CHRISTCHURCH CITY COUNCIL

PRK_0227_BLDG_007 EQ2
Linwood Nursery – Glasshouses (×5)
320 Linwood Ave, Linwood

QUALITATIVE ASSESSMENT REPORT

FINAL

- Rev C
- 23 May 2013

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Contents

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Approval

	Signature	Date	Name	Title
Author		15/04/2013	Nick Calvert	Senior Structural Engineer
Approver		15/04/2013	Alex Martin	Project Manager

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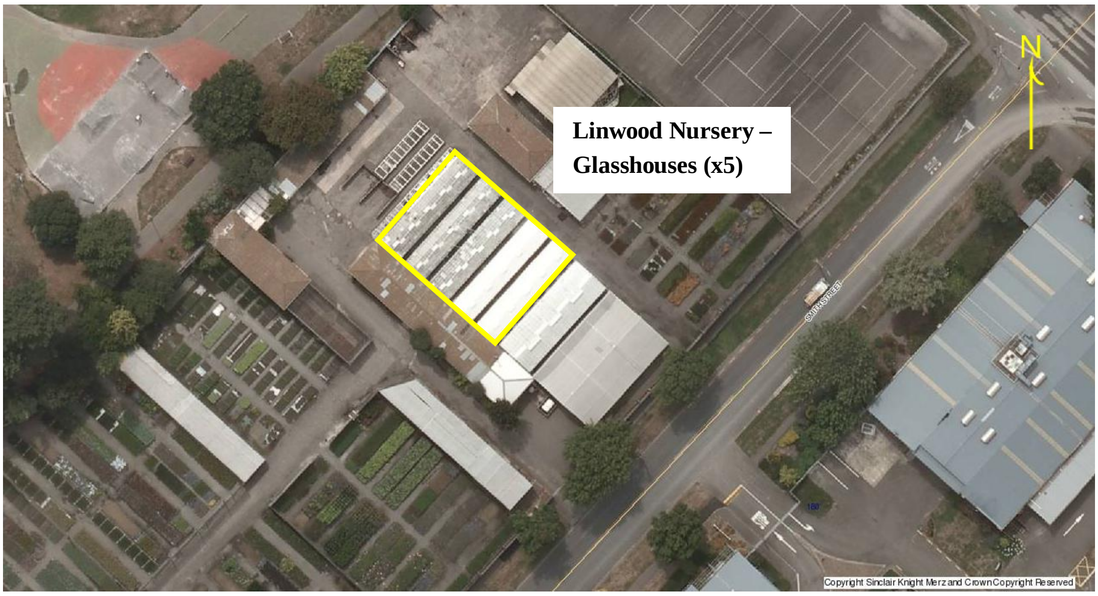
1. Executive Summary

1.1. Background

A Qualitative Assessment was carried out on Linwood Nursery – Glasshouses (×5) located at 320 Linwood Ave, Linwood. Linwood Nursery – Glasshouses (×5) is made up of a series of 5 glasshouses orientated in a north-south direction each comprised of concrete walls and a glass clad aluminium frame structure. An aerial photograph illustrating the location of Linwood Nursery – Glasshouses (×5) is shown in Figure 1 below. A detailed description outlining the building's age and construction type is given in Section 5 of this report.



- **Figure 1 Aerial Photograph of Linwood Nursery – Glasshouses (×5) at 320 Linwood Ave**

The qualitative assessment includes a summary of the building damage as well as an initial assessment of the current Seismic Capacity compared with current seismic code loads using the Initial Evaluation Procedure (IEP).

This Qualitative report for the building structure is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on 4 April 2012.

1.2. Key Damage Observed

Key damage present prior to the earthquakes includes:-

- Significant cracking and spalling of the concrete walls
- Corrosion of fixings of the aluminium structure

This damage is thought to be likely to have occurred prior to the earthquakes.

1.3. Critical Structural Weaknesses

No critical structural weaknesses have been identified.

1.4. Indicative Building Strength (from IEP and CSW assessment)

Based on the information available, and using the NZSEE Initial Evaluation Procedure, the building's original capacity has been assessed to be in the order of 46%NBS and due to the limited damage that has occurred as a result of the earthquake/s, the post earthquake capacity remains the same.

The building has been assessed to have a seismic capacity in the order of 46% NBS and is therefore not earthquake prone.

1.5. Recommendations

It is recommended that:

a) There is no damage to the building that would cause it to be unsafe to occupy.
b) We consider that barriers around the building are not necessary.

Since the qualitative assessment is less than 67% we recommend that a quantitative assessment be carried out due to the margin of error inherent in the IEP assessment.

2. Introduction

Sinclair Knight Merz was engaged by Christchurch City Council to prepare a qualitative assessment report for the building located at 320 Linwood Ave, Linwood following the magnitude 6.3 earthquake which occurred in the afternoon of the 22nd of February 2011 and the subsequent aftershocks.

The Qualitative Assessment uses the methodology recommended in the draft Engineering Advisory Group document "Guidance on Detailed Engineering Evaluation of Earthquake affected Non-residential Buildings in Canterbury" (July 2011). The qualitative assessment broadly includes a summary of the building damage as well as an initial assessment of the current Seismic Capacity compared with current seismic code loads.

A qualitative assessment involves inspections of the building and a desktop review of existing structural and geotechnical information, including existing drawings and calculations, if available.

The purpose of the assessment is to determine the likely building performance and damage patterns, to identify any potential critical structural weaknesses or collapse hazards, and to make an initial assessment of the likely building strength in terms of percentage of new building standard (%NBS).

This report describes the structural damage observed during our inspection and indicates suggested remediation measures. The inspection was undertaken from floor levels and was a visual inspection only. Our report reflects the situation at the time of the inspection and does not take account of changes caused by any events following our inspection. A full description of the basis on which we have undertaken our visual inspection is set out in Section 7.2.

The NZ Society for Earthquake Engineering (NZSEE) Initial Evaluation Procedure (IEP) was used to assess the likely performance of the building in a seismic event relative to the New Building Standard (NBS). 100% NBS is equivalent to the strength of a building that fully complies with current codes. This includes a recent increase of the Christchurch seismic hazard factor from 0.22 to 0.3[1].

At the time of this report, no intrusive site investigation, detailed analysis, or modelling of the building structure had been carried out. The building description below is based solely on our visual inspections.

[1] http://www.dbh.govt.nz/seismicity-info

3. Compliance

This section contains a summary of the requirements of the various statutes and authorities that control activities in relation to buildings in Christchurch at present.

3.1. Canterbury Earthquake Recovery Authority (CERA)

CERA was established on 28 March 2011 to take control of the recovery of Christchurch using powers established by the Canterbury Earthquake Recovery Act enacted on 18 April 2011. This act gives the Chief Executive Officer of CERA wide powers in relation to building safety, demolition and repair. Two relevant sections are:

Section 38 – Works

This section outlines a process in which the chief executive can give notice that a building is to be demolished and if the owner does not carry out the demolition, the chief executive can commission the demolition and recover the costs from the owner or by placing a charge on the owners' land.

Section 51 – Requiring Structural Survey

This section enables the chief executive to require a building owner, insurer or mortgagee carry out a full structural survey before the building is re-occupied.

We understand that CERA will require a detailed engineering evaluation to be carried out for all buildings (other than those exempt from the Earthquake Prone Building definition in the Building Act). It is anticipated that CERA will adopt the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011. This document sets out a methodology for both qualitative and quantitative assessments.

The qualitative assessment is a desk-top and site inspection assessment. It is based on a thorough visual inspection of the building coupled with a review of available documentation such as drawings and specifications. The quantitative assessment involves analytical calculation of the buildings strength and may require non-destructive or destructive material testing, geotechnical testing and intrusive investigation.

It is anticipated that factors determining the extent of evaluation and strengthening level required will include:

- The importance level and occupancy of the building
- The placard status and amount of damage
- The age and structural type of the building
- Consideration of any critical structural weaknesses

- The extent of any earthquake damage

3.2. Building Act

Several sections of the Building Act are relevant when considering structural requirements:

3.2.1. Section 112 – Alterations

This section requires that an existing building complies with the relevant sections of the Building Code to at least the extent that it did prior to any alteration. This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

3.2.2. Section 115 – Change of Use

This section requires that the territorial authority (in this case Christchurch City Council (CCC)) be satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'. Regarding seismic capacity 'as near as reasonably practicable' has previously been interpreted by CCC as achieving a minimum of 67%NBS however where practical achieving 100%NBS is desirable. The New Zealand Society for Earthquake Engineering (NZSEE) recommend a minimum of 67%NBS.

3.2.3. Section 121 – Dangerous Buildings

The definition of dangerous building in the Act was extended by the Canterbury Earthquake (Building Act) Order 2010, and it now defines a building as dangerous if:

- in the ordinary course of events (excluding the occurrence of an earthquake), the building is likely to cause injury or death or damage to other property; or
- in the event of fire, injury or death to any persons in the building or on other property is likely because of fire hazard or the occupancy of the building; or
- there is a risk that the building could collapse or otherwise cause injury or death as a result of earthquake shaking that is less than a 'moderate earthquake' (refer to Section 122 below); or
- there is a risk that that other property could collapse or otherwise cause injury or death; or
- a territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

3.2.4. Section 122 – Earthquake Prone Buildings

This section defines a building as earthquake prone if its ultimate capacity would be exceeded in a 'moderate earthquake' and it would be likely to collapse causing injury or death, or damage to other property. A moderate earthquake is defined by the building regulations as one that would generate ground shaking 33% of the shaking used to design an equivalent new building.

3.2.5. Section 124 – Powers of Territorial Authorities

This section gives the territorial authority the power to require strengthening work within specified timeframes or to close and prevent occupancy to any building defined as dangerous or earthquake prone.

3.2.6. Section 131 – Earthquake Prone Building Policy

This section requires the territorial authority to adopt a specific policy for earthquake prone, dangerous and insanitary buildings.

3.3. Christchurch City Council Policy

Christchurch City Council adopted their Earthquake Prone, Dangerous and Insanitary Building Policy in 2006. This policy was amended immediately following the Darfield Earthquake of the 4th September 2010.

The 2010 amendment includes the following:

- A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
- A strengthening target level of 67% of a new building for buildings that are Earthquake Prone. Council recognises that it may not be practicable for some repairs to meet that target. The council will work closely with building owners to achieve sensible, safe outcomes;
- A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
- Repair works for buildings damaged by earthquakes will be required to comply with the above.

The council has stated their willingness to consider retrofit proposals on a case by case basis, considering the economic impact of such a retrofit.

We anticipate that any building with a capacity of less than 33%NBS (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67%NBS of new building standard as recommended by the Policy.

If strengthening works are undertaken, a building consent will be required. A requirement of the consent will require upgrade of the building to comply ‘as near as is reasonably practicable’ with:

- The accessibility requirements of the Building Code.
- The fire requirements of the Building Code. This is likely to require a fire report to be submitted with the building consent application.

3.4. Building Code

The building code outlines performance standards for buildings and the Building Act requires that all new buildings comply with this code. Compliance Documents published by The Department of Building and Housing can be used to demonstrate compliance with the Building Code.

After the February Earthquake, on 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

a) Hazard Factor increased from 0.22 to 0.3 (36% increase in the basic seismic design load)
b) Serviceability Return Period Factor increased from 0.25 to 0.33 (80% increase in the serviceability design loads when combined with the Hazard Factor increase)

The increase in the above factors has resulted in a reduction in the level of compliance of an existing building relative to a new building despite the capacity of the existing building not changing.

4. Earthquake Resistance Standards

For this assessment, the building's earthquake resistance is compared with the current New Zealand Building Code requirements for a new building constructed on the site. This is expressed as a percentage of new building standard (%NBS). The new building standard load requirements have been determined in accordance with the current earthquake loading standard (NZS 1170.5:2004 Structural design actions - Earthquake actions - New Zealand).

The likely capacity of this building has been derived in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) guidelines 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes' (AISPBE), 2006. These guidelines provide an Initial Evaluation Procedure that assesses a buildings capacity based on a comparison of loading codes from when the building was designed and currently. It is a quick high-level procedure that can be used when undertaking a Qualitative analysis of a building. The guidelines also provide guidance on calculating a modified Ultimate Limit State capacity of the building which is much more accurate and can be used when undertaking a Quantitative analysis.

The New Zealand Society for Earthquake Engineering has proposed a way for classifying earthquake risk for existing buildings in terms of %NBS and this is shown in Figure 2 below.

Description	Grade	Risk	%NBS	Existing Building Structural Performance	Improvement of Structural Performance	
					Legal Requirement	NZSEE Recommendation
Low Risk Building	A or B	Low	Above 67	Acceptable (improvement may be desirable)	The Building Act sets no required level of structural improvement (unless change in use) This is for each TA to decide. Improvement is not limited to 34%NBS.	100%NBS desirable. Improvement should achieve at least 67%NBS
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally. Improvement recommended		Not recommended. Acceptable only in exceptional circumstances
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement	Unacceptable	Unacceptable

- **Figure 2: NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE Guidelines**

Table 1 below compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% risk of exceedance in 50 years (i.e. 0.2% in the next year).

- **Table 1: %NBS compared to relative risk of failure**

Percentage of New Building Standard (%NBS)	Relative Risk (Approximate)
>100	<1 time
80-100	1-2 times
67-80	2-5 times
33-67	5-10 times
20-33	10-25 times
<20	>25 times

5. Building Details

5.1. Building description

Linwood Nursery – Glasshouses (×5) is made up of five glasshouses orientated in a north-south direction. Each glasshouse has 125mm thick, 850mm high concrete walls and an aluminium framed structure fixed to the top of the walls. The structures are clad with glass panes which clip in to place, allowing for some movement. The longitudinal concrete walls of each structure are tied in to the northern wall of the Linwood Nursery – Potting Shed (PRK_0227_BLDG_004). Five vents, approximately 560mmx280mm in size, are located near the base of each longitudinal wall. (see Photo 1, Photo 2 and Photo 3)

Internal concrete decking runs the length of the glasshouses, adjacent to the east and west walls of each structure. The decking is supported off concrete beams and steel posts and is tied in to the adjacent glasshouse wall. The deck is used for the storage of plants. (see Photo 2 and Photo 4)

The walls of each glasshouse are supported on concrete strip footings. No internal floor slab is present. A concrete path runs down the centre of each glasshouse.

5.2. Gravity Load Resisting system

Gravity load from the aluminium frame structure is transferred to the concrete walls in bearing and then down to ground via the strip footings.

5.3. Seismic Load Resisting system

Lateral load in the aluminium frame structure is transmitted to the concrete wall through fixings between a longitudinal angle at the base of the frame and the longitudinal walls. In the across direction, lateral load from the aluminium structure is resisted through cantilever action in the longitudinal concrete walls and shear in the walls at the north end of the structure. In the along direction lateral loads are resisted by shear in the longitudinal walls.

For the lateral analysis of this building the ‘across direction’ has been taken as north-south whereas the ‘along direction’ has been taken as east-west.

5.4. Geotechnical Conditions

A geotechnical desktop study was carried out for this site. The main conclusions from this report are:

- The site has been assessed as NZS1170.5 Class D (deep or soft soil) from adjacent borehole logs.
- Liquefaction risk is low at this site.

The full geotechnical desktop study can be found in Appendix 4.

6. Damage Summary

6.1. Damage Summary

SKM undertook an inspection of the building from floor level on the 4 April 2012. For the purpose of this inspection the glasshouses were labelled A, B, C, D and E from west to east. 'Longitudinal walls' are those located on the east and west sides of the glasshouses. The following areas of damage were observed during the time of inspection.

1) Spalling was found at the top of the longitudinal walls of a number of the glasshouses. In some locations the reinforcement was exposed (see Photo 5). Spalling was found at the following locations:
    a. along the length of the eastern wall of Glasshouse B
    b. at the north end of the western wall of Glasshouse C
    c. halfway along and at the north end of the western wall of Glasshouse D
    d. along the length of the eastern wall of Glasshouse D
    e. at the north end of the eastern wall of Glasshouse E
2) Concrete was spalling off the face of the longitudinal walls at the following locations:
    a. exterior of the western wall of Glasshouse A, adjacent to a vent, half way along the wall (see Photo 6). This damage is likely to have been caused by the recent earthquakes.
    b. external face of the eastern wall of Glasshouse B, near the southern end of the structure (see Photo 7)
    c. interior of the eastern wall of Glasshouse C, adjacent to a vent, half way along the wall (see Photo 8)
3) Regular vertical cracking was found in the longitudinal walls of Glasshouse A, C and E (see Photo 9). The cracks were up to 0.5mm in width and spaced at approximately 1m centres. Some additional vertical cracking was observed adjacent to vents in the walls (see Photo 10).
4) Horizontal cracking, located approximately 350mm from the top of the wall, was found at the north end of the east wall of Glasshouse B (see Photo 11). The cracking was approximately 0.5mm in width.
5) Cracking and spalling was found at the top of the northern concrete wall in Glasshouse A and D (see Photo 12).
6) Concrete was spalling off the vertical edge of the concrete wall, adjacent to the door, at the north end of Glasshouse B, C and E (see Photo 13).
7) The aluminium framed structure had shifted laterally, half way along the east side of Glasshouse C (see Photo 15) and the west side of Glasshouse E (see Photo 16). Fixings at

these locations were missing or had corroded and sheared. Corrosion of the fixings is expected to have occurred prior to the recent earthquakes, however shearing of the bolts and lateral movement of the aluminium structure is likely to have happened in the recent earthquakes.

8) One broken pane of glass was observed at the south end of Glasshouse B, on the west side of the structure, and at the north end of Glasshouse E (see Photo 19). This is likely to have occurred in the recent earthquakes.
9) Some glass panes in Glasshouse A had slipped, allowing weather to penetrate the roof of the structure (see Photo 20). This is likely to have occurred in the recent earthquakes.
10) Concrete troughs are located in the south west corner of Glasshouse C, in the place of the concrete decking (see Photo 21). The troughs have significant cracking and spalling along the eastern edge (see Photo 22).

Photos of the above damage can be found in Appendix 1 – Photos.

