*Christchurch City Council*

# Avonheath Courts BE 1401 EQ2

## Detailed Engineering Evaluation

## Quantitative Assessment Report

*Christchurch City Council*

# Avonheath Courts

# Quantitative Assessment Report

## Beachville Road Pensioners' Cottages

## 86 Beachville Road, Redcliffs, Christchurch 8081

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# Summary

Avonheath Courts
BE 1401 EQ2

Detailed Engineering Evaluation
Quantitative Report - Summary
Final

86 Beachville Road, Redcliffs, Christchurch

**Background**

This is a summary of the detailed seismic assessment of the Avonheath Courts housing complex and is based on the Detailed Engineering Evaluation Procedure document issued by the Structural Advisory Group on 19 July 2011, visual inspections, available drawings and calculations.

The purpose of the assessment is to determine if the buildings located at this site are classed as being earthquake prone in accordance with the Building Act 2004.

**Key Damage Observed**

Blocks A and B have suffered significant damage primarily due to lateral spread and differential settlement of the land. The extent of damage varies along the site with slight land damage at the south end of the site which increases towards the extreme north end (river estuary).

Block C appears to have performed well during the recent Canterbury earthquakes. The building shows very little sign of movement induced damage.

**Critical Structural Weaknesses**

We have identified the following Critical Structural Weakness:

- Lateral Spreading – It has been noted that the northern area of the site has suffered from lateral spreading, which has resulted in amplified settlements occurring to particular areas, most notably the north end of Blocks A and B.

**Indicative Building Strength**

- The seismic performance of Block A and Block B is governed by the capacity of the timber shear walls with an expected capacity of ≤33%NBS, allowing for damage, in the longitudinal (north-south) direction. The buildings are therefore considered to be Earthquake Prone Buildings in accordance with the Building Act 2004.

- The seismic performance of Block C has been calculated as a strength of 21%NBS. As the seismic capacity is less than 34%NBS, Block C is assessed as being Earthquake Prone in terms of the Building Act and is considered to be a grade D - high risk building in accordance with NZSEE guidelines.

## Recommendations

- It is recommended that Blocks A and B are either replaced or repaired and strengthened to at least 67%NBS.
- It is recommended that Block C is strengthened to at least 67%NBS, subject to detailed geotechnical advice regarding the site as a whole.

# Contents

# 1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council (CCC) to undertake a detailed seismic assessment of the Avonheath Courts housing complex, located at 86 Beachville Road, Redcliffs, Christchurch, following the Canterbury earthquake sequence that began during September 2010.

The purpose of the assessment is to determine if the building is classed as being earthquake prone in accordance with the Building Act 2004.

The seismic assessment and reporting have been undertaken based on the qualitative and quantitative procedures detailed in the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) [3] [4].

# 2 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.

## 2.1 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 to carry out a full structural survey before the building is re-occupied.

We understand that CERA 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). CERA have adopted the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011. This document sets out a methodology for both initial qualitative and detailed quantitative assessments.

It is anticipated that a number of factors, including the following, will determine the extent of evaluation and strengthening level required:

1. The importance level and occupancy of the building.
2. The placard status and amount of damage.
3. The age and structural type of the building.
4. Consideration of any critical structural weaknesses.

Christchurch City Council requires any building with a capacity of less than 34% of New Building Standard (including consideration of critical structural weaknesses) to be strengthened to a target of 67% as required under the CCC Earthquake Prone Building Policy.

## 2.2 Building Act

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

**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 the alteration. This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

The Earthquake Prone Building policy for the territorial authority shall apply as outlined in Section 2.3 of this report.

**Section 115 – Change of Use**

This section requires that the territorial authority is satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'.

This is typically interpreted by territorial authorities as being 67% of the strength of an equivalent new building or as near as practicable. This is also the minimum level recommended by the New Zealand Society for Earthquake Engineering (NZSEE).

**Section 121 – Dangerous Buildings**

This section was extended by the Canterbury Earthquake (Building Act) Order 2010, and defines a building as dangerous if:

1. 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
2. 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
3. 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

4. There is a risk that other property could collapse or otherwise cause injury or death; or
5. A territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

**Section 122 – Earthquake Prone Buildings**

This section defines a building as earthquake prone (EPB) 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 loads 33% of those used to design an equivalent new building.

**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.

**Section 131 – Earthquake Prone Building Policy**

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

## 2.3 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 on 4 September 2010.

The 2010 amendment includes the following:

1. A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
2. A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;
3. A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
4. 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.

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.

Where an application for a change of use of a building is made to Council, the building will be required to be strengthened to 67% of New Building Standard or as near as is reasonably practicable.

## 2.4 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.

On 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

- increase in the basic seismic design load for the Canterbury earthquake region (Z factor increased to 0.3 equating to an increase of 36 – 47% depending on location within the region);
- Increased serviceability requirements.

## 2.5 Institution of Professional Engineers New Zealand (IPENZ) Code of Ethics

One of the core ethical values of professional engineers in New Zealand is the protection of life and safeguarding of people. The IPENZ Code of Ethics requires that:

*Members shall recognise the need to protect life and to safeguard people, and in their engineering activities shall act to address this need.*

*1.1 Giving Priority to the safety and well-being of the community and having regard to this principle in assessing obligations to clients, employers and colleagues.*

*1.2 Ensuring that responsible steps are taken to minimise the risk of loss of life, injury or suffering which may result from your engineering activities, either directly or indirectly.*

All recommendations on building occupancy and access must be made with these fundamental obligations in mind.

# 3 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 loadings are in accordance with the current earthquake loading standard NZS1170.5 [1].

A generally accepted classification of earthquake risk for existing buildings in terms of %NBS that has been proposed by the NZSEE 2006 [2] is presented in Figure 1 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 required under Act) | Unacceptable | Unacceptable |

**Figure 1: 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 |

## 3.1 Minimum and Recommended Standards

Based on governing policy and recent observations, Opus makes the following general recommendations:

### 3.1.1 Occupancy

The Canterbury Earthquake Order[1] in Council 16 September 2010, modified the meaning of "dangerous building" to include buildings that were identified as being EPB's. As a result of this, we would expect such a building would be issued with a Section 124 notice, by the Territorial Authority, or CERA acting on their behalf, once they are made aware of our assessment. Based on information received from CERA to date and from the DBH guidance document dated 12 June 2012 [6], this notice is likely to prohibit occupancy of the building (or parts thereof), until its seismic capacity is improved to the point that it is no longer considered an EPB.

### 3.1.2 Cordoning

Where there is an overhead falling hazard, or potential collapse hazard of the building, the areas of concern should be cordoned off in accordance with current CERA/territorial authority guidelines.

### 3.1.3 Strengthening

Industry guidelines (NZSEE 2006 [2]) strongly recommend that every effort be made to achieve improvement to at least 67%NBS. A strengthening solution to anything less than 67%NBS would not provide an adequate reduction to the level of risk.

It should be noted that full compliance with the current building code requires building strength of 100%NBS.

### 3.1.4 Our Ethical Obligation

In accordance with the IPENZ code of ethics, we have a duty of care to the public. This obligation requires us to identify and inform CERA of potentially dangerous buildings; this would include earthquake prone buildings.

[1] This Order only applies to buildings within the Christchurch City, Selwyn District and Waimakariri District Councils authority

# 4 Background Information

## 4.1 Building Description

The site is located at Redcliffs, an outer coastal suburb of Christchurch. The Avonheath Courts housing complex refers to three distinct sub-blocks (Blocks A to C) which are structurally independent. The documentation received from CCC indicates that the buildings were designed and constructed in 1972. The site compromises a two-storey unit (Block C) located to the south, and two single storey units located along the western (Block A) and eastern (Block B) boundary. Refer to 'Appendix 2 – Original Drawings' for location plans identifying the aforementioned blocks and the varying storey heights.

**Block A** - This block is a single storey, timber framed structure with a flat timber framed roof. All elevations are clad in concrete block veneer. Block A compromises 5 separate housing units with two adjoining units being separated by a reinforced masonry party wall, presumably for fire resistance.


Photo 1 – Block A East Elevation

**Block B** –This block is similar in construction to Block A.

**Block C** – This block is a two-storey reinforced masonry building with timber trusses supporting a lightweight roof. The building contains 3 separate housing units numbered from 11 to 13 and a carport on the ground floor and four units numbered respectively from 14 to 17 on the upper level.


