APPENDIX 3:

ENGINEERING SERVICES REPORT AND GEOTECHNICAL SITE SUITABILITY REPORT (BLOXAM BURNETT AND OLLIVER)

PUTAUAKI TRUST
Kawerau Industrial Development

Geotechnical Site Suitability Report

March 2012

BLOXAM BURNETT & OLLIVER

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Kawerau Industrial Development

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March 2012

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CONTENTS

1.0 INTRODUCTION

This site suitability geotechnical report has been prepared for the Kawerau District Council to assist in their application for a proposed plan change to rezone an area of land near Kawerau to industrial. The site is located off State Highway 34, just northeast of Kawerau, and opposite the Tasman Pulp and Paper mill, as shown in Figure 1 below.

Figure 1: Site Location Plan Google 2012

The site contains two areas to be investigated, Areas A and B, as shown in Figure 2 below. These areas are proposed to be rezoned to industrial. Areas C and E contain an existing Super Skid site and associated stormwater ponds, and have not been investigated as part of this report. Area D is proposed to be rezoned to rural, and therefore does not require investigation, however the results of a previous investigation in this area are briefly discussed in this report.

Figure 2: Site Plan

This report presents the results of a geotechnical investigation undertaken at the site. It assesses the suitability of the in situ soils to found industrial structures, to accept stormwater soakage, and makes general geotechnical recommendations. Characteristic information that is expected to be indicative of the site in general has been obtained, but further investigations will be required to allow detailed design work to proceed. Additional investigations supporting detailed designs should be tailored to the geometric and functional specifics of any proposed structure, in particular the loads imposed on the soils and acceptability of the resulting deformations under static and seismic conditions.

2.0 SITE DESCRIPTION

The site is located on State Highway 34, northeast of Kawerau, and opposite the Tasman Pulp and Paper Mill. The areas investigated currently consist of fenced farmland. Railway spur lines to the Super Skid site cross the proposed industrial area in an east/west direction, separating Areas A and B, and high voltage overhead transmission lines and a high pressure gas main cross the proposed industrial area in a north/south direction.

The site generally falls to the northeast at a relatively flat grade, draining into the catchment of the Rangitaiki River rather than the Tarawera River. The surface is undulating with bumps and hollows over a height range of approximately two metres. The surface is further scoured with shallow "bull holes" created by livestock in the loose surface soils. Fill has been placed to form the McKee Road overbridge over State Highway 34 (constructed between Areas A and D), and the railway spur line underpass under State Highway 34 (between Areas A and B). Other areas of minor cut and fill are expected to be present as typically found on farmland.

3.0 SITE GEOLOGY

The Institute of Geological and Nuclear Sciences geological map "Geology of the Rotorua Area" (2010) shows Tauranga Group alluvial deposits of recent Quaternary age in the vicinity of the site. The Tauranga Group deposits contain a wide range of materials, and local knowledge suggests that alluvial pumice deposits may be present in the surface soils at this site.

Seismicity and volcanism are considered to be significant issues at this site. There are many known faults near the site, and the surrounding area has geothermal activity and relatively recent volcanic activity. Of the faults, the closest mapped fault is the Edgecumbe Fault, approximately two kilometres northwest of the site. The 1987 Edgecumbe earthquake produced significant areas of regional settlement, with up to two metres close to the epicentre. Reported settlements reduced rapidly with offset from the epicentre, with none reported in Kawerau in the New Zealand Society for Earthquake Engineering reconnaissance report. However, the potential for significant displacements in the general region is noted, as is the broad similarity between the simplified soil profile shown in the NZSEE report and the specifics of this site. Some of the two metres "settlement" may be more attributable to fault movement, but the quantum is noted.

The site is close to the Okataina Volcanic Centre, which contains many recently active vents between Rotorua and Kawerau. The closest vent is Putauaki (Mount Edgecumbe) approximately 2.5 km southeast of the site, and the most recently active vent is Mount Tarawera (approximately 25 km southwest of the site).

4.0 FIELDWORK

Fieldwork was undertaken at the site on the 26th of January 2012. The location of all fieldwork is shown on the geotechnical testing location plan in Figure 3 below, and in the Site Plan in Appendix A. The fieldwork consisted of seven Cone Penetration Tests (CPT's), and two hand augured exploratory boreholes (BH), which were also used for soakage testing. The CPT's had a target depth of 20 m, with actual testing depths ranging from 10.3 m to 20 m depth. The boreholes were excavated to 2.0 and 2.3 m depth, with drilling stopped due to very slow progress. The soakage tests were undertaken when the boreholes were at 1.5 m depth, and tested in general accordance with the Building Code E1. Summary CPT results, exploratory borehole logs, and soakage test results are attached in Appendix B.

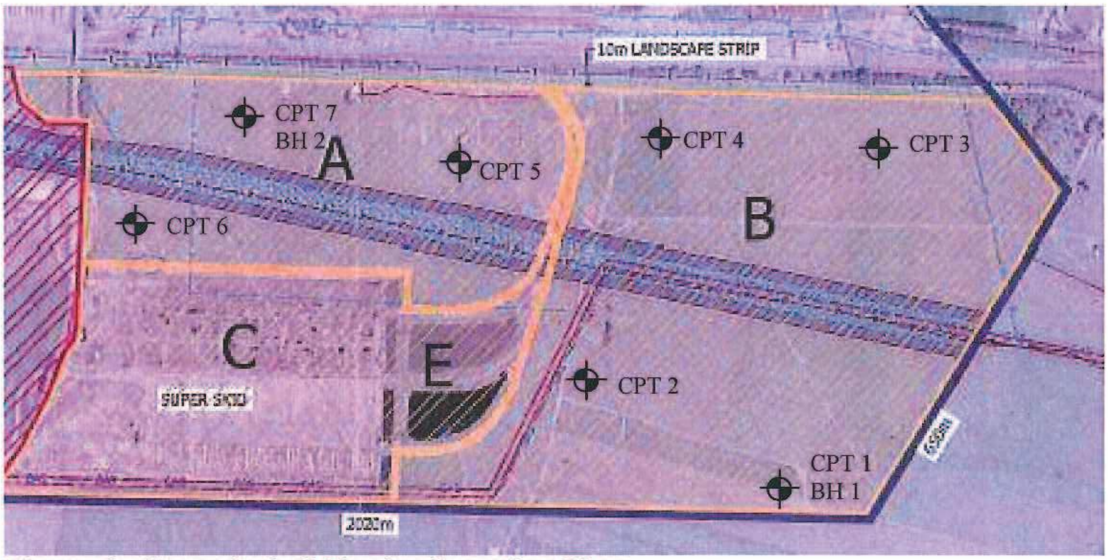


Figure 3: Geotechnical Testing Location Plan

5.0 SUBSOIL MATERIALS AND CONDITIONS

5.1 TOPSOIL

Topsoil was encountered at the surface of both boreholes down to 0.2 to 0.3 m depth. The topsoil was described as loose and dry, with frequent pumice gravels up to 5 mm diameter. The CPT soil behaviour index (shown in Figure 4 below) indicates that sand and gravely sand is present in the surficial soils, this is a reasonably good match for the soil description given in the borehole logs.

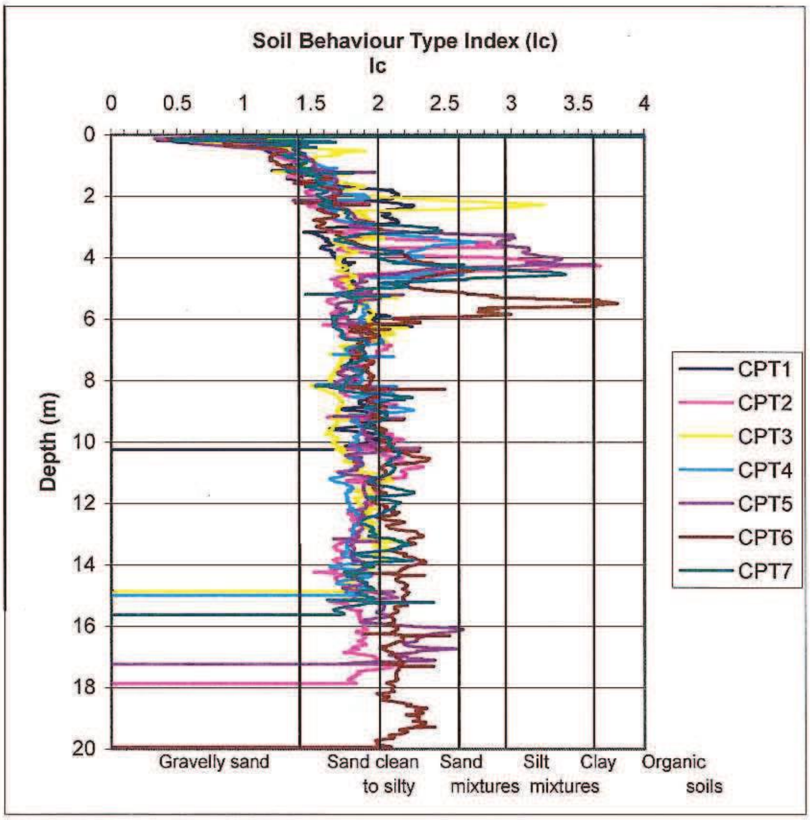


Figure 4: CPT Soil Behaviour

5.2 VOLCANIC ASH/LAPILLI

A layer of dark grey sand with occasional gravels up to 5 mm diameter was encountered below the topsoil in both boreholes. This layer had a sharp transition with the lighter coloured alluvial materials encountered below it. The layer was 0.15 to 0.2 m thick, and was described as being loose. Again, the CPT results indicating sand and gravely sand at this depth is a reasonably good match for this soil description.

5.3 ALLUVIUM

Alluvium was encountered in the boreholes below the ash/lapilli layer, from between 0.35 to 0.5 m below ground level, and extended to the base of both boreholes (2.0 and 2.3 m depth) and all CPT's (10.3 to 20.0 m depth). The alluvium encountered in the boreholes was typically described as fine to coarse grained sand, with frequent pumice gravels up to 20 mm diameter, light grey, and moist. The density increased with depth, with loose deposits encountered closer to the surface, and dense material encountered in the base of the boreholes (2.0 to 2.3 m depth).