7. Initial Seismic Evaluation

7.1. The Initial Evaluation Procedure Process

This section covers the initial seismic evaluation of the building as detailed in the NZSEE 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes'. The IEP grades buildings according to their likely performance in a seismic event. The procedure is not yet recognised by the NZ Building Code but is widely used and recognised by the Christchurch City Council as the preferred method for preliminary seismic investigations of buildings[2].

The IEP is a coarse screening process designed to identify buildings that are likely to be earthquake prone. The IEP process ranks buildings according to how well they are likely to perform relative to a new building designed to current earthquake standards, as shown in Table 2. The building rank is indicated by the percent of the required New Building Standard (%NBS) strength that the building is considered to have. Earthquake prone buildings are defined as "having regard to its condition and to the ground on which it is built, and because of its construction, the building—

(a) will have its ultimate capacity exceeded in a moderate earthquake (as defined in the regulations); and

(b) would be likely to collapse causing—

(i) injury or death to persons in the building or to persons on any other property; or

(ii) damage to any other property."[2]

"A moderate earthquake means, in relation to a building, an earthquake that would generate shaking at the site of the building that is of the same duration as, but that is one-third as strong as, the earthquake shaking (determined by normal measures of acceleration, velocity, and displacement) that would be used to design a new building at that site"[2]. A building having a strength less than one third of the New Building Standard correlates to an increased risk of approximately 20 times that of 100% NBS3. Buildings that are identified to be earthquake prone are required by law to be followed up with a detailed assessment and strengthening work within 30 years of the owner being notified that the building is potentially earthquake prone[2].

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[2] http://resources.ccc.govt.nz/files/EarthquakeProneDangerousAndInsanitaryBuildingsPolicy2010.pdf

[3] NZSEE 2006, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes,* p 2-2

Table 2: IEP Risk classifications

Description	Grade	Risk	%NBS	Structural performance
Low risk building	A+	Low	> 100	Acceptable. Improvement may be desirable.
	A		100 to 80	
	B		80 to 67	
Moderate risk building	C	Moderate	67 to 33	Acceptable legally. Improvement recommended.
High risk building	D	High	33 to 20	Unacceptable. Improvement required.
	E		< 20	

The IEP is a simple desktop study that is useful for risk management. No detailed calculations are done and so it relies on an inspection of the building and its plans to identify the structural members and describe the likely performance of the building in a seismic event. A review of the plans is also likely to identify any critical structural weaknesses. The IEP assumes that the building was properly designed and built according to the relevant codes at the time of construction. The IEP method rates buildings based on the code used at the time of construction and some more subjective parameters associated with how the building is detailed and so it is possible that %NBS derived from different engineers may differ.

This assessment describes only the likely seismic Ultimate Limit State (ULS) performance of the building. The ULS is the level of earthquake that can be resisted by the building without catastrophic failure. The IEP does not attempt to estimate Serviceability Limit State (SLS) performance of the building, or the level of earthquake that would start to cause damage to the building[4]. This assessment concentrates on matters relating to life safety as damage to the building is a secondary consideration. SLS performance of the building can be estimated by scaling the current code levels if required.

The relevant codes for determining %NBS are primarily:

- AS/NZS 1170 Structural Design Actions
- NZS 3101:2006 Concrete Structures Standard
- NZS 3404:1997 Steel Structures Standard

[4] NZSEE 2006, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes*, p2-9

7.2. Available Information, Assumptions and Limitations

Following our inspection on the 4 April 2012, SKM carried out a preliminary structural review. The structural review was undertaken using the available information which was as follows:

- SKM site measurements and inspection findings of the building. Please note no intrusive investigations were undertaken.

The assumptions made in undertaking the assessment include:

- The soil on site is class D as described in AS/NZS1170.5:2004, Clause 3.1.3, Soft Soil. This is based on our geotechnical desktop study that was carried out for this site.
- Standard design assumptions for typical office and factory buildings as described in AS/NZS1170.0:2002
  - 50 year design life, which is the default NZ Building Code design life.
  - Structure importance level 1. This level of importance is relevant to ‘structures presenting a low degree of hazard to life and other property’.
- Ductility level of 1.25, based on our assessment and code requirements at the time of design.

A site hazard factor, Z, of 0.3 was used in accordance with that stated in NZBC, Clause B1 Structure, Amendment 11 effective from 1 August 2011

This IEP was based on our visual inspection of the building only. Since it is not a full design and construction review, it has the following limitations:

- It is not likely to pick up on any original design or construction errors (if they exist)
- Other possible issues that could affect the performance of the building such as corrosion and modifications to the building will not be identified
- The IEP deals only with the structural aspects of the building. Other aspects such as building services are not covered.
- The IEP does not involve a detailed analysis or an element by element code compliance check.

7.3. Survey

There was no visible settlement of the structure, nor were there any significant ground movement issues around the building. The land at the site has been classified as green by CERA. The combination of these factors means that we do not recommend that any survey be undertaken at this point.

7.4. Critical Structural Weaknesses

No critical structural weaknesses for the building were observed during our visual inspection.

7.5. Qualitative Assessment Results

The building has had its seismic capacity assessed using the Initial Evaluation Procedure based on the information available. The building's capacity is expressed as a percentage of new building standard (%NBS) and is in the order of that shown below in Table 3. The capacity is subject to confirmation by a quantitative analysis.

Table 3: Qualitative Assessment Summary

Item	%NBS
Building's likely Seismic Capacity	46

Refer to Appendix 2 – IEP Reports for the full IEP assessment form.

8. Further Investigation

A building consent is not likely to be required for the repair of the damage noted in Section 11.

If a quantitative assessment is carried out further geotechnical investigation will be required as summarised below.

The geotechnical investigations recommended for the site are as follows:

- Four CPTs to refusal spread evenly throughout the site

Building consent drawings shows a fuel tank was proposed to be built at the corner of the nursery building and shade house. The presence and location of the fuel tank would need to be confirmed before undertaking any additional investigations.

Refer to Appendix 4 for the geotechnical desktop study.

9. Conclusion

A qualitative assessment was carried out on Linwood Nursery – Glasshouses (×5) located at 320 Linwood Ave, Linwood. Significant spalling was present on the majority of the concrete walls. The building has been assessed to have a seismic capacity in the order of 46% NBS and is therefore not earthquake prone.

No critical structural weaknesses were identified.

It is recommended that:

a) There is no damage to the building that would cause it to be unsafe to occupy.

b) We consider that barriers around the building are not necessary.

Since the qualitative assessment is less than 67% we recommend that a quantitative assessment be carried out due to the margin of error inherent in the IEP assessment.

10. Limitation Statement

This report has been prepared on behalf of, and for the exclusive use of, SKM's client, and is subject to, and issued in accordance with, the provisions of the contract between SKM and the Client. It is not possible to make a proper assessment of this report without a clear understanding of the terms of engagement under which it has been prepared, including the scope of the instructions and directions given to, and the assumptions made by, SKM. The report may not address issues which would need to be considered for another party if that party's particular circumstances, requirements and experience were known and, further, may make assumptions about matters of which a third party is not aware. No responsibility or liability to any third party is accepted for any loss or damage whatsoever arising out of the use of or reliance on this report by any third party.

Without limiting any of the above, in the event of any liability, SKM's liability, whether under the law of contract, tort, statute, equity or otherwise, is limited in as set out in the terms of the engagement with the Client.

It is not within SKM's scope or responsibility to identify the presence of asbestos, nor the responsibility of SKM to identify possible sources of asbestos. Therefore for any property pre-dating 1989, the presence of asbestos materials should be considered when costing remedial measures or possible demolition.

There is a risk of further movement and increased cracking due to subsequent aftershocks or settlement.

Should there be any further significant earthquake event, of a magnitude 5 or greater, it will be necessary to conduct a follow-up investigation, as the observations, conclusions and recommendations of this report may no longer apply Earthquake of a lower magnitude may also cause damage, and SKM should be advised immediately if further damage is visible or suspected.

11. Appendix 1 – Photos

Photo 1: Exterior of a glasshouse structure

Photo 2: Interior of a glasshouse structure

Photo 3: The south end of a glasshouse structure showing the connection to the wall of the Potting Shed (Building 4)

Photo 4: The underside of the concrete decking showing the supporting steel posts and ties to the adjacent walls

	
Photo 5: Typical spalling found at the top of the longitudinal concrete walls	Photo 6: Spalling found adjacent to a vent, half way along the western wall of Glasshouse A
	
Photo 7: Spalling found at the southern end of the eastern wall of Glasshouse B	Photo 8: Spalling found adjacent to a vent half way along the eastern wall of Glasshouse C

	
Photo 9: Typical vertical cracking found in the longitudinal walls of Glasshouses A, C and E	Photo 10: Typical vertical cracking found adjacent to vents in the longitudinal walls of Glasshouses A, C and E
	
Photo 11: Horizontal cracking located approximately 350mm from the top of the eastern wall of Glasshouse B	Photo 12: Typical cracking and spalling found at the top of the northern wall of Glasshouse A and D

	
Photo 13: Typical spalling found on the vertical edge of the northern wall, adjacent to the door in Glasshouse B, C and E	Photo 14: Fixings between sections of the aluminium frame structure
	
Photo 15: Lateral displacement mid-way along the east side of Glasshouse C	Photo 16: Lateral displacement mid-way along the west side of Glasshouse E

Photo 17: Bolt missing at an aluminium frame connection, mid-way along the east side of Glasshouse D

Photo 18: A loose bolt at an aluminium frame connection, mid-way along the west side of Glasshouse C

Photo 19: Broken glass pane at the south end of Glasshouse B

Photo 20: Glass pane which has shifted in Glasshouse A

Photo 21: Concrete troughs at the south end of Glasshouse C

Photo 22: Detail of cracking to the sides of the concrete toughs in Glasshouse C

12. Appendix 2 – IEP Reports

Table IEP-1 **Initial Evaluation Procedure – Step 1**

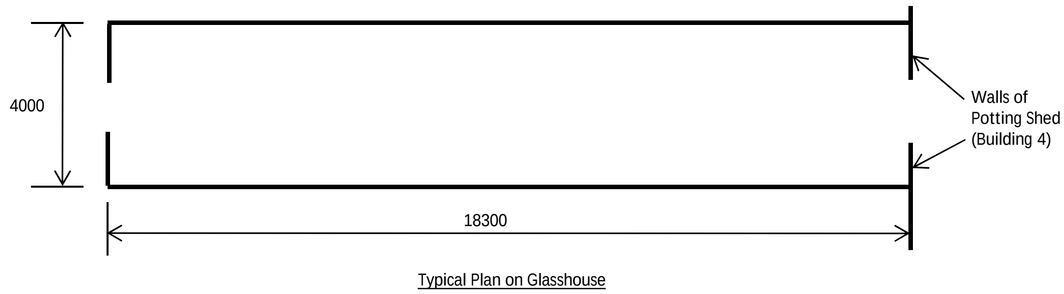
(Refer Table IEP - 2 for Step 2; Table IEP - 3 for Step 3, Table IEP - 4 for Steps 4, 5 and 6)

Building Name:	Linwood Nursery - Glasshouses (×5)	Ref.	ZB01276.024
Location:	320 Linwood Ave, Linwood, Christchurch	By	OAK
		Date	15/04/2013

Step 1 - General Information

1.1 Photos (attach sufficient to describe building)

1.2 Sketch of building plan



1.3 List relevant features

Linwood Nursery - Glasshouses (×5) is made up of five glasshouses orientated in a north-south direction. Each glasshouse has 125mm thick, 850mm high concrete walls and an aluminium framed structure fixed to the top of the walls. The structures are clad with glass panes which clip in to place, allowing for some movement. The longitudinal concrete walls of each structure are tied in to the northern wall of the Potting Shed (Building 4). Five vents approximately 560mmx280mm in size, are located near the base of each longitudinal wall.

Internal concrete decking runs the length of the glasshouses, adjacent to the east and west walls of each structure. The decking is supported off concrete beams and steel posts and is tied in to the adjacent glasshouse wall. The deck is used for the storage of plants.

The walls of each glasshouse are supported on concrete strip footings. No internal floor slab is present. A concrete path runs down the centre of each glasshouse.

1.4 Note information sources

Tick as appropriate

Visual Inspection of Exterior	✓
Visual Inspection of Interior	✓
Drawings (note type)	
Specifications	
Geotechnical Reports	✓
Other (list)	

Table IEP-2 Initial Evaluation Procedure – Step 2

(Refer Table IEP - 1 for Step 1; Table IEP - 3 for Step 3, Table IEP - 4 for Steps 4, 5 and 6)

Building Name:	Linwood Nursery - Glasshouses (×5)	Ref.	ZB01276.024
Location:	320 Linwood Ave, Linwood, Christchurch	By	OAK
Direction Considered:	**Longitudinal & Transverse**	Date	15/04/2013

(Choose worse case if clear at start. Complete IEP-2 and IEP-3 for each if in doubt)

Step 2 - Determination of (%NBS)b

2.1 Determine nominal (%NBS) = (%NBS)nom

Pre 1935			○	See also notes 1, 3
1935-1965			◉	
1965-1976	Seismic Zone;	A	○	
		B	○	
		C	○	See also note 2
1976-1992	Seismic Zone;	A	○	
		B	○	
		C	○	
1992-2004			○	

b) Soil Type

From NZS1170.5:2004, Cl 3.1.3

A or B Rock	○
C Shallow Soil	○
D Soft Soil	◉
E Very Soft Soil	○

From NZS4203:1992, Cl 4.6.2.2
(for 1992 to 2004 only and only if known)

a) Rigid	○	N-A
b) Intermediate	○	

c) Estimate Period, T

building Ht = 1.5 meters

	Longitudinal	Transverse	
Ac =	700		m2

Can use following:

		Longitudinal	Transverse
$T = 0.09h_n^{0.75}$	for moment-resisting concrete frames	○ MRCF	◉ MRCF
$T = 0.14h_n^{0.75}$	for moment-resisting steel frames	○ MRSF	○ MRSF
$T = 0.08h_n^{0.75}$	for eccentrically braced steel frames	○ EBSF	○ EBSF
$T = 0.06h_n^{0.75}$	for all other frame structures	○ Others	○ Others
$T = 0.09h_n^{0.75}/A_c^{0.5}$	for concrete shear walls	◉ CSW	○ CSW
$T <= 0.4sec$	for masonry shear walls	○ MSW	○ MSW

Where hn = height in m from the base of the structure to the uppermost seismic weight or mass.
Ac = ΣAi(0.2 + Lwi/hn)2
Ai = cross-sectional shear area of shear wall i in the first storey of the building, in m2
lwi = length of shear wall i in the first storey in the direction parallel to the applied forces, in m
with the restriction that lwi/hn shall not exceed 0.9

Longitudinal	Transverse	
0.0	0.1	Seconds

d) (%NBS)nom determined from Figure 3.3

Longitudinal	2.8	$(\%NBS)_{nom}$
Transverse	2.8	$(\%NBS)_{nom}$

			Factor
Note 1:	For buildings designed prior to 1965 and known to be designed as public buildings in accordance with the code of the time, multiply (%NBS)nom by 1.25.	No	1
	For buildings designed 1965 - 1976 and known to be designed as public buildings in accordance with the code of the time, multiply (%NBS)nom by 1.33 - Zone A or 1.2 - Zone B	No	1
Note 2:	For reinforced concrete buildings designed between 1976 -1984 (%NBS)nom by 1.2	No	1
Note 3:	For buildings designed prior to 1935 multiply (%NBS)nom by 0.8 except for Wellington where the factor may be taken as 1.	No	1

Longitudinal	2.8	$(\%NBS)_{nom}$
Transverse	2.8	$(\%NBS)_{nom}$

Continued over page

Building Name:	Linwood Nursery - Glasshouses (×5)	Ref.	ZB01276.024
Location:	320 Linwood Ave, Linwood, Christchurch	By	OAK
Direction Considered:	**Longitudinal & Transverse**	Date	15/04/2013

(Choose worse case if clear at start. Complete IEP-2 and IEP-3 for each if in doubt)

2.2 Near Fault Scaling Factor, Factor A
If T < 1.5sec, Factor A = 1

a) Near Fault Factor, N(T,D) 1
(from NZS1170.5:2004, Cl 3.1.6)

b) Near Fault Scaling Factor = 1/N(T,D) **Factor A** 1.00

2.3 Hazard Scaling Factor, Factor B

Select Location: Christchurch

a) Hazard Factor, Z, for site
(from NZS1170.5:2004, Table 3.3)

Z = 0.3
Z 1992 = 0.8

Auckland	0.6	Palm Nth	1.2
Wellington	1.2	Dunedin	0.6
Christchurch	0.8	Hamilton	0.67

b) Hazard Scaling Factor

For pre 1992 = 1/Z

\# For 1992 onwards = Z 1992/Z

(Where Z 1992 is the NZS4203:1992 Zone Factor from accompanying Figure 3.5(b))

Factor B 3.33

2.4 Return Period Scaling Factor, Factor C

a) Building Importance Level 1
(from NZS1170.0:2004, Table 3.1 and 3.2)

b) Return Period Scaling Factor from accompanying Table 3.1 **Factor C** 2.00

2.5 Ductility Scaling Factor, D

a) Assessed Ductility of Existing Structure, μ
(shall be less than maximum given in accompanying Table 3.2)

Longitudinal	1.25	μ Maximum = 2
Transverse	1.25	μ Maximum = 2

b) Ductility Scaling Factor

For pre 1976 = k_μ

For 1976 onwards = 1

(where k_μ is NZS1170.5:2005 Ductility Factor, from accompanying Table 3.3)

Longitudinal	**Factor D**	1.14
Transverse	**Factor D**	1.14

2.6 Structural Performance Scaling Factor, Factor E

Select Material of Lateral Load Resisting System

Longitudinal: Concrete
Transverse: Concrete

a) Structural Performance Factor, S_p
from accompanying Figure 3.4

Longitudinal	Sp	0.90
Transverse	Sp	0.90

b) Structural Performance Scaling Factor

Longitudinal	$1/S_p$	**Factor E**	1.11
Transverse	$1/S_p$	**Factor E**	1.11

2.7 Baseline %NBS for Building, $(\%NBS)_b$
(equals $(\%NSB)_{nom}$ x A x B x C x D x E)

Longitudinal	23.7	$(\%NBS)_b$
Transverse	23.7	$(\%NBS)_b$

Table IEP-3 **Initial Evaluation Procedure – Step 3**

(Refer Table IEP - 1 for Step 1; Table IEP - 2 for Step 2, Table IEP - 4 for Steps 4, 5 and 6)

Building Name:	Linwood Nursery - Glasshouses (×5)	Ref.	ZB01276.024
Location:	320 Linwood Ave, Linwood, Christchurch	By	OAK
Direction Considered:	**a) Longitudinal**	Date	15/04/2013

(Choose worse case if clear at start. Complete IEP-2 and IEP-3 for each if in doubt)

Step 3 - Assessment of Performance Achievement Ratio (PAR)

(Refer Appendix B - Section B3.2)

Critical Structural Weakness

Effect on Structural Performance (Choose a value - Do not interpolate) — **Building Score**

3.1 Plan Irregularity

	Severe	Significant	Insignificant		Building Score
Effect on Structural Performance	○	○	◉	**Factor A**	1
Comment					

3.2 Vertical Irregularity

	Severe	Significant	Insignificant		Building Score
Effect on Structural Performance	○	○	◉	**Factor B**	1
Comment					

3.3 Short Columns

	Severe	Significant	Insignificant		Building Score
Effect on Structural Performance	○	○	◉	**Factor C**	1
Comment					

3.4 Pounding Potential

(Estimate D1 and D2 and set D = the lower of the two, or =1.0 if no potential for pounding)

a) Factor D1: - Pounding Effect

Select appropriate value from Table

Note:
Values given assume the building has a frame structure. For stiff buildings (eg with shear walls), the effect of pounding may be reduced by taking the co-efficient to the right of the value applicable to frame buildings.