Figure 2 – Location Map

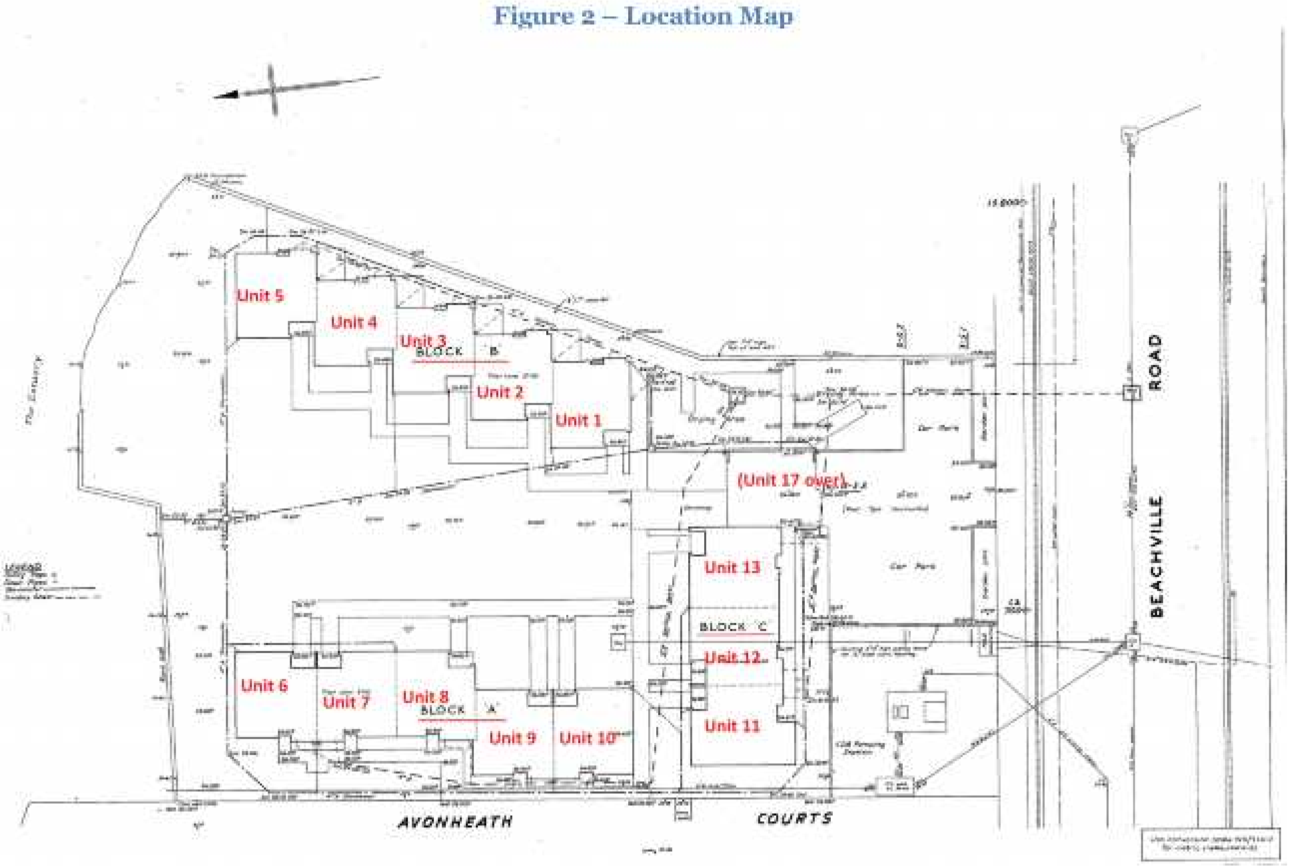



Figure 3 - Sub-block ground floor reference plan

## 4.2 Gravity Load Resisting System

### Block A (Block B similar)

The gravity load system is a combination of reinforced concrete masonry walls and timber framing. The flat roof areas are constructed using butynol roofing on plywood sarking on timber roof joists with lightweight aluminium cladded skylights formed in the roof.

### Block C

The gravity load system is a combination of reinforced concrete masonry and timber framed walls. The pitched roof is clad in lightweight Trimline aluminium roofing on building paper and plywood sarking on timber truss rafters. The building has a precast concrete first floor which is tied into masonry walls via perimeter beams with specific reinforcement detailing.

The 'Vibradek' floor system was used as the main floor system, incorporating prestressed ribs, hollow-core tiles, combined with insitu concrete topping to form a composite section.

The walls are reinforced concrete masonry with an approximate height of 2.65-2.75m throughout and are typically 190mm thick, excluding 3 walls in the longitudinal direction, which are 150mm thick.

The walls are founded on reinforced concrete foundation beams having an inverted T-shaped profile. The ground floor consists of a 100mm thick reinforced concrete slab–on-grade. There appears to be no mechanical connection between the ground slab and perimeter walls or strip footings.

## 4.3 Seismic Force Resisting System

### Block A (Block B similar)

The main seismic resisting elements are reinforced concrete masonry walls in the transverse (east-west) direction and timber shear walls in the longitudinal (north-south) direction. The single storey blocks appear to lack sufficient connections between the roof diaphragm (plywood sarking) and supporting walls and therefore the lateral forces have been apportioned on a tributary area basis, assuming that timber roof joists act as collectors to a moderate extent.

### Block C

Level 1 (ground floor) seismic forces in both the longitudinal (east-west) and transverse (north-south) directions are resisted by the in-plane capacity of the reinforced concrete masonry walls, whilst the lateral load resisting system on Level 2 (upper floor) is a combination of timber shear walls in the longitudinal and reinforced concrete masonry walls in the transverse direction.

The insitu concrete topping of the suspended precast concrete slab is mesh reinforced and is assumed to provide a rigid diaphragm which can distribute forces to the wall bracing elements.

## 4.4 Survey

### 4.4.1 Post 22 February 2011 Rapid Assessment

A structural (Level 2) Rapid assessment of the Avonheath Courts housing complex was undertaken on 3rd March 2011 by Opus International Consultants (Opus).

### 4.4.2 Further Inspections

Opus undertook a further walkover inspection on 20th September 2012. The visual inspection was supplementary to the recommendation for a quantitative assessment of the structures within the housing complex. Block A and Block B were not occupied at the time of inspection.

## 4.5 Original Documentation

Copies of the following construction drawings were provided for Avonheath Courts/ Beachville Road Pensioners' Cottages by CCC:

- Site plan and design levels dated February 1972
- Sewer longitudinal sections and car park cross-sections dated February 1972
- Block C foundation details, wall elevations, 1st floor plan, 1st floor details, stair details dated February 1972
- Block A - architectural plans and elevations dated 1973
- Block B - architectural plans and elevations dated 1973

The drawings have been used to confirm the structural systems, investigate potential critical structural weaknesses (CSW) and identify details which required particular attention. Copies of the design calculations were not provided.

# 5 Structural Damage

The following damage has been noted:

**Block A and Block B**

- differential settlement of the slab foundations
- separation of the slab foundations at the interface with adjacent units
- settlement of walls and separation of walls at the interface with adjacent units

**Block C**

- hairline cracks to suspended ceiling
- separation of perimeter reinforced concrete beams from top of reinforced masonry walls

**Surrounding Area Damage:**

- pavement cracking due to ground settlement
- lateral spread and settlement of built-up ground

Refer to Appendix 1– Photographs for general overview of damaged structures.

# 6 General Observations

Blocks A and B have suffered significant damage primarily due to lateral spread and differential settlement of the land. This has caused up to 20mm of separation between Units 1 to 5. In Units 6 to 10, the damage has been concentrated in Unit 8, with a separation of up to approximately 70mm.

Some temporary structural repairs were observed in some units.

The extent of damage varies along the site with slight land damage at the south end of the site which increases towards the extreme north end (river estuary).

Block C appears to have performed well during the recent Canterbury earthquakes. Overall the building shows very little sign of movement induced damage.

# 7 Detailed Seismic Assessment

The detailed seismic assessment has been based on the NZSEE 2006 [2] guidelines for the "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" together with the "Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure" [3] draft document prepared by the Engineering Advisory Group on 19 July 2011, and the SESOC guidelines "Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes" [5] issued on 21 December 2011.

## 7.1 Critical Structural Weaknesses

The term Critical Structural Weakness (CSW) refers to a component of a building that could contribute to increased levels of damage or cause premature collapse of a building. During the initial qualitative stage of the assessment the following potential CSW's were identified for each of the buildings and have been considered in the quantitative analysis.

We have identified the following Critical Structural Weakness:

- Lateral Spreading– It has been noted that the northern area of the site has suffered from lateral spreading, which has resulted in amplified settlements occurring to particular areas, most notably the north end of Blocks A and B.

## 7.2 Potential Structural Hazards

The following are potential structural hazards which have been identified in the Avonheath Courts housing complex. The nature of a structural hazard is to cause localised failure and damage but not influence the structure beyond the immediate area.

**Block A (Block B similar)**

- Ceiling diaphragms – Due to age of the building it can be assumed that the detailing of the horizontal roof/ceiling diaphragms may not meet the requirements of NZS 3604:2011.
- Inadequate connections of the ceiling diaphragm to the walls.
- Inadequate foundations – lack of mechanical connections between the floor slab and foundations.

**Block C**

- Inadequate connections of the ground floor diaphragm to the foundations - it has been noted that the ground floor slab is not tied into the foundations.
- Lack of adequate connections between the top storey ceiling diaphragm and supporting walls.