The CPT soil behaviour index closely matches the borehole descriptions, with gravely to silty sands indicated from the base of the ash/lapilli layer down to 1.5 m below ground level, and clean to silty sands indicated from 1.5 m to 2.0 m below ground level. It is noted that inaccuracies in CPT textural and strength interpretations can occur in pumice material, as the CPT tends to crush sands and gravels. The use of boreholes in site specific geotechnical investigations would provide greater certainty with regard to material type and strength.

Clay layers within the top 5 m of soil were reported in the interpretations of five of the seven CPT's undertaken. These layers were typically about 1.0 m thick, with one weak clay layer reported in CPT 3 that was only 0.3 m thick. The clay layer in CPT 3 was reported closer to the surface than the other clay layers, starting at 2.14 m below ground level. The clay layers reported in the other CPT's were first encountered between 3.5 and 4.2 m depth below ground level. The strength of these layers ranged from firm to very stiff, with undrained shear strength interpretations of 20 to 151 kPa. Some of the CPT interpretations also suggest that organic soils may be present in places (CPT 2 at 4.2 m depth, and CPT 6 at 5.6 m depth). These clay layers are assumed to be alluvial in origin, as they are variable over the site. However it is possible that they may be derived from ash deposits that have eroded in some areas.

From 5.9 m to the base of the CPT's (a maximum of 20 m) the CPT results typically indicate that sands are present. The materials encountered and inferred are consistent with recent alluvial deposits of the Tauranga Group.

The density of the alluvium from CPT inferred SPT "N" values is shown in Figure 5 below. These results show that the materials at the site were typically very loose to medium dense from ground level down to 5.8 m below ground level. From 5.8 m to the base of the CPT's the soil density ranged from medium dense to very dense. Six of the seven CPT's stalled before they reached their target depth of 20 m due to very dense material being encountered.

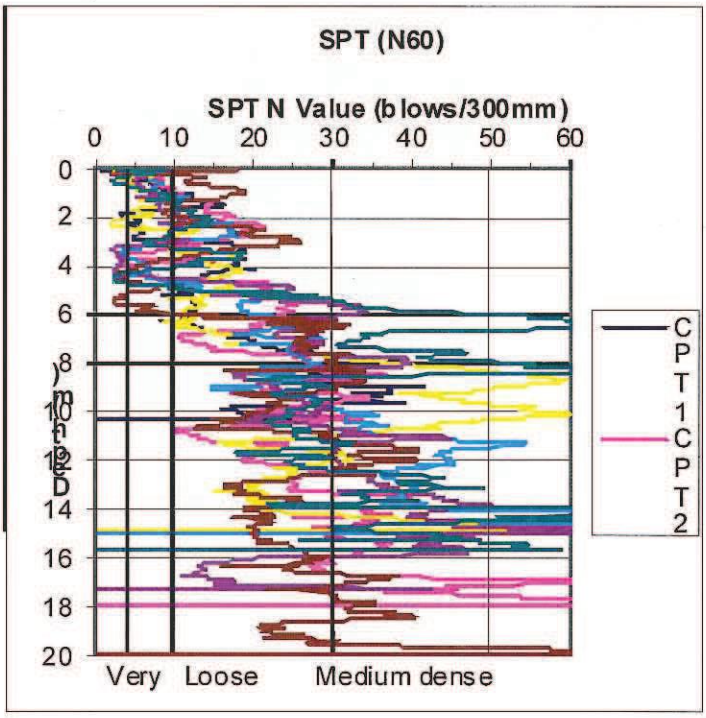
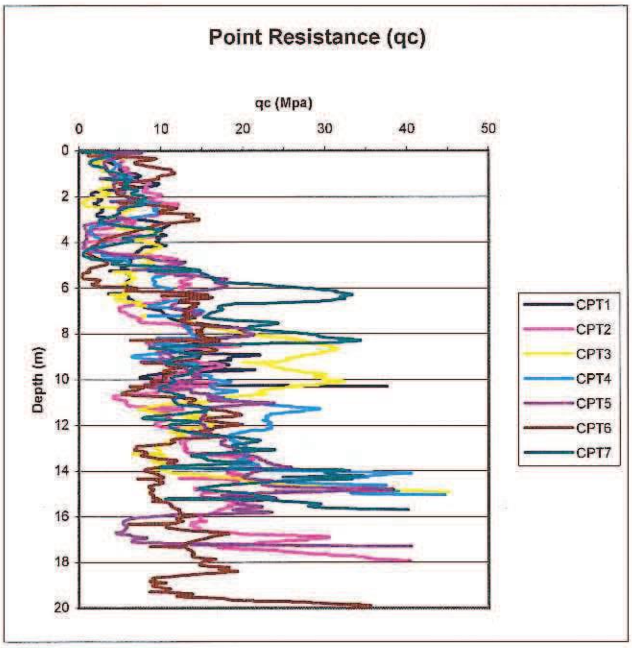


Figure 5: CPT Inferred SPT Values

CPT point resistance (q_c) and sleeve friction resistance (f_s) data are shown in Figures 6 and 7 below. There is significant variation in the CPT results, however an overall pattern does appear in the CPT results, with weaker layers encountered in the near surface soils of all CPT's (0 to 6 m depth). The impact of the weaker soil layers on foundation design will depend on the depth these layers are encountered at, and their thickness. Excavation of the very loose and loose surfical soils is expected to be required.



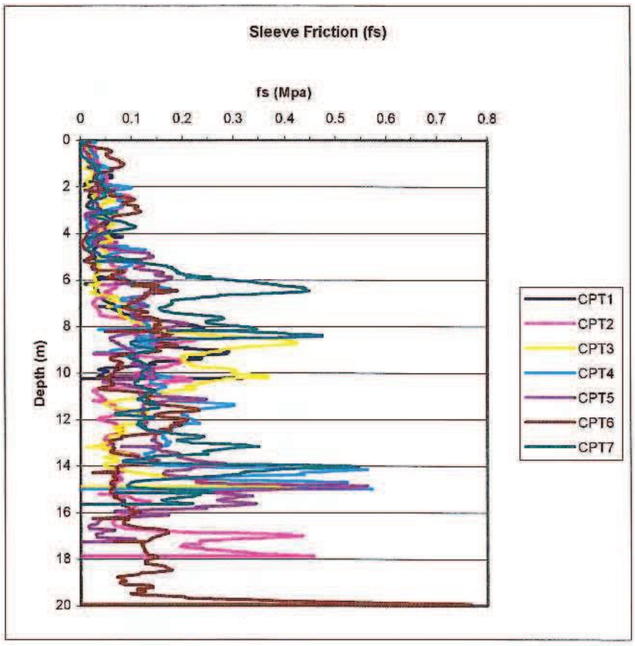


Figure 6: CPT Point Resistance

Figure 7: Sleeve Friction

Table 1: Sand CPT Data Summary for Near Surface Soils From 0.4 m to 5.0 m Depth

	q_c (MPa)	f_s (kPa)
Minimum	0.5	0
Maximum	14.6	143

5.4 PREVIOUS GEOTECHNICAL INVESTIGATION IN AREA D

An earlier geotechnical investigation has been undertaken in Area D for Mighty River Power. This investigation found similar materials to that encountered on the rest of the site – loose surficial sands grading to medium dense/dense sands with depth.

5.5 GROUNDWATER

Groundwater was not encountered in the exploratory boreholes during drilling. Groundwater was reported in the CPT's between 3.5 and 9.7 m depth. The watertable level of the CPT's in Area A was much lower than in Area B (5.5 to 9.7 m, compared with 3.5 to 4.5 m depth respectively). As the fieldwork was undertaken in January, higher groundwater levels are expected in winter.

5.6 SOAKAGE

The soakage tests indicate a soakage rate of approximately 1.1 m/h (the lower result of the two soakage tests). This rate falls within Category A of NZS 4610:1982, and is considered to be rapid to very rapid draining.

The Farm Manager has advised that site stormwater typically drains quickly into the soils, with no discernable overland flow path apparent due to the undulating topography. He also advised that the Super Skid soakage ponds occasionally overflow into the site, this issue would need to be addressed as part of the site development.

6.0 DESIGN CONSIDERATIONS

6.1 LIQUEFACTION POTENTIAL

Any unconsolidated sandy soils below the watertable are at risk of liquefying during seismic events, and therefore a preliminary liquefaction assessment has been undertaken using standard liquefaction analysis software, based on CPT data available. The following data has been adopted in the assessment:

Input data:

Site subsoil class:	D	(deep or soft soil, ex NZS 1170.5)
Zone factor:	0.29	(for Kawerau, ex NZS 1170.5)
Importance level:	2	("normal" structures, ex AS/NZS 1170.0)
Building design life:	50 years	(conventional Building Act limit)
Earthquake magnitude:	7.5	(implicit in NZS 1170.5 magnitude-weighted data)
Peak Ground Acceleration:	0.32 g	(derived from data above, to NZS 1170.5)
Groundwater level:	As shown on CPT readings and at 1.5m depth to account for winter groundwater level.	

Liquefaction is predicted at all CPT sites. Liquefaction-induced vertical settlement is also predicted at all CPT sites, peaking at 220 mm at CPT6. Assessment of liquefaction risk and consequential settlement is limited by the depth of the CPT traces. Assessment suggests that consequential settlements typically occur over almost the entire depth of the trace. This suggests that deeper data may suggest increased settlements.

Estimated liquefaction-induced settlements are very comparable using two different CPT-based analysis tools, but this information should only be used to indicate that risk of liquefaction and its consequences does exist at this site. Specific liquefaction consequences should be assessed for individual structures, considering their importance and ability to accommodate the consequences of liquefaction, as well as the specifics of their location; surcharge imposed by construction of any proposed building; and the benefits of any proposed ground improvement.

There is no obvious "free face" or other obvious potential reason to expect earthquake-induced liquefaction to precipitate lateral spread displacements, but the risk should not be ignored when considering the specifics of individual development proposals, the geotechnical investigations for which should be tailored to facilitate the assessment of such risks.

The design of foundations and selection and detailing of finishes should make allowance for predicted settlements resulting from a site specific liquefaction assessment. So too should the detailing of utility services approaching and exiting buildings, with particular emphasis on the interface between ground-supported utility services and those supported by the building.

6.2 BUILDING FOUNDATIONS

The preferred foundations for industrial buildings constructed at this site are typically expected to impose static (unfactored) design bearing pressures of about 100 kPa on the subsoils via local strip footings. Some structures, with a heavier intended use, may impose greater foundation design bearing pressures. The buildings are expected to have concrete slab floors.

