CHRISTCHURCH CITY COUNCIL
BU 0822-014 EQ2
Linwood Resource Centre
322 Linwood Avenue, Linwood

QUANTITATIVE ASSESSMENT REPORT
FINAL

- Rev C
- 01 July 2013

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Contents

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1. Executive Summary

1.1. Background

A Quantitative Assessment was carried out on building BU 0822-014 EQ2 located at 322 Linwood Avenue, Linwood. This building is a single storey timber framed house that has been converted into a resource centre. An aerial photograph illustrating the building's location is shown below in Figure 1. Detailed descriptions outlining the building's age and construction type is given in Section 5 of this report.

- **Figure 1: Aerial Photograph of Building BU-0822-014 EQ2 Located at 322 Linwood Ave**

This Quantitative 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 inspection on 26 March 2012, intrusive investigations on 28 May 2012, and subsequent calculations.

SKM prepared an assessment in September 2012 based on limited information available at the time. We have reassessed based on new information which has become available and on further consideration of the assumptions made. A peer review of our previous report was prepared by others. We have addressed the comments which were raised in the peer review where appropriate.

1.2. Key Damage Observed

Key damage observed includes:-

- Hairline cracking to internal wall linings.
- The eastern block wall around the rear entrance has pulled away from the building.
- A building consent is not likely to be required to repair the damage noted above.

1.3. Critical Structural Weaknesses

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

1.4. Indicative Building Strength

As described in the Engineering Advisory Group's "Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings" (from July 2011) we have assessed the percentage of new building standard seismic resistance using the quantitative method. Our assessment included consideration of geotechnical conditions, existing earthquake damage to the building and structural engineering calculations to assess both strength and ductility/resilience.

The assessments were based on the following:

- On-site investigation to assess the extent of existing earthquake damage including limited intrusive investigation.
- No geotechnical investigation has been undertaken. We have based this report on our knowledge of the area surrounding the site and the absence of liquefaction ejecta on the site at the time of inspection.
- Assessment of the strength of the existing structures taking account of the current condition.

Any building that is found to have a seismic capacity less than 33% of the new building standard is required to be strengthened up to a capacity of at least 67%NBS.

No drawings were available for the existing piles, and excavation to reveal their geometry below ground was not able to be undertaken as part of our initial intrusive investigations. The buildings original capacity has been estimated to be in the order of 68%NBS and post-earthquake capacity also in the order of 68%NBS.

The building has been assessed to have a seismic capacity in the order of 68% NBS and is therefore not likely to be earthquake prone.

1.5. Recommendations

Based on the findings of this quantitative assessment indicating the building is in the order of 68% NBS and strengthening is not required.

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.

2. Introduction

Sinclair Knight Merz was engaged by the Christchurch City Council to carry out a Quantitative Assessment of the seismic performance of building BU 0822-014 EQ2 located at 322 Linwood Avenue, Linwood.

The scope of this quantitative analysis includes the following:

- Analysis of the seismic load carrying capacity of the building compared with current seismic loading requirements or New Buildings Standard (NBS). It should be noted that this analysis considers the building in its damaged state where appropriate.
- Identify any critical structural weaknesses which may exist in the building and include these in the assessed %NBS of the structure.
- Preparation of a summary report outlining the areas of concern in the building as well as identifying strengthening concepts to 67%NBS for any areas which have insufficient capacity if the building is found to be an earthquake prone building.

The recommendations from the Engineering Advisory Group[1] were followed to assess the likely performance of the structures 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[2].

The initial qualitative assessment identified that the seismic capacity of the building was likely to be less than 67% of the New Building Standard (NBS). A quantitative assessment was recommended to confirm the initial assessment findings and to determine a more accurate seismic rating of the building.

At the time of this report, intrusive site investigations had been carried out as recommended by the previous qualitative report. Construction drawings were not made available, and this has been considered in our evaluation of the building. The building description in Section 5 is based on our visual inspections and intrusive investigations.

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[1] EAG 2011, *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury - Draft*, p 10

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

3. Compliance

This section contains a brief 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	Improvement of Structural Performance: 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 provides an indication of the risk of failure for an existing building with a given percentage NBS, relative to the risk of failure for a new building that has been designed to meet current Building Code criteria (the annual probability of exceedance specified by current earthquake design standards for a building of 'normal' importance is 1/500, or 0.2% in the next year, which is equivalent to 10% probability of exceedance in the next 50 years).

- **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

Building BU 0822-014 EQ2 is a 1970's single storey house that has been converted into a resource centre. It is constructed from timber framing and is externally clad with summerhill stone. The roof structure consists of timber framing and steel corrugate cladding. The ground floor is also a timber structure that is supported on concrete piles and a concrete perimeter shallow pad footing.

5.2. Gravity Load resisting System

Our evaluation was based on our site investigation conducted on the 26 March 2012 and intrusive investigations on 28 May 2012. These investigations allowed us to verify the structural system of the building.

Building BU 0822-014 EQ2 is a single storey timber framed structure. The roof structure consists of 100x45 joists at 600mm centres, and 75x45 purlins at 600mm centres. The roof is clad with timber boards and steel corrugated sheets. The roof structure is supported on the timber framed walls of the building. The timber studs in the walls were found to be 100x45 at 400 centres. The walls are supported by the floor joists and 100x45 bearers at 1500mm centres, which are then supported by the concrete piles at 1500 centres (except for external walls which rest on a concrete perimeter ground beam).

5.3. Seismic Load Resisting system

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

Lateral loads acting across and along the building will be resisted by the timber bracing walls. These walls are constructed from timber framing and plasterboard linings.

Seismic forces will be transferred from the walls to the subfloor system below. The joists and bearers must transfer forces from the walls to the concrete piles and perimeter beams..

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.
- It is expected that the allowable bearing capacity of a shallow pad footing on this site will be in the region of 200 kPa. We estimate a conservative ultimate bearing capacity to be in the order of 400 kPa. However, these may be revised by a site specific investigation.
- Liquefaction risk is low at this site.

Unless a change of use is intended for the site we do not believe that any further geotechnical investigations are required. Specific ground investigation should be undertaken if significant alterations or new structures are proposed. If any excavations are required on the site further investigation of the potential for contamination should be undertaken. The full geotechnical desktop study can be found in Appendix 4 – Geotechnical Desk Study.

5.5. Building Damage

General

1) No visual evidence of settlement was noted at this site.

External Damage

1) Hairline cracking present along the joints in the soffit linings (typical) (PHOTO 2, 3 & 4).

2) The eastern summerhill stone wall at the rear entrance has pulled away from the main structure (PHOTO 5 & 6).

Internal Damage

1) Northern Meeting Room
   - Vertical hairline cracking present at the wall lining joints above the door and northern window (PHOTO 8, 9 & 10).
   - Hairline cracking present between the ceiling lining and wall lining joint in the SW corner.
   - Vertical hairline crack present in the wall lining joint above the fire place on the eastern wall (PHOTO 11 & 12).
2) North-East Office (1)
   - Hairline crack present in the ceiling lining joint located in the centre of the room (PHOTO 14).
3) North-East Office (2)
   - Hairline cracking present between the wall lining and window architrave joint (PHOTO 16).
4) Bathroom
   - This room was being used as a storage room at the time of our inspection and as a result we were unable to inspect this room fully. No major damage was noted.
5) Toilet
   - Hairline cracking present between the wall lining and window architrave joint.
   - Hairline cracking present between the wall lining and wall timber trim located at mid height of the wall (PHOTO 18).
   - Vertical hairline cracking present above the eastern window (PHOTO 19).
6) Wash Area
   - Vertical hairline cracking present at wall lining joint above the window (PHOTO 21).
   - Hairline cracking present along the timber scotia and ceiling lining joint (PHOTO 21).

- Hairline cracking present along the timber wall trim and wall lining joint (PHOTO 22).

7) Kitchen
   - Hairline cracks present in the wall lining joints above the door (PHOTO 24).
   - Hairline cracking present along the scotia and ceiling lining joint (PHOTO 25).
   - Hairline cracking present along the timber wall trim and wall lining joint (PHOTO 26).
   - Hairline cracking at timber wall trim joints (PHOTO 27).
8) South-West Lounge (2)
   - Hairline cracking present at ceiling lining joints (PHOTO 29).
9) North-West Office
   - Hairline cracking present along the timber wall trim and wall lining joint (PHOTO 31 &32).
   - Hairline cracking present along the wall lining joint in the NE corner (PHOTO 33).
10) North-West Office (3)
   - Hairline cracking present in wall lining joint above the window on the western side (PHOTO 35).
11) Front Entrance / Hallway
   - Hairline cracking present at ceiling lining joints (PHOTO 37 & 38).
   - Hairline cracking present between the ceiling lining and wall lining joint outside the bathroom (storage room) and near the main entrance (PHOTO 38).

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

6. Available Information and Assumptions

6.1. Available Information

Following our inspections on the 26 March 2012 and 28 May 2012, SKM carried out a seismic review on the structure. This review was undertaken using the available information which was as follows:

- SKM site measurements and inspection findings for the Linwood Resource Centre (which did not include details of piles below ground).
- CCC Facilities Rebuild Earthquake Damage Assessment, Linwood Resource Centre, Prop 0822-014, dated 15th May 2013.

6.2. Survey

The building was not surveyed.

6.3. Assumptions

The assumptions made in undertaking the assessment include:

- The building was built according to good practice at the time.
- 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.
  - It is expected that the allowable bearing capacity of a shallow pad footing on this site will be in the region of 200 kPa. We estimate a conservative ultimate bearing capacity to be in the order of 400 kPa. However, these may be revised by a site specific investigation.
  - Liquefaction risk is low at this site.

  The full geotechnical desktop study can be found in Appendix 4 – Geotechnical Desk Study.
- 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 2. This level of importance is described as ‘normal’ with medium or considerable consequence for loss of human life, or considerable economic, social or environmental consequence of failure.
- The building is located in earthquake Zone 2 according to NZS3604:2011.

- Site hazard factor, Z = 0.3, NZBC, Clause B1 Structure, Amendment 11 effective from 1 August 2011

- Our previous approach only used the subfloor elements beneath the braced walls; this is deemed to be an overly conservative approach. The flooring is likely to provide diaphragm action even though it may not have been specifically designed for this purpose at the time of design. We have now assumed the tongue and groove flooring acts as a diaphragm. By doing this it will engage the entire subfloor structure to act against lateral loads.

- The capacity of the bracing walls is assumed to be 0.7 x 3kN/m for walls with plasterboard on one side, in accordance with the NZSEE publication 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes' (AISPBE), 2006. A 3kN/m capacity is equivalent to the published ultimate bracing capacity of modern GIB1 bracing systems, whereas as the additional strength reduction factor of 0.7 has been applied to reflect greater uncertainty in historical construction practices. We have assumed that nominal nailed fixings have been provided between the bottom plates and floor to connect the bracing walls to the subfloor, as required by GIB1 bracing systems. The bracing capacity of double-sided plasterboard bracing walls has been calculated using the published capacity of GIB2 bracing walls, multiplied by a strength reduction factor of 0.7. This is slightly more conservative than the NZSEE guidelines. Assumed material properties are recorded in Table 2 below, with capacities expressed at bracing units (20 bracing units = 1 kN).

- Based on our knowledge of typical historical pile construction methods, we have assumed that the conical concrete piles are shallow founded 200mm below ground level, and set into a 400x400x75 thick site concrete footing. Lateral resistance is assumed to be provided by a combination of passive pressure on the buried footing, and the mass of the pile and floor above. There is a degree of uncertainty associated with our evaluation of the lateral load capacity of the piles, however this will have a negligible effect on the final %NBS of the structure as the shallow concrete perimeter footing is the main sub floor load resisting element.

- We have assumed that the perimeter footings are built according to the requirements of NZS3604. This is a reasonable assumption as the rest of the building appears to be of standard construction. The capacity of the perimeter foundations has been taken to be 300BUs/m in accordance with Table 5.11 of NZS3604.

- **Table 2: Material Properties**

Material	Lateral Load Resistance in Earthquake
Gib lining (single side)	42 BUs/m
Gib lining (double side)	57 BUs/m
Piles	~7.5 BUs/pile
Perimeter foundation	300Bus/m

The detailed engineering analysis is a post construction evaluation. Since it is not a full design and construction monitoring, it has the following limitations:

- It is not likely to pick up on any concealed construction errors (if they exist)
- Other possible issues that could affect the performance of the building such as corrosion and modifications to the structure will not be identified unless they are visible and have been specifically mentioned in this report.
- The detailed engineering evaluation deals only with the structural aspects of the structure. Other aspects such as building services are not covered.

6.4. The Detailed Engineering Evaluation (DEE) process

The DEE is a procedure written by the Department of Building and Housing's Engineering Advisory Group and 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[3].

The procedure of the DEE is as follows:

1) Qualitative assessment procedure
    a. Determine the building's status following any rapid assessment that have been done
    b. Review any existing documentation that is available. This will give the engineer an understanding of how the building is expected to behave. If no documentation is available, site measurements may be required

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

c. Review the foundations and any geotechnical information available. This will include determining the zoning of the land and the likely soil behaviour, a site investigation may be required

d. Investigate possible Critical Structural Weaknesses (CSW) or collapse hazards

e. Assess the original and post earthquake strength of the building (this assessment is subsequently superseded by the quantitative assessment)

2) Quantitative procedure

a. Carry out a geotechnical investigation if required by the qualitative assessment

b. Analyse the building according to current building codes and standards. Analysis accounts for damage to the building.

The DEE assessment 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 3. 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 less than 33 %NBS strength which correlates to an increased risk of approximately 20 times that of 100% NBS[4]. Buildings that are identified to be earthquake prone are required by law to be strengthened within 30 years of the owner being notified that the building is potentially earthquake prone[5]. This timeframe is likely to be adjusted by CERA and Table 3 below contains the likely new recommendations.

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

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

- **Table 3: DEE 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 DEE method rates buildings based on the plans (if available) and other information known about the building 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 DEE does also consider Serviceability Limit State (SLS) performance of the building and or the level of earthquake that would start to cause damage to the building but this result is secondary to the ULS performance.

The NZ Building Code describes that the relevant codes for NBS are primarily:

- AS/NZS 1170 parts 0, 1 and 5 Structural Design Actions
- NZS 3101:2006 Concrete Structures Standard
- NZS 3404:1997 Steel Structures Standard
- NZS 2606:1993 Timber Structures Standard
- NZS 3604:2011 Timber Framed Buildings
- NZS 4230:1990 Design of Reinforced Concrete Masonry Structures

7. Results and Discussions

7.1. Critical Structural Weaknesses

This building has no critical structural weaknesses

7.2. Analysis Results

The equivalent static force method was used to analyse the seismic capacity of the building, based on the method prescribed by NZS3604:2011. The results of the analysis are reported in the following table as %NBS. The results below are calculated for the building in its damaged state. The building results have been broken down into their seismic resisting elements.

(%NBS = the reliable strength / new building standards)

- **Table 4: DEE Results**

Building	Seismic Resisting Element	Action	Seismic Rating %NBS
Linwood Resource Centre, Building 14	Gib lined timber framed walls (along direction overall)	Shear/racking	68%
	Gib lined timber framed walls (across direction overall)	Shear/racking	74%
	Sub floor structure cantilevered (both directions)	Shear in fixings	100%

7.3. Recommendations

The quantitative assessment carried out on BU 0822-014 EQ2 Linwood Resource Centre, Building 14 has a seismic capacity of 68%NBS. As the building is found to be above 34% it is not earthquake prone. No further action is required.

8. Conclusion

SKM carried out a quantitative assessment on building BU 0822-014 EQ2 located at 322 Linwood Avenue, Linwood. This assessment concluded that the building is not likely to be earthquake prone.

Table 5: Quantitative assessment summary

Description	Grade	Risk	%NBS	Structural performance
Linwood Resource Centre	B	Low	68	Acceptable, Improvement may be desirable.

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.

9. 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.

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.