## 7.3 Seismic coefficient parameters

The seismic design parameters based on current design requirements from NZS1170.5:2004 and the NZBC clause B1 for this building are:

- Site soil class E, clause 3.1.3 NZS 1170.5:2004;
- Site hazard factor, Z = 0.3, B1/VM1 clause 2.2.14B;
- Return period factor Ru = 1.0 from Table 3.5, NZS 1170.5:2004, for an Importance Level 2 structure with a 50 year design life;
- $\mu$ = 1.25 for reinforced masonry and $\mu$ = 3.0 for timber walls

## 7.4 Quantitative Assessment Methodology

All units have been assessed using the equivalent static method as outlined in NZS 1170.5 – 2004.

**Block A and Block B**

With the lack of dependable roof diaphragm connections, the single storey unit has been assessed on the basis of tributary area. The walls have been assessed in the two primary orthogonal directions.

**Block C**

The 2-storey unit has been assessed in two primary orthogonal directions. Upper storey seismic forces have been distributed to the walls on a tributary area basis. Upper storey seismic loads are then assigned at the first floor level and are distributed with the first floor level seismic loads through the first floor slab diaphragm to the walls below on a relative stiffness basis. The torsional effects resulting from the application of the 10% eccentricity of load and the 30% of seismic load applied in the orthogonal direction, in accordance with NZS1170.5, have been allowed for.

## 7.5 Review of Critical Structural Weaknesses

Most of the critical structural weaknesses identified in the qualitative assessment will have an effect on the capacity of the building. These have been considered in the assessment tables.

## 7.6 Limitations and Assumptions in Results

Our analysis is based on a quantitative assessment of the building in its undamaged state. The damage observed on-site to structural elements has been considered qualitatively in this report as it difficult to accurately quantify the detrimental effect of this damage as a %NBS.

Where damage was considered severe enough to comprise the integrity of the affected structural elements it is our understanding that the relevant units have already been closed. There may have been additional damage to the buildings that was unable to be observed during assessments that could cause the capacity of the buildings to be reduced; therefore the current capacity of the buildings may be lower than that stated.

The results have been reported as a %NBS and the stated value is that obtained from our analysis and assessment. Despite the use of best national and international practice in this analysis and assessment, this value contains uncertainty due to the many assumptions and simplifications which are made during the assessment. These include:

a. Simplifications made in the analysis, including boundary conditions such as foundation fixity.
b. Assessments of material strengths based on limited drawings, specifications and site inspections
c. The normal variation in material properties which change from batch to batch.
d. Approximations made in the assessment of the capacity of each element, especially when considering the post-yield behaviour.

## 7.7 Assessment

A summary of the structural performance of the buildings are shown in the following table. Note that the values given represent the worst performing elements in the building, as these effectively define the building's capacity.

Other elements within the buildings may have significantly greater capacity when compared with the governing elements. This will be considered further when developing the strengthening options.

Table 2: Summary of Seismic Performance

| Structural Element/System | Failure Mode, or description of limiting criteria based on elastic capacity of critical element. | % NBS based on calculated capacity |
|---|---|---|
| Block A (Block B similar) | | |
| Reinforced masonry wall | In-plane rocking<br>Shear-friction<br>In-plane shear capacity<br>Out-of-plane bending capacity | 76%<br>100%<br>100%<br>62% |
| Timber shear wall | Bracing capacity of timber shear walls<br>- Wall line 1 (critical)<br>- Wall line 2<br>- Wall line 3 | <br>35%<br>36%<br>78% |
| Block C | | |
| Upper level | | |
| Reinforced masonry shear walls | In-plane shear capacity<br>Out-of-plane bending capacity | ≥100%<br>55% |
| Timber shear walls | Bracing capacity of timber shear walls | 25% |
| Lower level | | |
| Reinforced masonry shear walls | Shear Friction<br>Shear capacity of critical shear wall<br>In-plane bending capacity of critical shear wall<br>Out-of plane bending capacity | 41%<br>37%<br>21%<br>≥100% |

## 7.8 Discussion of Results

The results outlined in the previous table are an assessment of the structures in an undamaged state.

### Block A (Block B similar)

The assessment has assumed the timber rafters/ceiling joists have reasonable capacity to transfer lateral loads to walls. The calculated seismic capacity of the structure is 35% NBS, which is limited by the lowest timber shear wall capacity in the longitudinal direction. The lack of suitable connections between the roof diaphragm and supporting walls, and hence the structurally-independent nature of the walls, resulted in the calculated %NBS being specific to each wall line as detailed in Table 2.

The northern part of the site has sustained extensive ground deformations. A number of walls have separated from the ceiling and are out of plumb. Noticeable differential settlement and "spreading" has occurred between Units and both the roof/ceiling and slab/foundations have suffered localised damage around connection points. This will result in a reduction of the calculated capacity of the affected walls.

The seismic performance of the single storey buildings have a calculated strength of 35% NBS however this will be somewhat reduced by the observed damage. The building is therefore assessed as an Earthquake Prone Building (EPB) as defined by the Building Act 2004. It is considered to be a Grade D – high risk structure in accordance with NZSEE Risk Classifications outlined in Section 3 of this report. As such, the building will require seismic improvement to a target strength of 67%NBS in accordance with the recommendations of the Christchurch City Council

**Block C**

The seismic capacity for the longitudinal direction has been limited by the in-plane capacity of the lower level reinforced masonry shear walls (21%NBS) and bracing capacity of the upper level timber shear walls (25%NBS), giving an overall rating for the building of 21%NBS.

As the seismic capacity is less than 34%NBS, Block C is assessed as being Earthquake Prone in terms of the Building Act and is considered to be a grade D - high risk building in accordance with NZSEE guidelines. As such, the building will require seismic improvement to a target strength of 67%NBS in accordance with the recommendations of the Christchurch City Council.

# 8 Geotechnical Summary

## 8.1 General

A site specific Geotechnical Desk Study has been undertaken for this building complex to assess the available ground investigation data and the potential geotechnical hazards that may be present at the site, and determine whether further subsurface geotechnical investigations are necessary, refer *Appendix 3 - Geotechnical Desktop Report*. A summary of the outcomes of the desktop study is outlined below:

- The area is generally underlain by very loose or soft fill interlayered with silt, sand and clay to 3.0m below ground level (B.G.L) and medium to dense sand to approximately 22.0 m B.G.L. Stiff to very stiff sandy silts/clay were encountered below 22.0m B.G.L, interpreted as being the Riccarton Formation.

- Damage has occurred to the units at Avonheath Courts due to the Canterbury Earthquake Sequence following the 4 September 2010 earthquake. The damage extends to and includes the concrete slab on grade foundations.

- The crack and gap monitoring results carried out indicate that the site is at high risk of further lateral spreading, particularly after significant seismic events. The monitoring also indicates the site may be experiencing creep type movement of the order of millimetres per year, although this is difficult to confirm due to the margin of error. Creep type movement may be expected at the site now that residual soil shear strengths have been reached along the defined failure surfaces over which the lateral spreading occurs towards the estuary.

- The summation of the crack widths in the ground and the path adjacent to the east side of Units 6 to 10 is approximately 0.95 m, indicating the site has suffered lateral spreading, mainly northwards towards the estuary.

- By comparison, the differential settlements recorded from the floor survey suggest that the buildings may have been affected less by lateral spreading than the surrounding ground. The reason for this is not known at this stage although we consider that it may be related to the depth to the underside of the building foundations and the founding medium. If near surface soils above the foundation level are susceptible to lateral spreading then it may be reasonable to expect the ground outside of the building footprints to show more severe signs of lateral spreading than the buildings themselves.

- Foundation repair and re-levelling solutions for the complex will likely need to be undertaken for the units.

- Seismically induced settlement / lateral spreading is inferred from the site observations and the monitoring carried out to date on the site as a result of the February 2011 event and subsequent events. The areas around the paths and the edge of the unit foundations appear to be the main areas where ground cracking has been observed. Seismically induced liquefaction has also occurred on the site as evidenced by the sand boils and areas of heave, particularly towards the southern part of the site.

- Further monitoring and site specific investigations and assessment are required to confirm both the liquefaction and lateral spreading potential of the site.

- Observed damage suggests the western side of the site is TC3 with severe lateral movement (greater than 500mm). Careful consideration of lateral spread would be required for design of both shallow foundations and piled foundations.

## 8.2 Liquefaction

GNS Science indicates an elevated risk of seismic activity is expected in the Canterbury region as a result of the earthquake sequence following the 4 September 2010 earthquake. Recent advice2 indicates there is a 13% probability of another Magnitude 6 or greater earthquake occurring in the next 12 months in the Canterbury region. Therefore there is currently a risk of liquefaction and further differential settlements occurring at this site, dependant on the location of the epicentre. It is expected that the probability of occurrence is likely to decrease with time, following periods of reduced seismic activity.

## 8.3 Further Work

A site specific investigation comprising initially Cone Penetrometer Tests (CPT's) and test pits is recommended to assess both the lateral spreading and liquefaction potential of the site and to identify conceptual foundation repair, re-level or rebuild options.