Shallow Foundations

The minimum basic allowable bearing pressure for a strip footing for the surficial cohesionless soils characteristic of this site has been assessed as being of the order of 100 kPa, before modification for footing width and depth. This assumes that soft zones and any organic layers at or close to foundation levels are removed and replaced, and that the subgrade is compacted prior to placing foundations. The near surface soils are therefore expected to typically have adequate bearing capacity to support appropriately detailed lightweight industrial structures, providing that the risk of vertical displacement described below is seen as acceptable and that the specific soils at individual building sites are investigated and found to be comparable.

The relatively low shear strength of the near surface clay lenses suggested by the CPT results indicates that settlement of these materials under static loading could be an issue where they occur. Earthquake-induced settlement due to liquefaction of the subsoils is also expected, as described above. This should be investigated at each proposed building site prior to building design, and any predicted settlement considered in the design and detailing of foundation and utility services. Detailed site testing for each site should further investigate the presence and impact of any organic soils.

Shallow spread or strip footings are typically expected to be adequate to found lightweight industrial structures. For heavier structures, or where settlement due to static or seismic effects cannot be accepted, a stiffened raft foundation has the potential to smear differential settlement effects over the area of a building. Piled foundations may be required if design levels are to be retained. Excavation of surficial very loose and organic material, and replacement with competent fill should be allowed for below all foundations and on-grade concrete slabs, as outlined above.

The recent Christchurch earthquakes have highlighted the advantages of using slab and footing reinforcement which is ductile and placed in sufficient quantities to address the risk of uncontrolled crack width. Both are seen as appropriate at this site. So too is careful detailing and location of slab and footing joints. Cost implications are minor, and robustness advantages are significant.

Utility services should be detailed to accommodate expected absolute and differential displacements, with particular attention being warranted for details at building perimeters.

Piled Foundations

Where piled foundations are required, deep geotechnical investigations will be required to support detailed design. We note that the limited indicative liquefaction assessment undertaken to date suggests that the depth of soils contributing to earthquake-inducted settlement typically approaches the full depth of the CPT traces, and that this trend may continue with deeper investigations. Consideration of negative skin friction resulting from settlement in the design of deep foundations is warranted, irrespective of the cause of settlement.

6.3 STORMWATER DRAINAGE

As described in Section 2 above, the site falls generally to the northeast at a flat grade, with an undulating surface of bumps and hollows. These undulations are likely to be removed by development, resulting in the potential for runoff to adjacent land to the northeast if this is not captured and directed to soakage devices.

The rapid to very rapid soakage of the soils tested at this site indicates that stormwater disposal to ground will be an appropriate solution to manage the effects of development on stormwater runoff. The similar results obtained from the two soakage tests undertaken, with one test undertaken in each of Area A and B, combined with advice from the Farm Manager that the site has very good soakage, gives confidence in the test results obtained.

Groundwater levels measured in the CPT's ranged from 3.5 to 9.7 m below ground level. Winter watertable levels are expected to be approximately one metre above this, from 2.5 to 8.7 m depth below ground level.

Further detailed testing will be required at each stormwater soakage site to allow detailed stormwater soakage designs to be developed.

6.4 EARTHWORKS AND ROAD FORMATION

The near surface soils at the site are generally suitable for re-contouring and road formation. Finished contours for building platforms, yard areas and roads will need to integrate with proposals for other infrastructure including stormwater drainage and sewerage.

6.5 VOLCANIC ERUPTIONS

There is a risk of volcanic activity in the vicinity of the site. A map of the area affected by the 1886 Tarawera Rift eruption (A.P. Thomas, 1887) showed that the site was covered in one foot of ash (300 mm) during this eruption. This precedent is noted, and it is recommended that potential ash load be considered in structure designs. Methods of addressing this issue could be through roof angle or structure design.

7.0 FURTHER INVESTIGATIONS

Further geotechnical investigation will be required at each building platform location to confirm soil strengths and stiffness, identify any concerns, and facilitate appropriate foundation design. Liquefaction and settlement concerns are expected. Hence detailed assessments of the consequences of liquefaction and settlements induced by static loads should be undertaken. Further soakage testing will be required in each stormwater soakage location to allow the detailed design of these systems to be developed.

8.0 CONCLUSION

Site areas A and B site are generally considered to be suitable for their proposed use for industrial development. The near surface soils at the site are expected to have sufficient strength to support industrial buildings on conventional slab and footing foundations, but the risks and consequences of earthquake-inducted liquefaction and settlement under static loads are noted and warrant further investigation to support the development of detailed design proposals. Stiffened or deep foundations may be warranted. Slab / footing / pile detailing should cater for potential static and seismic displacements and loads. Utility services which may be adversely affected by absolute or differential static or earthquake displacement should be detailed to accommodate predicted displacements and loads.

Comparable liquefaction / settlement concerns may well be present at practically all potential industrial land sites in the Kawerau/Edgecumbe/Whakatane area. As such we have assumed that these concerns represent risks which need to be identified and accommodated in the proposed development, rather than fundamentally affecting the suitability of this site for industrial purposes. However careful geotechnical investigation and interpretation is seen as essential to ensure that developments are technically appropriate.

The rapid to very rapid permeability of the site soils suggest that stormwater disposal to ground will be an acceptable solution at this site.

9.0 LIMITATION

The recommendations and options contained in this report are based on our visual reconnaissance of the site, information from geological maps, data from the field investigation, and the results and interpretation of in situ testing of soil at the site. Inferences about the nature and continuity of the subsoils away from and beyond test locations are made, but cannot be guaranteed by the limited geotechnical investigations to date.

The information in this report should be supplemented with more detailed geotechnical investigation and assessment tailored to each building site and the specific loads, performance demands and functional requirements of the building proposed for that site.

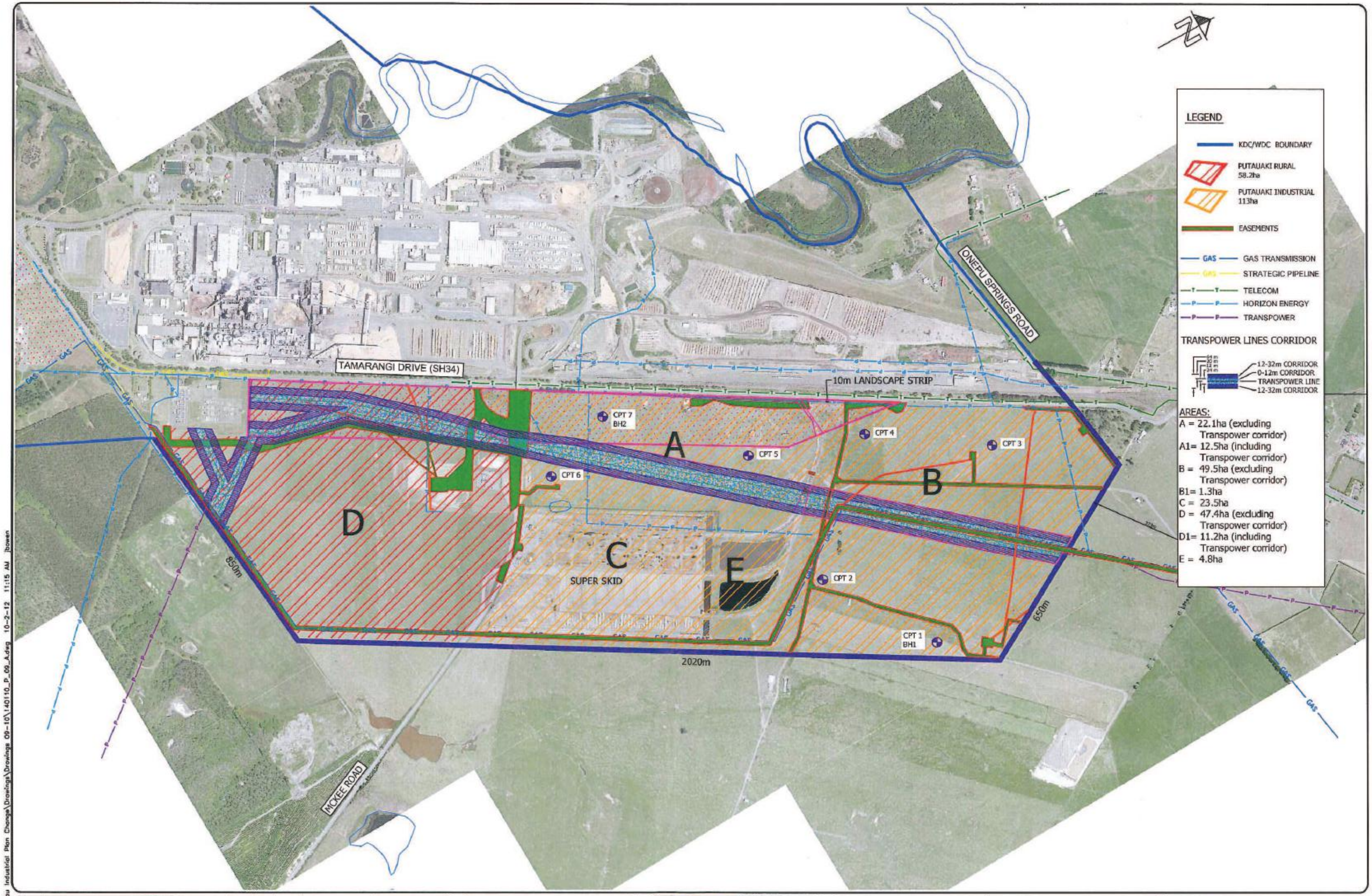
This report has been prepared for the particular project described in the report and no responsibility is accepted for the use of any part of this report in any other context or for any other purposes.

Appendix A

Site Plan

Appendix B

Fieldwork Results



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client

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drawing title GEOTECHNICAL TESTING LOCATION PLAN

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revision A

original sheet size A1

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LOG OF INVESTIGATION

DEPTH	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE REFERENCE	WATER LEVEL
		Topsoil, frequent pumice gravels upto 5mm ø, loose, dark brown, dry		
0.5 1.0 1.5 2.0		Sand (f-cg), dark grey, loose, moist, occasional gravels up to 5mm ø. 0.35m sharp transition to light reddish brown, occasional pumice gravels upto 20mm ø. 0.5m coarse grained, light grey. very moist, medium dense 50mm band of slight orange brown staining. slighty silty, loose, occasional pumice gravels upto 5mm ø. f-cg, silty, light brown, medium dense.	#1 0.8-1m	
2.5		End of Borehole 2.3m. Very slow drilling.		