Factor D1 1

Table for Selection of Factor D1

	Severe	Significant	Insignificant
Separation	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Alignment of Floors within 20% of Storey Height	○ 0.7	○ 0.8	◉ 1
Alignment of Floors not within 20% of Storey Height	○ 0.4	○ 0.7	○ 0.8

b) Factor D2: - Height Difference Effect

Select appropriate value from Table

Factor D2 1

Table for Selection of Factor D2

	Severe	Significant	Insignificant
Separation	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Height Difference > 4 Storeys	○ 0.4	○ 0.7	◉ 1
Height Difference 2 to 4 Storeys	○ 0.7	○ 0.9	○ 1
Height Difference < 2 Storeys	○ 1	○ 1	○ 1

Factor D 1

(Set D = lesser of D1 and D2 or..
set D = 1.0 if no prospect of pounding)

3.5 Site Characteristics - (Stability, landslide threat, liquefaction etc)

	Severe	Significant	Insignificant		Building Score
Effect on Structural Performance	○ 0.5	○ 0.7	◉ 1	**Factor E**	1

3.6 Other Factors

For < 3 storeys - Maximum value 2.5,

otherwise - Maximum value 1.5. No minimum. **Factor F** 2

Record rationale for choice of Factor F:

Reinforcement of concrete walls is expected to be governed by minimum requirements rather that design for specific seismic load.
No damage observed can be attributed to the earthquakes.

3.7 Performance Achievement Ratio (PAR)
(equals A x B x C x D x E x F)

PAR 2

(Refer Table IEP - 1 for Step 1; Table IEP - 2 for Step 2, Table IEP - 4 for Steps 4, 5 and 6)

Building Name:	Linwood Nursery - Glasshouses (x5)	Ref.	ZB01276.024
Location:	320 Linwood Ave, Linwood, Christchurch	By	OAK
Direction Considered:	**b) Transverse**	Date	15/04/2013

(Choose worse case if clear at start. Complete IEP-2 and IEP-3 for each if in doubt)

Step 3 - Assessment of Performance Achievement Ratio (PAR)

(Refer Appendix B - Section B3.2)

Critical Structural Weakness

Effect on Structural Performance (Choose a value - Do not interpolate) — **Building Score**

3.1 Plan Irregularity

Effect on Structural Performance	Severe	Significant	Insignificant	
	○	○	◉	**Factor A** 1
Comment				

3.2 Vertical Irregularity

Effect on Structural Performance	Severe	Significant	Insignificant	
	○	○	◉	**Factor B** 1
Comment				

3.3 Short Columns

Effect on Structural Performance	Severe	Significant	Insignificant	
	○	○	◉	**Factor C** 1
Comment				

3.4 Pounding Potential

(Estimate D1 and D2 and set D = the lower of the two, or =1.0 if no potential for pounding)

a) Factor D1: - Pounding Effect
Select appropriate value from Table

Note:
Values given assume the building has a frame structure. For stiff buildings (eg with shear walls), the effect of pounding may be reduced by taking the co-efficient to the right of the value applicable to frame buildings.

Factor D1 1

Table for Selection of Factor D1	Severe	Significant	Insignificant
Separation	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Alignment of Floors within 20% of Storey Height	○ 0.7	○ 0.8	◉ 1
Alignment of Floors not within 20% of Storey Height	○ 0.4	○ 0.7	○ 0.8

b) Factor D2: - Height Difference Effect
Select appropriate value from Table

Factor D2 1

Table for Selection of Factor D2	Severe	Significant	Insignificant
Separation	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Height Difference > 4 Storeys	○ 0.4	○ 0.7	◉ 1
Height Difference 2 to 4 Storeys	○ 0.7	○ 0.9	○ 1
Height Difference < 2 Storeys	○ 1	○ 1	○ 1

Factor D 1

(Set D = lesser of D1 and D2 or..
set D = 1.0 if no prospect of pounding)

3.5 Site Characteristics - (Stability, landslide threat, liquefaction etc)

Effect on Structural Performance	Severe	Significant	Insignificant	
	○ 0.5	○ 0.7	◉ 1	**Factor E** 1

3.6 Other Factors

For < 3 storeys - Maximum value 2.5,
otherwise - Maximum value 1.5. No minimum.

Factor F 2

Record rationale for choice of Factor F:
Reinforcement of concrete walls is expected to be governed by minimum requirements rather that design for specific seismic load.
No damage observed can be attributed to the earthquakes.

3.7 Performance Achievement Ratio (PAR)
(equals A x B x C x D x E x F)

PAR 2

Table IEP-4 **Initial Evaluation Procedure – Steps 4, 5 and 6**

(Refer Table IEP - 1 for Step 1; Table IEP - 2 for Step 2, Table IEP - 3 for Step 3)

Building Name:	Linwood Nursery - Glasshouses (×5)	Ref.	ZB01276.024
Location:	320 Linwood Ave, Linwood, Christchurch	By	OAK
Direction Considered:	**Longitudinal & Transverse**	Date	15/04/2013

(Choose worse case if clear at start. Complete IEP-2 and IEP-3 for each if in doubt)

Step 4 - Percentage of New Building Standard (%NBS)

	Longitudinal	Transverse
4.1 Assessed Baseline (%NBS)$_b$ (from Table IEP - 1)	23	23
4.2 Performance Achievement Ratio (PAR) (from Table IEP - 2)	2.00	2.00
4.3 PAR x Baseline (%NBS)$_b$	46	46

4.4 Percentage New Building Standard (%NBS)
(Use lower of two values from Step 4.3)

46

Step 5 - Potentially Earthquake Prone?
(Mark as appropriate)

%NBS ≤ 33 — NO

Step 6 - Potentially Earthquake Risk?

%NBS < 67 — YES

Step 7 - Provisional Grading for Seismic Risk based on IEP

Seismic Grade — **C**

Evaluation Confirmed by

Signature

JAMES CARTER — Name

1017618 — CPEng. No

Relationship between Seismic Grade and % NBS :

Grade:	A+	A	B	C	D	E
%NBS:	> 100	100 to 80	80 to 67	67 to 33	33 to 20	< 20

13. Appendix 3 – CERA Standardised Report Form

Location

Building Name:	Linwood Nursery - Glasshouses (X 5)	Reviewer:	James Carter
	Unit / No: 320 / Street: Linwood Ave	CPEng No:	1017618
Building Address:	320 Linwood Ave	Company:	SKM
Legal Description:		Company project number:	ZB01276.024
		Company phone number:	03 940 4900
GPS south: (Degrees / Min / Sec)		Date of submission:	24-May
GPS east:		Inspection Date:	4/04/2012
		Revision:	C
Building Unique Identifier (CCC):	PRK_0227_BLDG_007	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:		Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	
Ground floor split?	no		Ground floor elevation above ground (m):	
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	
Building height (m):	2.50		height from ground to level of uppermost seismic mass (for IEP only) (m):	2
Floor footprint area (approx):				
Age of Building (years):	50		Date of design:	1935-1965
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	other (specify)		Brief strengthening description:	Propagating plants
Use (upper floors):				
Use notes (if required):				
Importance level (to NZS1170.5):	IL1			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	other (note)	describe system	Aluminium frame with glass cladding
Floors:			
Beams:			
Columns:			
Walls:	load bearing concrete	#N/A	

Lateral load resisting structure

Lateral system along:	concrete shear wall	**Note: Define along and across in detailed report!**	note total length of wall at ground (m):	36.6
Ductility assumed, μ:	1.25		wall thickness (m):	0.125
Period along:		0.00 from parameters in sheet	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	other (note)		describe system	Cantilevered concrete walls
Ductility assumed, μ:	1.25			
Period across:		0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

north (mm):		leave blank if not relevant
east (mm):		
south (mm):		
west (mm):		

Non-structural elements

Stairs:			
Wall cladding:			
Roof Cladding:	Other (specify)	describe	Glass panes
Glazing:			
Ceilings:	none		
Services(list):			

Available documentation

Architectural	none	original designer name/date	
Structural	none	original designer name/date	
Mechanical	none	original designer name/date	
Electrical	none	original designer name/date	
Geotech report	none	original designer name/date	

Damage

Site: (refer DEE Table 4-2)

Site performance:	1	Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):	Damage to building is primarily pre-existing		
Across	Damage ratio:	0%		
	Describe (summary):	Damage to building is primarily pre-existing		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

$$Damage\_Ratio = \frac{(\% NBS\,(before) - \% NBS\,(after))}{\% NBS\,(before)}$$

Recommendations

Level of repair/strengthening required:	minor structural	Describe:	Repairs to spalling of concrete walls required. It is anticipated that a quantitative analysis of the building will show that little or no strengthening will be required.
Building Consent required:	no	Describe:	
Interim occupancy recommendations:	full occupancy	Describe:	

Along	Assessed %NBS before:	46%	%NBS from IEP below	If IEP not used, please detail assessment methodology:	Qualitative Assessment carried out includes NZSEE IEP (refer to SKM report) and some preliminary calculations on reinforcement required for seismic loading to current standard
	Assessed %NBS after:	46%			
Across	Assessed %NBS before:	46%	%NBS from IEP below		
	Assessed %NBS after:	46%			

14. Appendix 4 – Geotechnical Desk Study

Sinclair Knight Merz
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Christchurch City Council - Structural Engineering Service

Geotechnical Desk Study

SKM project number	ZB01276
SKM project site number	019 to 032 inclusive
Address	320 Linwood Ave
Report date	03 April 2012
Author	Ross Roberts \ Ananth Balachandra
Reviewer	Leah Bateman
Approved for issue	Yes

1. Introduction

This report outlines the geotechnical information that Sinclair Knight Merz (SKM) has been able to source from our database and other sources in relation to the property listed above. We understand that this information will be used as part of an initial qualitative Detailed Engineering Evaluation (DEE) and will be supplemented by more detailed information and investigations to allow detailed scoping of the repair or rebuild of the building.

2. Scope

This geotechnical desk top study incorporates information sourced from:

- Published geology
- Publically available borehole records
- Liquefaction records
- Aerial photography
- Council files
- A preliminary site walkover

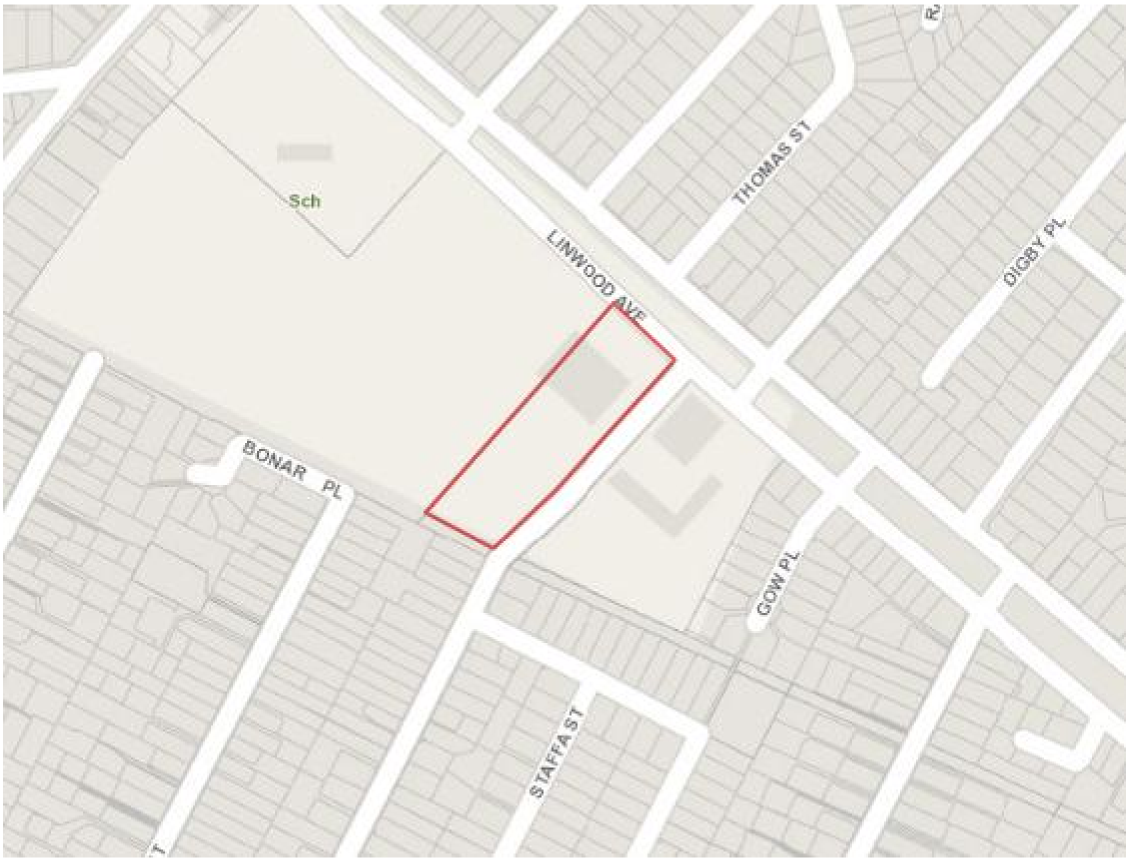
3. Limitations

This report was prepared to address geotechnical issues relating to the specific site in accordance with the scope of works as defined in the contract between SKM and our Client. This report has been prepared on behalf of, and for the exclusive use of, our Client, and is subject to, and issued in accordance with, the provisions of the contract between SKM and our Client. The findings presented in this report should not be applied to another site or another development within the same site without consulting SKM.

The assessment undertaken by SKM was limited to a desktop review of the data described in this report. SKM has not undertaken any subsurface investigations, measurement or testing of materials from the site. In preparing this report, SKM has relied upon, and presumed accurate, any information (or confirmation of the absence thereof) provided by our Client, and from other sources as described in the report. Except as otherwise stated in this report, SKM has not attempted to verify the accuracy or completeness of any such information.

This report should be read in full and no excerpts are to be taken as representative of the findings. It must not be copied in parts, have parts removed, redrawn or otherwise altered without the written consent of SKM.