10. Appendix 1 – Photos

PHOTO 1: North Elevation of Building 14

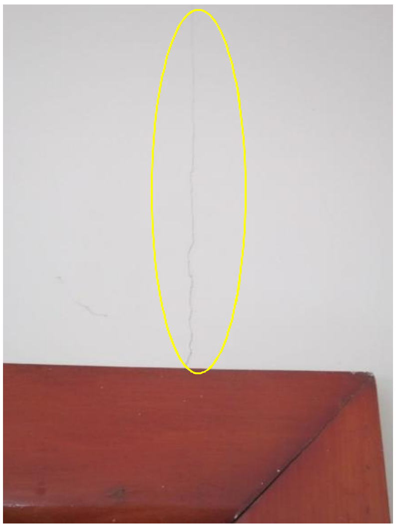
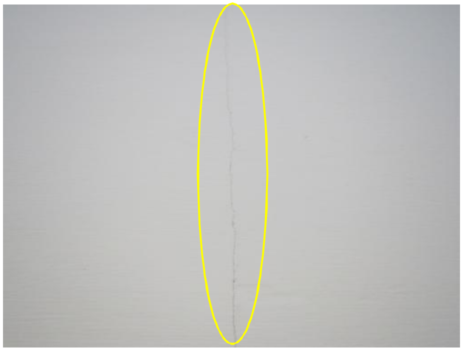
PHOTO 2: Hairline Cracking at Soffit Lining Joints

PHOTO 3: Hairline Cracking at Soffit Lining Joints

PHOTO 4: Close up of Photo 3

PHOTO 5: Gap between Eastern Wall at the Rear Entrance and the Main Structure

PHOTO 6: Gap between Eastern Wall at the Rear Entrance and the Main Structure

	
PHOTO 7: Northern Meeting Room – South Wall	PHOTO 8: Hairline Cracking in Wall Lining Joint above Door
	
PHOTO 9: Hairline Cracking in Wall Lining Joint above Window	PHOTO 10: Close up of Photo 9

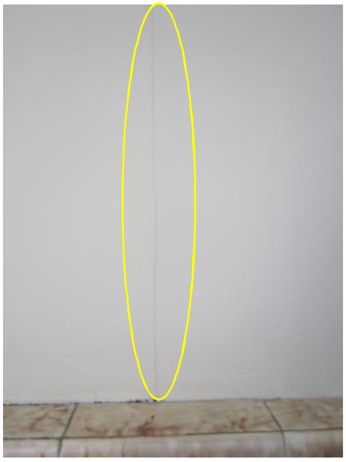
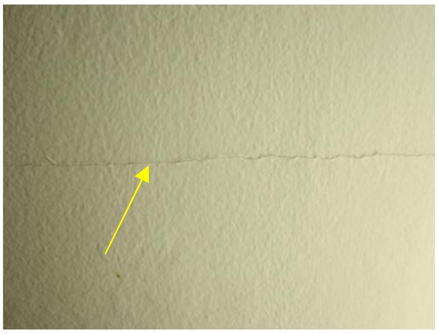
	
PHOTO 11: Hairline Cracking in Wall Lining Joint above Fireplace	PHOTO 12: Close up of Photo 11
	
PHOTO 13: North-East Office (1)	PHOTO 14: L1 - Hairline Crack in Ceiling Lining Joint – Near Centre of Room
	
PHOTO 15: North East Office (2)	PHOTO 16: Hairline Cracking between Wall Lining and window Architrave

	
PHOTO 17: Toilet	PHOTO 18: Hairline Cracking around Wall Trim
	
PHOTO 19: Hairline Crack in Wall Lining Joint above the Window	PHOTO 20: Wash Area

	
PHOTO 21: Hairline Crack in Wall Lining Joint above Window & Hairline Cracking along Scotia	PHOTO 22: Hairline Cracking along Timber Wall Trim

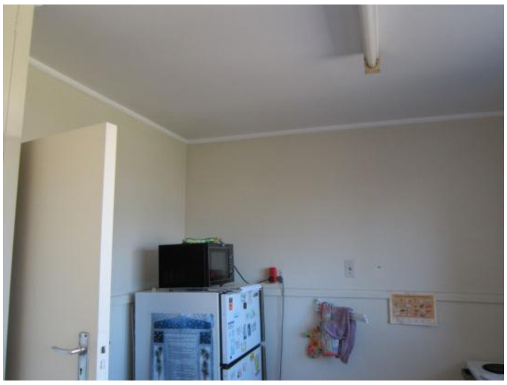



PHOTO 23: Kitchen	PHOTO 24: Hairline Crack at wall Lining Joint above Door

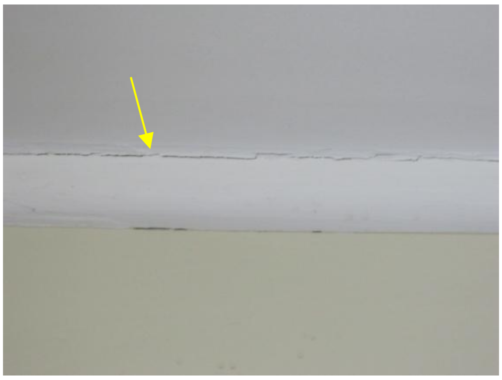


PHOTO 25: Hairline Crack along Scotia	PHOTO 26: Hairline Cracking along Timber Wall Trim

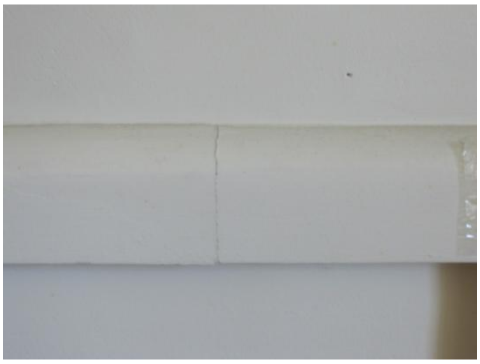



PHOTO 27: Close up of Photo 26	PHOTO 28: South-West Lounge (2)

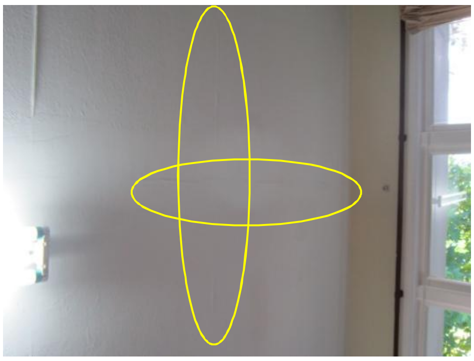



PHOTO 29: Hairline Cracking at Ceiling Lining Joints	PHOTO 30: North- West Office

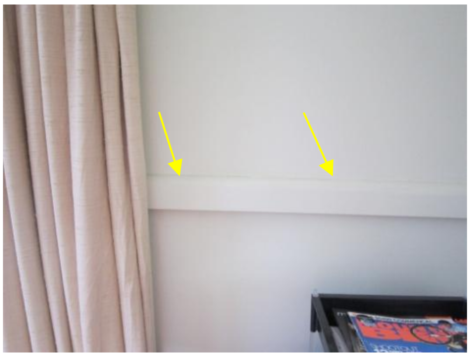
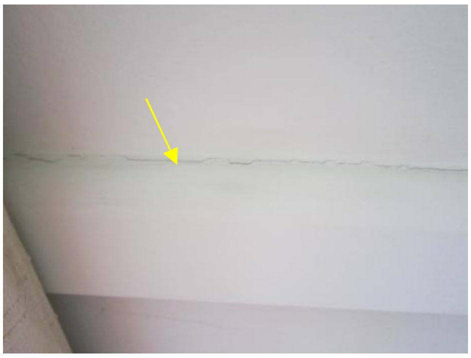
	

PHOTO 31: Hairline Cracking along Timber Wall Trim	PHOTO 32: Close up of Photo 31

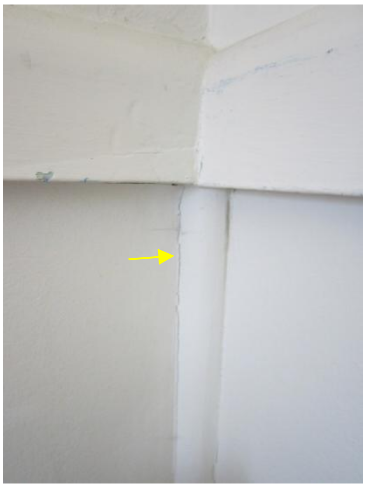



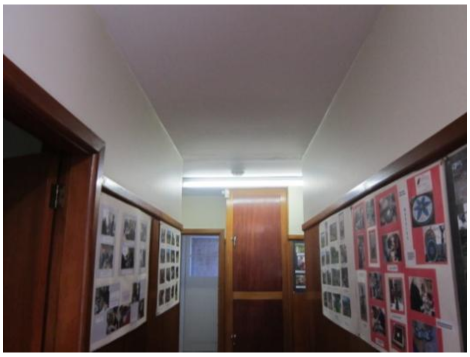
PHOTO 33: Hairline Cracking in NE Corner	PHOTO 34: North-west Office (3)
	
PHOTO 35: Hairline Crack in Wall Lining Joint above Window	PHOTO 36: Front Entrance / Hallway

	
PHOTO 37: Hairline Cracking in Ceiling Lining	PHOTO 38: Hairline Cracking around Ceiling Lining and Between Wall and Ceiling Lining Joint

11. Appendix 2 - Quantitative Calculations

SKM

Job No. ZB01276.004

Calc. Series BLG 14

Client Christchurch City Council

Page

Job Name PU - 0822 - 014

By NLC

Calcs Title Linwood Resource Centre

Date 13-6-13

Background

SKM completed a quantitative DEE for this building which a final report was issued on 27-09-12, the %NBS was found to be 10%.
During this time a peer review of our DEE was completed, and commented on our conservative approach

Scope

Update calcs, in response to some of the peer reviews comments

Summary

Walls	- Along	68% NBS
	- Across	74% NBS
Subfloor	- Along	100% NBS
	- Across	100% NBS

Contents

- Calcs
- Bracing Schedule
- NZS3604 information
- CHD peer review

* Taking tongue and groove floor as diaphragm. this will engage all the lined walls (not just the ones beneath the piles)

Demand (ground floor) = 2080 BU

Capacity, Along = 1413 BU %NBS = 68% NBS
Across = 1544 BU %NBS = 74% NBS

ground floor
Along = 68% NBS
Across = 74% NBS

* Subfloor

- Due to diaphragm from T & G flooring we are able to engage all the internal piles

- We are also able to engage all the perimeter walls, and will use a different capacity for these walls

Subfloor capacity, Along = [piles] + [perimeter foundation]

[piles] = 56 × 0.38 kN/pile
= 425 BUs (from previous DEE calcs)

- have assumed the perimeter foundation is as per the dimensions in NZS3604

[perimeter foundation] (from table 5.11 NZS3604)
= 8.150 × 300 BUs +
13.780 × 300 BUs
= 6579 BUs

- demand = 3130 BUs as per DEE calcs

= 425 + 6579
= 7004 BUs ≥ 3130 BUs

subfloor
Along = 100% NBS

Across = [piles] + [perimeter foundation]

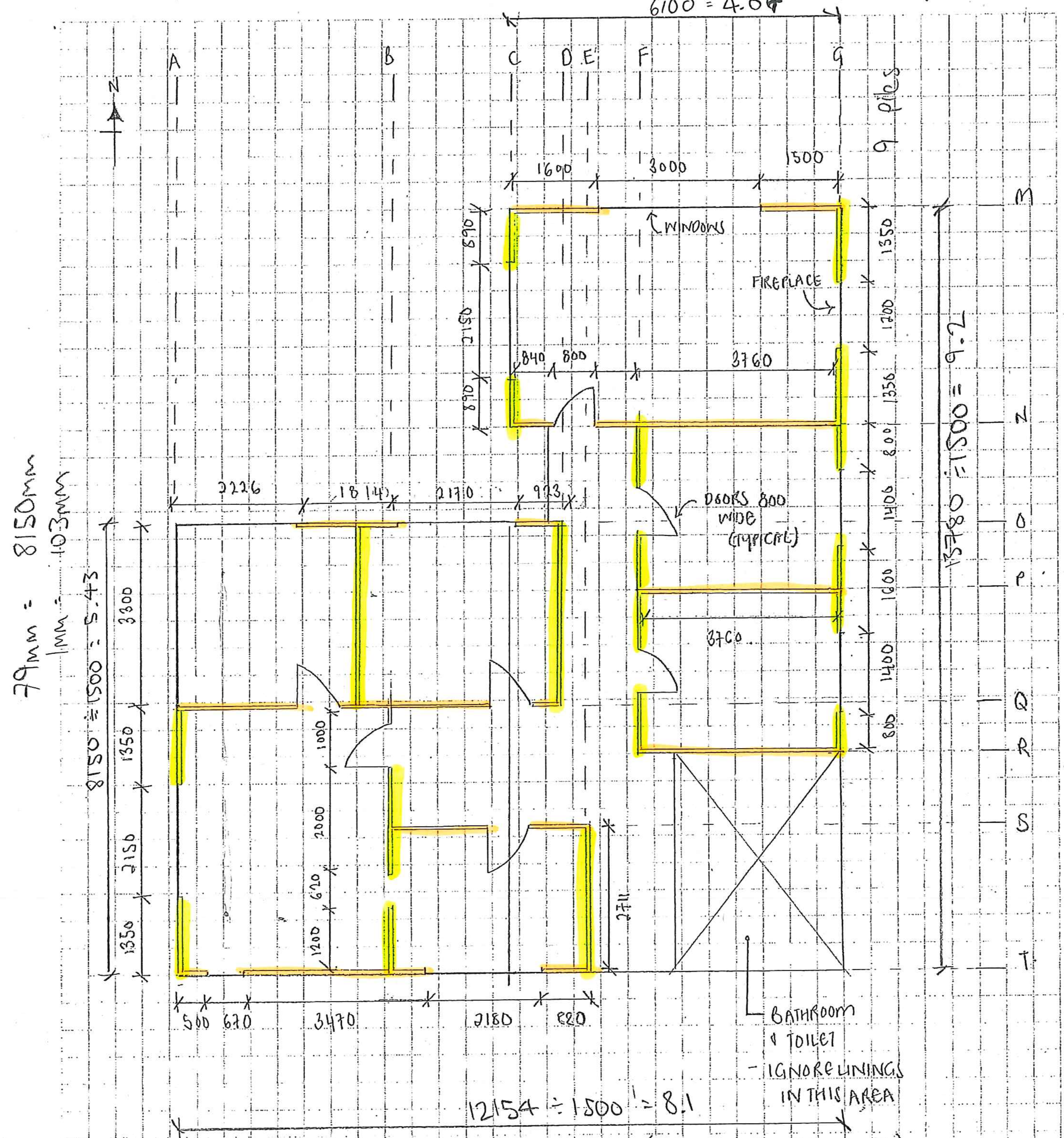
[perimeter foundation]
= 6.1 × 300 BUs +
12.15 × 300 BUs
= 5475 BUs

= 425 + 5475
= 5900 BUs ≥ 3130 BUs

Across = 100% NBS

SKM

Client: CCC - PANEL	Job No.: ZB01276-04
Job Name: BU-0822-014	Calc. Series: BLD 14
Calcs Title: LINWOOD RESOURCE CENTRE - PLAN	Page: 01
	By: KW
	Date: 06/04/12.



FLOOR PLAN OF BLD 14 (LINWOOD RESOURCE CENTRE)

+ STUD HEIGHT = 2.4m
+ Concrete PILES SPACED AT 1500 CRS (MIN).
+ FOOTPRINT AREA = 145m²

GIB EzyBrace® 2011 Software

GIB

SINGLE OR UPPER STOREY WALLS ALONG									V06/11
Lines		*Bracing Elements*							
1	2	3	4	5	6	7	8	9	10
Line Total Check	Line Label	Bracing Element No.	Available Wall Length L (m)	Angle to Bracing line (degrees)	Element Height H (m)	Bracing Type	Supplier	Bracing Units Achieved	
								W	E
-41	a	1	1.35		2.4	GS1*0.7	NZSEE	57	57
		2	1.35		2.4	GS1*0.7	NZSEE	57	57
402	b	1	3.3		2.4	GS2*0.7	NZSEE	198	198
		2	2.2		2.4	GS2*0.7	NZSEE	132	132
		3	1.2		2.4	GS2*0.7	NZSEE	72	72
-80	c	1	1.78		2.4	GS1*0.7	NZSEE	75	75
198	d	1	3.3		2.4	GS2*0.7	NZSEE	198	198
-41	e	1	2.7		2.4	GS1*0.7	NZSEE	113	113
264	f	1	1.1		2.4	GS2*0.7	NZSEE	66	66
		2	1.1		2.4	GS2*0.7	NZSEE	66	66
		3	1.1		2.4	GS2*0.7	NZSEE	66	66
		4	1.1		2.4	GS2*0.7	NZSEE	66	66
248	g	1	1.35		2.4	GS1*0.7	NZSEE	57	57
		2	2.15		2.4	GS1*0.7	NZSEE	90	90
		3	1.6		2.4	GS1*0.7	NZSEE	67	67
		4	0.8		2.4	GS1*0.7	NZSEE	34	34

					Wind	Earthq.
Totals Achieved	W	523%	EQ		1413	1413
Timber Floor, design limit of 120 BU/m			***accepted***		*OK*	*not enough*
Totals Required (from Demand)					270	2080

GIB EzyBrace® 2011 Software

SINGLE OR UPPER STOREY WALLS ACROSS

V06/11

Lines		*Bracing Elements*							
1	2	3	4	5	6	7	8	9	10
Line Total Check	Line Label	Bracing Element No.	Available Wall Length L (m)	Angle to Bracing line (degrees)	Element Height H (m)	Bracing Type	Supplier	Bracing Units Achieved W	Bracing Units Achieved E
-5	m	1	1.5		2.4	GS1*0.7	NZSEE	63	63
		2	1.6		2.4	GS1*0.7	NZSEE	67	67
261	n	1	3.76		2.4	GS2*0.7	NZSEE	226	226
		2	0.84		2.4	GS1*0.7	NZSEE	35	35
-22	o	1	0.9		2.4	GS1*0.7	NZSEE	38	38
		2	1.8		2.4	GS1*0.7	NZSEE	76	76
226	p	1	3.76		2.4	GS2*0.7	NZSEE	226	226
330	q	1	5.5		2.4	GS2*0.7	NZSEE	330	330
158	r	1	3.76		2.4	GS1*0.7	NZSEE	158	158
-14	s	1	2.9		2.4	GS1*0.7	NZSEE	122	122
204	t	1	0.88		2.4	GS1*0.7	NZSEE	37	37
		2	3.47		2.4	GS1*0.7	NZSEE	146	146
		3	0.5		2.4	GS1*0.7	NZSEE	21	21

					Wind	Earthq.
Totals Achieved	W	572%	EQ		1544	1544
Timber Floor, design limit of 120 BU/m			***accepted***		*OK*	*not enough*
Totals Required (from Demand)					270	2080

5.5.3.2 Two-storey buildings with timber ground floors

Two-*storey* buildings with timber ground floors shall be as follows:

(a) In all wind and earthquake zones, buildings with a height (measured from the underside of the *bottom plate* of the lowest floor to the top of the *roof*) exceeding 1.7 times the width shall be attached to a continuous *foundation wall* around the entire perimeter. The continuous *foundation wall* may be stepped, provided *wall framing* is constructed to directly support the building to the *foundation wall*.