LOCATION

NOTES
Excavated to 1.5 m with 100mmø auger. caved to 1.02m during presoaking. Excavated back to 1.5m and undertook soakage test. caved to 1.02m again. Excavated to base of borehole with 60mmø auger.

WEATHER WHEN LOGGED: Fine

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CLIENT: Putauaki Trust
PROJECT: Industrial Plan Change
JOB NO.: 140110
LOGGED BY: TH
TEST NO.: BH1
DATE: 26/1/12
SHEET 1 OF 1

LOG OF INVESTIGATION

DEPTH	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE REFERENCE	WATER LEVEL
		Topsoil, dark brown, loose, frequent pumice gravels upto 5mm ∅, dry.		
0.5		Sand (f-cg), dark grey, loose, moist. medium grained, brown, rare gravels. coarse grained, light brown, occasional pumice gravels upto 20mm ∅	#2 0.7-0.8m	
1.0		light grey, frequent pumice gravels upto 20mm ∅. very moist, frequent pumice gravels upto 7mm ∅.		
1.5		Medium to coarse grained, light grey and light brown, medium dense, occasional pumice gravels upto 10mm ∅. 1.7m light grey, rare gravels upto 7mm ∅. 1.8m dense		
2.0				
		End of borehole 2.0m. very slow drilling.		

LOCATION

NOTES
Excavated to 1.5m with 100mm ∅ auger. caved to 1.04m during presoaking. Excavated back to 1.5m and undertook soakage test. caved to 1.08m. Excavated to base of borehole with 50mm ∅ auger.

WEATHER WHEN LOGGED: Fine

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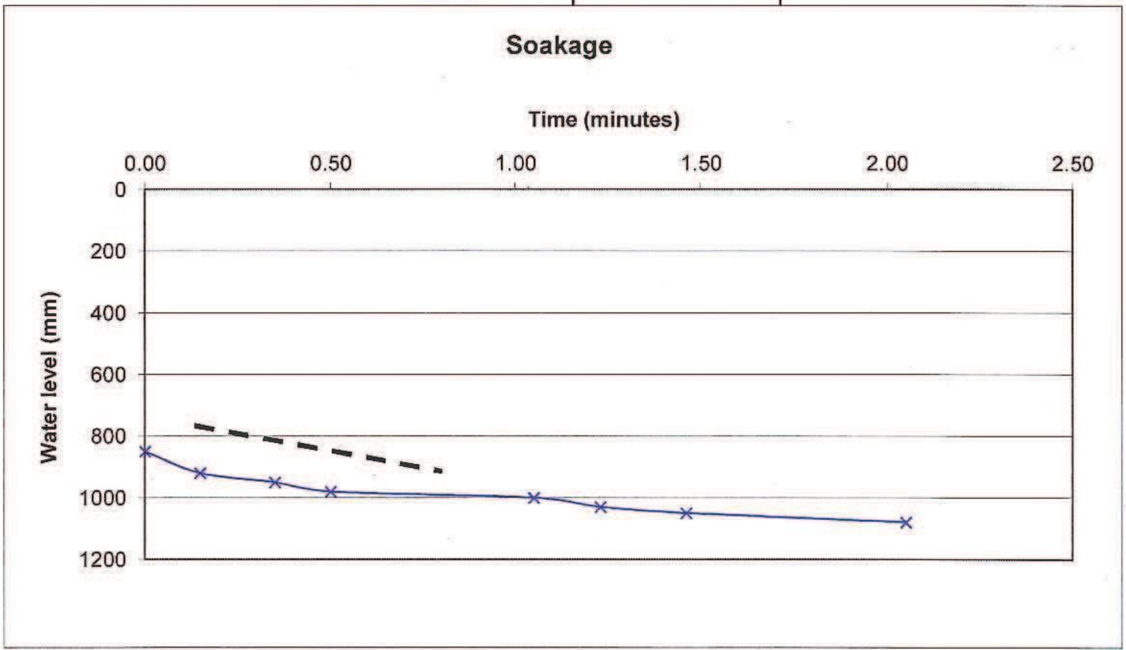
CLIENT: Putauaki Trust PROJECT: Industrial Plan Change
JOB NO.: 140110 LOGGED BY: TH
TEST NO.: BH2 DATE: 26/1/12 SHEET 1 OF 1

Soakage Test	Clock Time h	m	s	Depth to water (mm)	Acc Time (Min)	Slope on Graph (mm/h)
BH 1	9	34	45	640	0.00	
	9	35	10	760	0.25	28800
	9	35	33	830	0.48	18261
	9	35	49	850	0.64	7500
	9	36	7	870	1.22	2069
	9	36	30	930	1.45	15652
	9	37	30	970	2.45	2400
	9	37	50	1000	3.05	3000
	9	39	0	1020	4.15	1091
	Average from 0.25-1.45 minutes (middle section of graph)					**8500** E1 soakage rate

Excavated to 1.5m depth. Presoaked with 20 litres water. Borehole almost empty after two minutes.
Borehole had caved to 1.02m depth. Excavated back down to 1.5m for soakage test.
Borehole again caved to 1.02m depth following the application of 20 litres of water.



Testing was in accordance with the New Zealand Building Code E1 with the following exceptions:
Presoaked till empty.

Input data:

Hole diameter	0.1	m		
For gradient:	8500	mm/h		
		Average water level	0.175	m
		Water drop	0.17	m

Infiltration rate from first principles (volume of water and area available) for 8500 mm/h gradient

Volume of water	0.0013	m3
/ Area available for infiltration	0.06	m2
/ time	0.02	hr
Infiltration	1.063	m/hr

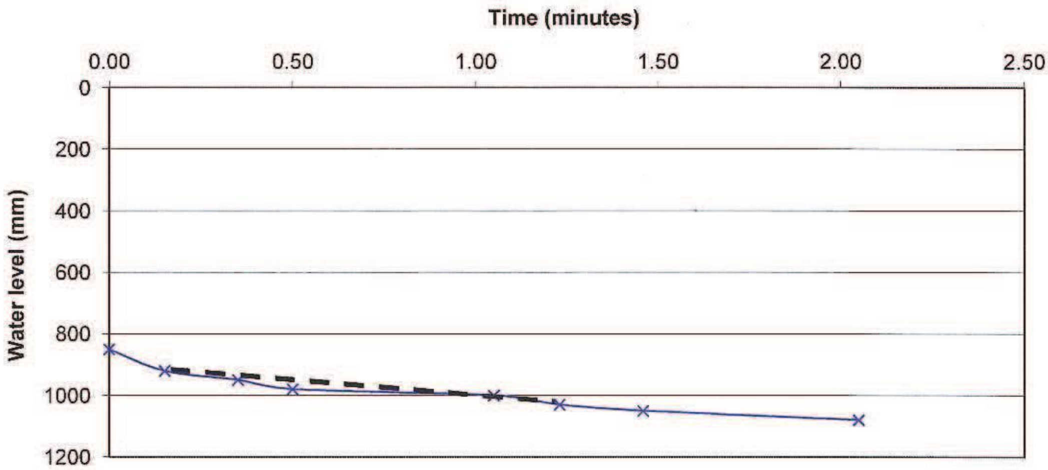
Infiltration rate from ACC soakage design manual worksheet 1 (Diameter x gradient x 1000)/4 x d)

Diameter	0.1	m
Gradient	0.1417	m/min
d	0.175	m
Infiltration	20.238	l/m2/min
	1.214	m/hr

d = distance between the midpoint of the last two readings and the base of the borehole

Soakage Test	Clock Time h	m	s	Depth to water (mm)	Acc Time (Min)	Slope on Graph (mm/h)	
BH2	12	6	45	850	0.00		
	12	7	0	920	0.15	28000	
	12	7	20	950	0.35	9000	
	12	7	35	980	0.50	12000	
	12	7	50	1000	1.05	2182	
	12	8	8	1030	1.23	10000	
	12	8	31	1050	1.46	5217	
	12	8	50	1080	2.05	3051	
Average from 0.15-1.23 minutes (middle section of graph)						**6111**	E1 soakage rate

Excavated to 1.5m depth. Presoaked with 20 litres water. Borehole almost empty after two minutes.
Borehole had caved to 1.04m depth. Excavated back down to 1.5m for soakage test.
Borehole again caved to 1.08m depth following the application of 20 litres of water.



Testing was in accordance with the New Zealand Building Code E1 with the following exceptions:
Presoaked till empty.

Input data:				
Hole diameter	0.1	m		
For gradient:	6111	mm/h		
		Average water level	0.105	m
		Water drop	0.11	m