We recommend the following:

- A total of four CPT's, comprising three CPT's to 25m depth or refusal. Due to restricted access to parts of the site, a small rig will need to be used for this work. The CPT rig may not penetrate shallow gravel or dense sand layers if encountered on site.

- Three test pits to expose the underside of the foundation of the units and to determine the bearing capacity of shallow soils. A hand auger Scala test should then be carried out within the test pit to 3m depth or refusal.

- The crack / gap monitoring program be continued on a bi-yearly basis or following a significant seismic event. This can be reviewed and reduced to yearly monitoring if the results stabilise to don't indicate ground movement.

- Assessment of the above information.

Refer to *Appendix 3 - Geotechnical Desktop Report* for details.

[2] GNS Science reporting on Geonet Website: http://www.geonet.org.nz/canterbury-quakes/aftershocks/ updated on 07 September 2012.

# 9 Conclusions

a) The seismic performance of Block A and Block B is governed by the capacity of the timber shear walls with an expected capacity of ≤33%NBS, allowing for damage, in the longitudinal (north-south) direction. The buildings are therefore considered to be Earthquake Prone Buildings in accordance with the Building Act 2004.

Block A and Block B require both repair and strengthening. The Canterbury Earthquake Recovery Authority (CERA) land zone map released 23 June 2011 has classified the site as TC3 meaning that moderate land deformations are likely to occur in a future small to medium sized earthquake, and significant land deformations are likely to occur in a future moderate to large earthquake.

Due to the severe ground damage (spreading) and differential settlement of foundations of Blocks A and B, combined with a moderate to high degree of distributed damage to structural elements, reinstatement of the above buildings may be impractical and/or prohibitively expensive.

b) The seismic performance of Block C is governed by the capacity of the lower level shear walls with an expected capacity of 21% in the transverse direction. The building is therefore considered to be Earthquake Prone in accordance with the Building Act 2004. Subject to detailed geotechnical advice, regarding the site as a whole, strengthening of Block C is a practical proposition.

# 10 Recommendations

- It is recommended that Blocks A and B are either replaced or repaired and strengthened to at least 67%NBS.
- It is recommended that Block C is strengthened to at least 67%NBS, subject to detailed geotechnical advice regarding the site as a whole.

# 11 Limitations

a) This report is based on an inspection of the structure with a focus on the damage sustained from the 22 February 2011 Canterbury Earthquake and aftershocks only. Some non-structural damage is mentioned but this is not intended to be a comprehensive list of non-structural items.

b) Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practicing in this field at the time.

c) This report is prepared for the Christchurch City Council to assist with assessing remedial works required for council buildings and facilities. It is not intended for any other party or purpose.

# 12 References

[1] NZS 1170.5: 2004, Structural design actions, Part 5 Earthquake actions, Standards New Zealand.

[2] NZSEE (2006), Assessment and improvement of the structural performance of buildings in earthquakes, New Zealand Society for Earthquake Engineering.

[3] Engineering Advisory Group, Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure, Draft Prepared by the Engineering Advisory Group, Revision 5, 19 July 2011.

[4] Engineering Advisory Group, *Guidance on Detailed Engineering Evaluation of Non-residential buildings, Part 3 Technical Guidance*, Draft Prepared by the Engineering Advisory Group, 13 December 2011.

[5] SESOC (2011), Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes, Structural Engineering Society of New Zealand, 21 December 2011.

[6] DBH (2012), Guidance for engineers assessing the seismic performance of non-residential and multi-unit residential buildings in greater Christchurch, Department of Building and Housing, June 2012.

# Appendix 1 - Photographs

| Site Name | | |
|---|---|---|
| No. | Item description | Photo |
| Block A | | |
| 1. | Block A – East elevation |  |
| 2. | Block A – East elevation |  |
| 3. | Severe cracking of block veneer outside Unit 8 |  |

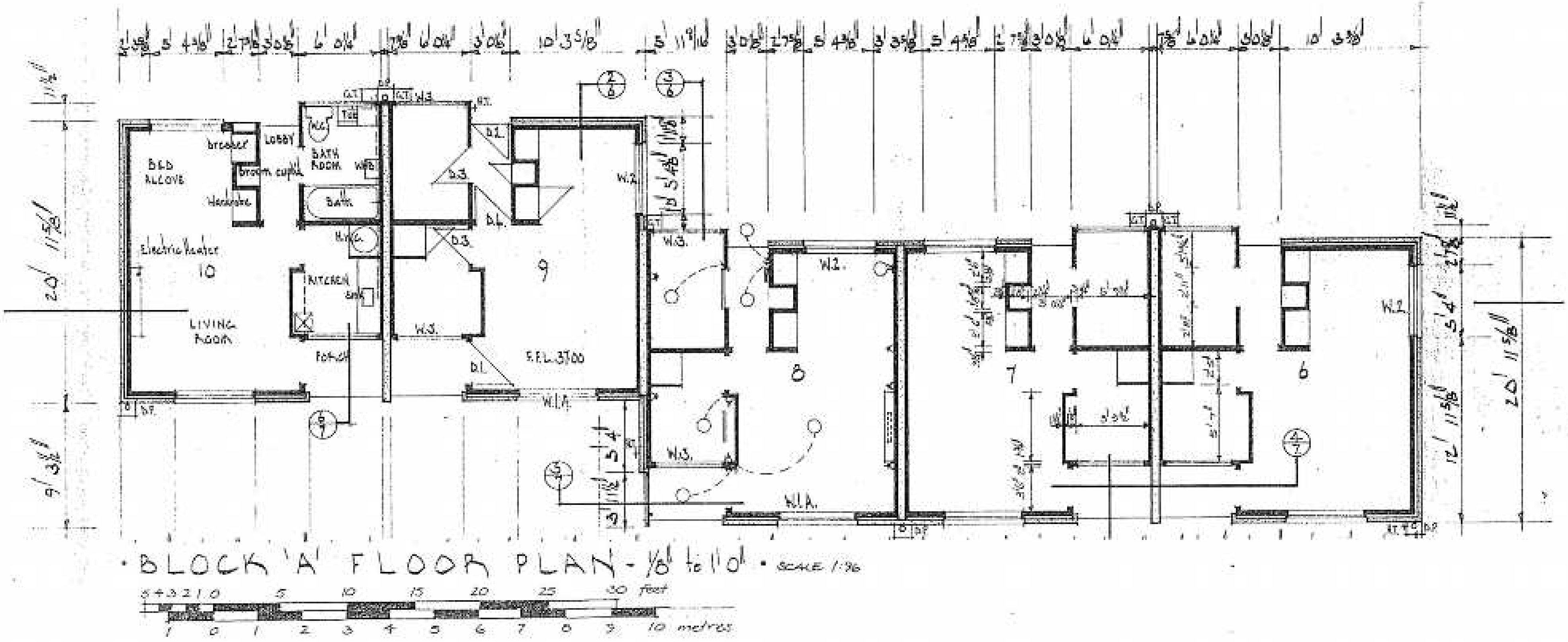
| | | |
|---|---|---|
| 4. | Separation between Units 8 and 9 due to lateral spreading |  |
| 5. | Loss of seating to roof along wall between Units 8 and 9 |  |
| 6. | Block A – Step cracking to northern end wall |  |

| | | |
|---|---|---|
| 7. | Block A – Significant ground damage /differential settlement of foundations induced by soil lateral spreading |  |
| Block B | | |
| 8. | Block B – West elevation |  |

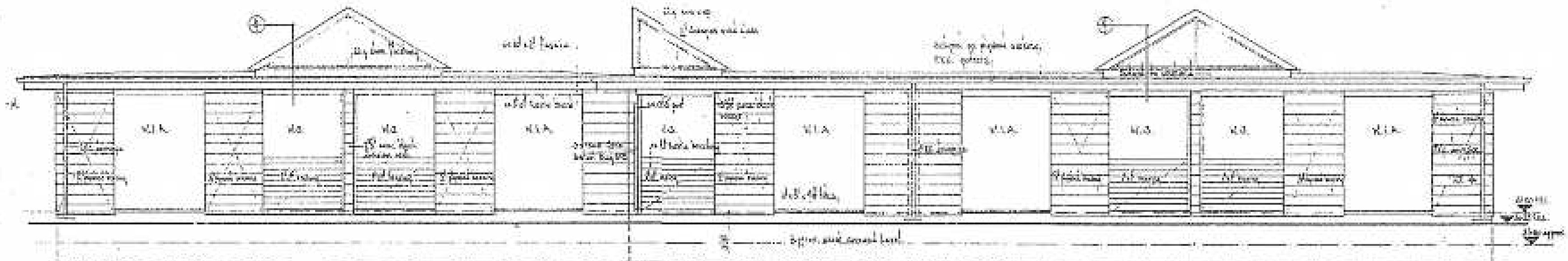
| Block C | | |
|---|---|---|
| 9. | Block C – South (street) elevation |  |
| 10. | Block C – North (Courtyard) elevation |  |
| 11. | Block C - Timber truss roof |  |

| | | |
|---|---|---|
| 12. | Cracking between block veneer and concrete bond beam |  |
| 13. | As above |  |
| 14. | Hairline cracking to suspended slab |  |