4. Site location

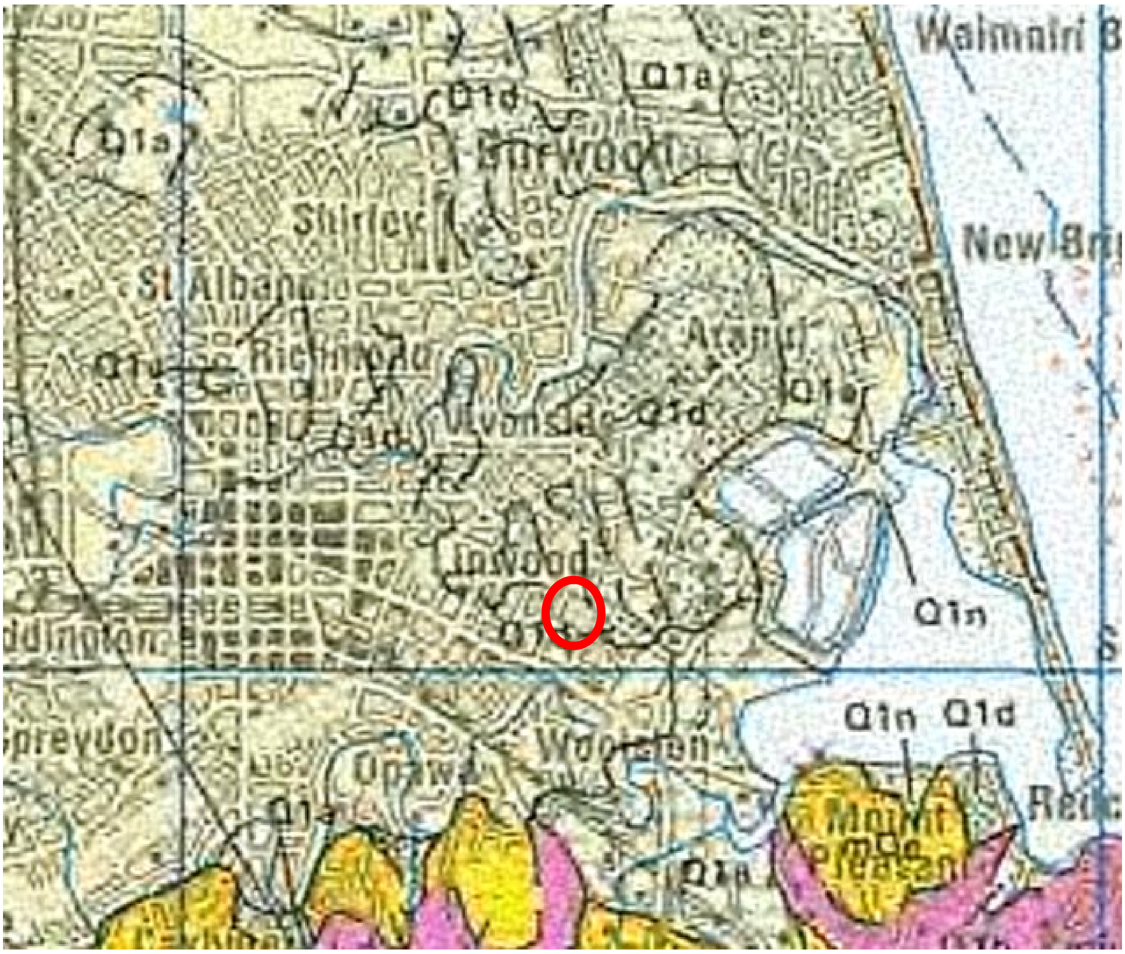


- **Figure 1 – Site location (courtesy of LINZ http://viewers.geospatial.govt.nz)**

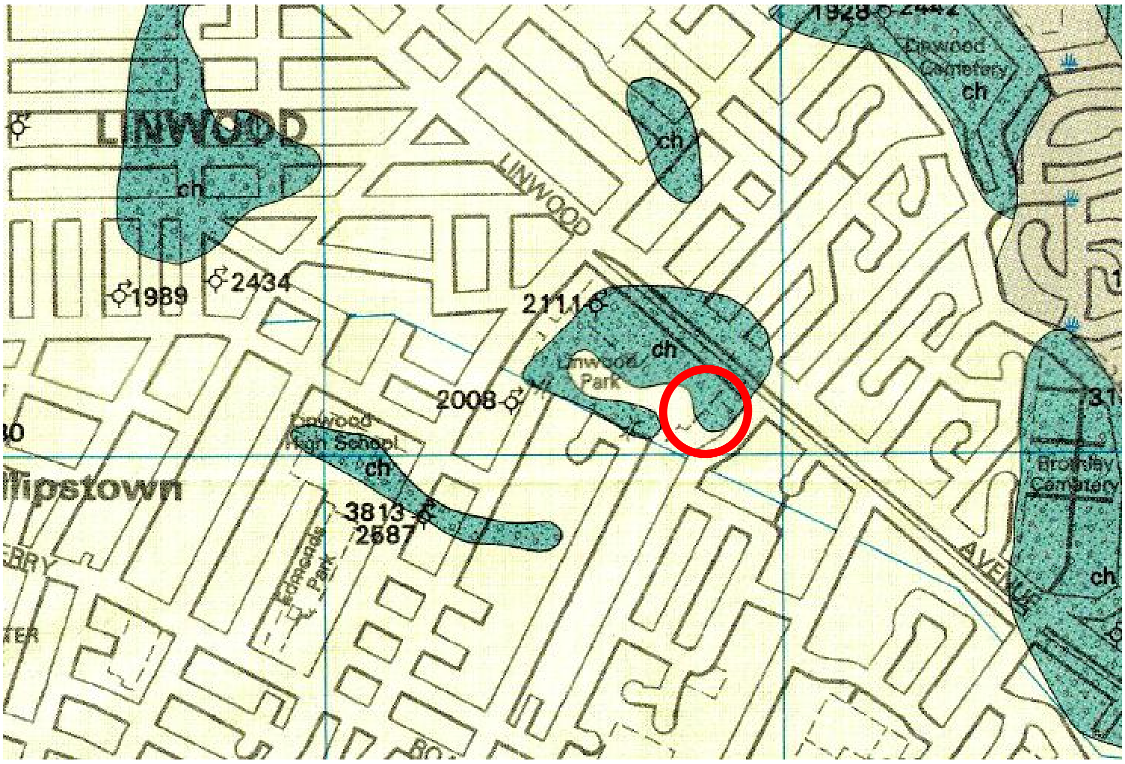
These structures are located on Linwood Avenue at grid reference 1573857 E, 5179462 N (NZTM).

5. Review of available information

5.1 Geological maps



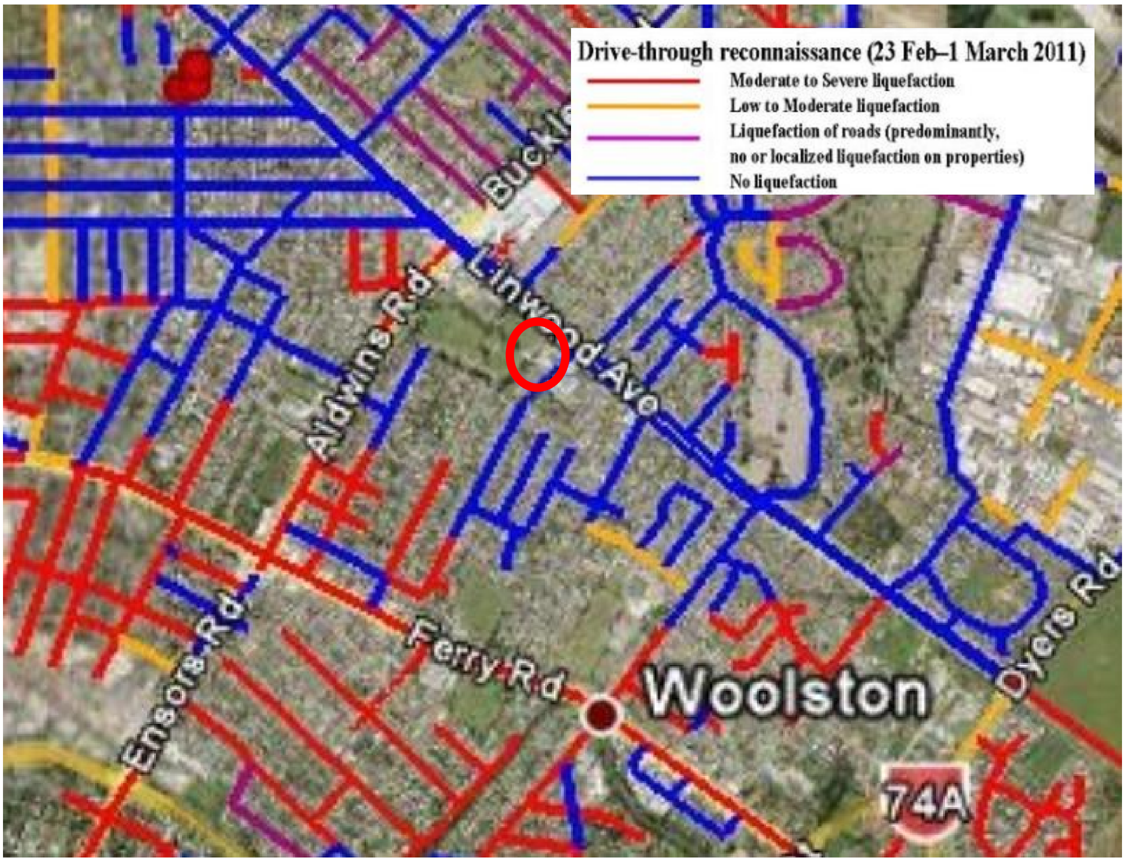
- **Figure 2 – Regional geological map (Forsyth et al, 2008). Site marked in red.**



- **Figure 3 – Local geological map (Brown et al, 1992). Site marked in red.**

The northern section of the site is shown to be underlain by Holocene deposits compromising sand of fixed and semi-fixed dunes and beaches of the Christchurch formation while the southern end of the site is shown to be underlain by Holocene deposits compromising of alluvial sand and silt over bank deposits forming the Springston Formation.

5.2 Liquefaction map



- **Figure 4 – Liquefaction map (Cubrinovski & Taylor, 2011). Site marked in red.**

Following the 22 February 2011 event drive through reconnaissance was undertaken from 23 February until 1 March by M Cubrinovsko and M Taylor of Canterbury University. Their findings show no liquefaction at this site.

5.3 Aerial photography

- **Figure 5 – Aerial photography from 24 Feb 2011 (http://viewers.geospatial.govt.nz/)**

There appears to be liquefied material present at the northern end of the tennis courts. This relates well with the inferred underlying geology of the site, where the northern end of the site was shown to be underlain by sand of fixed and semi-fixed dunes. No liquefied material can be seen in the southern end of the site from the aerial photographs.

5.4 CERA classification

A review of the LINZ website (http://viewers.geospatial.govt.nz/) shows that the site is:

- Zone: Green
- DBH Technical Category: N/A (Urban Non-residential) – adjacent properties to the north and south of the site are TC2

5.5 Historical land use

Reference to historical documents (e.g. Appendix A) shows that some of the site (the northern end) was recorded as marshland or swamp in 1856. The whole of Linwood Park, immediately to the left of the property, was recorded to be marshland or swamp in 1856. It is therefore likely that soft or liquefiable ground would be present near the site. Given the relatively low accuracy of these historical documents, it should be considered possible that old swamp deposits are present on the site.

5.6 Existing ground investigation data

- **Figure 6 – Local boreholes from Project Orbit and SKM files (https://canterburyrecovery.projectorbit.com/)**

Where available logs from these investigation locations are attached to this report (Appendix B), and the results are summarised in Appendix C.

5.7 Council property files

The available council property files for the site are building consent document relating to the alterations of the structure on site and construction of an underground fuel tank.

From available drawing, a 100mm concrete floor slab supported on hardfill and 200mm diameter concrete posthole footings to a depth of 1000mm beneath the walls of the structure are shown. A minimum reinforcement to a depth of 300mm below ground level for the post hole footing is specified. No indication on the depth of hardfill is provided in the available records. There is a possibility of contamination on site due to the presence of the fuel tank. However, no other hazards were identified in the available council records.

Detailed information about the geology underlying the site was not found in the available council records. However, ground investigation for the adjacent property to the east shows a ground profile consisting of 0.5m of topsoil or sandy clay and medium sand to be present from 0.5m to 3.8m. An allowable bearing capacity of 200 kPa was identified for the medium sand layer.

5.8 Site walkover

The main buildings were constructed using masonry blocks and metal clad roofs. A number of glasshouses and open frames were also present on site. No damage to any of the buildings was observed during the external site visit. Residents at the property mentioned that they saw no sign of liquefaction. Most of the site was asphalted, with no signs indicating that land damage occurred.

- **Figure 7 Overview of Linwood nursery building**

▪ **Figure 8 Overview of garden area**

6. Conclusions and recommendations

6.1 Site geology

The available geotechnical investigation data are at considerable distance away from the site. Additionally, from the local geological map it was inferred that a geological boundary between deposits forming the Christchurch formation and deposits forming the Springston formation is present beneath the site. An inference on possible underlying geology based on available investigation data is made below. However, the site geology would need to be confirmed through further investigation conducted closer to the site.

Depth range (mBLG)	Soil type
0 - 1	Sensitive fine grained soils (clay or silt)
1 - 8	Very stiff clays and loose to dense clayey sand
8 – 19	Dense sand
19 – 21	Interbedded clay and silt
21 - 23	Dense sand
23 +	Soft to firm clay or silt

6.2 Seismic site subsoil class

The site has been assessed as NZS1170.5 Class D (deep or soft soil) from adjacent borehole logs.

As described in NZS1170, the preferred site classification method is from site periods based on four times the shear wave travel time through material from the surface to the underlying rock. The next preferred methods are from borelogs including measurement of geotechnical properties or by evaluation of site periods from Nakamura ratios or from recorded earthquake motions. Lacking this information, classification may be based on boreholes with descriptors but no geotechnical measurements. The least preferred method is from surface geology and estimates of the depth to underlying rock.

In this case the second preferred method has been used to make the assessment utilising records from sites at least 50 m from the site. It is therefore possible that site specific investigation could revise the site class.

6.3 Building performance

The performance to date suggests that the existing foundations are adequate for their current purpose.

6.4 Ground performance and properties

There is significant uncertainty in the underlying geology, due to the likely geological boundary identified in section **Error! Reference source not found.** present beneath the site. Evidence of different geotechnical behaviour of the underlying soil layer could be seen by the fact that some liquefaction was observed near the north eastern part of the site but no significant land damage due to recent earthquakes was observed in the southern parts of the site. Conclusions about the location of the geological boundary or the geotechnical properties of the two geological units were not able to be made from available ground investigation data.

However, there was little damage to the structure or the surrounding land noted during the site inspection. It is also expected that the two formations are likely to comprise predominantly sand mixtures in which case the ground investigation data available for adjacent sites could be used to estimate the likely ground properties of the site. Therefore, for the purposes of carrying out a quantitative DEE, the following parameters are recommended for the shallow soil layer:

- Effective angle of friction = 35 degrees
- Apparent cohesion = 1 kPa
- Unit weight = 18 kPa
- Ultimate bearing capacity = 300 kPa

It is noted these parameters should not be used for consent or design purposes and ground conditions should be confirmed by a geotechnical investigation.

Liquefaction risk is expected to be low at this site. However, it should be noted that the liquefaction susceptibility of the two geological units underlying the site could be different but a reliable assessment of this could not be made using available information. If a more detailed assessment of liquefaction risk is needed, additional investigations on site would be required.

6.5 Further investigations

No further investigations are likely to be necessary in order to undertake a quantitative DEE for the site. However, due to concerns raised in section 6.4, additional investigations will be required for consenting and design purposes if this is to be carried out.

In which case, further investigations recommended are:

- Four CPTs to refusal spread evenly throughout the site

Building consent drawings shows a fuel tank was proposed to be built at the corner of the nursery building and shade house. The presence and location of the fuel tank would need to be confirmed before undertaking any additional investigations.

7. References

Brown LJ, Weeber JH, 1992. Geology of the Christchurch urban area. Scale 1:25,000. Institute of Geological & Nuclear Sciences geological map 1.

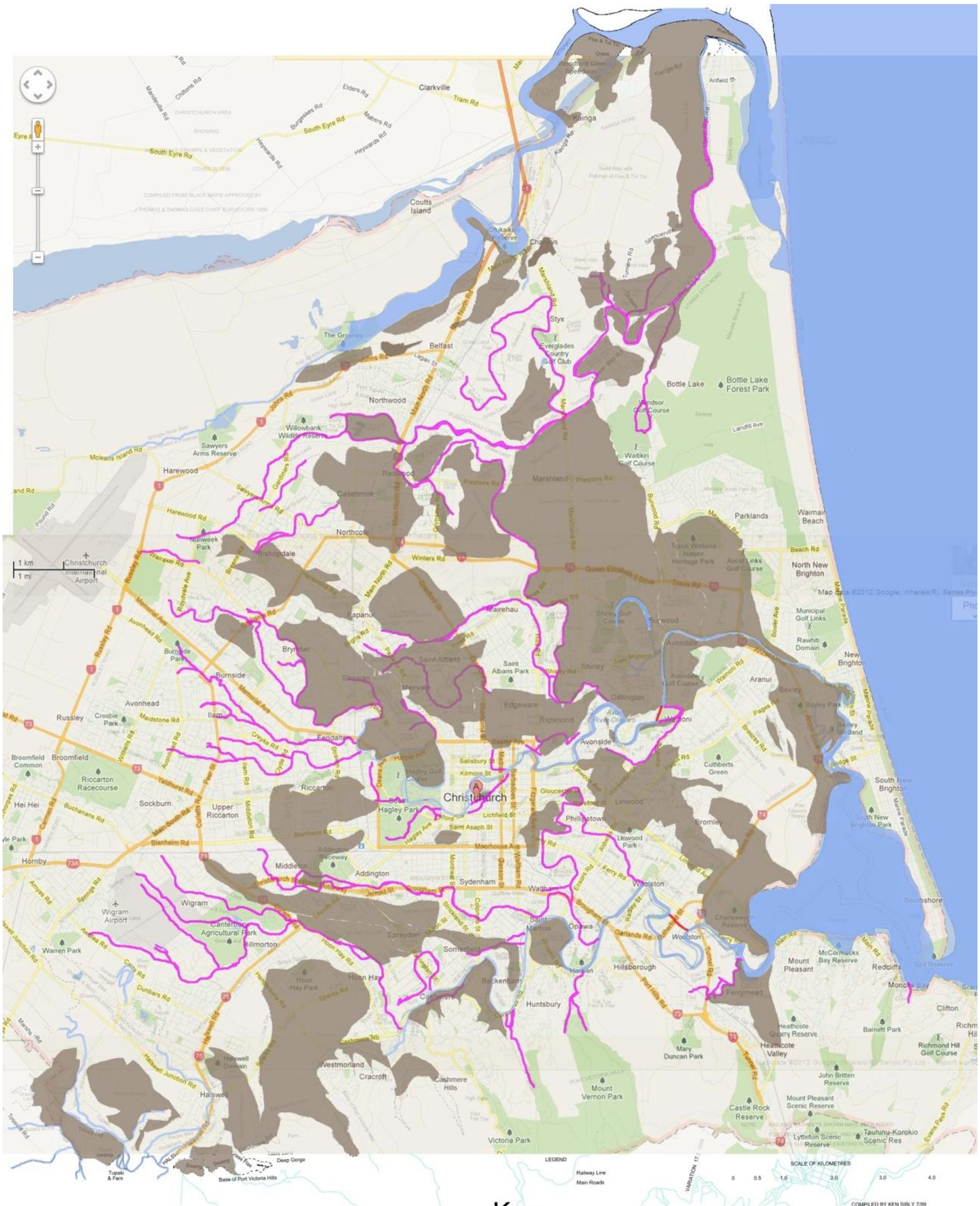
Cubrinovski & Taylor, 2011. Liquefaction map summarising preliminary assessment of liquefaction in urban areas following the 2010 Darfield Earthquake.

Forsyth PJ, Barrell DJA, Jongens R, 2008. Geology of the Christchurch area. Institute of Geological & Nuclear Sciences geological map 16.