(b) In all wind and earthquake zones, buildings with a height not exceeding 1.7 times the width may be supported by *bracing* systems complying with 5.4.3.

C5.5.3.2

(a) This clause is aimed at providing a substantial mass in the foundations to resist overturning of slender structures subjected to wind and earthquake forces.

(b) It may be necessary in some wind and earthquake zones to use foundation walls to reduce the number of braced piles.

5.5.4 Subfloor bracing element bracing capacity values

Subfloor bracing shall be rated for wind and earthquake *capacity* as set out in table 5.11.

Table 5.11 – Bracing capacity ratings of subfloor bracing elements (see 5.5.4)

Description of bracing element	Bracing capacity in the horizontal direction for earthquake and wind resistance
Reinforced concrete or reinforced masonry walls (greater than 1.5 m in length) If ratio of average wall height / Wall is: Less than 0.75 More than 0.75 but less than 1.5 More than 1.5 but less than 3.0 More than 3.0 but less than 4.5 More than 4.5	(BUs) 0 42 BUs per metre of wall 100 BUs per metre of wall 200 BUs per metre of wall 300 BUs per metre of wall
Exterior grade product test P21 Test (see 6.2.3)	As determined by test
Braced pile system (consisting of two piles and a diagonal brace)	120 BUs for earthquake 160 BUs for wind
Cantilever piles, (driven timber piles) rating per pile	30 BUs for earthquake 70 BUs for wind
Anchor piles rating per pile	120 BUs for earthquake 160 BUs for wind
NOTE – Reinforced concrete and reinforced masonry bracing capacities are based on the limitations of fixings between the timber structure and the concrete component. Masonry bracing capacity values from NZS 4229 cannot be used with this Standard unless SED is applied to the connections between the timber structure and the masonry wall.	

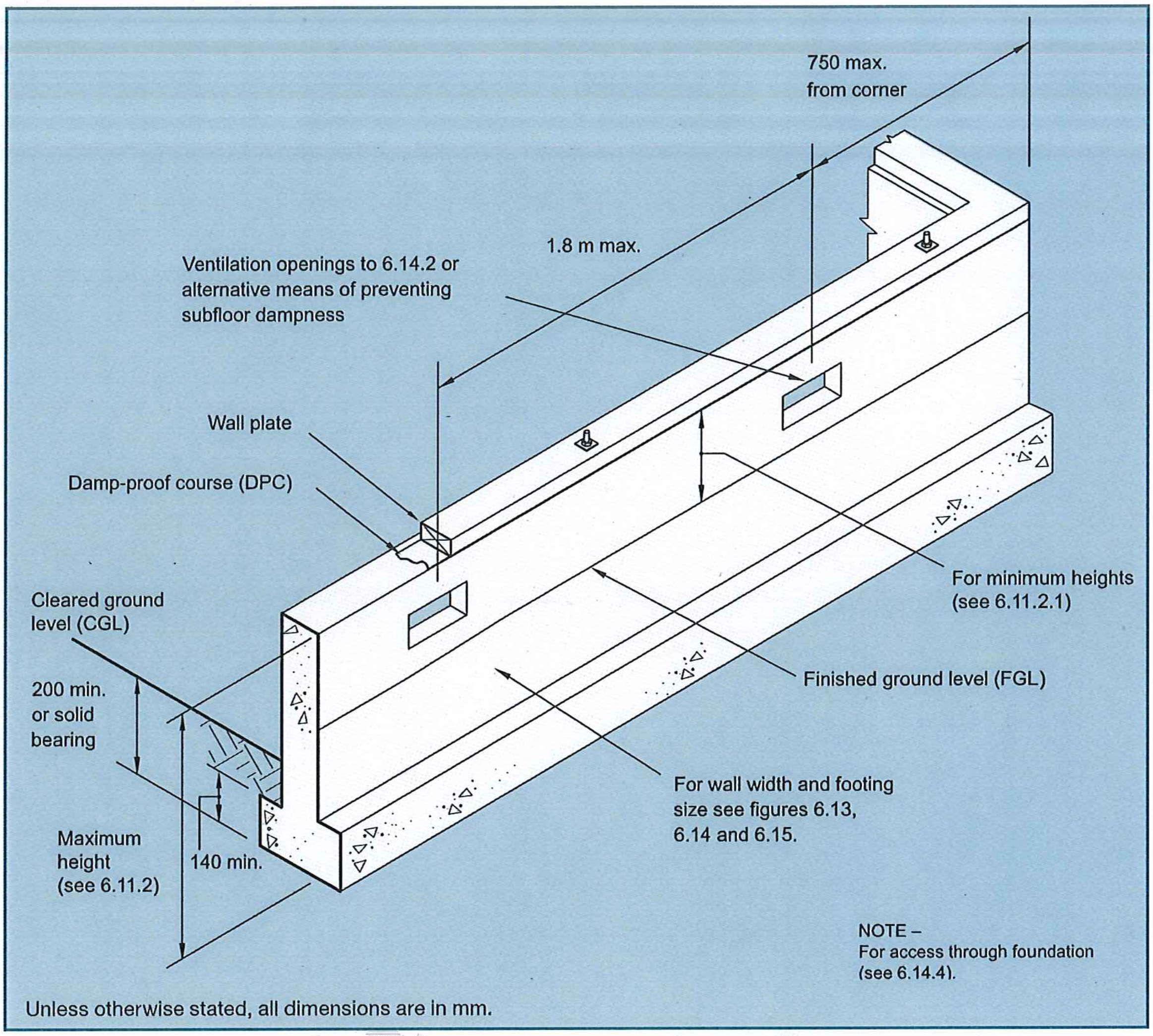


Figure 6.11 – Foundation walls (see 6.11.1 and 6.11.2)

6.11.3 Width of foundation walls

The width of a *foundation wall* shall be not less than shown in:

(a) Figure 6.13 for *cantilevered foundation walls*;

(b) Figure 6.14 for single-*storey foundation walls*; and

(c) Figure 6.15 for two-*storey foundation walls*.

Where the sides of a *foundation wall* are cast against earth, the thickness shall be increased so that there is a minimum cover of 75 mm to the *reinforcement*.

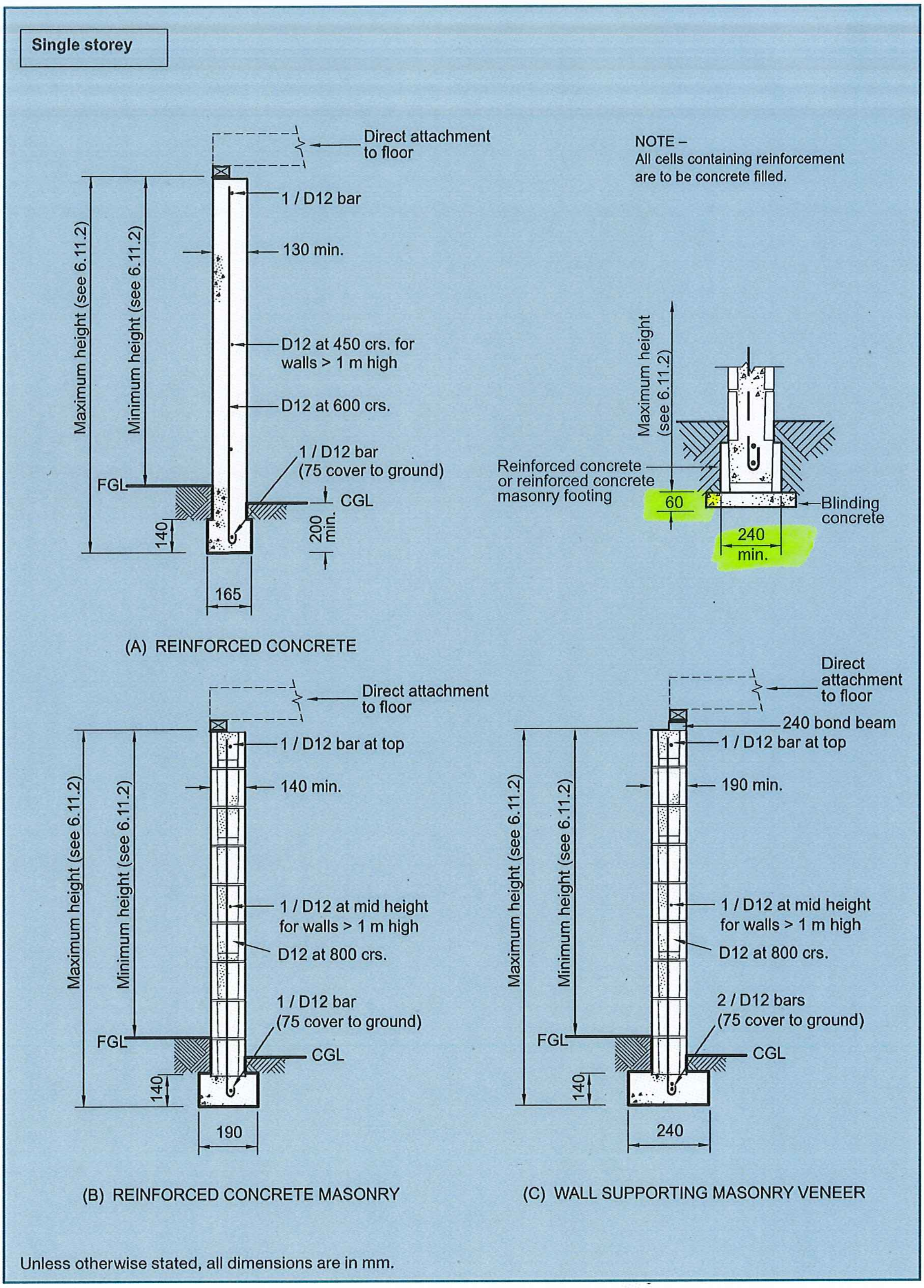


Figure 6.14 – Foundation walls (not cantilevered) for single-storey buildings (see 6.11.3, 6.11.4.2, and 6.11.7.1)

12. Appendix 3 – CERA Standardised Report Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Linwood Resource Centre	Reviewer:	Nick Calvert
Building Address:	Unit: / No: 180 / Street: Smith Street	CPEng No:	242062
Legal Description:		Company:	SKM
		Company project number:	ZB01276.004
		Company phone number:	09 940 4900
GPS south: (Degrees / Min / Sec)		Date of submission:	1-Jul
GPS east:		Inspection Date:	26/03/2012
		Revision:	C
Building Unique Identifier (CCC):	BU 0822-014 EQ2	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):	0.00
Ground floor split?	no	Ground floor elevation above ground (m):	0.00
Storeys below ground	0		
Foundation type:	driven precast piles	if Foundation type is other, describe:	
Building height (m):	4.00	height from ground to level of uppermost seismic mass (for IEP only) (m):	0.00
Floor footprint area (approx):	145		
Age of Building (years):	42	Date of design:	1965-1976
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	other (specify)	Brief strengthening description:	
Use (upper floors):	other (specify)		
Use notes (if required):	Community Centre		
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	frame system		
Roof:	timber truss	truss depth, purlin type and cladding	Rafter 100x45 at 600 crs, purlins 75x45 at 600crs, light weight corrugated steel cladding with timber boards
Floors:	timber	joist depth and spacing (mm)	Joists - approx 400 crs
Beams:	none	overall depth x width (mm x mm)	
Columns:			
Walls:		**Walls are load bearing timber frame**	

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	lightweight timber framed walls		note typical wall length (m)	
Ductility assumed, μ:	1.00			
Period along:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):	30		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):	0		estimate or calculation?	estimated
Lateral system across:	lightweight timber framed walls		note typical wall length (m)	
Ductility assumed, μ:	1.00			
Period across:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):	30		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):	0		estimate or calculation?	estimated

Separations:

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

Non-structural elements

Stairs:			
Wall cladding:	brick or tile	describe (note cavity if exists)	summerhill stone venner
Roof Cladding:	Metal	describe	corrugated steel cladding
Glazing:	timber frames		
Ceilings:	plaster, fixed		
Services(list):	lighting		

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:	slight	notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	damage observed does not deminish the capacity of the structure
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	yes	Describe:	Minor hairline cracking to wall and ceiling linings. Damage to veneer cladding

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

Recommendations

	Level of repair/strengthening required:	none	Describe:	
	Building Consent required:	no	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	
Along	Assessed %NBS before:	68%	%NBS / If IEP not used, please detail assessment methodology:	Gib Ezy Brace 2011, with 0.7 safety factor as recommended by NZSEE 2006 for plaster wall linings
	Assessed %NBS after:	68%		
Across	Assessed %NBS before:	74%	%NBS	
	Assessed %NBS after:	74%		

Official Use only:

Accepted By	
Date:	

13. Appendix 4 – Geotechnical Desk Study

Sinclair Knight Merz
142 Sherborne Street
Saint Albans
PO Box 21011, Edgeware
Christchurch, New Zealand

Tel: +64 3 940 4900
Fax: +64 3 940 4901
Web: www.globalskm.com

Christchurch City Council - Structural Engineering Service

Geotechnical Desk Study

SKM project number	ZB01276
SKM project site number	002 to 005 inclusive
Address	Linwood Resource Centre and Library, 180 Smith Street and 332 Linwood Ave
Report date	26 March 2012
Author	Ross Roberts / Ananth Balachandra
Reviewer	Leah Bateman
Approved for issue	Yes

1. Introduction

This letter 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 assessment of whether the building can be economically repaired, 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.