# Appendix 2 – Original drawings




·BLOCK 'A' FLOOR PLAN - 1/8" to 1'0" · SCALE 1:96

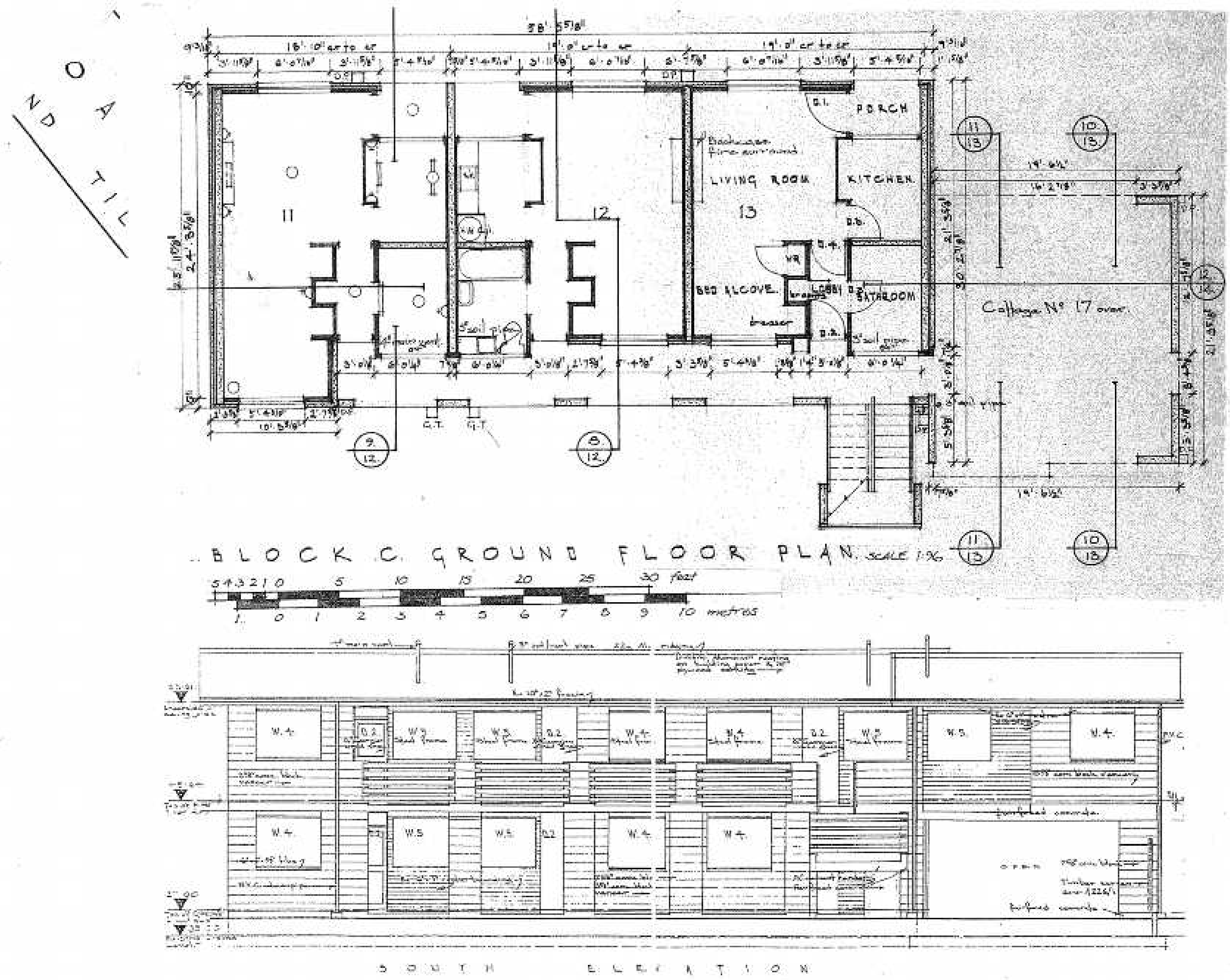


·EAST ELEVATION·

AVONHEATH COURTS

~~BEACHVILLE ROAD~~ PENSIONERS' COTTAGES.

JOB NUMBER: A 183
DATE: 1973




AVONHEATH COURTS

~~BEACHVILLE ROAD~~ PENSIONERS' COTTAGES.


AVONHEATH COURTS

~~BEACHVILLE ROAD~~ PENSIONERS' COTTAGES.

JOB NUMBER: A/89
DATE: 1973

# Appendix 3 – Geotechnical Desktop Report

20th November 2012

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Christchurch City Council

6-QUCC2.02_005HC

**Avonheath Courts- Phase 1 Geotech Assessment – Stage 1**

# 1 Introduction

The Units of Avonheath Courts on Beachville Road were subjected to severe ground shaking during the Magnitude 7.1 Darfield 2010 and Magnitude 6.3 Christchurch 2011 earthquake and subsequent aftershocks. This report summarises the findings of a geotechnical desktop study and site walkover completed by Opus International Consultants (Opus) for the Christchurch City Council.

The purpose of the geotechnical desk study is to assess the available ground investigation data and the potential geotechnical hazards that may be present at the site, and determine whether further subsurface geotechnical investigations are necessary.

Various geotechnical and structural inspections have been undertaken by Opus. It is our understanding this is the first reporting by a Geotechnical Engineer of this property following the earthquakes.

This geotechnical desk study forms part of a Detailed Engineering Evaluation prepared by Opus, and has been undertaken without the benefit of any site specific investigations and is therefore preliminary in nature.

# 2 Desktop Study

## 2.1 Site Description

The site comprising Avonheath Courts is located approximately 450m east of the Main Road / Beachville Road intersection in the Redcliffs suburb of Christchurch City. The estuary to the confluence of the Avon River and the Heathcote River is located immediately to the north of the site.

The Avonheath Courts site comprises three blocks of residential units, all of which have concrete breeze block type walls over a suspected concrete on grade type foundation and with a corrugated iron roof. There are two north to south oriented blocks, one on the west part and one on the east part of the site. The third block is a west to east oriented two storey block in the south part of the site.

The area around the units comprises pathways and landscaped areas.

The site is thought to have been located on a fill site to raise the land from the sea/tides and is generally flat and level throughout from Beachville Road to the northern boundary of the site. There is a low height (1.4m) stone masonry wall along the northern boundary of the site and this retains fill supporting the lawn area to the south. The top of the retaining wall is approximately 5.5m from the northern wall of unit 6.

## 2.2 Structural Drawings

No geotechnical report or records of ground investigations associated with the construction of the buildings are on Christchurch City Council's property file.

An internal floor level survey has been completed by Opus (refer to plans in Appendix A).

## 2.3 Regional Geology

The published geological map of the area, (Geology of the Christchurch Urban Area 1:25,000, Brown and Weeber, 1992) indicates the site is underlain by marine sand of semi and fixed dunes and beaches of the Christchurch Formation of Holocene age.

A groundwater table depth of less than 1m is also indicated on the published map by Brown and Weeber (1992).

## 2.4 Expected Ground Conditions

A review of the Environmental Canterbury (ECan) Wells database showed three wells located within approximately 60m of the property (refer to Site Location Plan and borehole logs in Appendix B).

Following the earthquake sequence, the Earthquake Commission (EQC) and Stronger Christchurch Infrastructure Rebuild Team (SCIRT) has undertaken additional borehole and cone penetrometer (CPT) testing in Christchurch. The cone penetration test holes CPT-RCL-02 and CPTu 01 to 04 (refer Appendix C), are indicated to have been carried out in the location of Beachville Road and Celia Street of the northern site boundary.

The stratigraphy below Avonheath Courts, inferred from the ECan boreholes and the EQC Investigations, is shown in Table 1 below;

| **Stratigraphy** | **Thickness (m)** | **Depth Encountered from (m) bgl** |
|---|---|---|
| Very loose or soft FILL / interlayered SILT/SAND/CLAY | 3.0m | Surface |
| Medium dense to dense SAND | 19m | 3.0m |
| Stiff to very stiff sandy SILT/CLAY (Riccarton Formation) | - | 22.0m |

**Table 1 Inferred Ground Conditions**

No groundwater levels are provided on the SCIRT CPT logs. The ECan boreholes simply state the water level is 'artesian' and this is likely to relate to the Riccarton Gravel Formation at depth.

### 2.5 Liquefaction Assessment

The 2004 Environment Canterbury (ECan) Solid Facts Liquefaction Study indicates the site is in an area designated as 'moderate liquefaction ground damage potential'. According to this study, based on a low groundwater table, ground damage is expected to be moderate and may be affected by up to 300mm of ground subsidence.

Inspection of maps provided by the EQC in the Canterbury Geotechnical Database (Project Orbit) has been carried out.