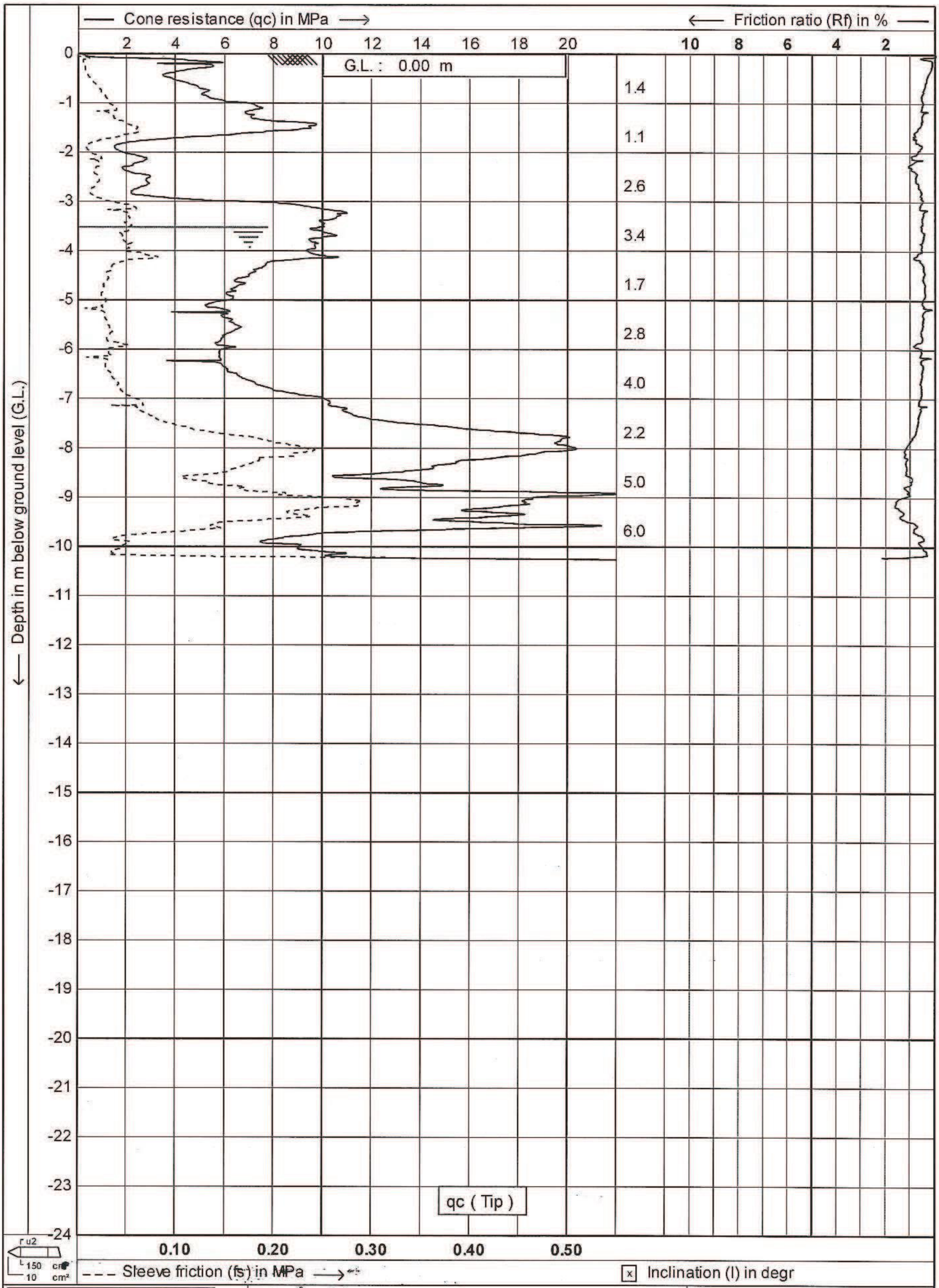
Infiltration rate from first principles (volume of water and area available) for 8500 mm/h gradient

Volume of water	0.0009	m3
/ Area available for infiltration	0.04	m2
/ time	0.02	hr
Infiltration	1.175	m/hr

Infiltration rate from ACC soakage design manual worksheet 1 (Diameter x gradient x 1000)/4 x d)

Diameter	0.1	m
Gradient	0.1019	m/min
d	0.105	m
Infiltration	24.250	l/m2/min
	1.455	m/hr

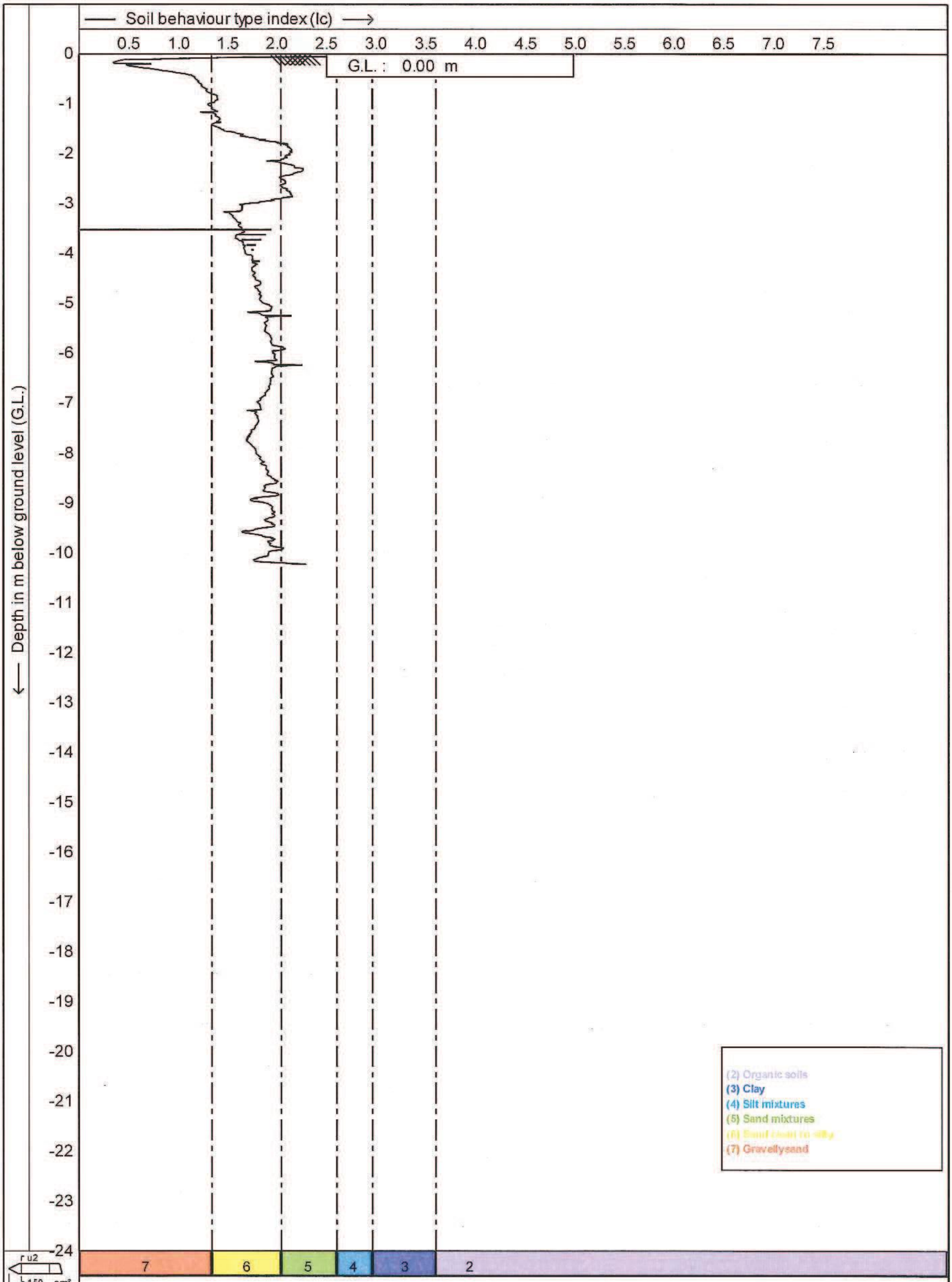
d = distance between the midpoint of the last two readings and the base of the borehole



Test according A.S.T.M. Standard D 5778-07	Date	: 26-1-2012
	Cone no.	: C10CFIIP.F57
Project : **Putauaki Trust Site Investigations**	Project no. :	**02BBO1**
Location: **Kawerau**	CPT no.	: **01** 1/14

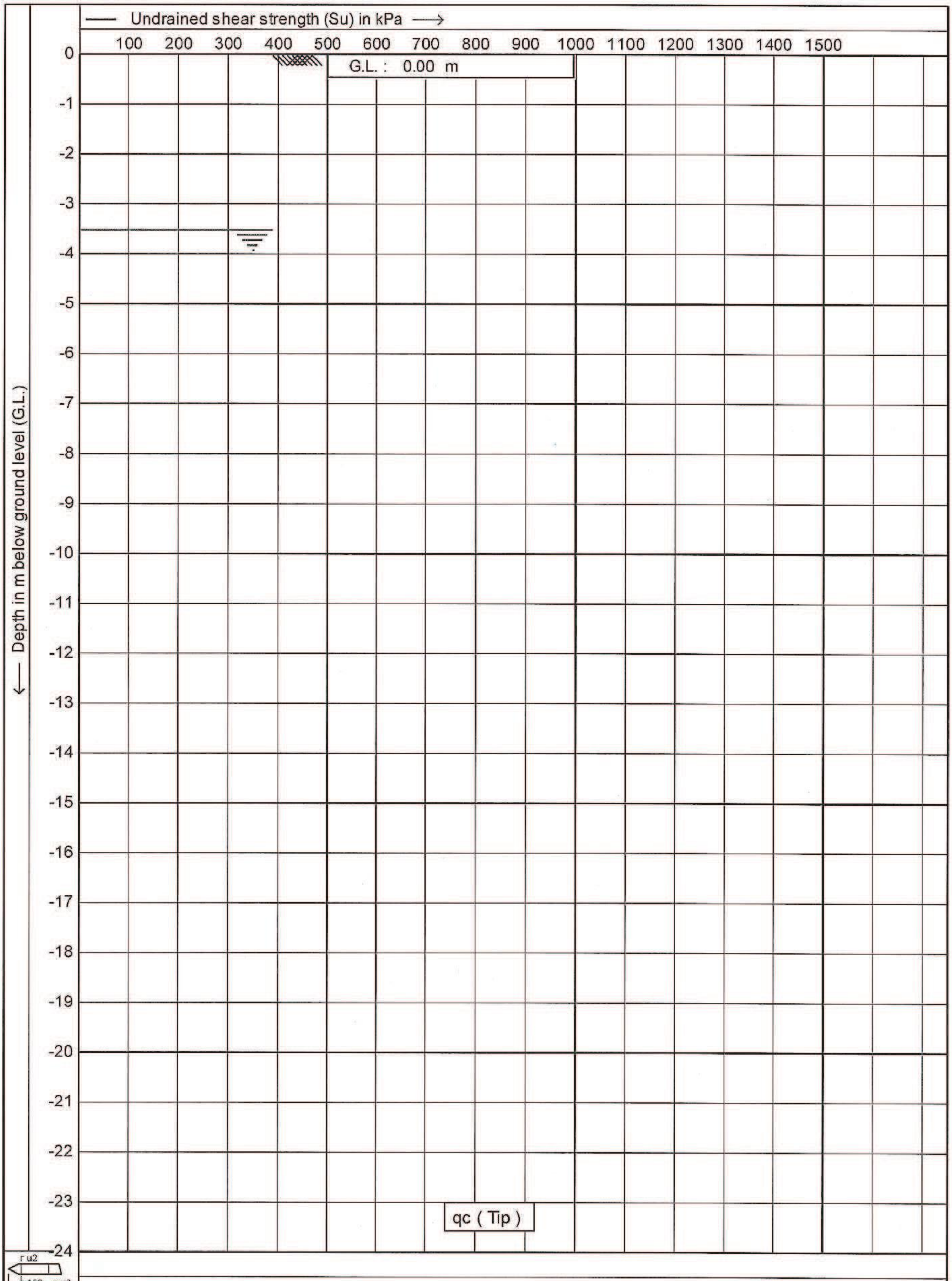
Perry Drilling Geo Probe Ground Drilling and Environmental Specialists

