problems can occur if the report is misinterpreted. To avoid these problems, Lane Piper Pty Ltd should be retained to work with the appropriate design professionals and to review the adequacy of their plans and specifications relative to the geotechnical matters.

This report should only be reproduced in its entirety. Reproduction of borehole or testpit logs alone without the entire report should not be permitted. Redrafting of the borehole or testpit logs for inclusion in drawings or other reports should not be allowed as errors in the drafting can occur. It is recommended that the report be made available in entirety to persons and organisations involved in the project such as contractors. Simply disclaiming responsibility for the accuracy of the subsurface or geotechnical information does not insulate the organisation from liability. The more information a contractor has available to him, the better able he is to avoid costly construction problems and costly adversarial situations.

Finally, geotechnical reports are based extensively on opinion and judgment and are less exact than other sciences. The report may contain a number of explanatory clauses or limitations on the results to inform the client about the restrictions of the report. These clauses are not meant to be exculpatory clauses to foist liability onto another person, but to identify where Lane Piper's and the client's responsibilities start and finish. Their use is to clarify where individual responsibilities lie and to allow the individual to take appropriate actions.