Land Information New Zealand (LINZ) geospatial viewer (http://viewers.geospatial.govt.nz/)

EQC Project Orbit geotechnical viewer (https://canterburyrecovery.projectorbit.com/)

Appendix A – Christchurch 1856 land use

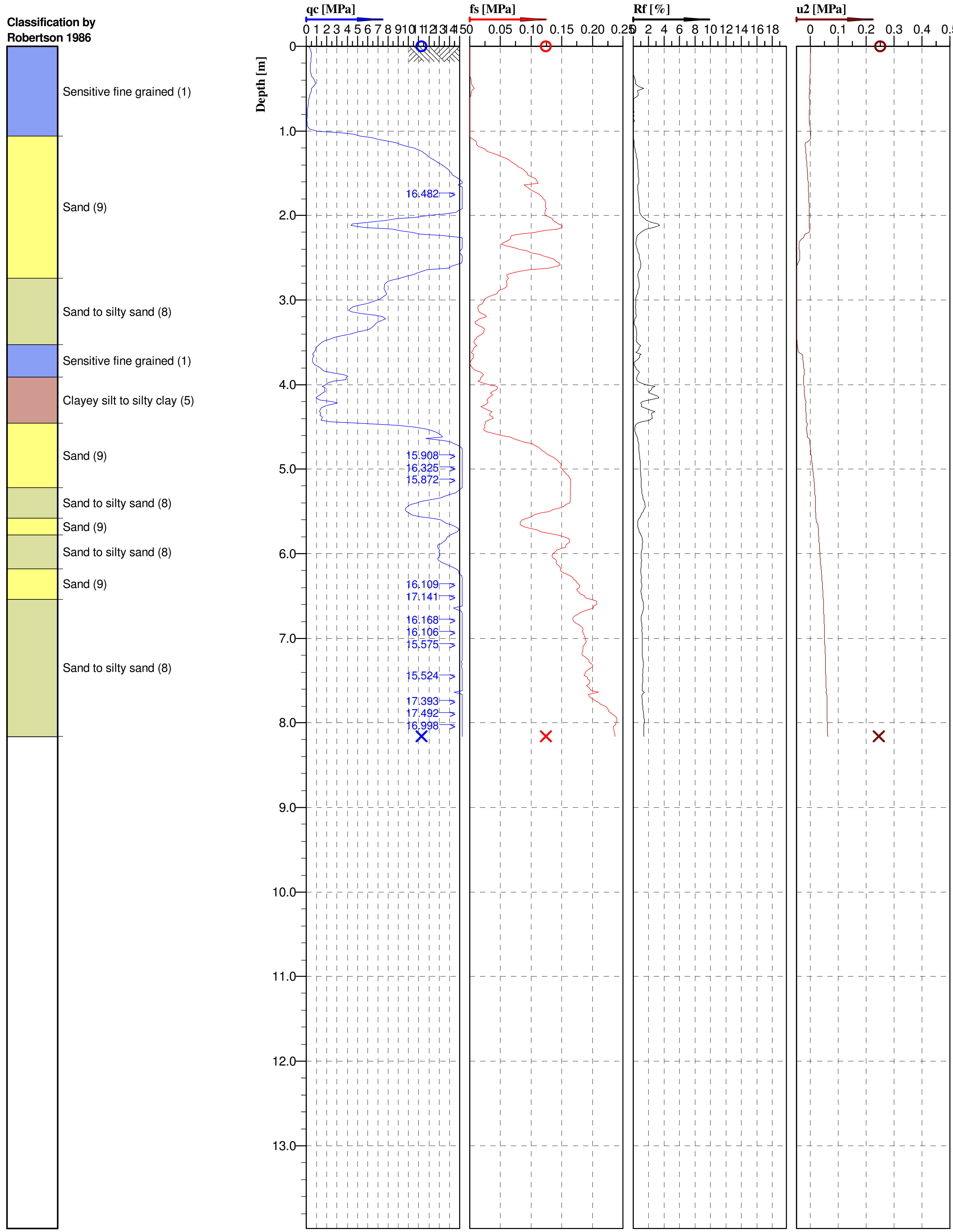


The swamps and previous creeks/rivers from 1856 have been overlayed onto a map of Christchurch in 2012

Key

- Previous creeks/rivers
- Existing creeks/rivers
- New creeks/rivers
- Swamp/Marshland

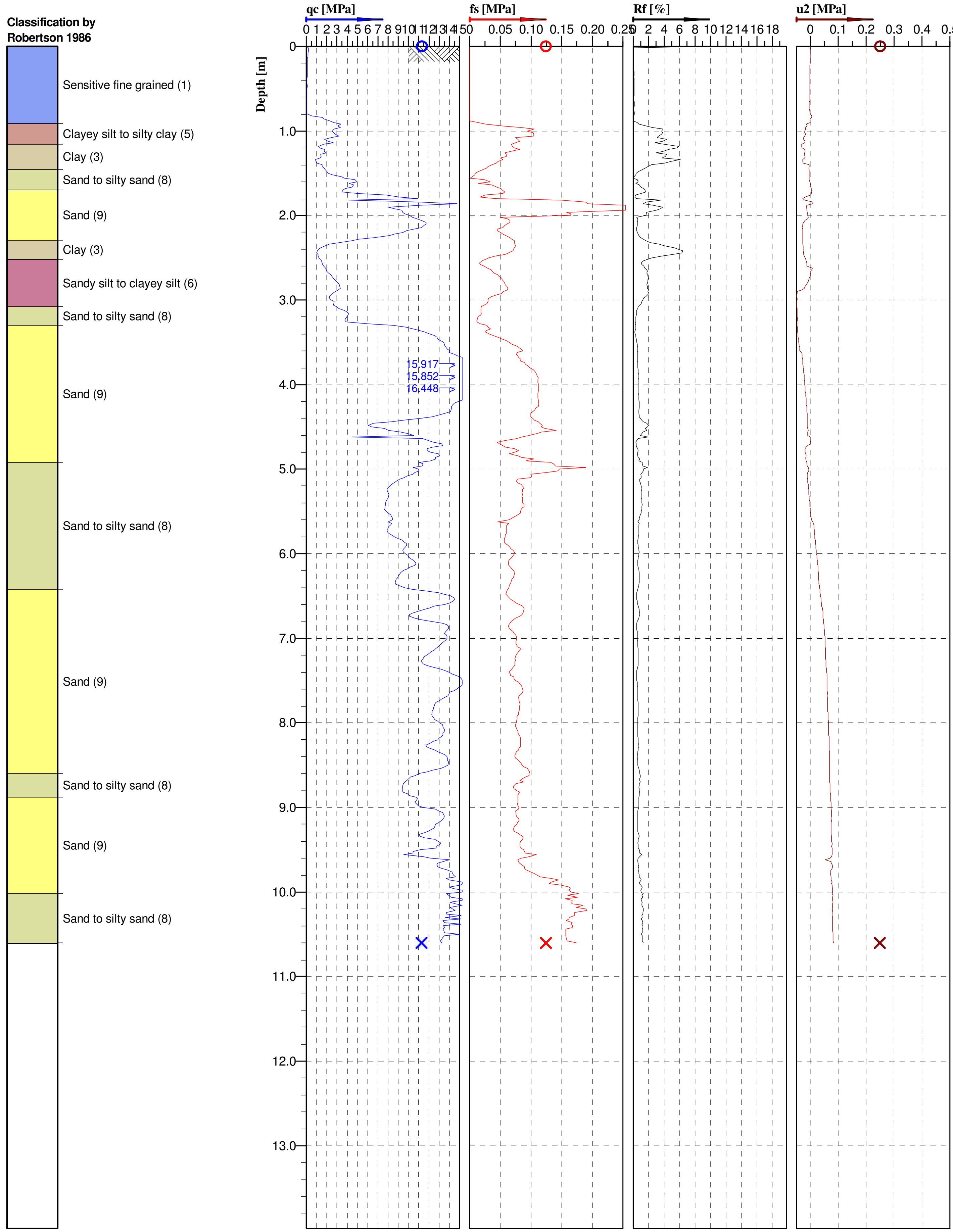
Appendix B – Existing ground investigation logs



U2

Cone No: 100KN 4341
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: LINWOOD	Position: X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no: CPT-LWD-35
Project ID:	Client: TONKIN & TAYLOR LTD	Date: 20/05/2011	Scale: 1 : 60
Project: EQC SITES		Page: 1/1	Fig:
		File: CPT-LWD-35.CPT	

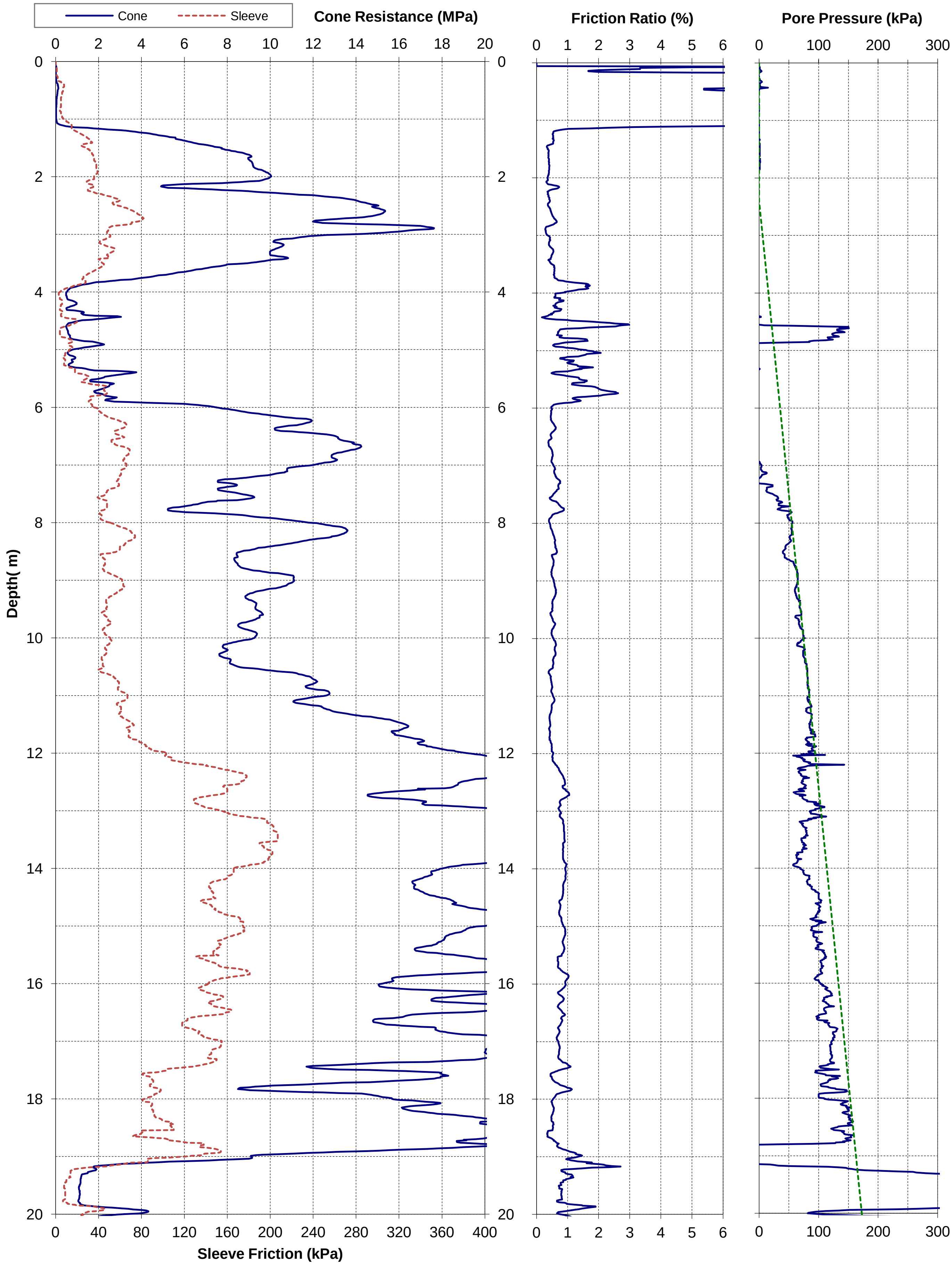


U2

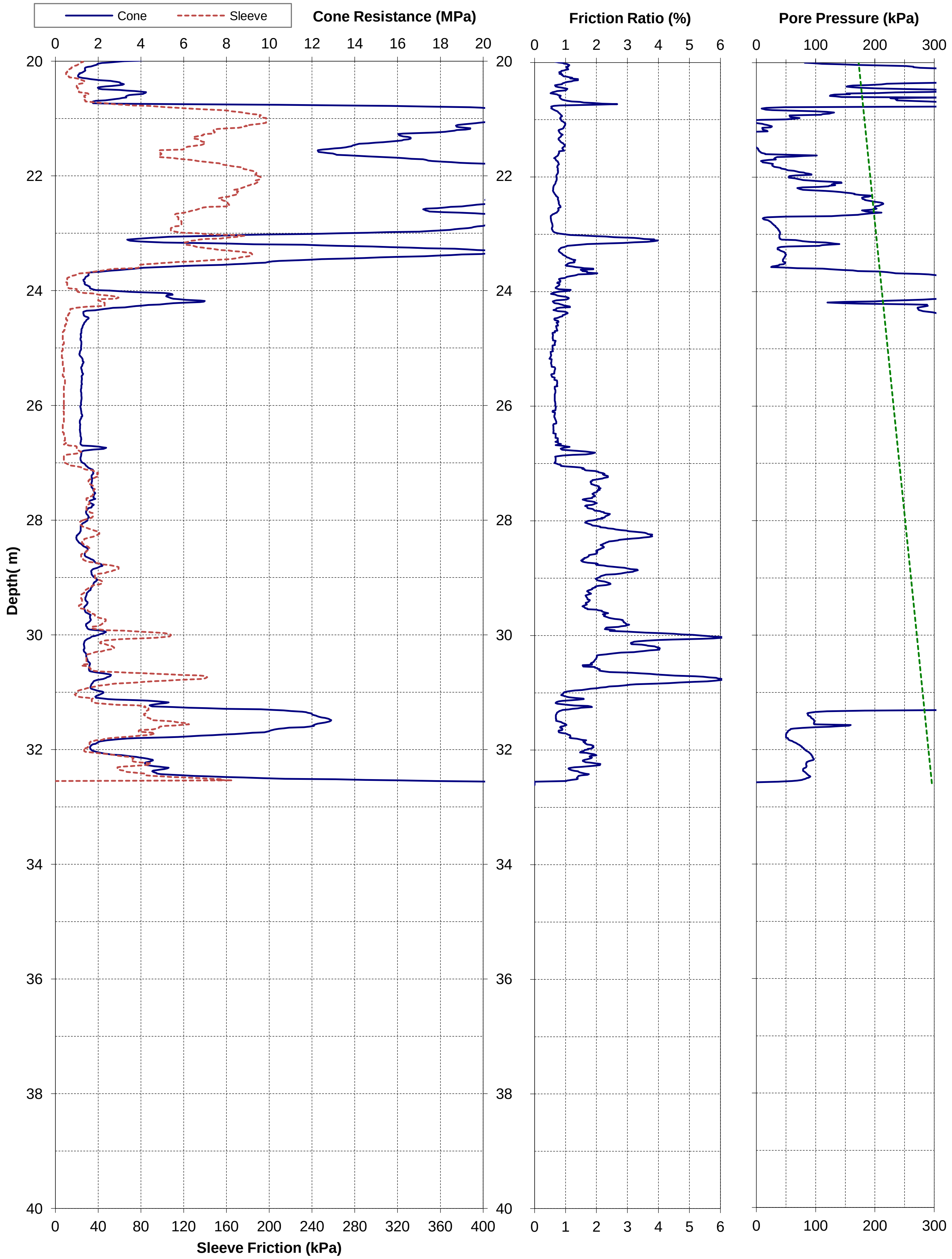
Cone No: 100KN 4341
Tip area [cm2]: 10
Sleeve area [cm2]: 150

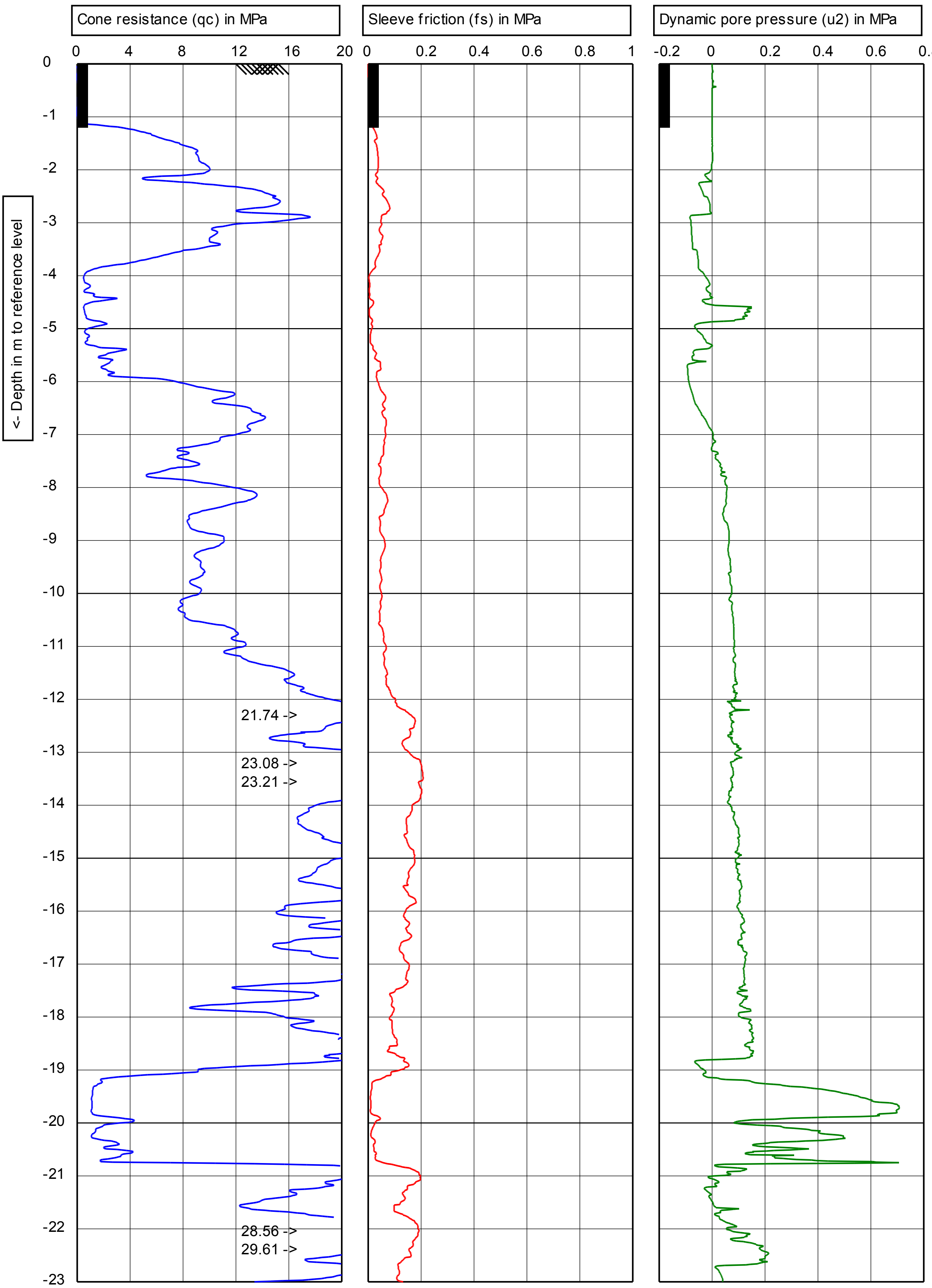
Location: LINWOOD	Position: X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no: CPT-LWD-34
Project ID:	Client: TONKIN & TAYLOR LTD	Date: 20/05/2011	Scale: 1 : 60
Project: EQC SITES		Page: 1/1	Fig:
		File: CPT-LWD-34.CPT	