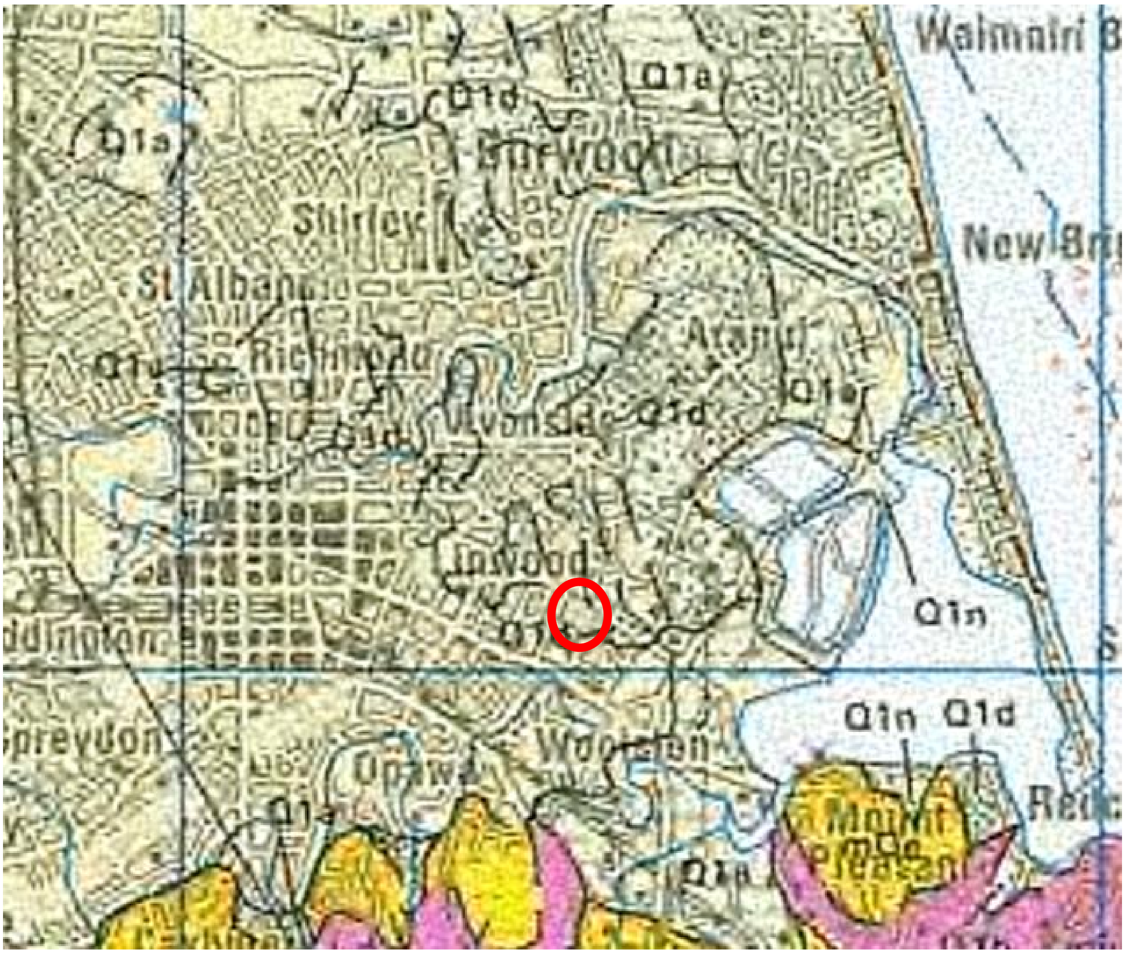
4. Site location

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

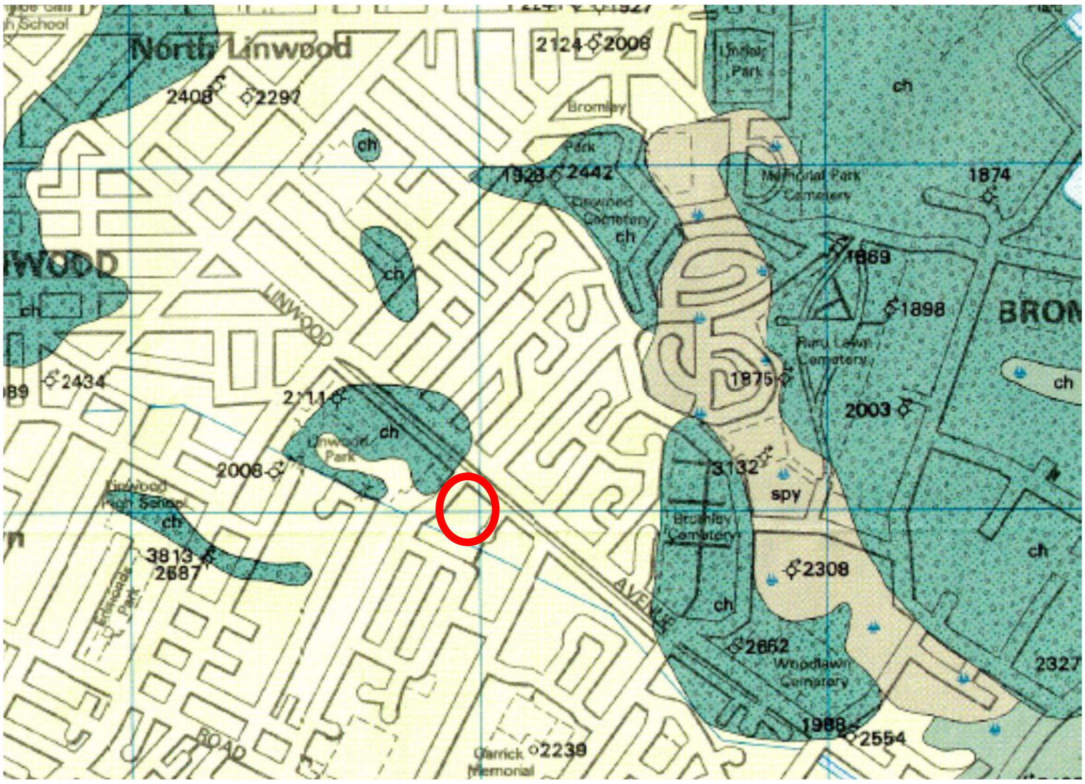
These structures are located on the corner of Linwood Avenue and Smith Street at grid reference 1573957 E, 5179440 N (NZTM).

5. Review of available information

5.1 Geological maps



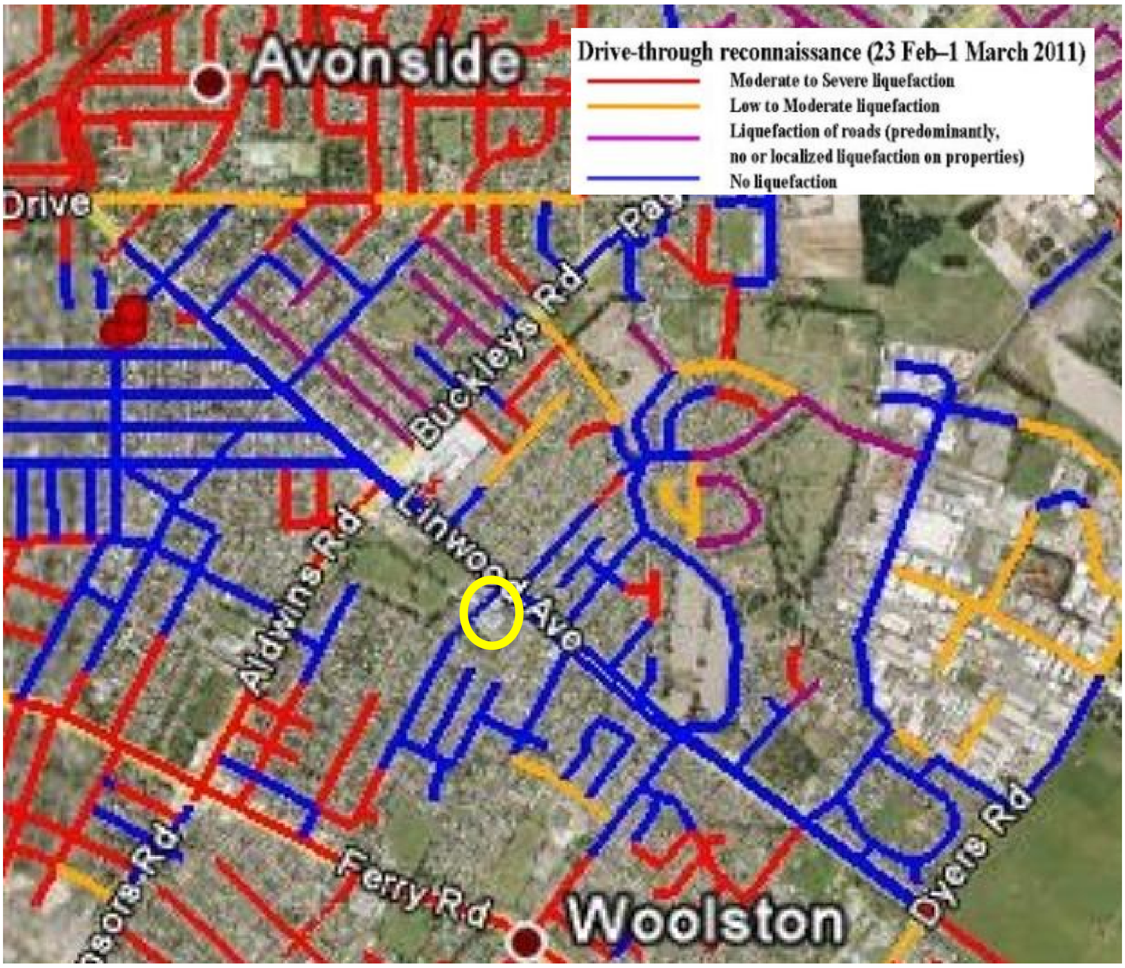
- **Figure 2 – Regional geological map (Forsyth et al, 2008). Site marked in red.**



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

The site is shown to be underlain by Holocene deposits comprising predominantly alluvial sand and silt overbank deposits of the Springston Formation. Immediately to the north west lies an area of Christchurch Formation sand of fixed dunes.

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 Feb until 1 Mar 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/)**

Aerial photography shows relatively little damage after the 22 Feb 2011 event. There appears to be a burst water main on Linwood Avenue, and what may be a single source of liquefied material in the tennis courts to the north west of the property. This coincides with a change of geology as identified on the geological maps.

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 are TC2

5.5 Historical land use

Reference to historical documents (eg Appendix A) shows that the site lies immediately south and east of land that was recorded as 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 records were limited to building consents applied for the demolition of existing garage and shed and reconstruction of a double door garage and other documents relating to the above construction. The Council foundation record identifies top soil or sandy clay to be present to a depth of 0.5 m and medium sand from a depth of 0.5 m to 3.8 m for the site. No ground investigation for depth greater than 3.8 m was found in the council property files. The ground water table was estimated to be between 1.6 m to 3 m. The council record identifies an allowable bearing pressure of 200 kPa for the sand layer with comments stating that the identified allowable bearing pressure is adequate for the proposed buildings.

Drawings for the utility shed showed 500 mm square pad footings at a depth of 1 m. Drawings for the new garage at 180 Smith Street show a 100 mm thick reinforced raft foundation with edge thickening to 150 mm. Piles are shown inconsistently in the record. One drawing identifies 250 mm diameter piles are shown at each corner and at 2 m centres along the edge of the slab (depth not recorded). Another shows 150 mm 'piles' to 400 mm depth at 1.2 m spacing.

The council property files identify possible contamination under the "old work shop area" due to the presence of two tanks containing flammable liquid, which have since been removed.

5.8 Site walkover

An engineer from SKM undertook a site walkover in the week commencing 12 March 2012.

The Linwood Resource Centre and Library were mostly constructed using masonry block with an iron roof, Figure 7 shows the overview of the site. The buildings were both in good condition, with no external evidence of structural damage. The majority of the land on the site was asphalt, which showed no signs of land damage. There was no evidence that liquefaction occurred on the site.

Residents report that the only damage occurred to the footpaths, with paving slabs being displaced,(Figure 8) and that no liquefaction occurred on the site.

- **Figure 7 Overview of Linwood Services Centre and Library**

- **Figure 8 Damaged paving slabs at Linwood Resource Centre**

6. Conclusions and recommendations

6.1 Site geology

An interpretation of the most relevant local investigation suggests that the site is underlain by:

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

Although detailed records of the existing foundations are not available, the performance to date suggests that they are adequate for their current purpose.

6.4 Ground performance and properties

It is expected that the allowable bearing capacity of a shallow pad footing on this site will be in the region of 200 kPa, as stated in the council records and supported by the findings of the nearby ground investigations. We estimate a conservative ultimate bearing capacity to be in the order of 400 kPa. However, these may be revised by a site specific investigation.

For the purposes of shallow foundation design, the following parameters are recommended for the near surface clayey sand:

- Effective angle of friction = 35 degrees
- Apparent cohesion = 1 kPa
- Unit weight = 18 kPa

Liquefaction risk is low at this site.

6.5 Further investigations

Unless a change of use is intended for the site we do not believe that any further geotechnical investigations are required. Specific ground investigation should be undertaken if significant alterations or new structures are proposed. If any excavations are required on the site further investigation of the potential for contamination should be undertaken.

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.

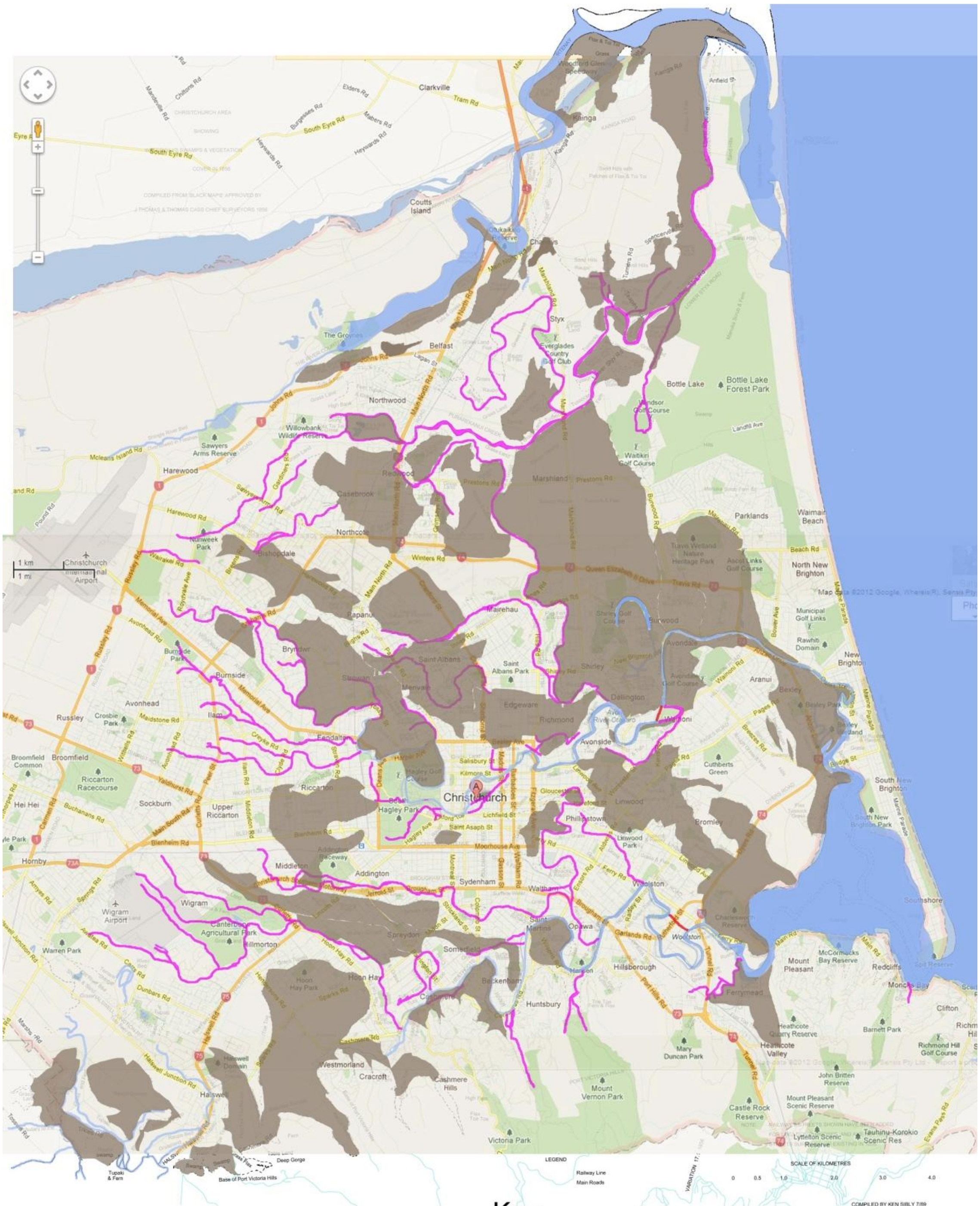
Cubrinovski & Taylor, 2011. Liquefaction map summarising preliminary assessment of liquefaction in urban areas following the 2010 Darfield Earthquake.