EQC examination of aerial photos around the 4 Sept 2010, 22 Feb 2011 and 13 June 2011 Earthquakes identified that the site lies in an area interpreted to have had minor observed liquefaction.

The post-earthquake ground surface observations has identified that the site lies in an area that experienced no lateral spreading but minor to moderate quantities of ejected material after the 4 Sept 2010, 22 Feb 2011 and 13 June 2011 Earthquakes.

Inspection of the Observed Crack Location Map has identified a number of cracks trending west to east and between 10mm to 200mm wide that were observed on site. These are shown to be in the central and northern part of the site and orientated west to east, refer Appendix D.

The vertical elevation change (LIDAR) map indicates that the southern part of the site has generally subsided by between 0.1m to 0.2m, whilst the northern part of the site by the estuary has uplifted generally between 0.1m to 0.5m.

### 2.6 Flood Risks and other hazards

The site area is shown to be part of the Flood Management Area. Floor levels are not currently available on the CCC website. Actual floor levels for each property will be set as part of the building consent process.

Contamination has been undertaken at Avonheath Courts in the past. It is recommended that a contamination assessment is included in the scope of works for any future land or foundation strengthening works.

## 3 Site Walkover Inspection

A walkover inspection of the exterior of the building and surrounding ground was carried out by an Opus Senior Geotechnical Engineer on 27th May 2011. The following observations were made (refer to the Walkover Inspection Plan and Site Photos attached to this report):

- Gap separations between the individual units of eastern block (Units 1 to 5) with estimated combined displacement up to 200mm.

- Gap separations between the individual units for western block (Units 6 to 10) with estimated combined displacement 300mm to 500mm.

- Cracks (less than 10mm) in block-work of north east corner of Unit 6 (photos 1 and 2).
- Crack (less than 10mm) in foundation on the western side of Unit 8 (photo 3).
- Translational crack (15mm north to south offset, 16mm west to east offset) in the pre-cast concrete block-work on the north east corner of Unit 9 (photo 4)
- Evidence of sand boils in Beachville Road and the site car park area (photo 5).
- Large crack (200mm wide) in ground adjacent to the northern wall of Unit 6 (photos 6 & 7). The footing is exposed to a depth of at least 900mm in this crack.
- Large cracks in lawn area, particularly in the northern part of the site between units 5 and 6 (photos 8 and 9).
- Numerous cracks in car park asphalt and also concrete path around the units, some of which have evidence of sand boils (photos 10 to 13).
- Cracking and rotational damage to corner of stone masonry retaining wall at northern part of site (photo 14).

The cracks in addition to the displacement observed between the units are likely to be associated with lateral spreading northwards towards the Estuary.

The above does not incorporate every single crack / feature on site, rather it is an assessment of some of the main features observed.

# 4 Monitoring

Crack and gap monitoring points were set up around key locations at the site on the 3rd June 2011 following the initial site walkover inspection. A total of 9 stations were installed and the crack /gap widths measured on the following dates:

- 3rd June 2011
- 10th June 2011
- 15th June 2011
- 24th June 2011
- 1st July 2011
- 11th July 2011
- 18th July 2011
- 16th September 2011
- 20th September 2012
- 30th October 2012

The locations of the monitoring stations are provided on the Site Walkover Observations Plan, whilst photos of the monitoring stations (as of 30th October 2012) and the monitoring results are provided in Appendix D.

Significant increases of crack / gap separations were recorded following the 13th June 2011 earthquake. The recorded increases after the June earthquake are summarised in Table 2 below:

| Station Number | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 |
|---|---|---|---|---|---|---|---|---|---|
| **Crack width increase - mm** | 6mm | 4mm | 41mm | 15mm | 32mm | 26mm | N/A* | 9mm | 5mm |

**Table 2: Crack increase of monitoring stations following 13th June 2011 event**

*=Station required re-establishment therefore measurements not obtained for period.

Minor on-going 'creep' type movement has been recorded in the period after the 13th June 2011 event, indicating the ground is continuing to move possibly in response to the subsequent aftershocks.

On the most recent monitoring visit (30th October 2012), measurements of all crack widths in the ground and path along the eastern side of Units 6 to 10 was also carried out to provide an estimate on the total lateral spreading that has occurred in this part of the site. Our summation of measured crack widths in the lawn area between unit 6 and the retaining wall in addition to cracks in the path on this part of the site is just under 0.95m.

# 5 Level Survey

An internal floor level survey was completed by Opus in October 2012. The survey plans are included in Appendix A. The differential settlement recorded within each unit is summarised in Table 3 below:

| Unit Number | Differential Settlement - mm | General Direction of fall |
|---|---|---|
| **1** | 46mm | South |
| **2** | 42mm | South |
| **3** | 78mm | South |
| **4** | 68mm | North |
| **5** | 98mm | North |
| **6** | 114mm | North |
| **7** | 47mm | North |
| **8** | 71mm | North |
| **9** | 44mm | North |
| **10** | 97mm | South |
| **11** | 46mm | South |
| **12** | 34mm | East |
| **13** | 26mm | East |

**Table 3: Differential settlement, direction of ground floor units**

# 6 Discussion

All Units are suspected to be constructed on concrete slab on grade type foundations. This would equate to a C2 type structures in accordance with the Department of Building and Housing "Revised guidance on repairing and rebuilding houses affected by the Canterbury Earthquake Sequence", dated November 2011.

Damage has occurred to the units at Avonheath Courts due to the Canterbury Earthquake Sequence following the 4 September 2010 earthquake. The damage extends to and includes the concrete slab on grade foundations.

The crack and gap monitoring results carried out indicate that the site is at high risk of further lateral spreading, particularly after significant seismic events. The monitoring also indicates the site may be experiencing creep type movement of the order of millimetres per year, although this is difficult to confirm due to the margin of error. Creep type movement may be expected at the site now that residual soil shear strengths have been reached along the defined failure surfaces over which the lateral spreading occurs towards the estuary.

The summation of the crack widths in the ground and the path adjacent to the east side of Units 6 to 10 is approximately 0.95 m, indicating the site has suffered lateral spreading, mainly northwards towards the estuary.

By comparison, the differential settlements recorded from the floor survey suggest that the buildings may have been affected less by lateral spreading than the surrounding ground. The reason for this is not known at this stage although we consider that it may be related to the depth to the underside of the building foundations and the founding medium. If near surface soils above the foundation level are susceptible to lateral spreading then it may be reasonable to expect the ground outside of the building footprints to show more severe signs of lateral spreading than the buildings themselves.

Foundation repair and relevelling solutions for the complex will likely need to be undertaken for the units.

Seismically induced settlement / lateral spreading is inferred from the site observations and the monitoring carried out to date on the site as a result of the February 2011 event and subsequent events. The areas around the paths and the edge of the unit foundations appear to be the main areas where ground cracking has been observed. Seismically induced liquefaction has also occurred on the site as evidenced by the sand boils and areas of heave, particularly towards the southern part of the site.

Further monitoring and site specific investigations and assessment are required to confirm both the liquefaction and lateral spreading potential of the site.

GNS Science indicates an elevated risk of seismic activity is expected in the Canterbury region as a result of the earthquake sequence following the 4 September 2010 earthquake. Recent advice[1] indicates there is a 13% probability of another Magnitude 6 or greater earthquake occurring in the next 12 months in the Canterbury region. Therefore there is currently a risk of liquefaction and further differential settlements occurring at this site, dependant on the location of the epicentre. It

[1] GNS Science reporting on Geonet Website: http://www.geonet.org.nz/canterbury-quakes/aftershocks/ updated on 07 September 2012.

is expected that the probability of occurrence is likely to decrease with time, following periods of reduced seismic activity.

Observed damage suggests the western side of the site is TC3 with severe lateral movement (greater than 500mm). Careful consideration of lateral spread would be required for design of both shallow foundations and piled foundations.

# 7 Recommendations

A site specific investigation comprising initially Cone Penetrometer Tests (CPT's) and test pits is recommended to assess both the lateral spreading and liquefaction potential of the site and to identify conceptual foundation repair, relevel or rebuild options.

We recommend the following:

- A total of four CPT's, comprising three CPT's to 25m depth or refusal. Due to restricted access to parts of the site, a small rig will need to be used for this work. The CPT rig may not penetrate shallow gravel or dense sand layers if encountered on site.
- A contamination assessment of the underlying shallow soils.
- Three test pits to expose the underside of the foundation of the units and to determine the bearing capacity of shallow soils. A hand auger Scala test should then be carried out within the test pit to 3m depth or refusal.
- The crack / gap monitoring program be continued on a bi-yearly basis or following a significant seismic event. This can be reviewed and reduced to yearly monitoring if the results stabilise to don't indicate ground movement.
- Assessment of the above information.

The Proposed Ground Investigation Plan is provided in Appendix F.

***Limitation***

This report has been prepared solely for the benefit of Christchurch City Council as our client with respect to the brief. The reliance by other parties on the information or opinions contained in the report shall, without our prior review and agreement in writing, be at such parties' sole risk.