Project:	Christchurch 2011 Earthquake - EQC Ground Investigations			Page:	1 of 2	CPT-BRY-18
Test Date:	17-Jun-2011	Location:	Bromley	Operator:	Opus	
Pre-Drill:	1.2m	Assumed GWL:	2.4mBGL	Located By:	Survey GPS	
Position:	2484276.9mE	5741214.3mN	2.39mRL	Coord. System:	NZMG & MSL	
Other Tests:				Comments:		



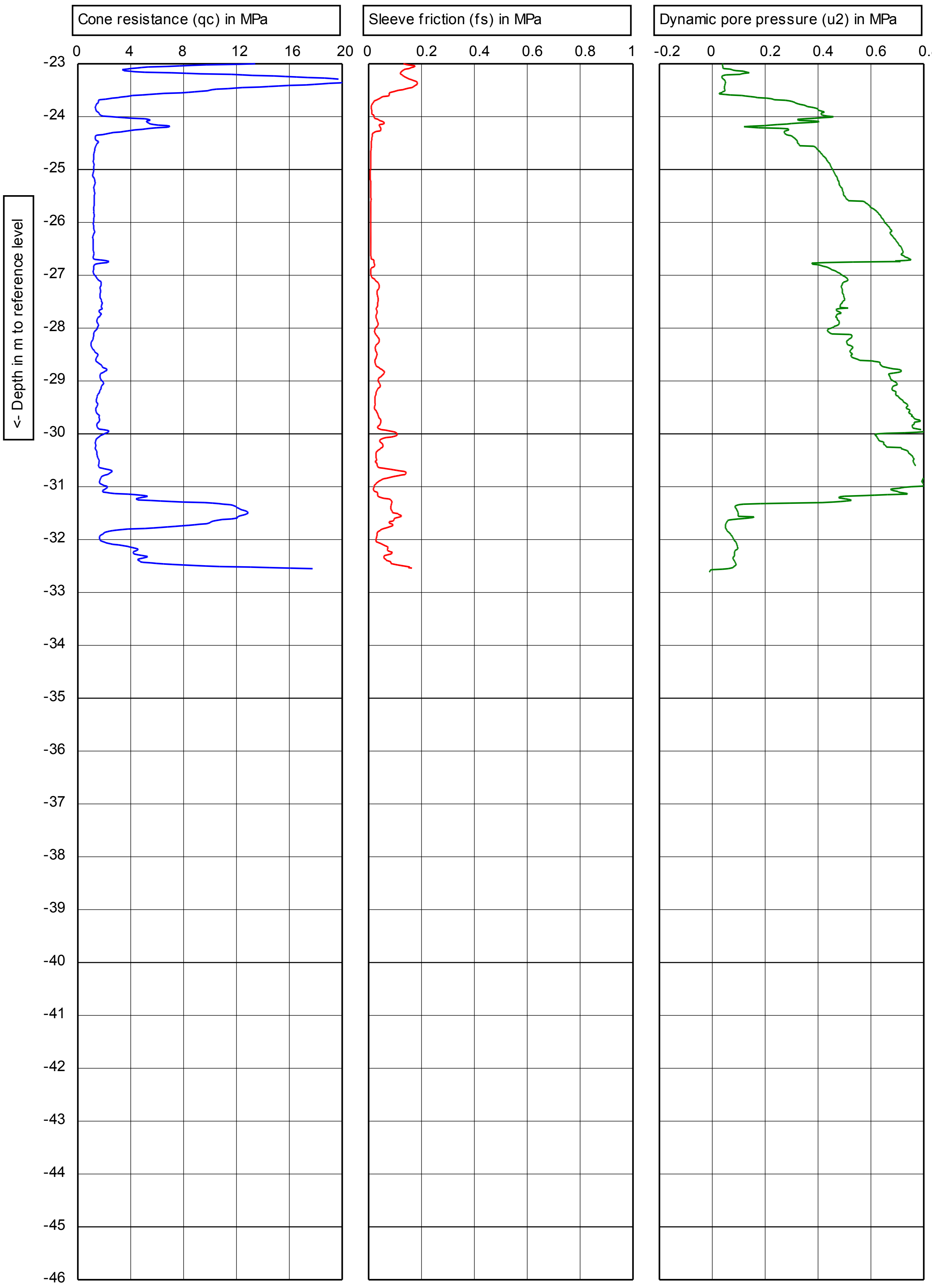
Project:	Christchurch 2011 Earthquake - EQC Ground Investigations			Page:	2 of 2	CPT-BRY-18
Test Date:	17-Jun-2011	Location:	Bromley	Operator:	Opus	
Pre-Drill:	1.2m	Assumed GWL:	2.4mBGL	Located By:	Survey GPS	
Position:	2484276.9mE	5741214.3mN	2.39mRL	Coord. System:	NZMG & MSL	
Other Tests:				Comments:		





OPUS HAMILTON LABORATORIES	u2 150 cm² 10 cm²	Test according to A.S.T.M standard D-5778-95	Predrill :	1.2
		G.L. **0** W.L.: **-60**	Date:	**17/06/2011**
		Project: **BRY**	Cone no.:	**C10CFIIP.C10204**
		Location: **GPS: E2484271 N5741215**	Project no.:	**2-68292.11**
		Position:	CPT no.:	**CPT-BRY-18** 1/12

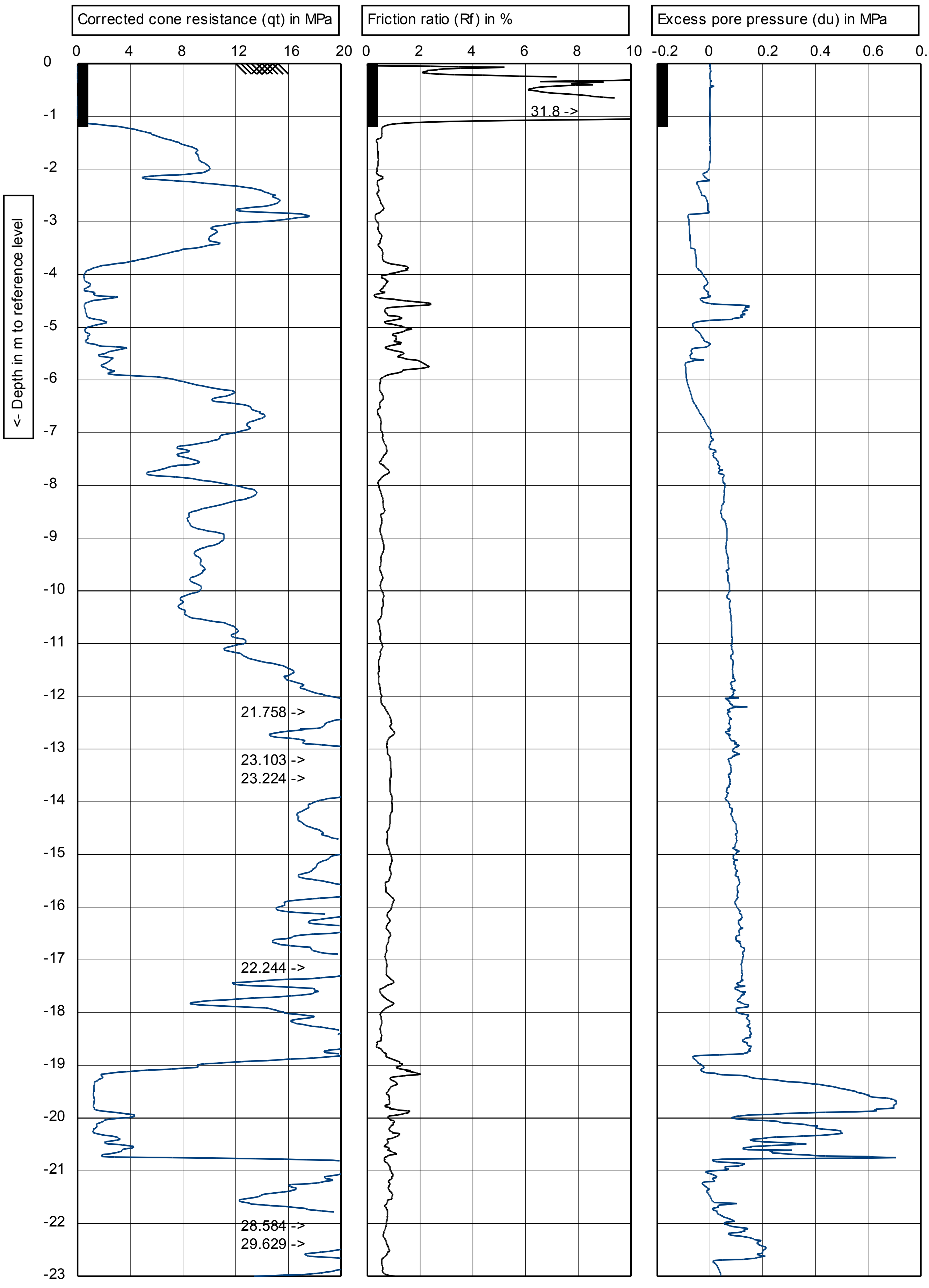
CPTask V1.20



Test according to A.S.T.M standard D-5778-95		Predrill :	1.2
G.L. **0**	W.L.: **-60**	Date:	**17/06/2011**
Project: **BRY**		Cone no.:	**C10CFIIP.C10204**
Location: **GPS: E2484271 N5741215**		Project no.:	**2-68292.11**
Position:		CPT no.:	**CPT-BRY-18** 2/12

OPUS HAMILTON LABORATORIES

CPTask V1.20



CPTask V1.20

OPUS HAMILTON LABORATORIES

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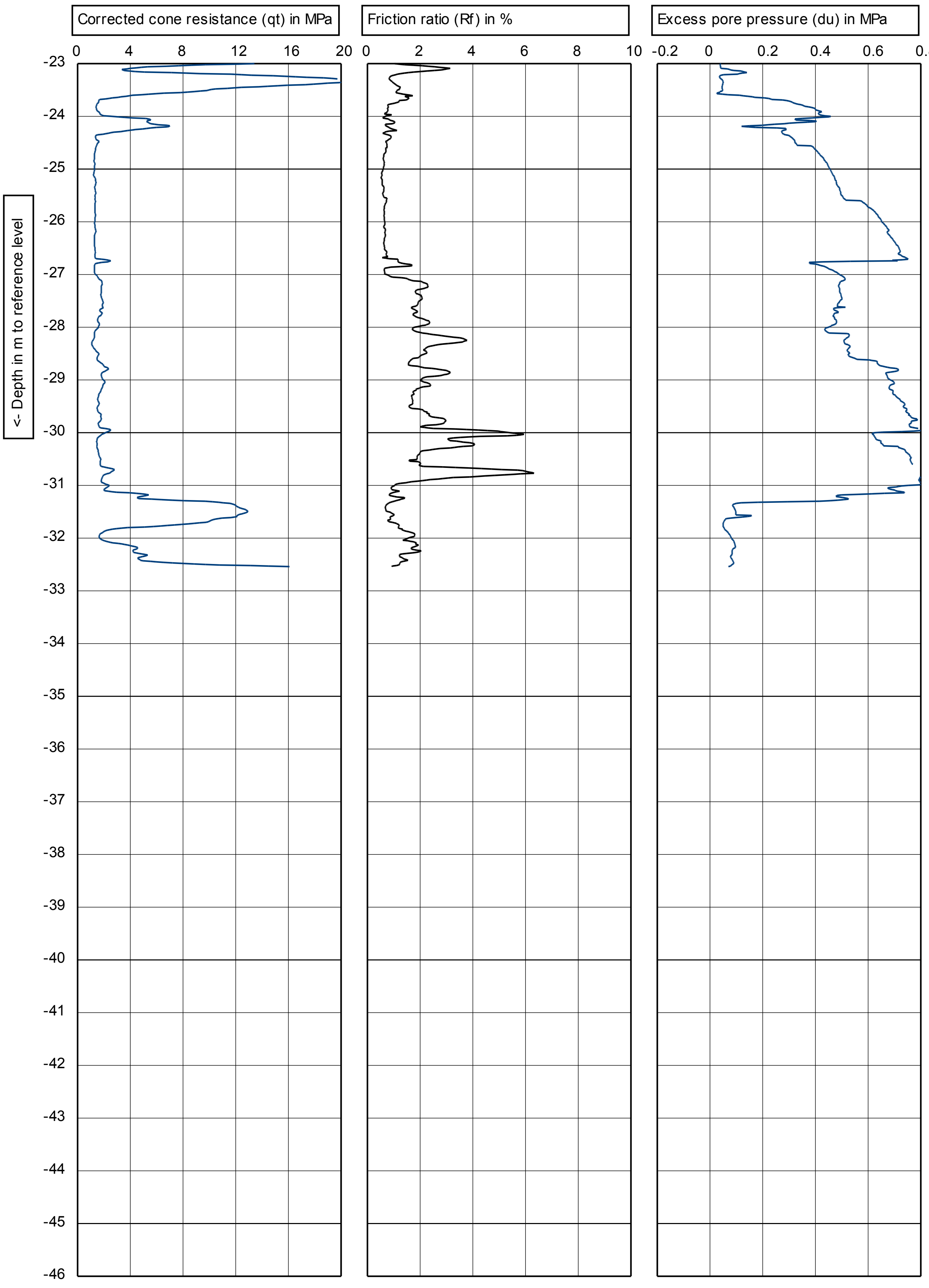
G.L. **0**	W.L.: **-60**	Date:	**17/06/2011**
Project: **BRY**		Cone no.:	**C10CFIIP.C10204**
Location: **GPS: E2484271 N5741215**		Project no.:	**2-68292.11**
Position:		CPT no.:	**CPT-BRY-18** **3/12**



OPUS HAMILTON LABORATORIES	Test according to A.S.T.M standard D-5778-95	Predrill :	1.2
u2 150 cm² 10 cm²	G.L. **0** W.L.: **-60**	Date:	**17/06/2011**
Project: **BRY**		Cone no.:	**C10CFIIP.C10204**
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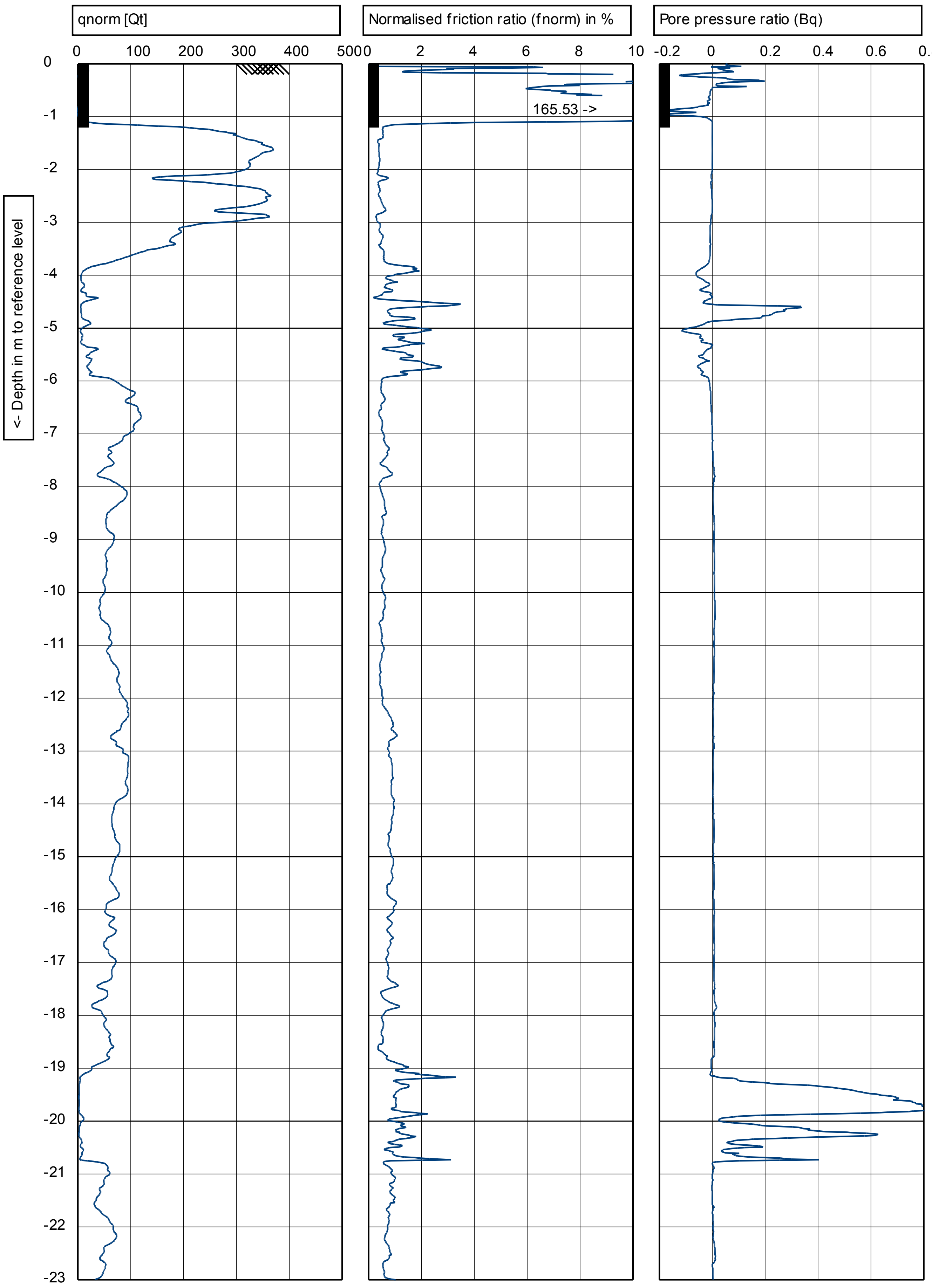
Position:		CPT no.:	**CPT-BRY-18** **4/12**