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

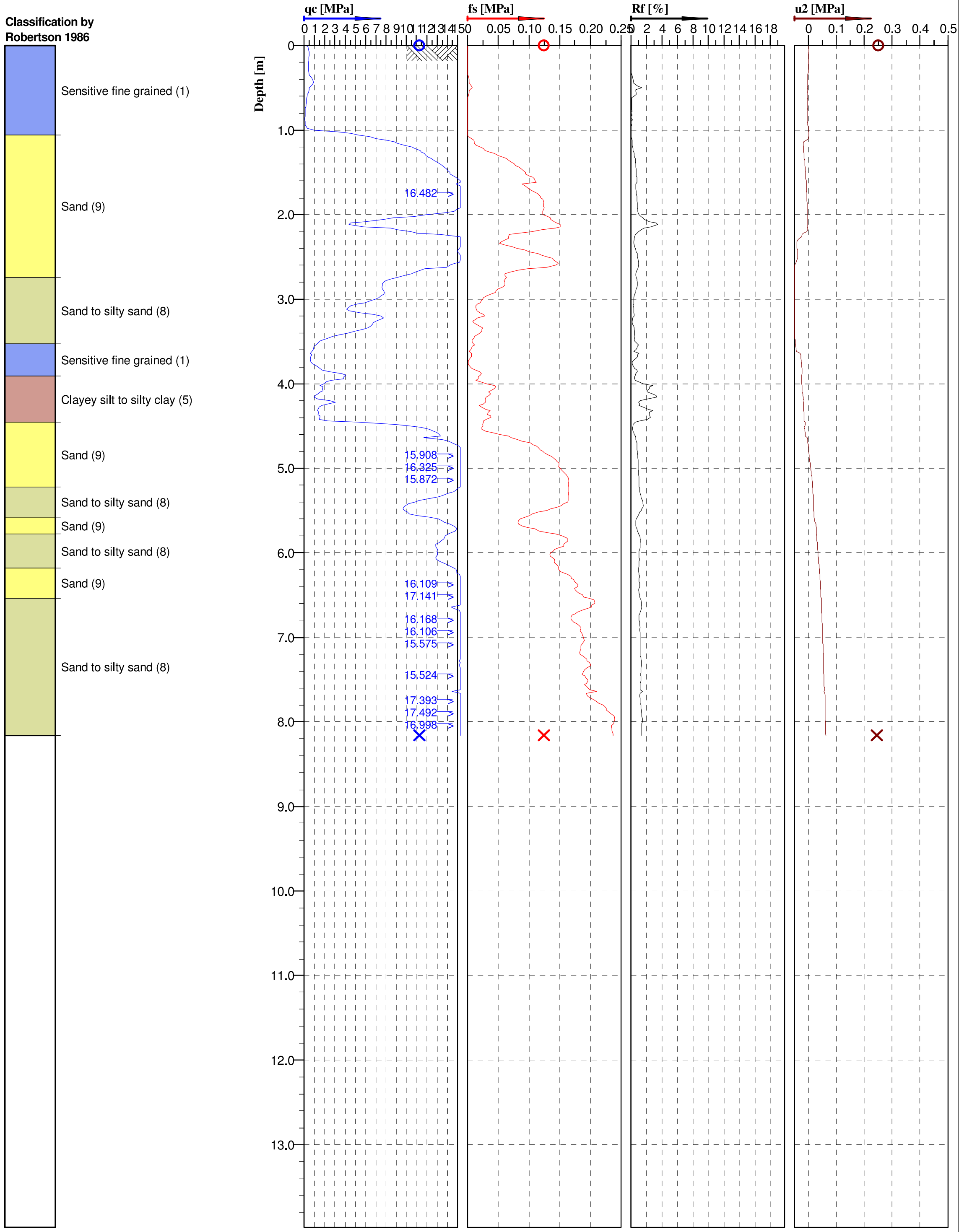


Key

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

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

Appendix B – Existing ground investigation logs

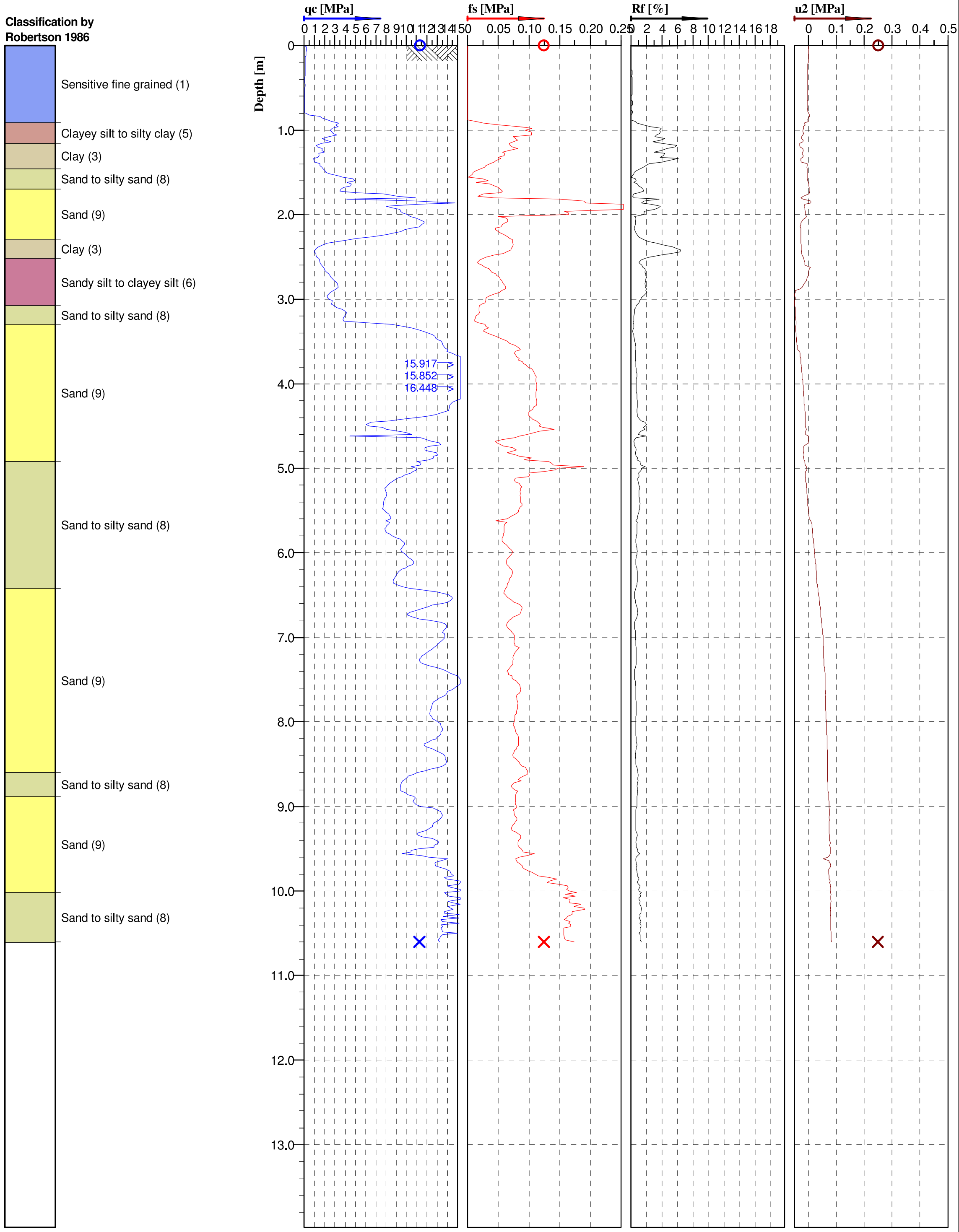


GEOTECH DRILLING

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

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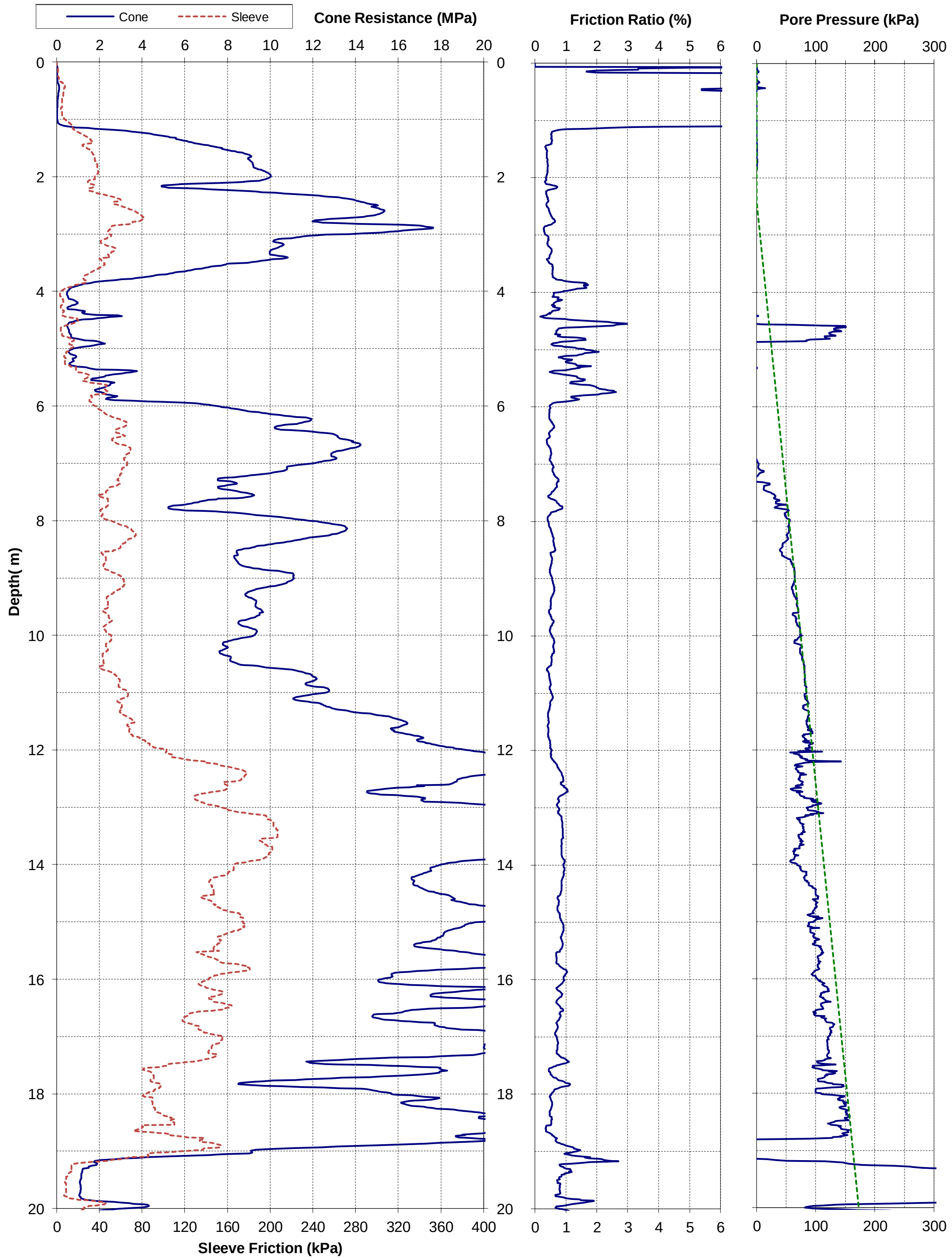
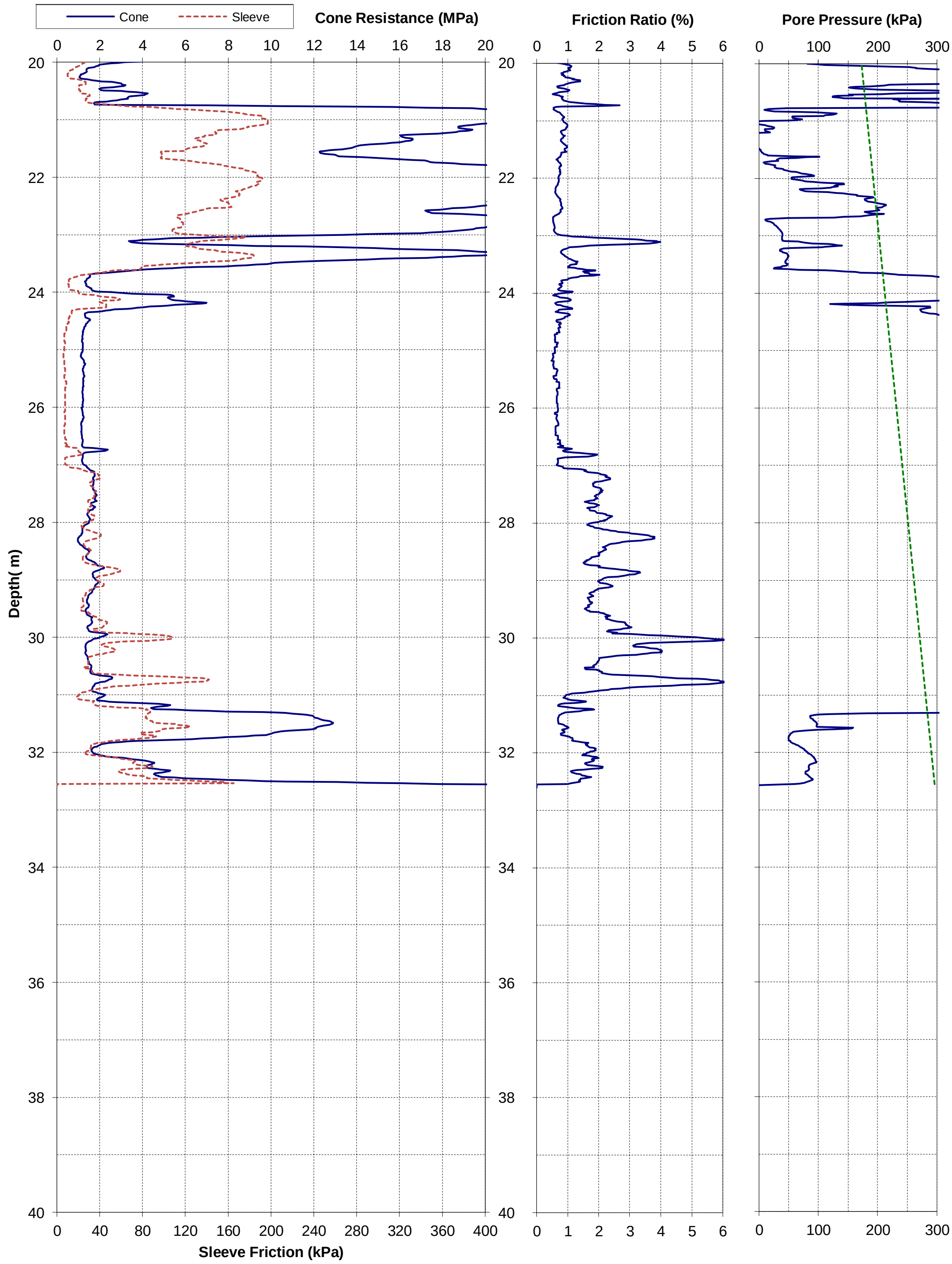
Project ID:	Client: TONKIN & TAYLOR LTD	Date: 20/05/2011	Scale: 1 : 60
Project: EQC SITES		Page: 1/1	Fig:
		File: CPT-LWD-34.CPT	

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:		



T+T Ref: 52000.3000 Printed: 7/10/2011 3:15 p.m. Template: CPT Graph Template v0.41.xls

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:	



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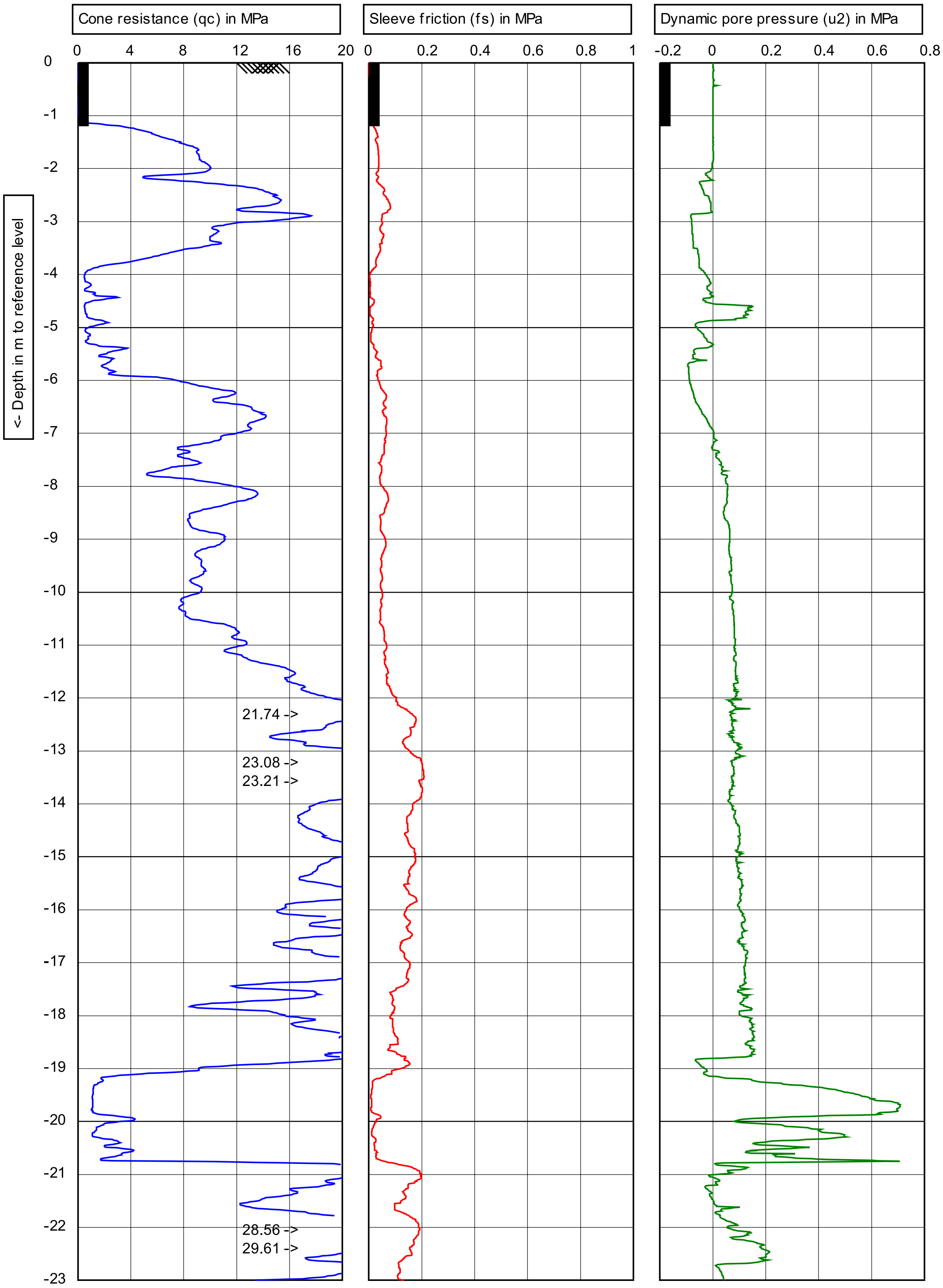