CPTask V1.31



Perry DRILLING GEO PROBE Ground Drilling and Environmental Specialists	Test according A.S.T.M. Standard D 5778-07	Date : **26-1-2012**
	Project : **Putauaki Trust Site Investigations**	Cone no. : **C10CFIIP.F57**
		Project no. : **02BBO1**
	Location: **Kawerau**	CPT no. : **01** **9/14**

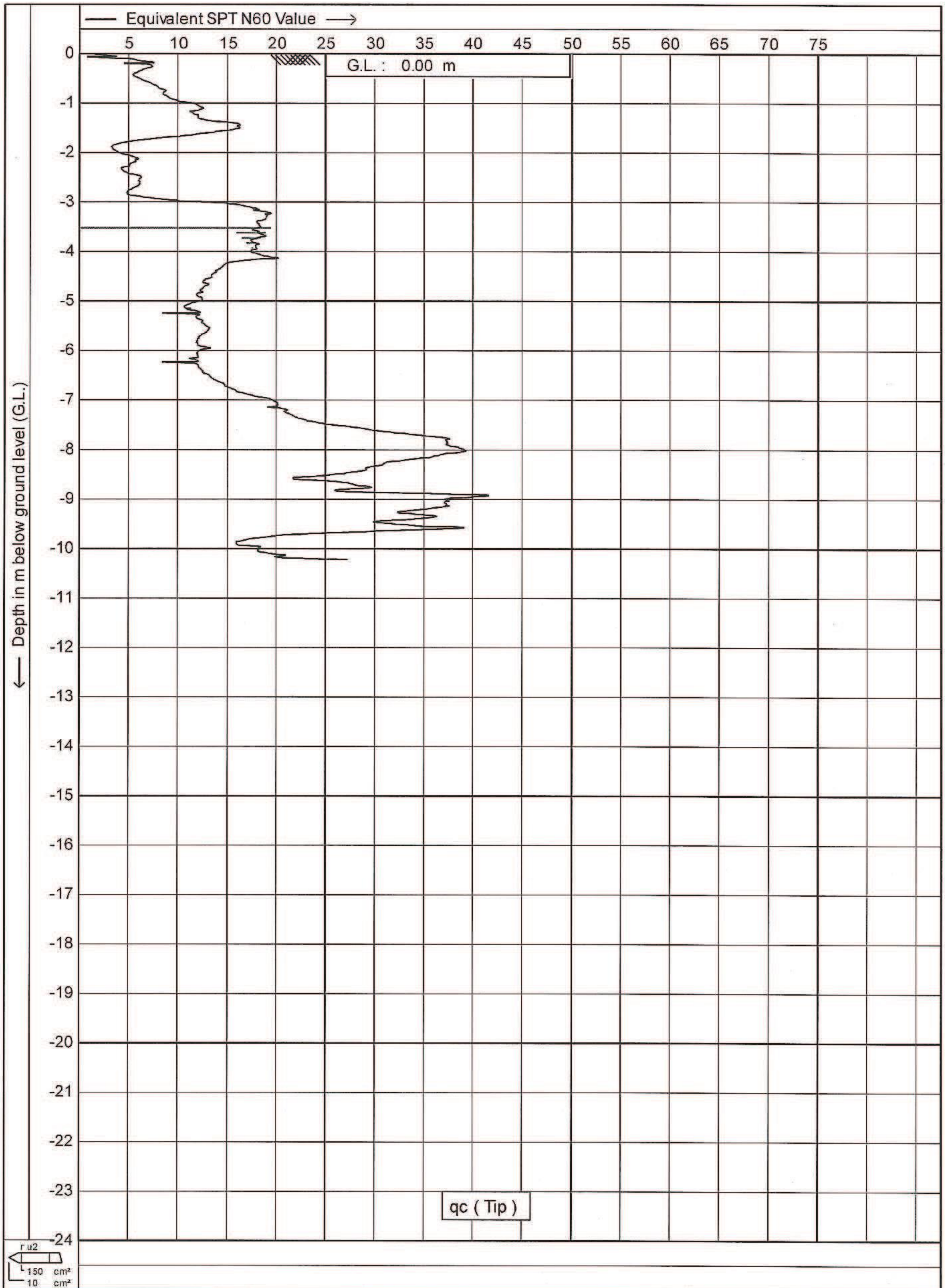
CPTask V1.31



CPTask V1.31

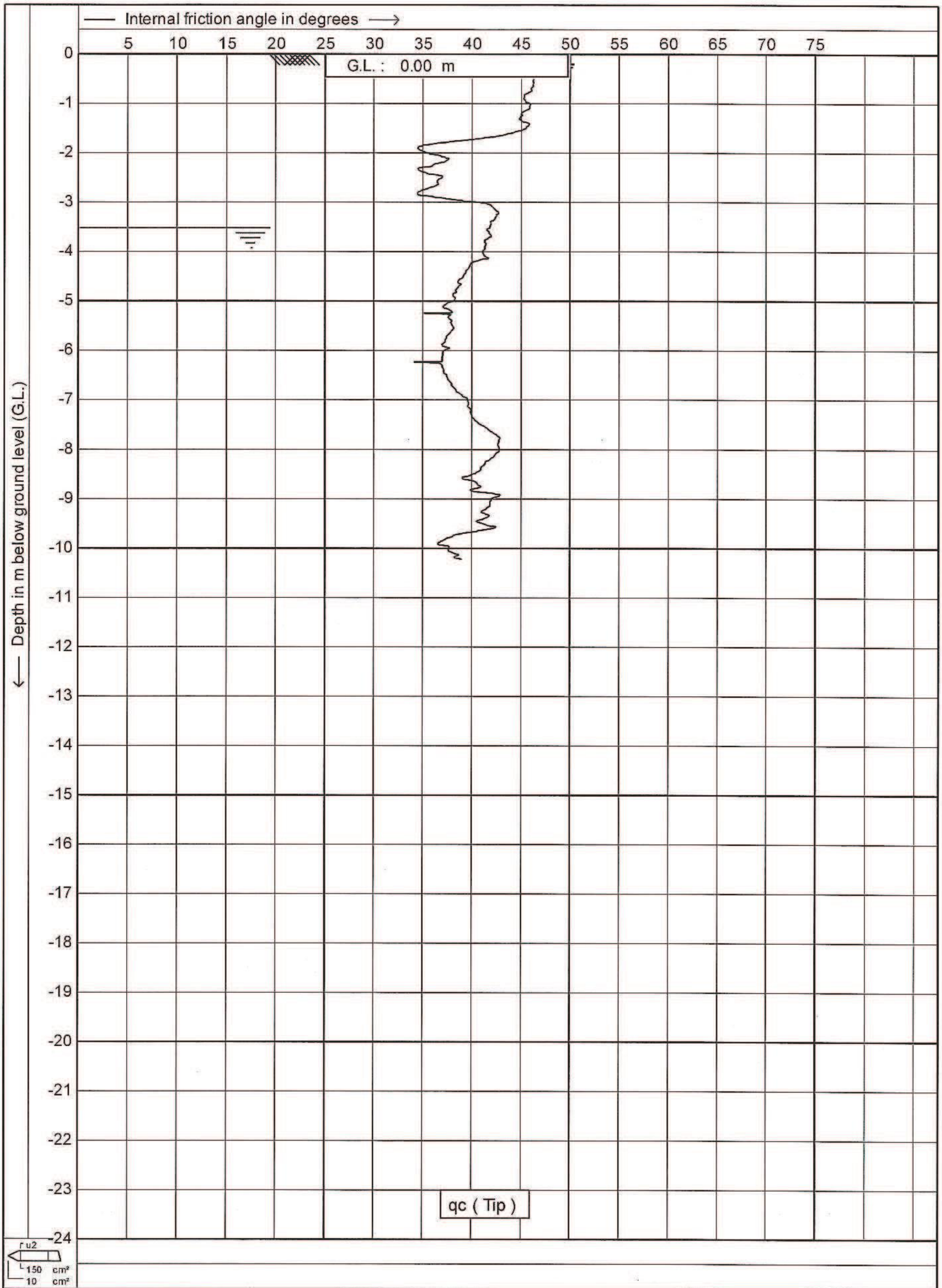
Penny Drilling — GEO PROBE — Ground Drilling and Environmental Specialists

Test according A.S.T.M. Standard D 5778-07	Date : **26-1-2012**
	Cone no. : **C10CFIIP.F57**
Project : **Putauaki Trust Site Investigations**	Project no. : **02BBO1**
Location: **Kawerau**	CPT no. : **01**