CPTask V1.20

OPUS HAMILTON LABORATORIES

u2 150 cm² 10 cm²

Test according to A.S.T.M standard D-5778-95		Predrill :	**1.2**
G.L. **0**	W.L.: **-60**	Date:	**17/06/2011**
Project: **BRY**		Cone no.:	**C10CFIIP.C10204**
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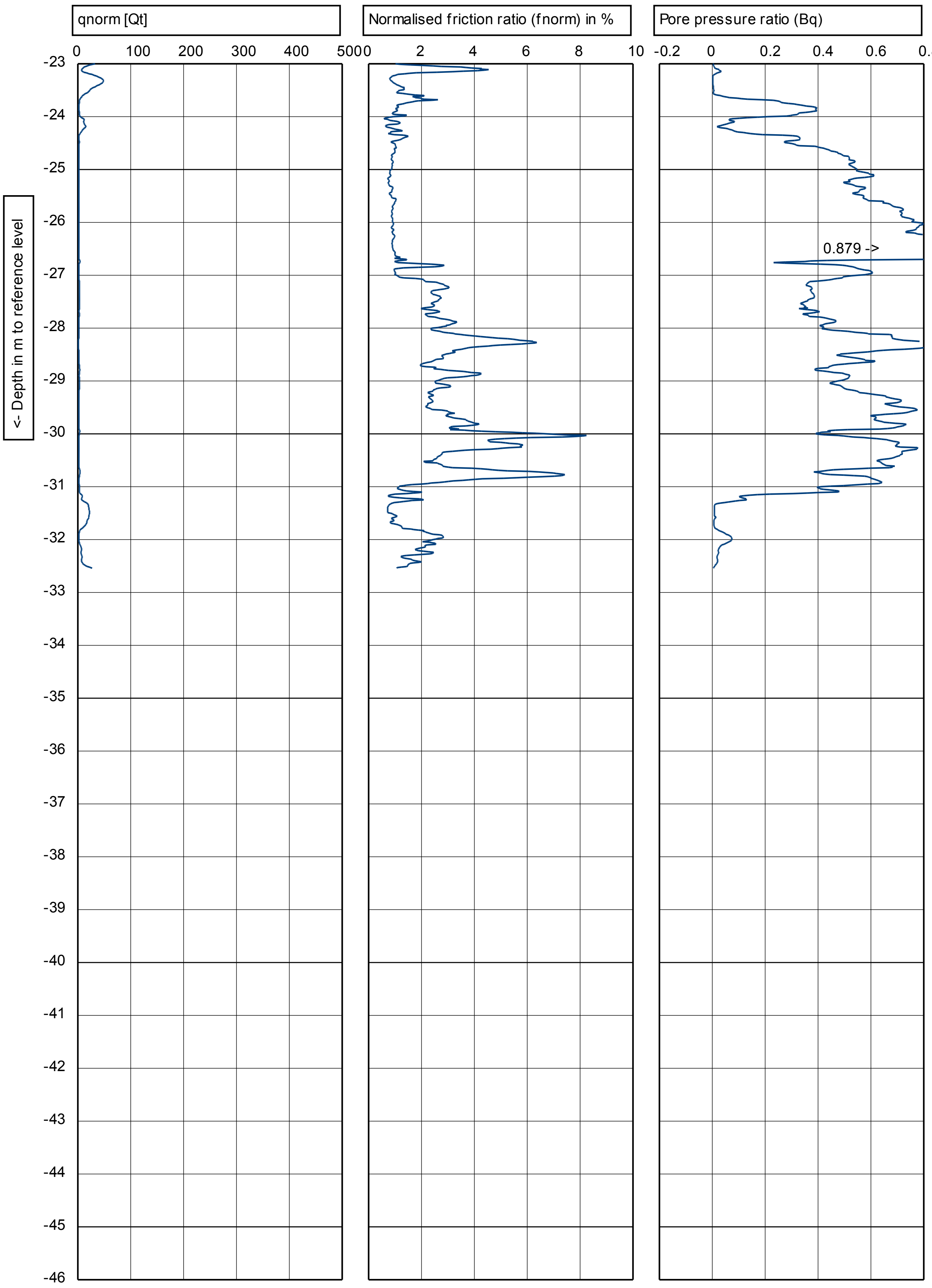
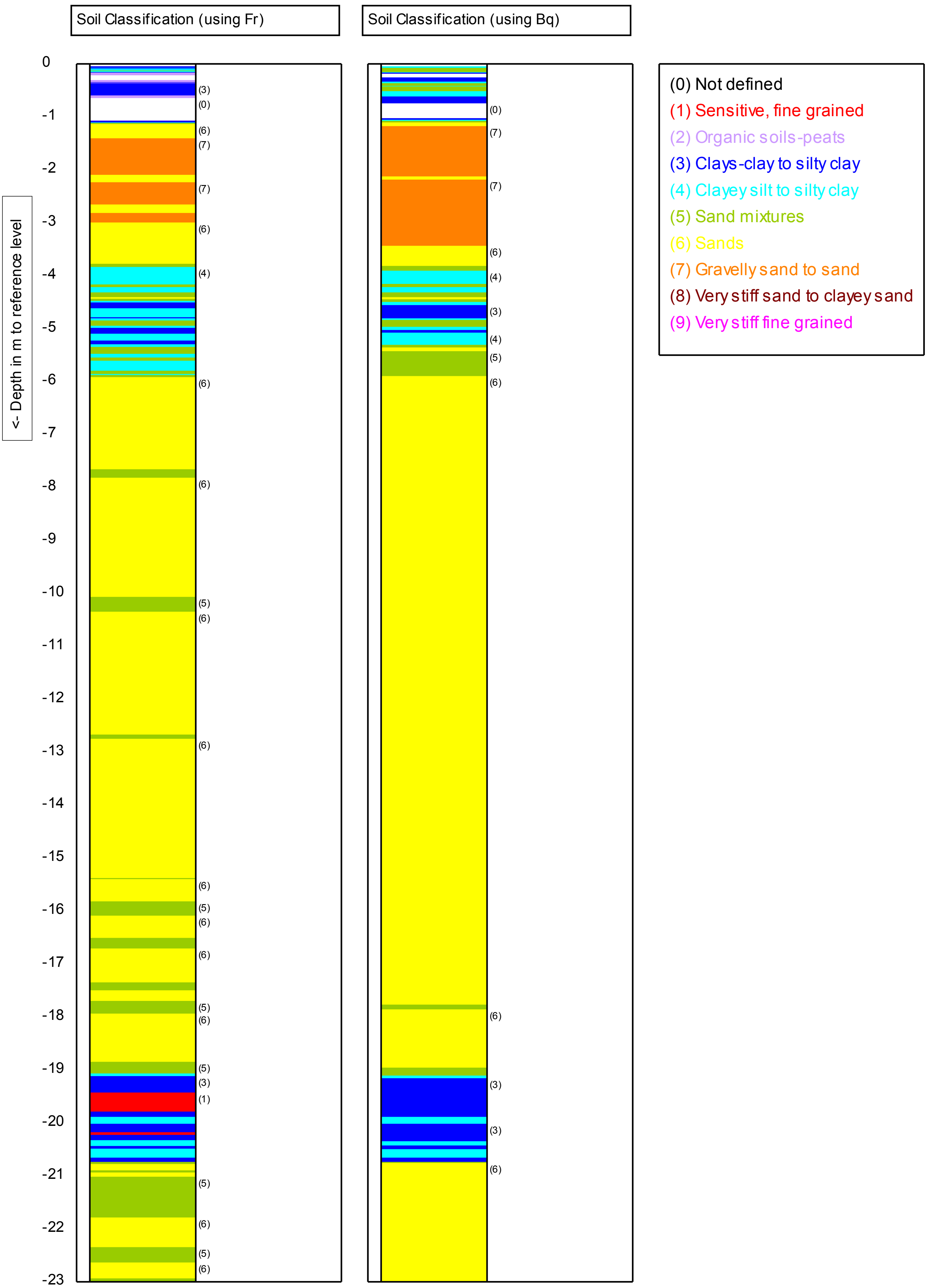
Position:		CPT no.:	**CPT-BRY-18** 5/12



OPUS HAMILTON LABORATORIES	u2 150 cm² 10 cm²	Test according to A.S.T.M standard D-5778-95	Predrill :	**1.2**
		G.L. **0** W.L.: **-60**	Date:	**17/06/2011**
	Project: **BRY**		Cone no.:	**C10CFIIP.C10204**
	Location: **GPS: E2484271 N5741215**		Project no.:	**2-68292.11**
	Position:		CPT no.:	**CPT-BRY-18** 6/12

CPTask V1.20



CPTask V1.20

OPUS HAMILTON LABORATORIES

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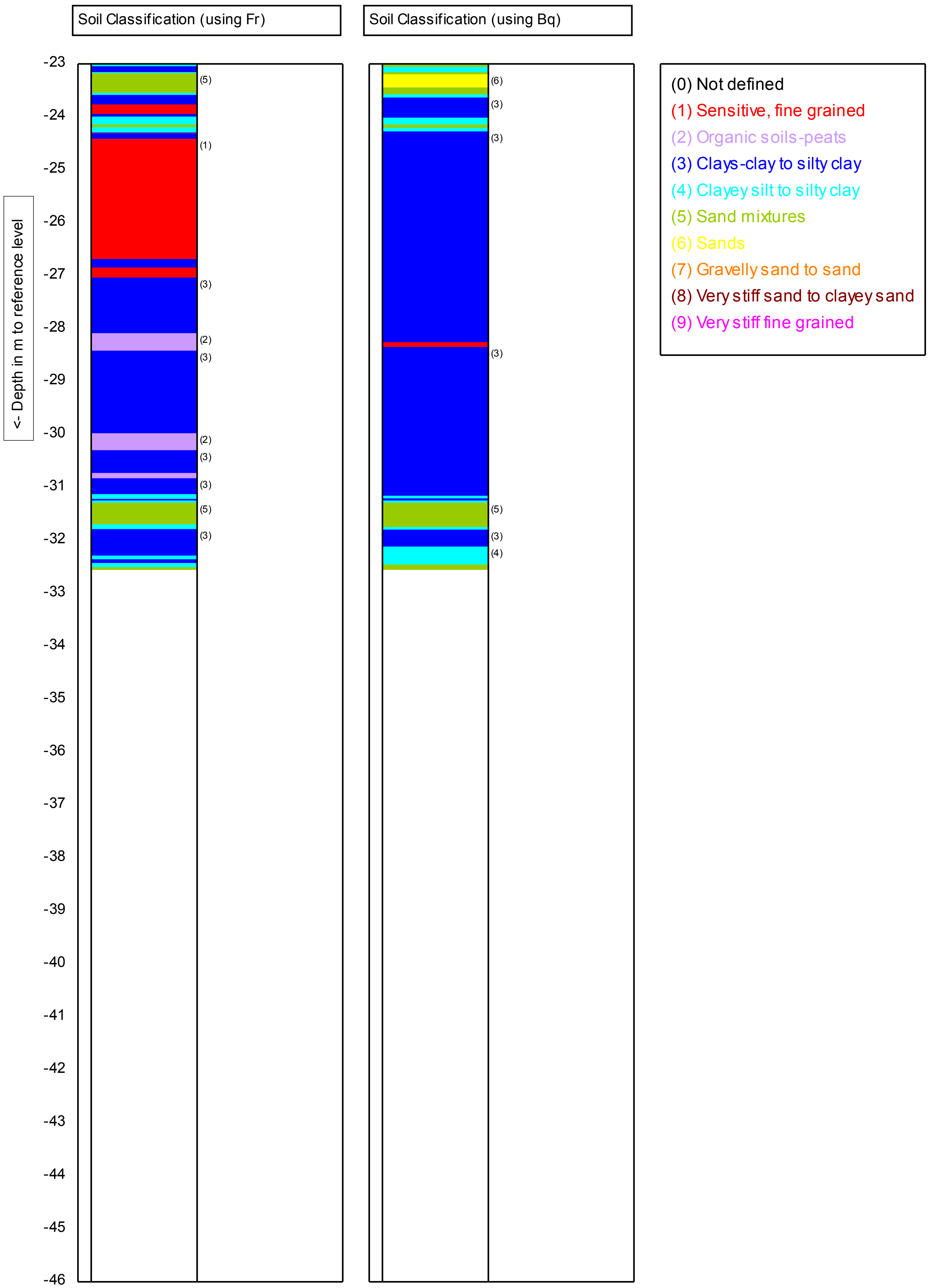
Position:		CPT no.:	**CPT-BRY-18** 7/12



CPTask V1.20

OPUS HAMILTON LABORATORIES

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Project: **BRY**			Cone no.:	**C10CFIIP.C10204**
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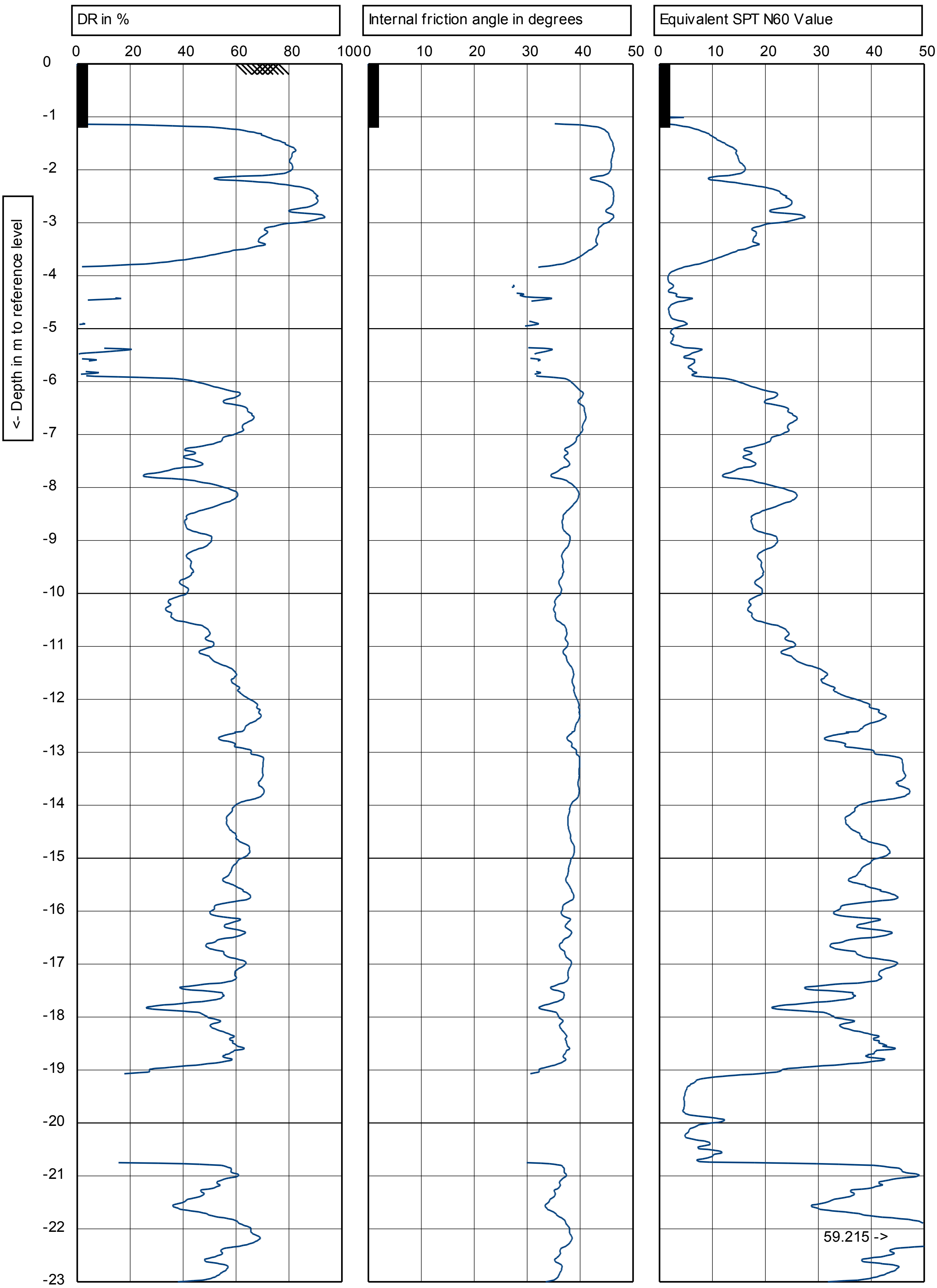
Position:			CPT no.:	**CPT-BRY-18** 8/12