**Attachments**

<u>Figures:</u>
Site Photographs
Site Location Plan
Site Walkover Observations Plan

<u>Appendices:</u>
Appendix A: Floor Level Survey Drawings
Appendix B: Environment Canterbury Borehole Logs
Appendix C: CPTu-01 to 04
Appendix D: Crack Monitoring Photos and Results
Appendix E: Proposed Ground Investigation Plan

Photo 1: Cracks in block-work of north east corner of Unit 6

Photo 2: Cracks in block-work of north east corner of Unit 6


Photo 3: Crack in foundation in western side of Unit 8.

Photo 4: Crack in block-work on north east corner of Unit 9


Photo 5: Sand boils in car park area, close to service trench


Photo 6: Crack in ground adjacent to northern wall of Unit 6

Photo 7: Crack in ground adjacent to northern wall of Unit 6

Photo 8: Cracks in lawn area in northern part of site.

Photo 9: Cracks in lawn area in northern part of site.

Photo 10: Crack in car park, with sand boil infill


Photo 11: Crack in path with sand boil infill, by Unit 9 and 10

Photo 12: Crack in path with sand boil infill, by Unit 7 and 8

Photo 13: Cracks in path on eastern side of units 7 and 8

Photo 14: Crack damage to retaining wall in north part of the site.


Key:

ECan Borehole Location

CPTu-01 to 04

Site Boundary


OPUS

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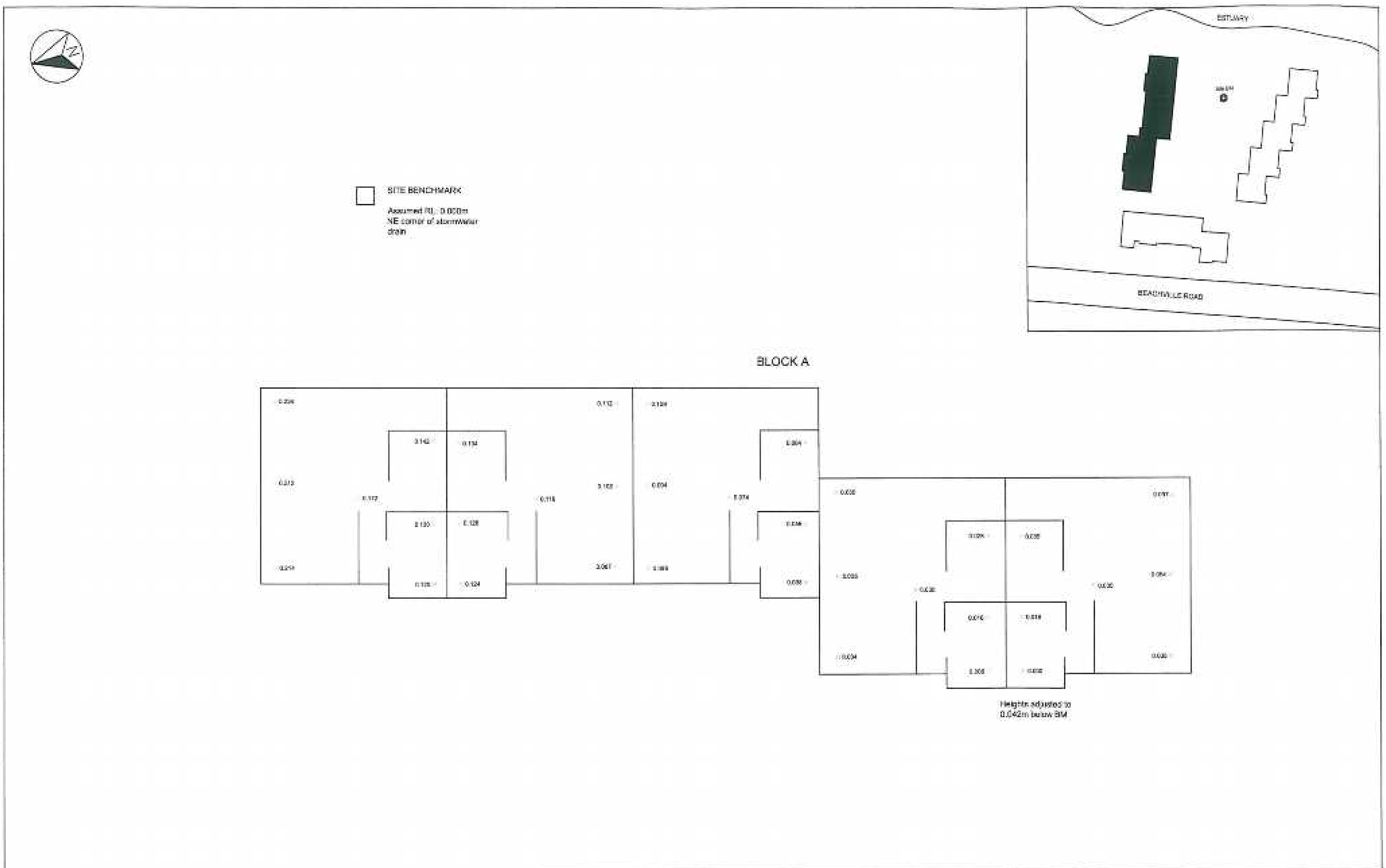
| | | | |
|---|---|---|---|
| **Project:** | Avonheath Courts, 86A Beachville Road<br>Geotechnical Desk Study | **Site Location Plan - NTS** | |
| **Project No.:** | 6-QUCC2.02 | **Drawn:** | Geotechnical Engineer |
| **Client:** | Christchurch CC | **Date:** | 31st October 2012 |

Site Walkover Observations Plan. (NTS)
(As of 27th May 2011)
Estuary →
N
Damage to retaining wall corner
Minor bulk in 1.4m high rock wall
Station ③
Station ⑥
Station ②
Station ④
Station ⑤
Station ⑧
Station ⑨
Station ①
Station ⑦
Unit ⑥
Unit ⑦
Unit ⑧
Unit ⑨
Unit ⑩
Unit ⑤
Unit ④
Unit ③
Unit ②
Unit ①
Units ⑪-⑬ - Ground floor
Units ⑭-⑰ - Upper floor.
60mm
100mm
50mm
30mm
20mm
25mm
~100mm
Area of Heave
Key:
50mm = Approximate Crack location and aperture width. (Where recorded).
Station ⑦ = Crack Monitoring Locations.
← Beachville Road →