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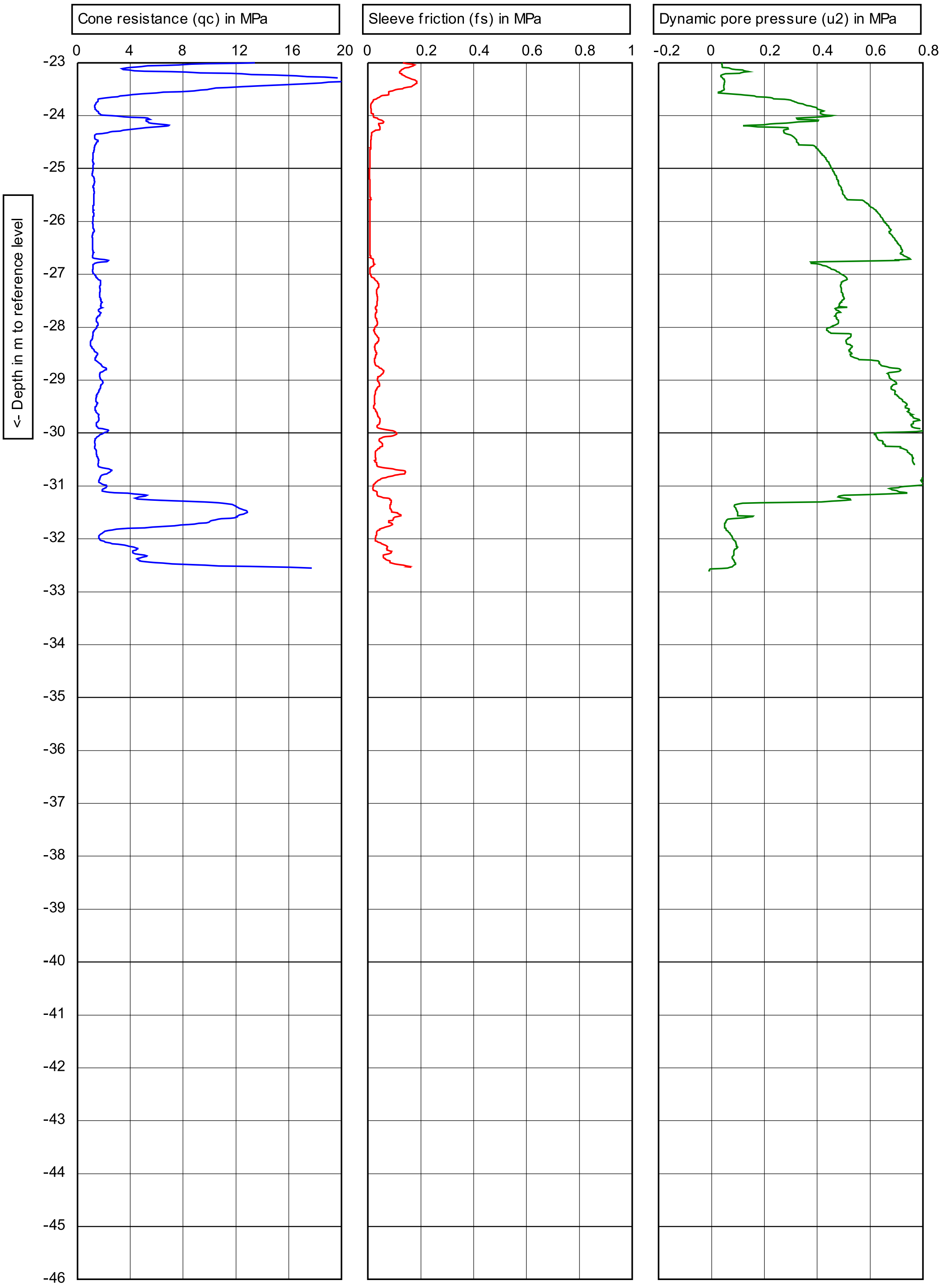
	G.L. 0	W.L.: -60	Date: 17/06/2011
	Project: BRY		Cone no.: C10CFIIP.C10204
	Location: GPS: E2484271 N5741215		Project no.: 2-68292.11
	Position:		CPT no.: CPT-BRY-18 1/12

CPTask V1.20



OPUS
HAMILTON LABORATORIES

u2
150 cm²
10 cm²

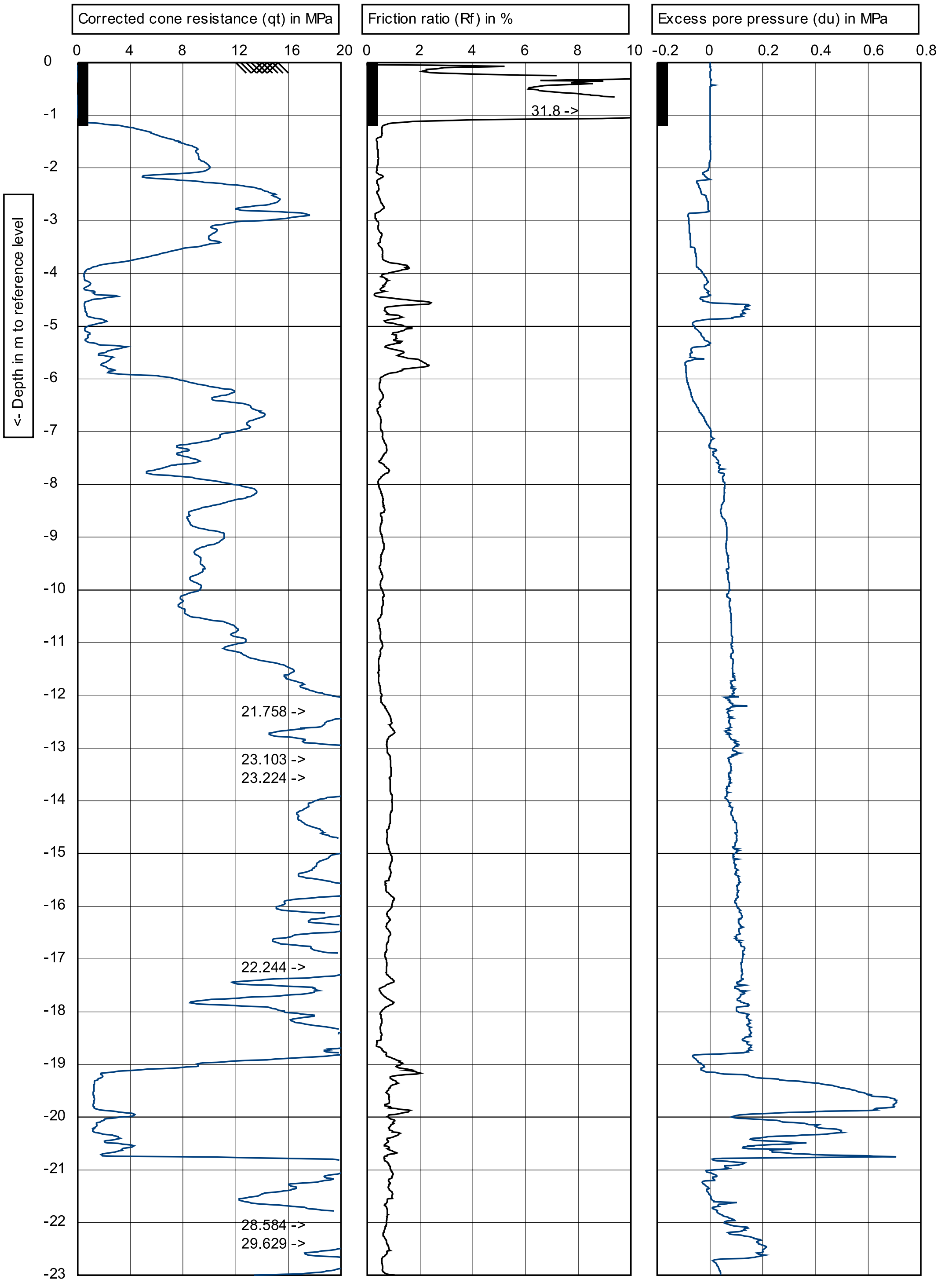
Test according to A.S.T.M standard D-5778-95

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

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

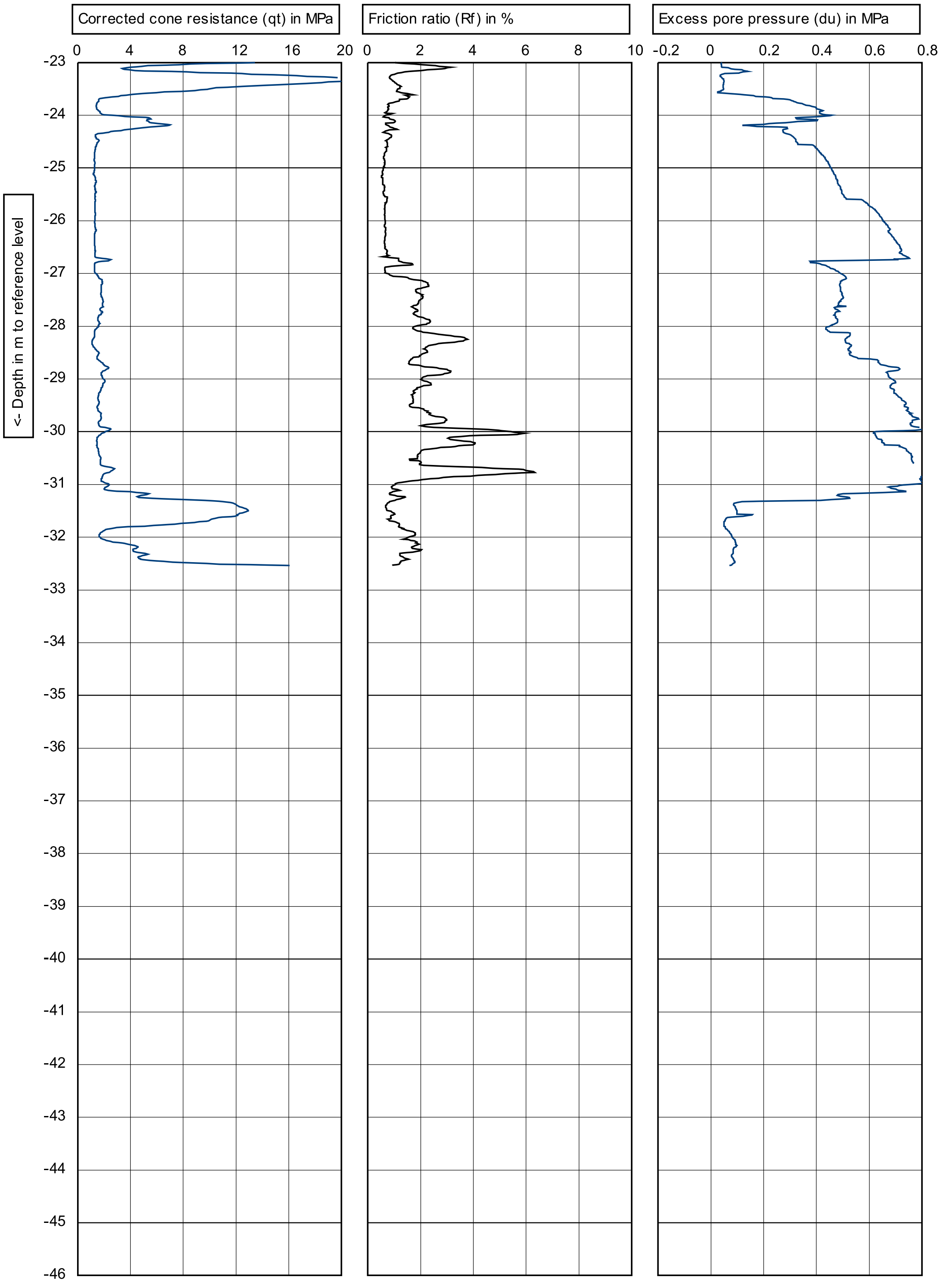
Predrill : **1.2**
Date: **17/06/2011**
Cone no.: **C10CFIIP.C10204**
Project no.: **2-68292.11**
CPT no.: **CPT-BRY-18** **2/12**

CPTask V1.20



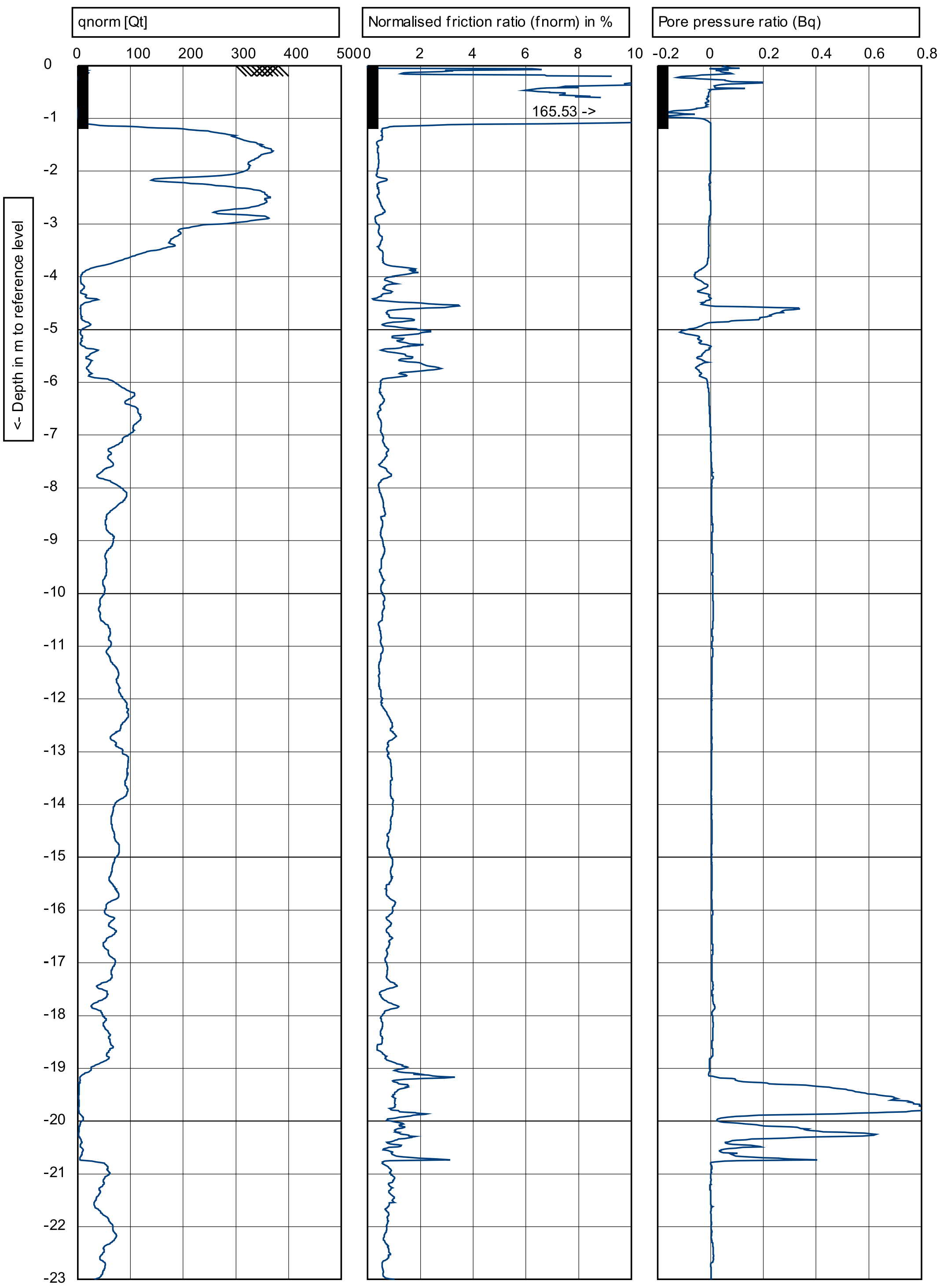
OPUS HAMILTON LABORATORIES	u2 150 cm² 10 cm²	Test according to A.S.T.M standard D-5778-95		Predrill :	1.2
		G.L. **0**	W.L.: **-60**	Date:	**17/06/2011**
	Project:	**BRY**		Cone no.:	**C10CFIIP.C10204**
	Location:	**GPS: E2484271 N5741215**		Project no.:	**2-68292.11**
	Position:			CPT no.:	**CPT-BRY-18** 3/12