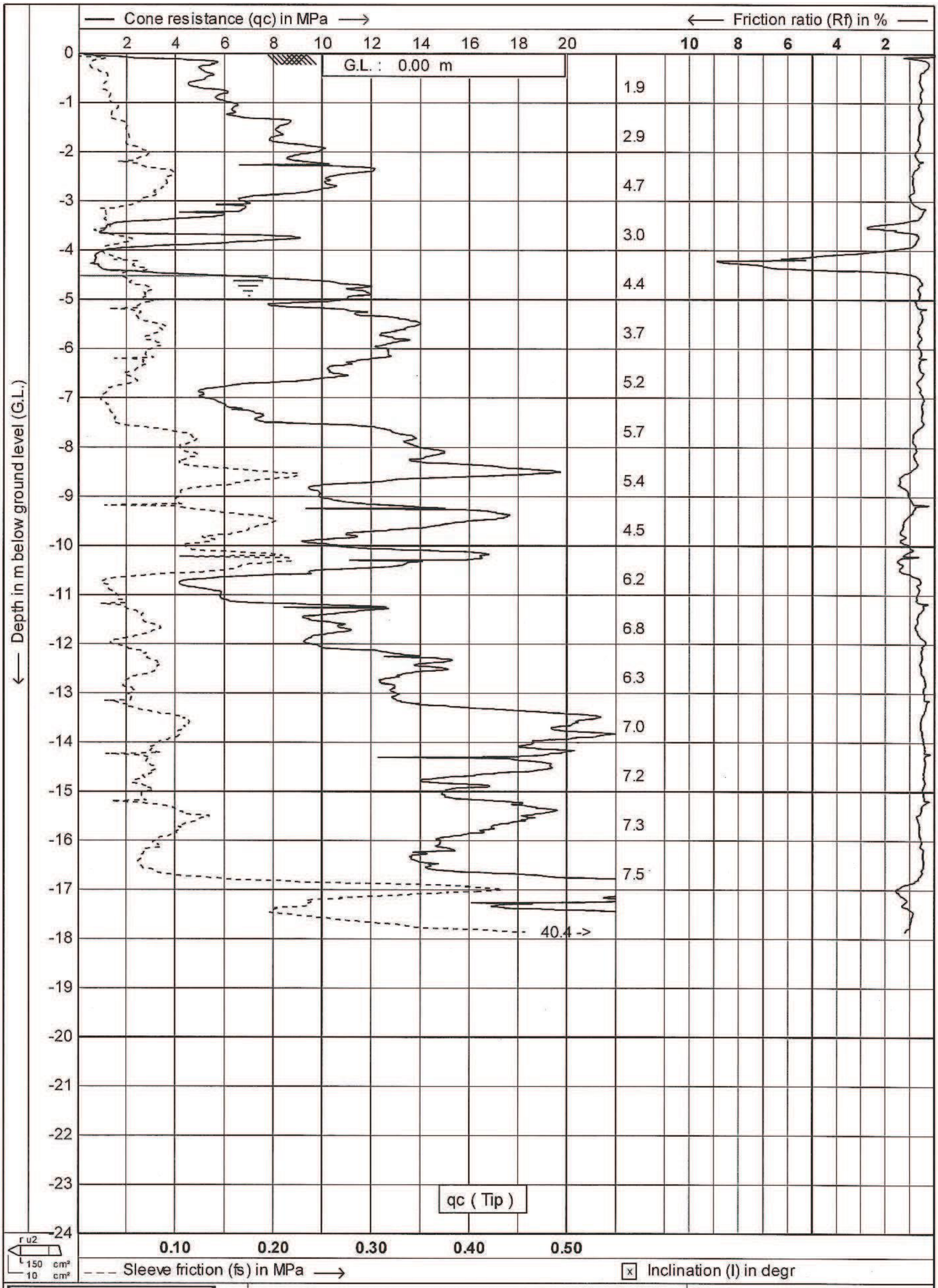
CPTask V1.31

Test according A.S.T.M. Standard D 5778-07		Date : **26-1-2012**
Project :	**Putauaki Trust Site Investigations**	Cone no. : **C10CFIIP.F57**
		Project no. : **02BBO1**
Location:	**Kawerau**	CPT no. : **01** **12/14**



Perry DRILLING GEO PROBE Ground Drilling and Environmental Specialists	Test according A.S.T.M. Standard D 5778-07	Date : **26-1-2012**
	Project : **Putauaki Trust Site Investigations**	Cone no. : **C10CFIIP.F57**
		Project no. : **02BBO1**
	Location: **Kawerau**	CPT no. : **01** **14/14**

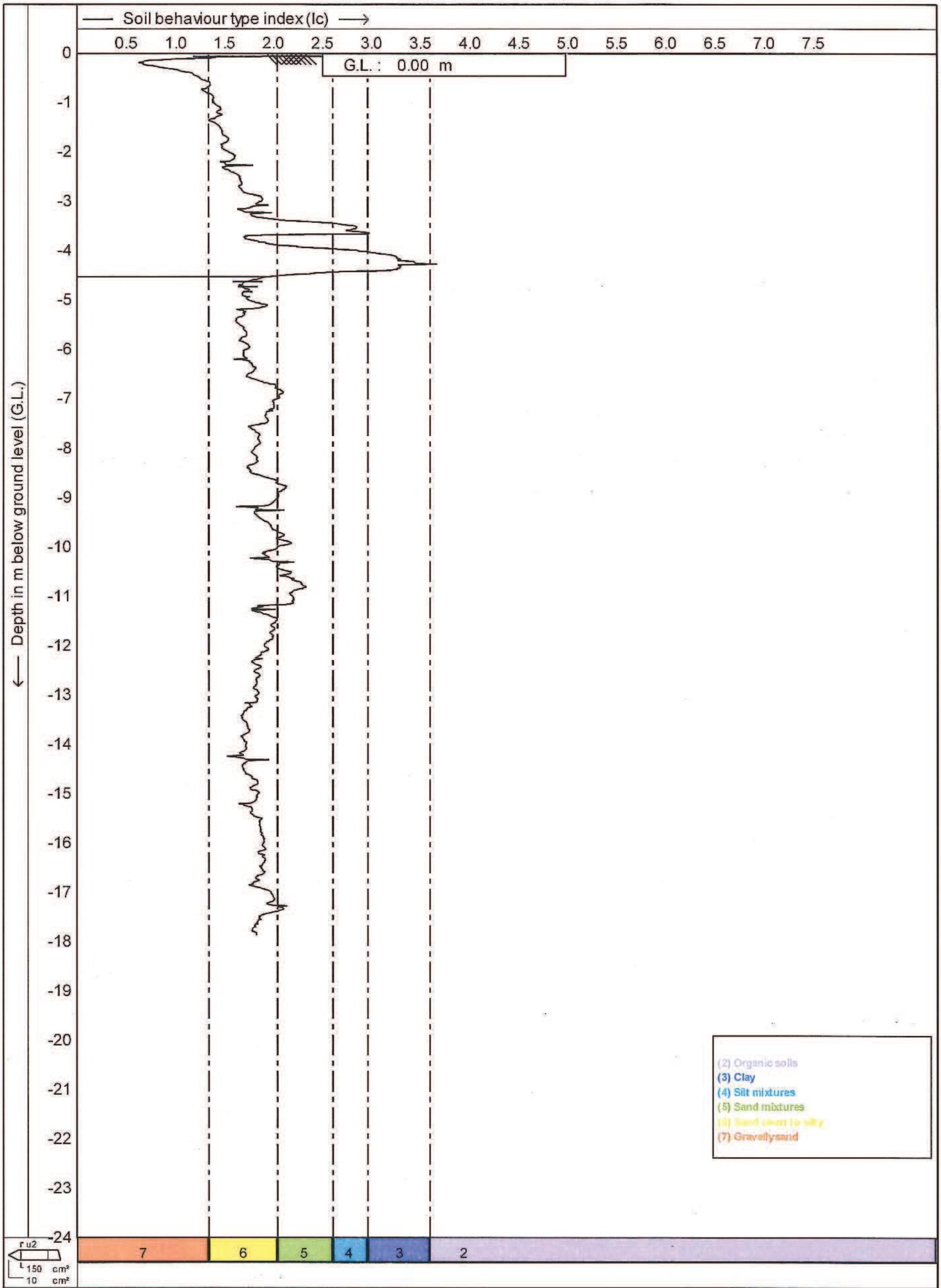
CPTask V1.31



CPTask V1.31

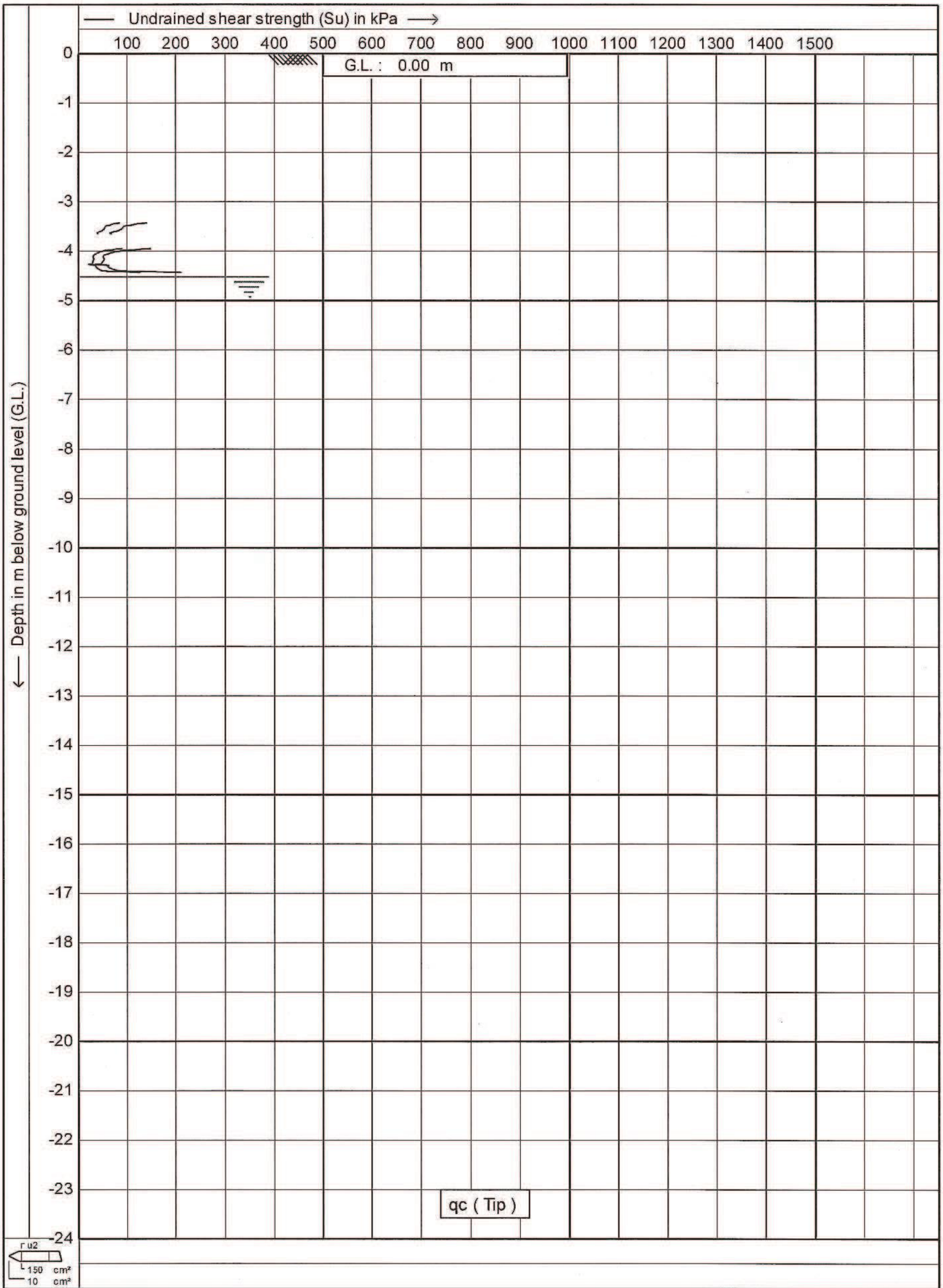
Perry Drilling GEO PROBE A division of Perry Engineering. Ground Drilling and Environmental Specialists