OPUS
HAMILTON LABORATORIES

u2 150 cm² 10 cm²

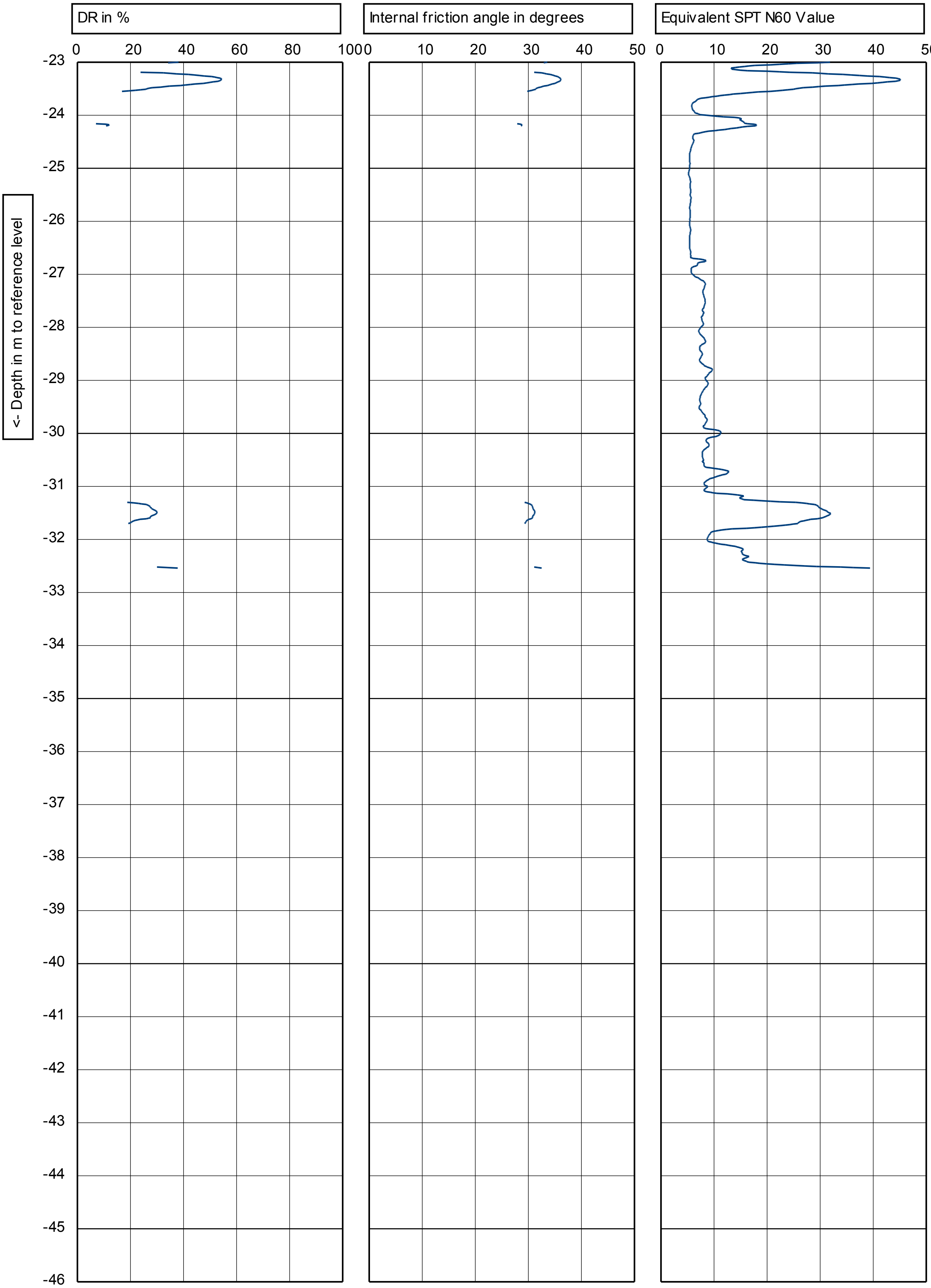
Test according to A.S.T.M standard D-5778-95

G.L. **0** W.L.: **-60**

Project: **BRY**
Location: **GPS: E2484271 N5741215**
Position:

Predrill : **1.2**
Date: **17/06/2011**
Cone no.: **C10CFIIP.C10204**
Project no.: **2-68292.11**
CPT no.: **CPT-BRY-18** **9/12**

CPTask V1.20



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OPUS HAMILTON LABORATORIES	Project: **BRY**		Cone no.: **C10CFIIP.C10204**
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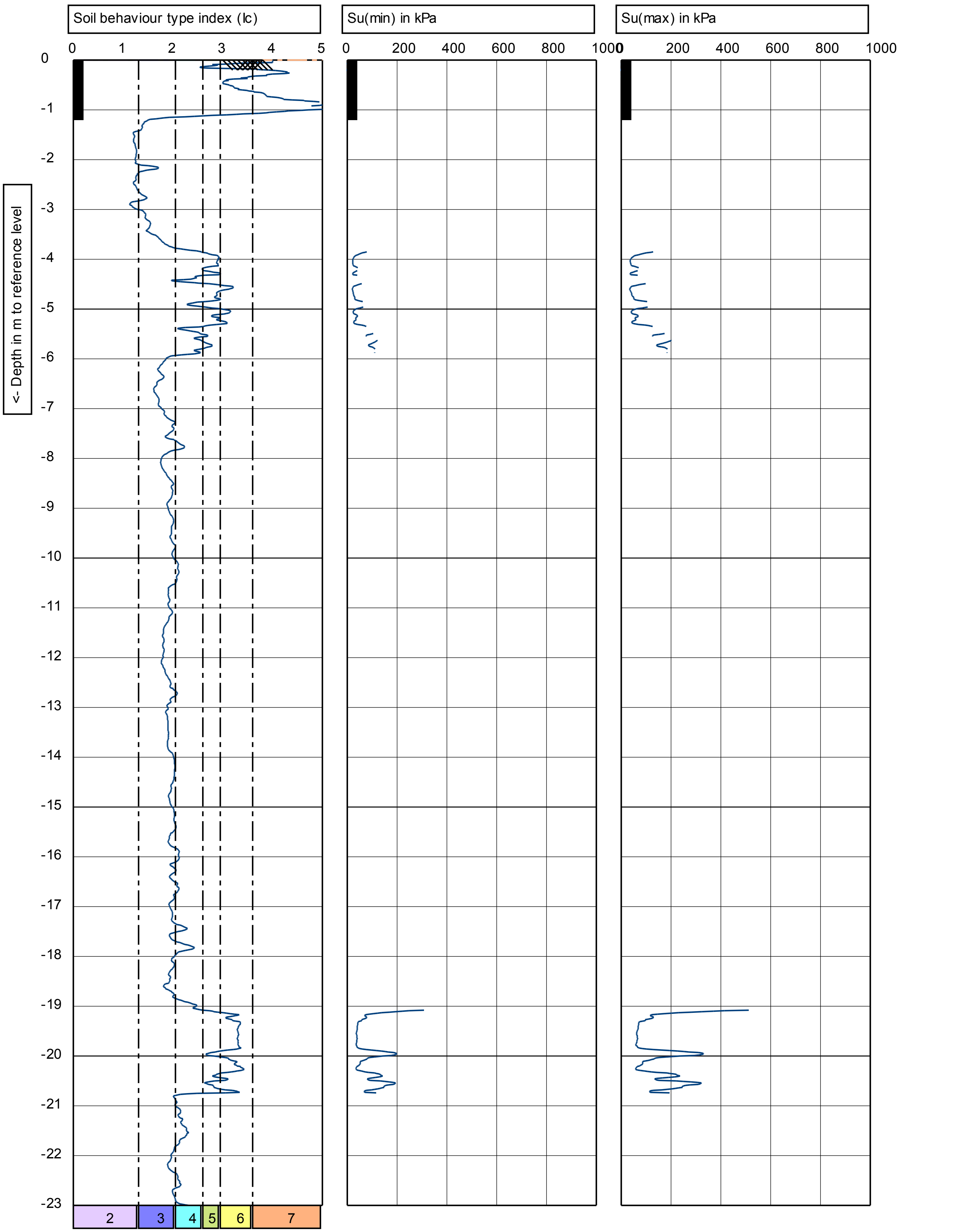
CPTask V1.20



OPUS HAMILTON LABORATORIES	u2 150 cm² 10 cm²	Test according to A.S.T.M standard D-5778-95	Predrill :	**1.2**
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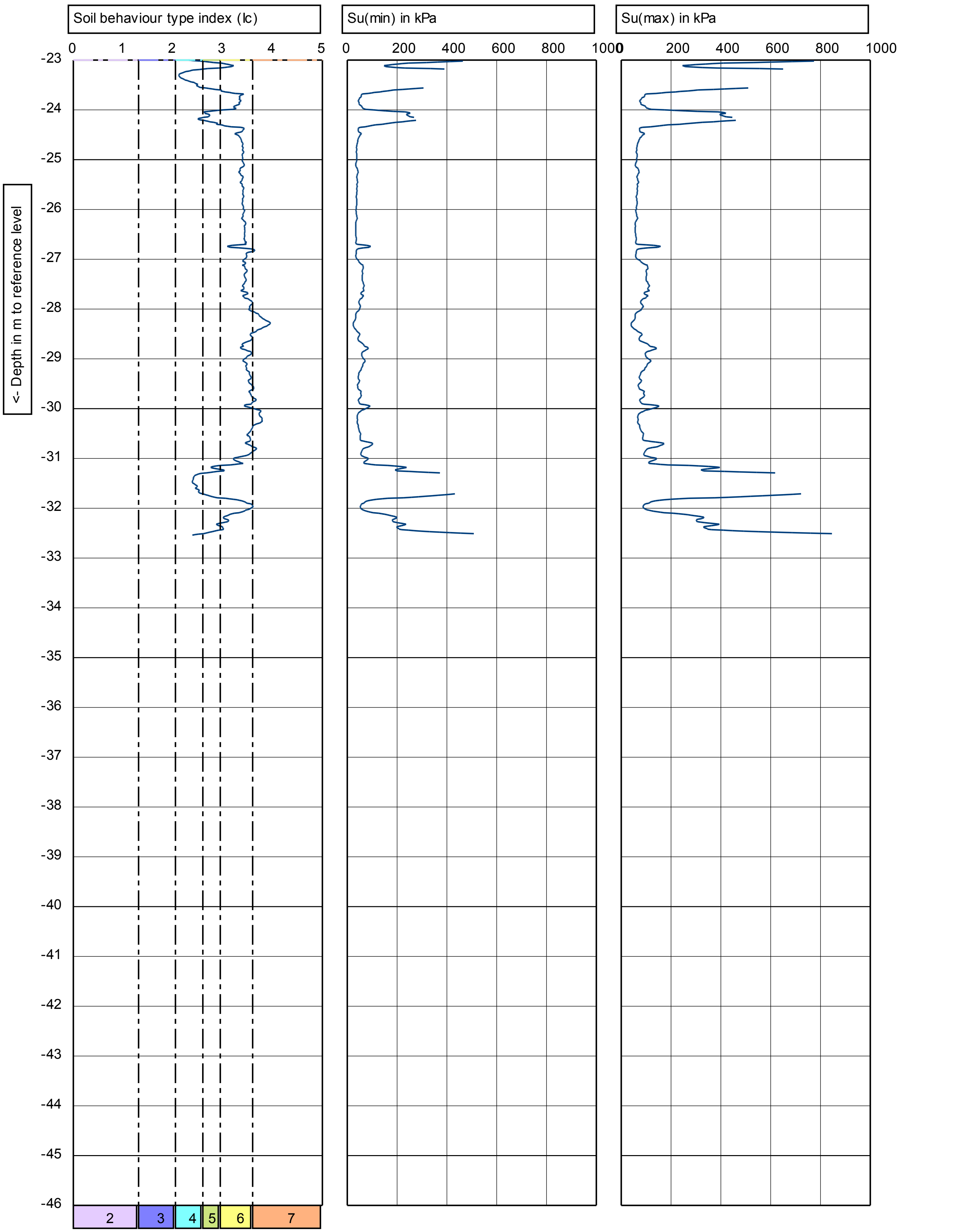

OPUS HAMILTON LABORATORIES

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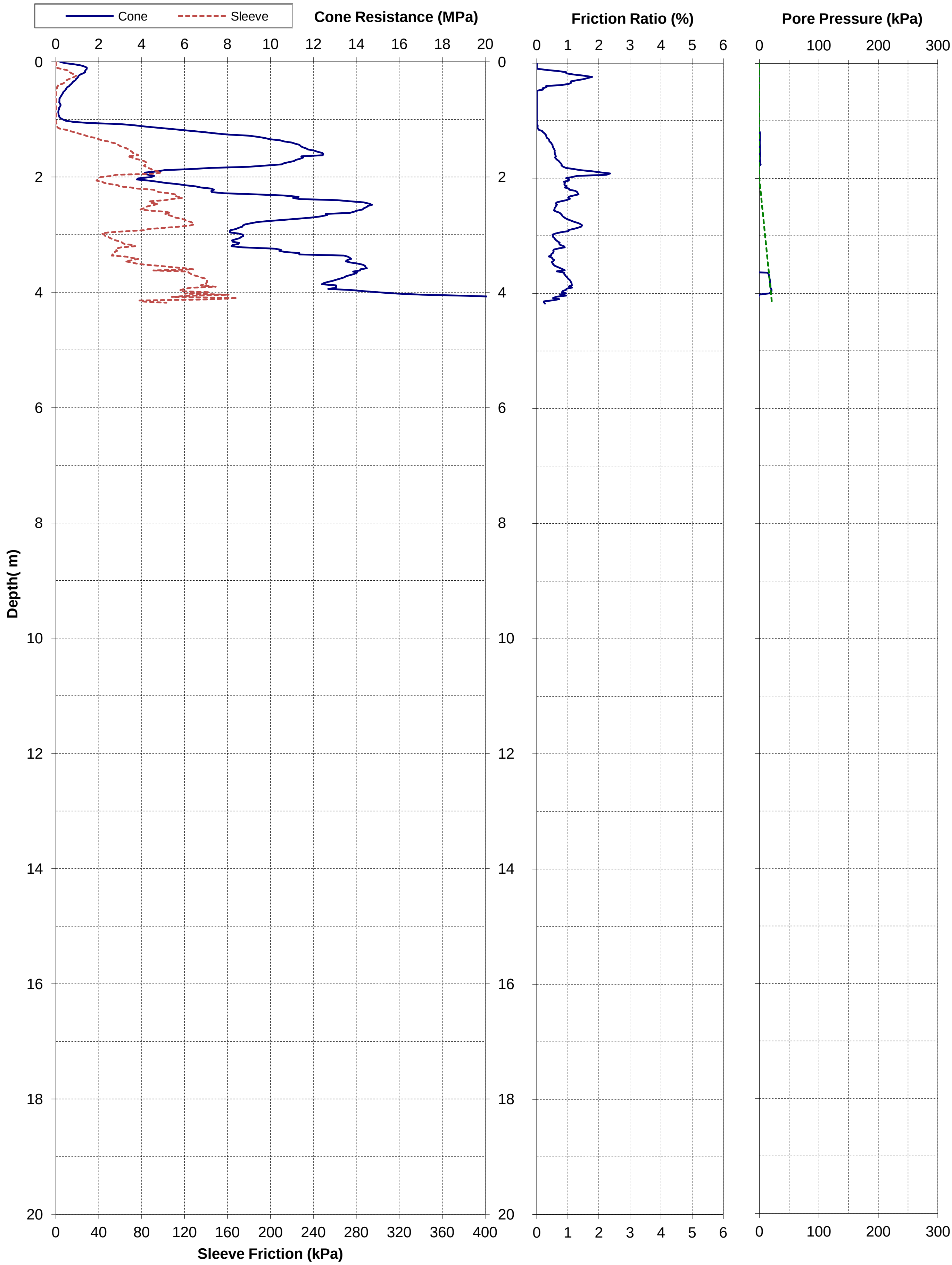
	Project: **BRY**	Cone no.:	**C10CFIIP.C10204**
	Location: **GPS: E2484271 N5741215**	Project no.:	**2-68292.11**
	Position:	CPT no.:	**CPT-BRY-18** 12/12

CPTask V1.20

Project:	Christchurch 2011 Earthquake - EQC Ground Investigations			**Page:**	1 of 1	**CPT-LWD-28**
Test Date:	19-May-2011	**Location:**	Linwood	**Operator:**	Geotech	
Pre-Drill:	1.2m	**Assumed GWL:**	2mBGL	**Located By:**	Survey GPS	
Position:	2483547.1mE	5741248.6mN	2.73mRL	**Coord. System:**	NZMG & MSL	
Other Tests:				**Comments:**		



Borelog for well M35/2111

Gridref: M35:836-414 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 3.1 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -66.4m Drill Date : 8/12/1944

Environment Canterbury Regional Council

Water Level: Artesian

Depth(m)	Full Drillers Description	Formation Code
-9.10m	Sand & clay	sp?
- 19.8m	Sand & gravel	sp?
- 26.8m	Blue sand & clay	ch
- 32.3m	Blue clay & peat	ch
- 40.5m	Blue shingle	ri
- 53.6m	Brown shingle	ri
- 53.9m	Blue sand	br
- 58.8m	Yellow clay	br
- 63.0m	Brown sand	br
- 66.4m	Brown shingle	li-1

Scale(m): -10, -20, -30, -40, -50, -60

Appendix C – Geotechnical Investigation Summary

- **Table 1 Summary of most relevant investigation data**

ID		1	2	3	4	5
Type *		CPT	CPT	CPT	CPT	BH
Ref		LWD-35	LWD-34	BRY-18	LWD-28	M35-2111
Depth (m)		8	11	32	32	66
Distance from site (m)		100	200	375	450	500
Ground water level (mBGL)		4	3	2.5	2	Artesian
Simplified recorded geological profile (depth below ground level to top of stratum, m)	0	N/A	N/A		N/A	
	1				MD	
	2				MD	
	3				MD	
	4				So	
	5				So	
	6				MD	
	7				MD	
	8				MD	
	9				MD	
	10				MD	
	11				D	
	12				D	
	13				D	
	14				D	
	15				D	
	16				D	
	17				D	
	18				D	
	19				F	
	20				F	
	21				D	
	22				D	
	23				St	
	24				St	
	25				St	
Greater depths				Clay to 31 m	Clay to 32 m	

*BH: Borehole, HA: Hand Auger, WW: Water Well, CPT: Cone Penetration Test

Sensitive or organic clay/silt | Clay to silty clay | Clayey silt to silt | Silty sand to silt

Clayey sand | Sand | Gravelly sand or gravel

VL = very loose, L = loose, MD = medium dense, D = dense, VD = very dense
VS = very soft, So = soft, F = firm, St = stiff, VS = very stiff, H = hard