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
		Location: GPS: E2484271 N5741215	Project no.: 2-68292.11
		Position:	CPT no.: CPT-BRY-18 / 4/12

CPTask V1.20



OPUS
HAMILTON LABORATORIES

CPTask V1.20

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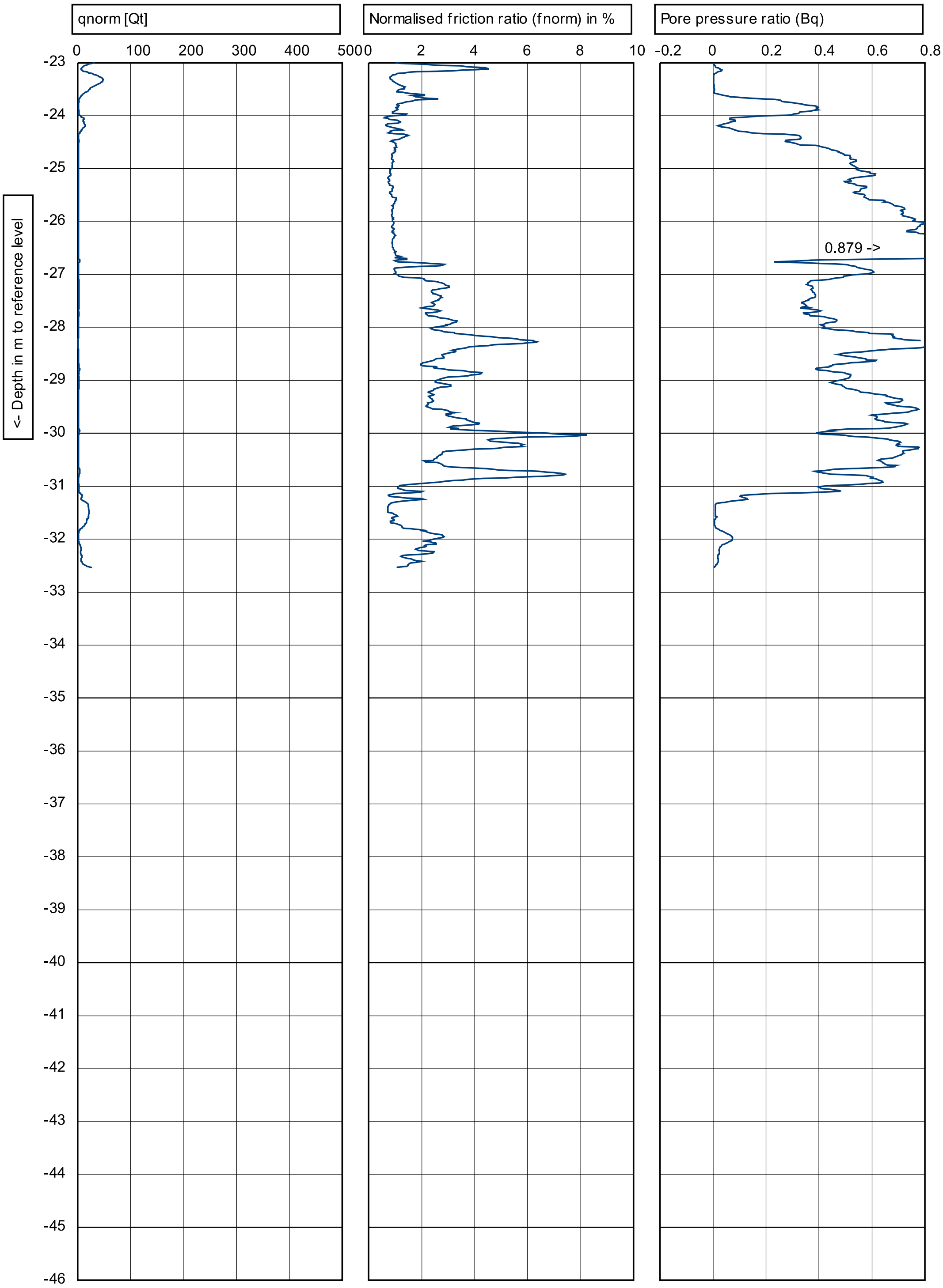
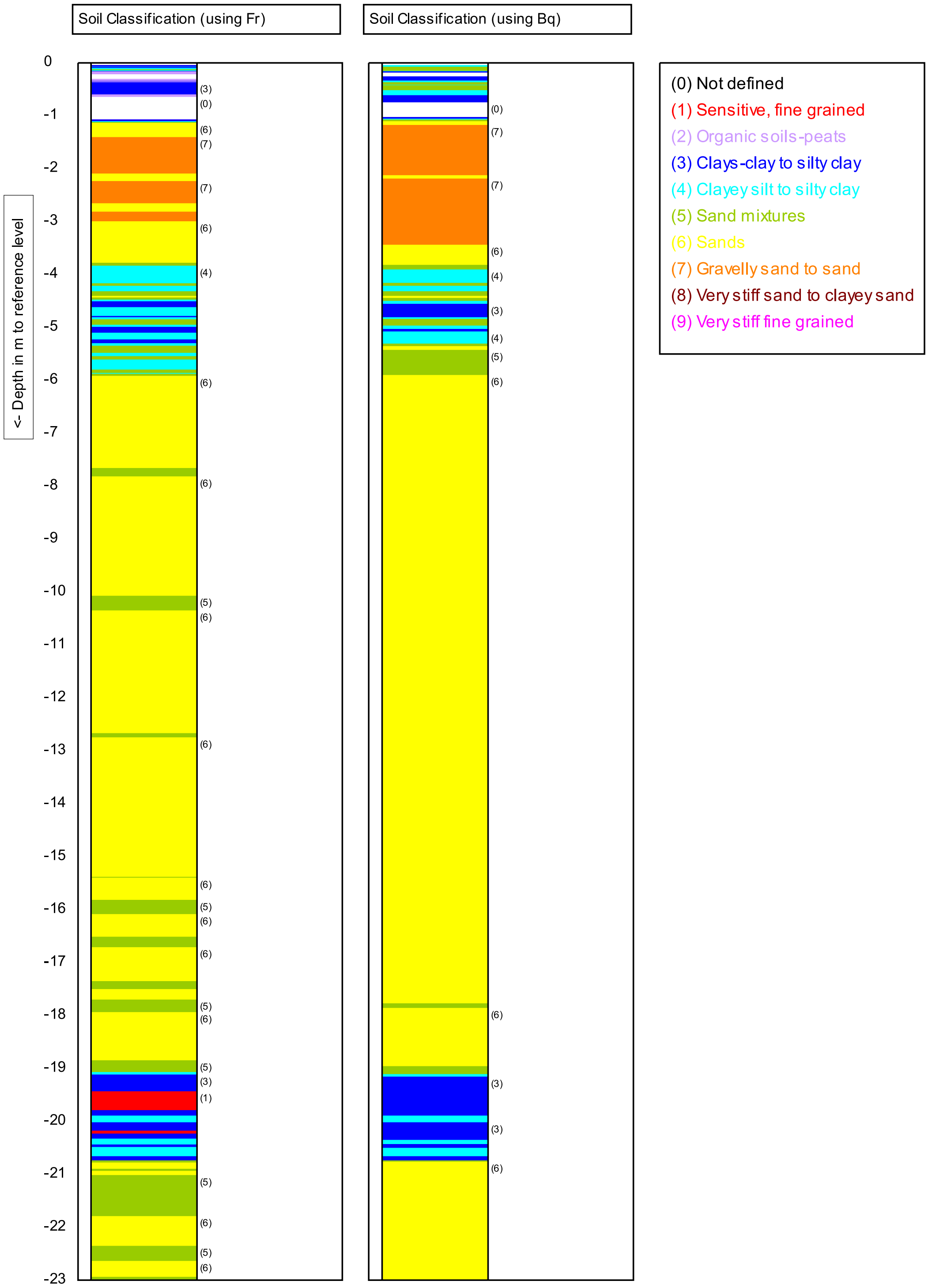
	G.L. **0**	W.L.: **-60**	Date:	**17/06/2011**	
Project: **BRY**			Cone no.:	**C10CFIIP.C10204**	
Location: **GPS: E2484271 N5741215**			Project no.:	**2-68292.11**	
Position:			CPT no.:	**CPT-BRY-18**	**5/12**



OPUS
HAMILTON LABORATORIES

CPTask V1.20

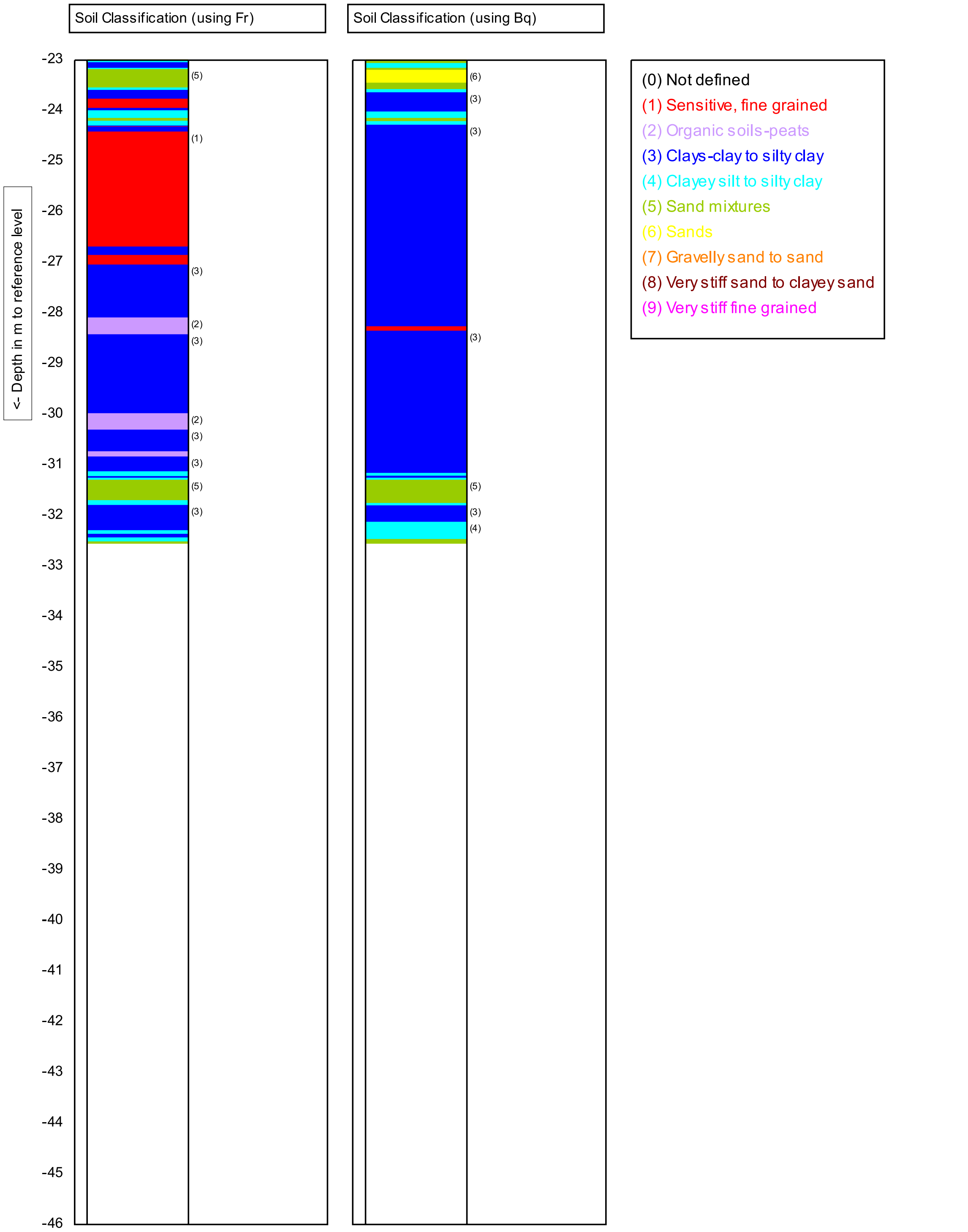
u2 150 cm² 10 cm²	Test according to A.S.T.M standard D-5778-95	Predrill :	1.2
G.L. **0**	W.L.: **-60**	Date:	**17/06/2011**
Project: **BRY**		Cone no.:	**C10CFIIP.C10204**
Location: **GPS: E2484271 N5741215**		Project no.:	**2-68292.11**
Position:		CPT no.:	**CPT-BRY-18** **6/12**



OPUS
HAMILTON LABORATORIES

CPTask V1.20

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 7/12



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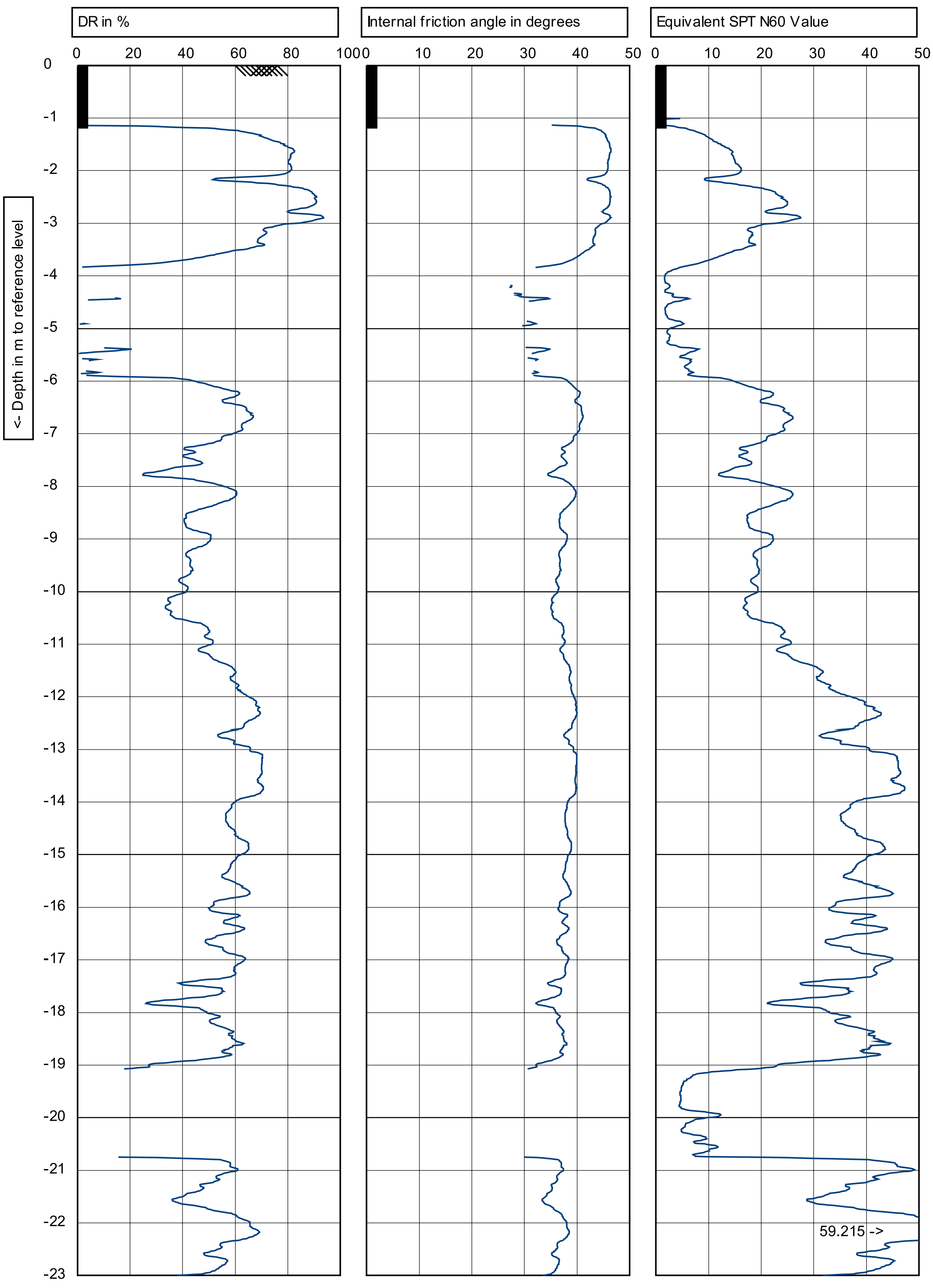
u2 150 cm² 10 cm²	G.L. 0	W.L.: -60	Date: 17/06/2011
	Project: BRY		Cone no.: C10CFIIP.C10204
	Location: GPS: E2484271 N5741215		Project no.: 2-68292.11
	Position:		CPT no.: CPT-BRY-18 8/12