	Test according A.S.T.M. Standard D 5778-07	Date : **26-1-2012**
	Project : **Putauaki Trust Site Investigations**	Cone no. : **C10CFIIP.F57**
		Project no. : **02BBO1**
	Location: **Kawerau**	CPT no. : **02** **1/14**



Perry DRILLING GEO PROBE Ground Drilling and Environmental Specialists	Test according A.S.T.M. Standard D 5778-07	Date : **26-1-2012**
		Cone no. : **C10CFIIP.F57**
	Project : **Putauaki Trust Site Investigations**	Project no. : **02BBO1**
	Location: **Kawerau**	CPT no. : **02** — **9/14**

CPTask V1.31



Test according A.S.T.M. Standard D 5778-07

Project : **Putauaki Trust Site Investigations**

Location: **Kawerau**

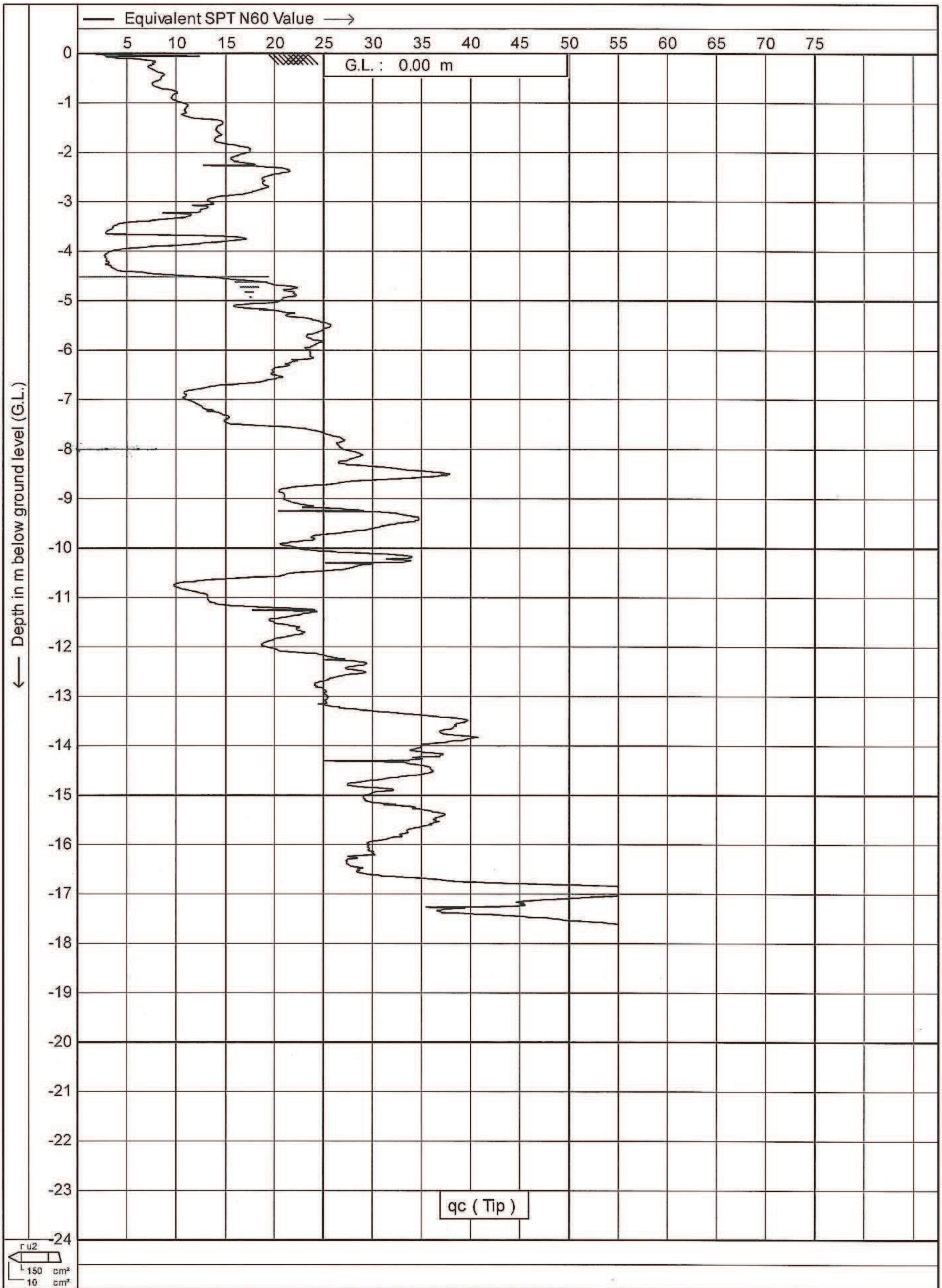
Date : **26-1-2012**

Cone no. : **C10CFIIP.F57**

Project no. : **02BBO1**

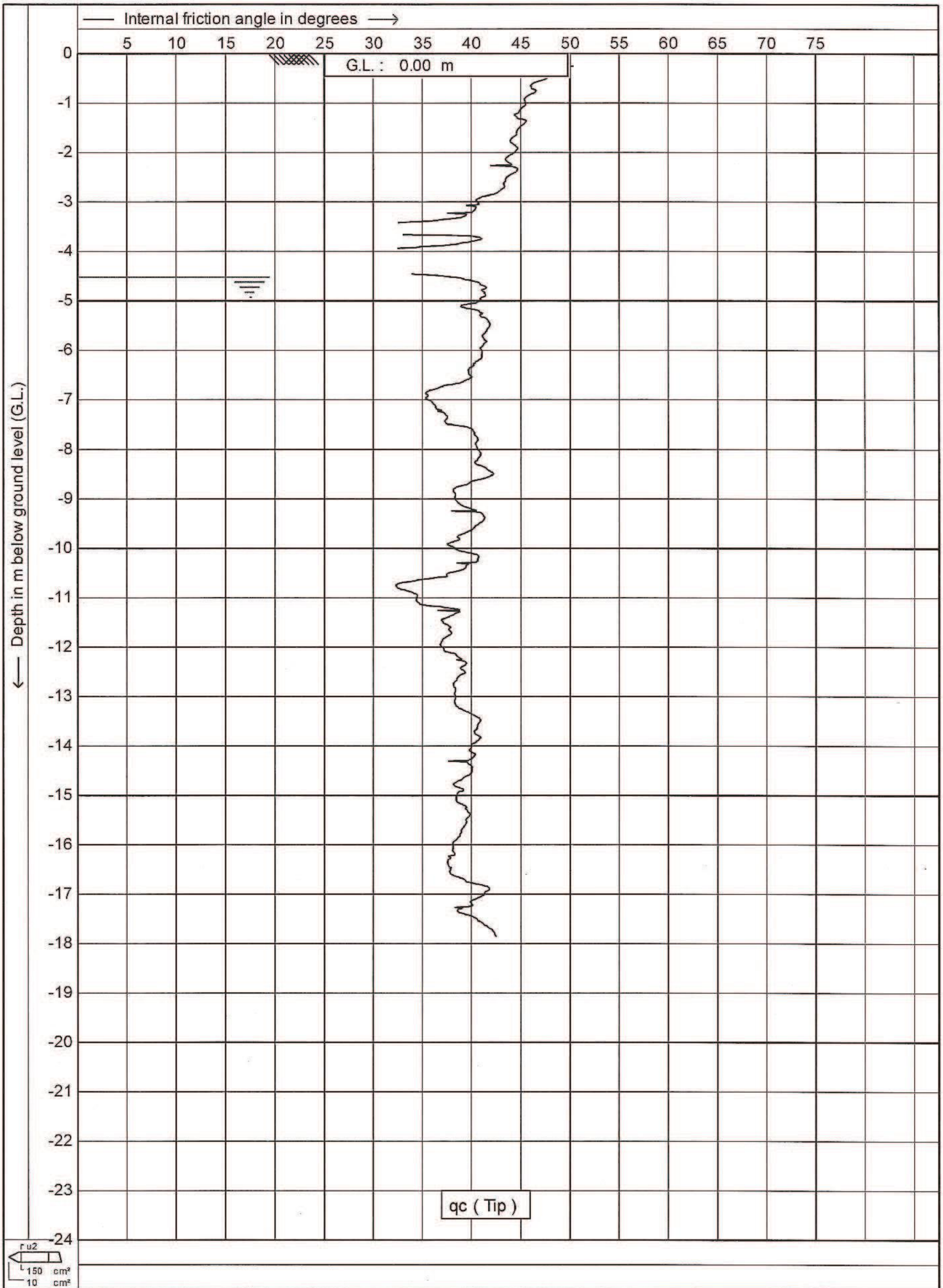
CPT no. : **02**

10/14



Perry Drilling / GEO PROBE, Ground Drilling and Environmental Specialists	Test according A.S.T.M. Standard D 5778-07	Date	: **26-1-2012**
	Project : **Putauaki Trust Site Investigations**	Cone no.	: **C10CFIIP.F57**
		Project no. :	**02BBO1**
	Location: **Kawerau**	CPT no.	: **02** **12/14**

CPTask V1.31



CPTask V1.31

Perry Drilling
GEO PROBE
A division of Perry Drilling
Ground Drilling and Environmental Specialists

Test according A.S.T.M. Standard D 5778-07	Date	: **26-1-2012**
Project : **Putauaki Trust Site Investigations**	Cone no.	: **C10CFIIP.F57**
	Project no. :	**02BBO1**
Location: **Kawerau**	CPT no.	: **02** **14/14**