CPTask V1.20



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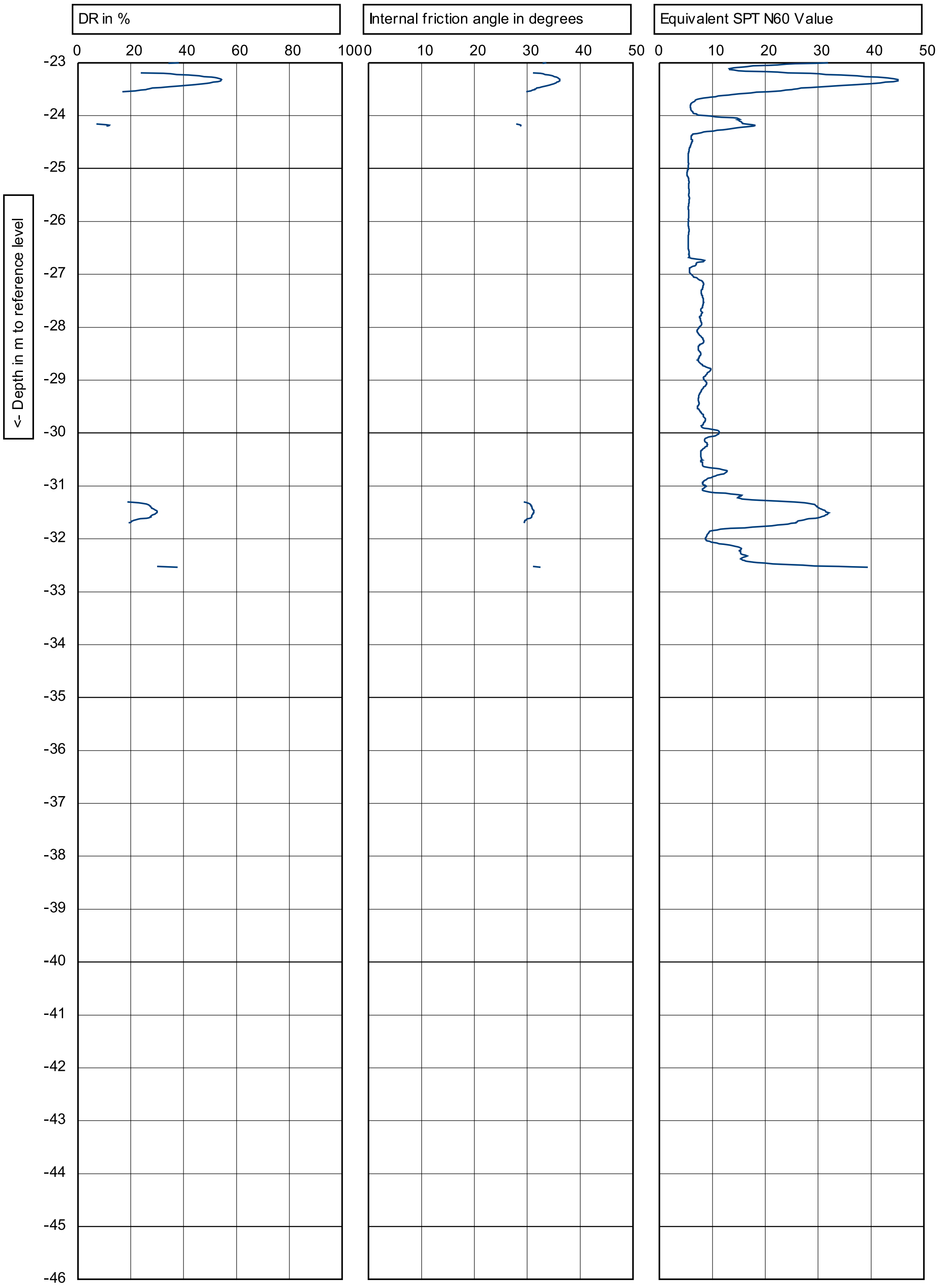
Project: BRY		Cone no.:	C10CFIIP.C10204
Location: GPS: E2484271 N5741215		Project no.:	2-68292.11
Position:		CPT no.:	CPT-BRY-18 9/12

OPUS
HAMILTON LABORATORIES



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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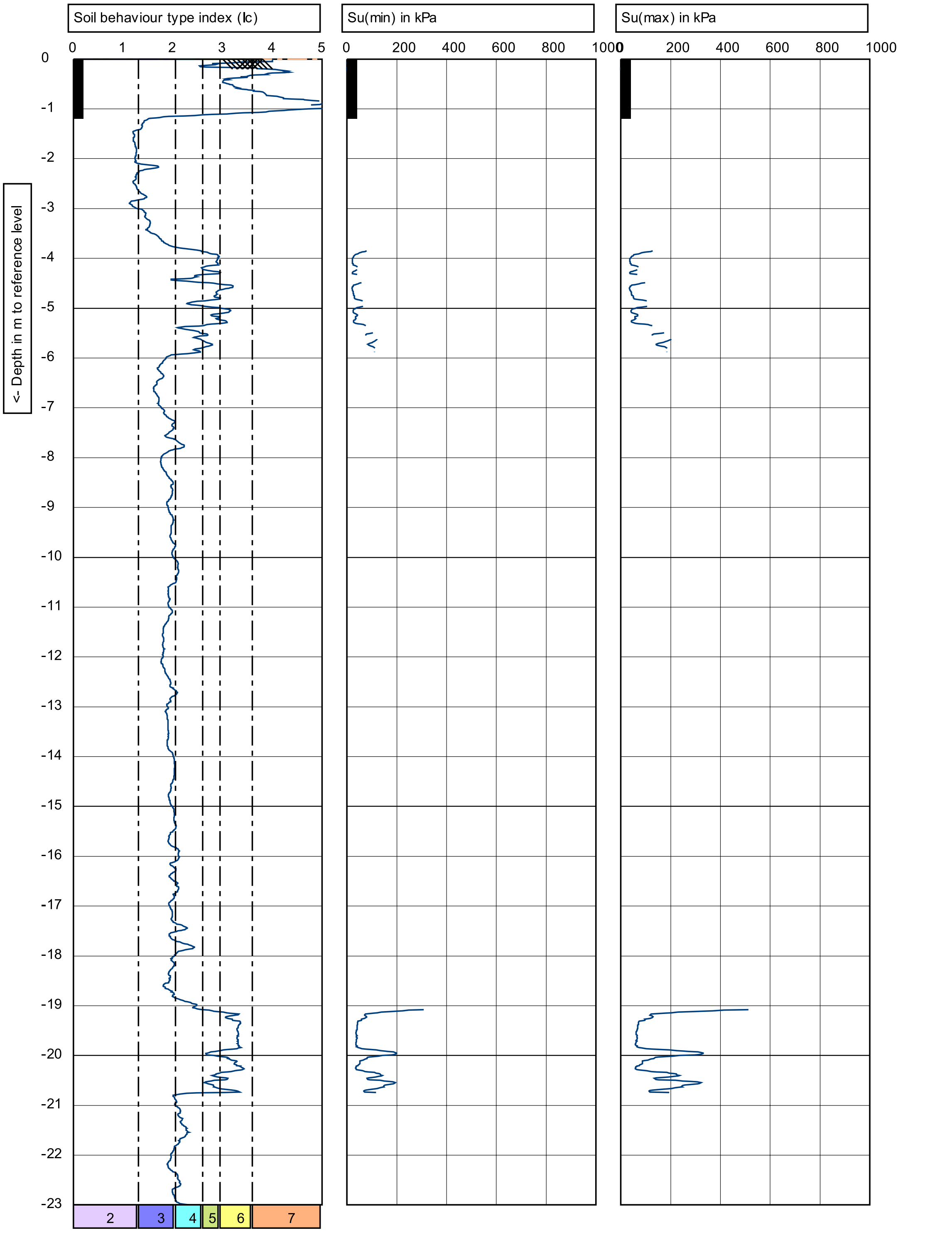
CPTask V1.20	Position:		CPT no.: CPT-BRY-18 10/12



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**
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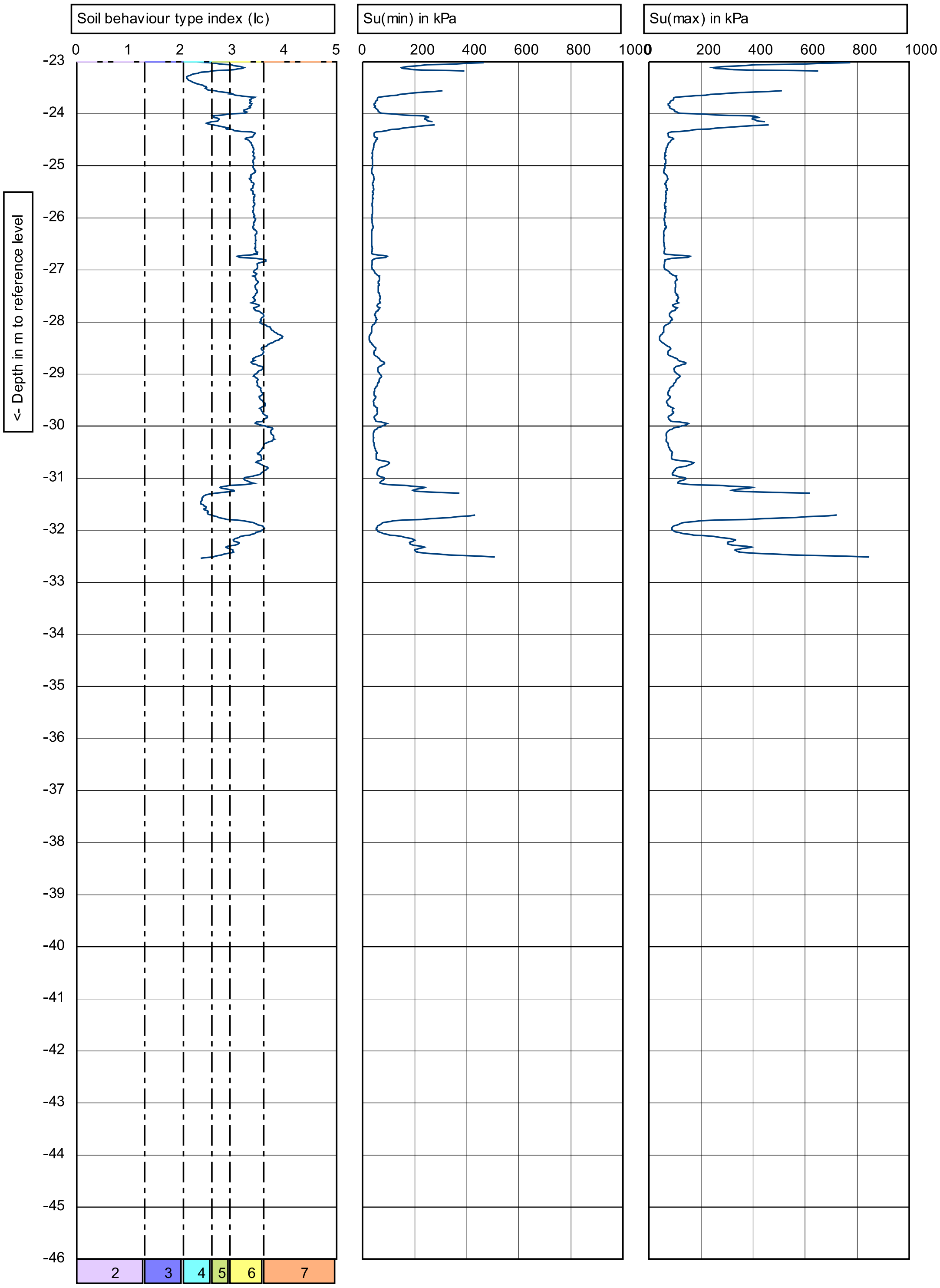
CPTask V1.20



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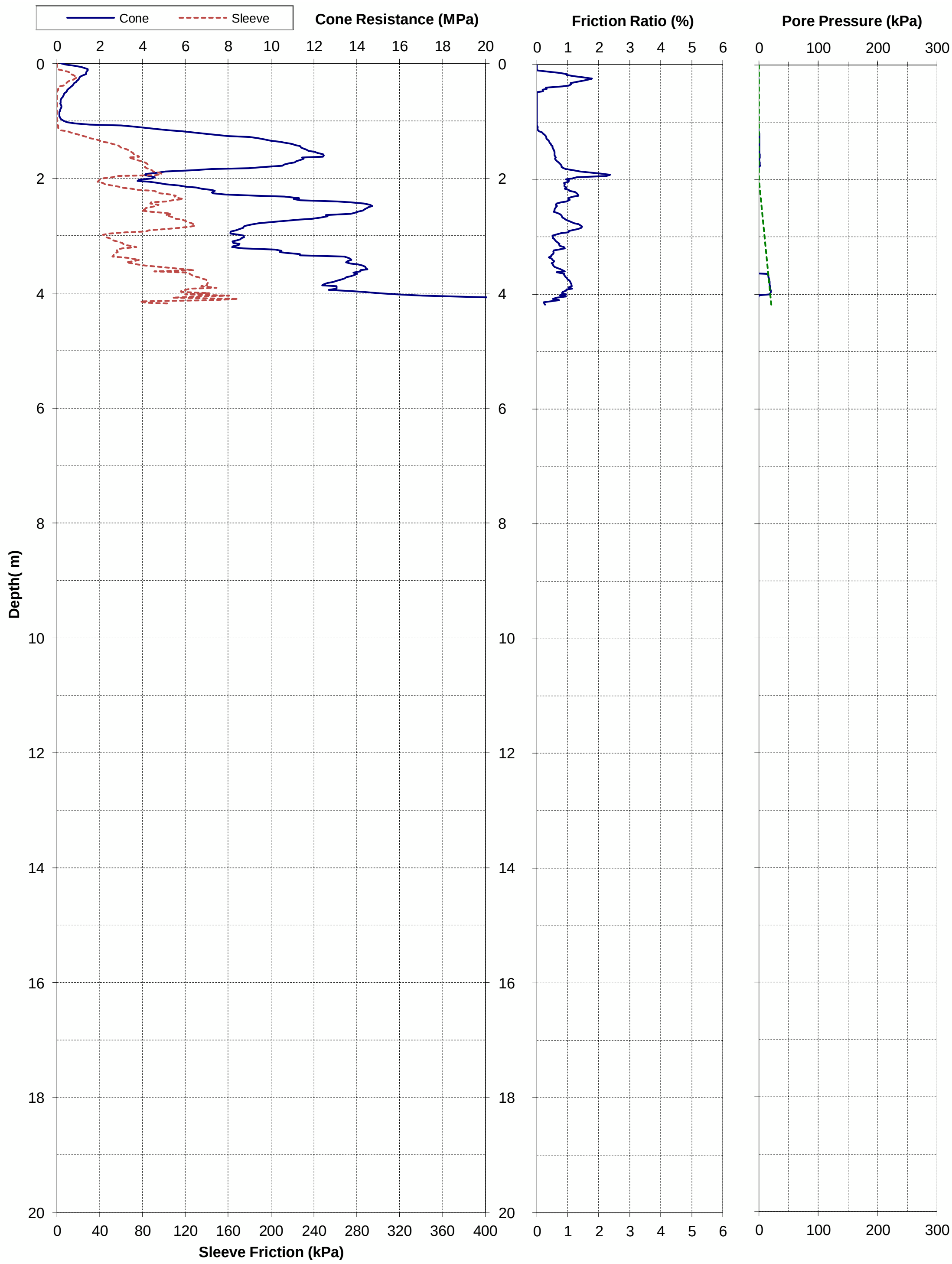
	Location: GPS: E2484271 N5741215		Project no.: 2-68292.11
	Position:		CPT no.: CPT-BRY-18 12/12

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:	



T+T Ref: 52000.3000 Printed: 7/10/2011 10:50 a.m. Template: CPT Graph Template v0.41.xls

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

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

Appendix C – Geotechnical investigation summary

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