## Appendix A:

**Site Plan and Floor Level Survey Drawings**

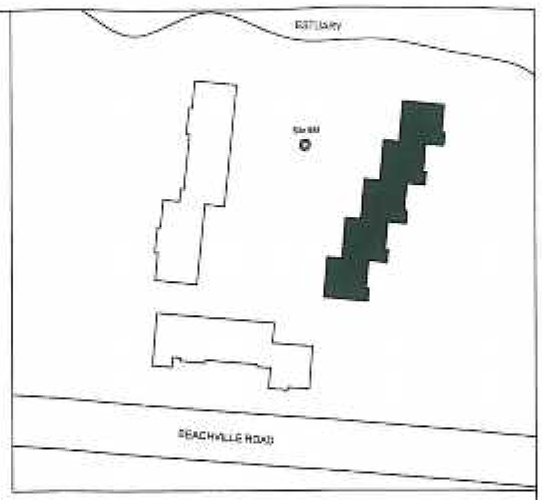


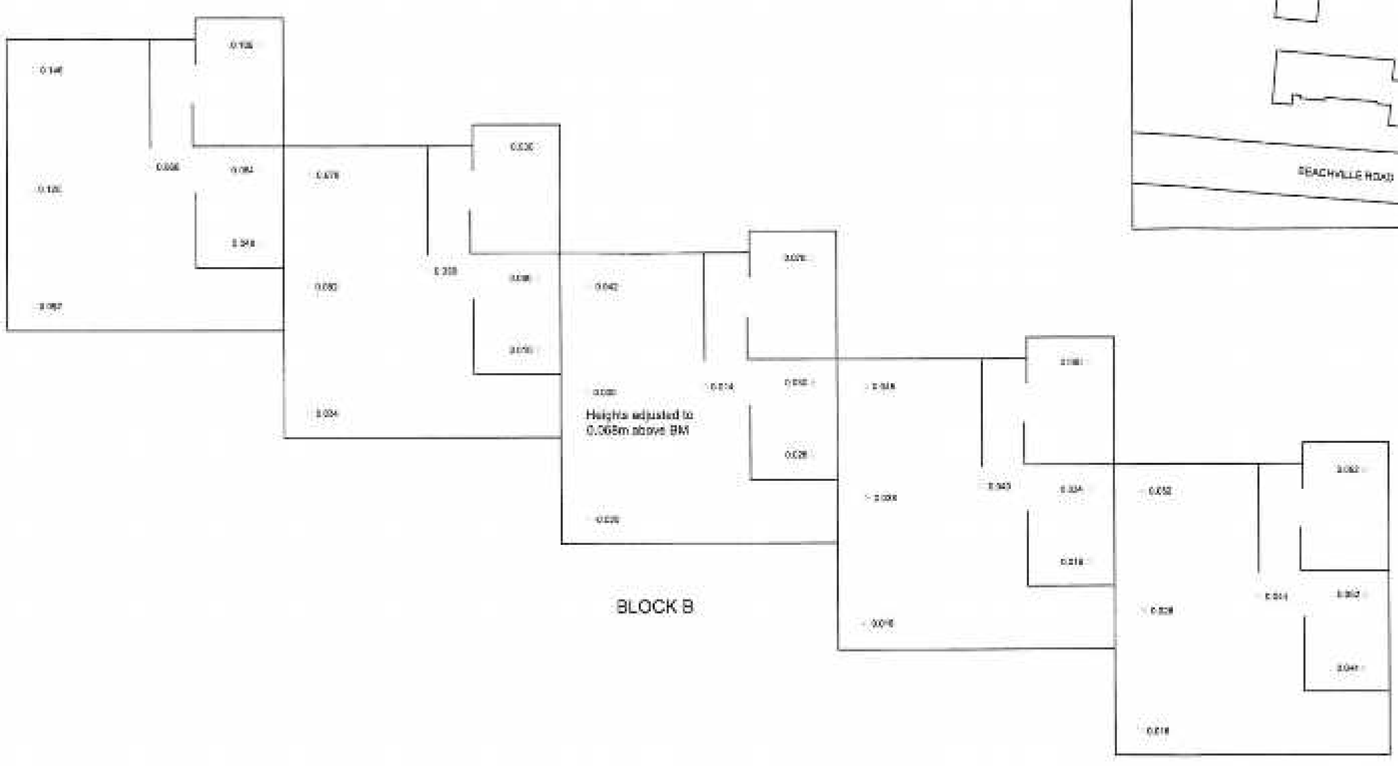

OPUS
Christchurch Office

CHRISTCHURCH CITY COUNCIL
AVON HEATH COURTS

FLOOR LEVEL SURVEY
BLOCK A

6-1365.01/702504 | 1/3 | R0




SITE BENCHMARK

Assumed RL: 0.000m
NE corner of stormwater drain

**OPUS**

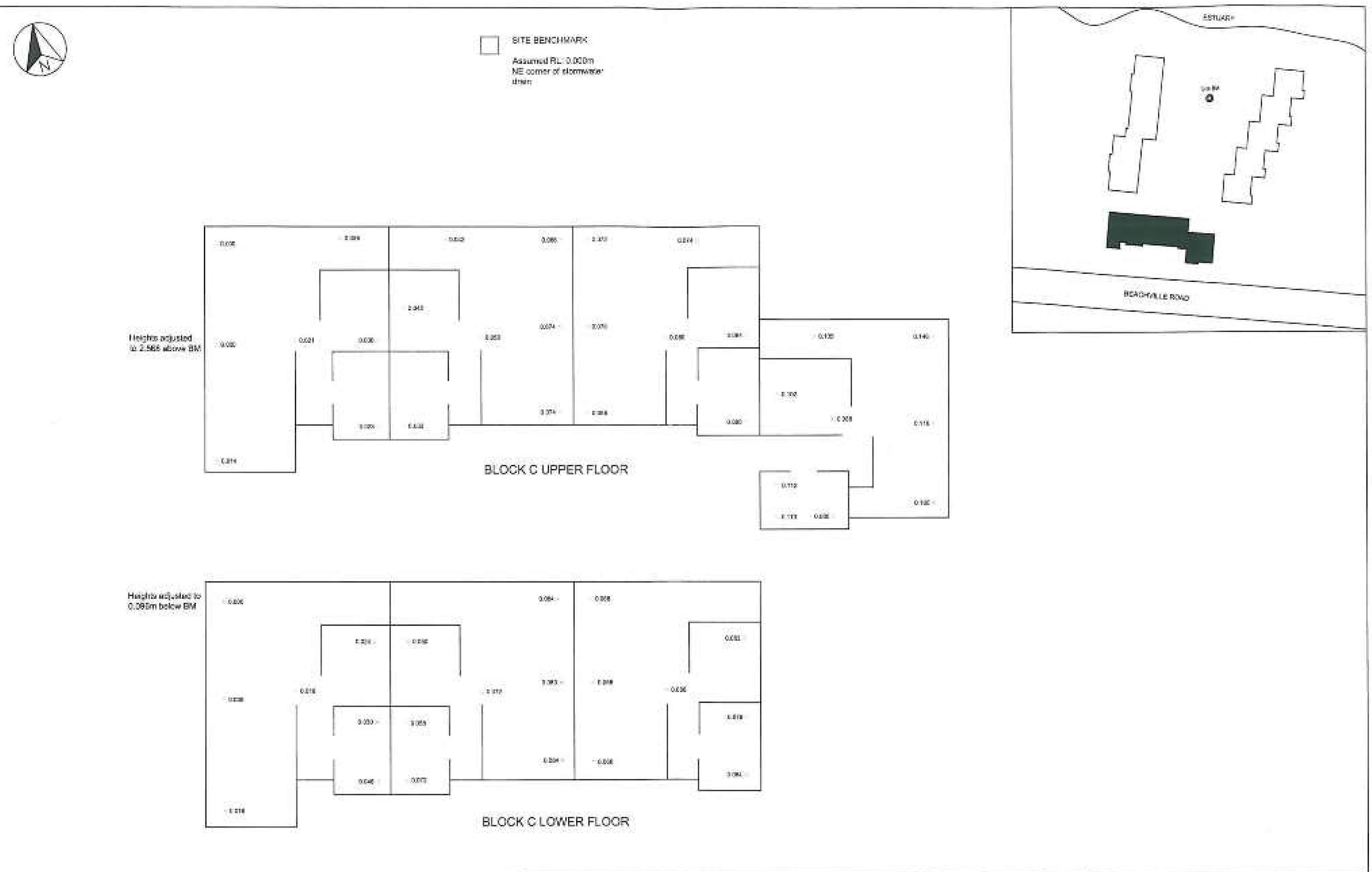
Christchurch Office

CHRISTCHURCH CITY COUNCIL

FLOOR LEVEL SURVEY
BLOCK B

6/1369/307/2604

R0




CHRISTCHURCH CITY COUNCIL

FLOOR LEVEL SURVEY

BLOCK C

OPUS

Christchurch Office

New Zealand

## Appendix B:

## Environment Canterbury Borehole Logs

# Borelog for well M36/1083

Gridref: M36:887-390 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 1.8 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Cable Tool
Drill Depth : -63.7m Drill Date : 27/10/1939

Environment Canterbury Regional Council

| Water Level | Depth(m) | Full Drillers Description | Formation Code |
|---|---|---|---|
| Artesian | 0 to -12.2m | Sand & clay | ch |
| | -12.2m to -25.3m | Blue sand | ch |
| | -25.3m to -28.7m | Blue clay | ri |
| | -28.7m to -32.0m | Yellow sand & clay | ri |
| | -32.0m to -35.1m | Blue clay & sand | ri |
| | -35.1m to -44.2m | Blue sand | br |
| | -44.2m to -45.7m | Blue sand & clay | br |
| | -45.7m to -47.8m | Brown clay | br |
| | -47.8m to -50.9m | Scoria | br |
| | -50.9m to -55.4m | Yellow sand & clay | br |
| | -55.4m to -55.7m | Brown sand | li-1 |
| | -55.7m to -56.9m | Brown gravel (Small) | li-2 |
| | -56.9m to -58.5m | Yellow sand & clay | |
| | -58.5m to -63.7m | Blue sand & clay | li-2 |

# Borelog for well M36/1039 page 1 of 2

Environment Canterbury Regional Council

Gridref: M36:887-390 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 1.8 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -104.4m Drill Date : 24/02/1939

Water Level: Artesian

| Depth(m) | Full Drillers Description | Formation Code |
|---|---|---|
| - 12.2m | Sand & clay | ch |
| - 23.2m | Blue sand | ch |
| - 30.5m | Blue clay & sand | ri |
| - 34.1m | Yellow sand & clay | ri |
| - 43.3m | Blue clay | br |
| - 44.5m | Blue sand | br |
| - 45.7m | Blue clay & sand | br |
| - 48.2m | Brown clay | br |
| - 50.6m | Scoria (Pumped 7.5 gpm) | br |
| - 52.2m | Yellow sand & clay | br |

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

# Borelog for well M36/1039 

Environment Canterbury Regional Council

Gridref: M36:887-390 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 1.8 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -104.4m Drill Date : 24/02/1939

Water Level: Artesian

| Depth(m) | Full Drillers Description | Formation Code |
|---|---|---|
| - 54.5m | Yellow sand & clay | br |
| - 56.1m | Brown gravel (Water) small | li? |
| - 57.6m | Brown sand | li? |
| - 60.7m | Yellow sand & clay | li? |
| - 64.0m | Blue sand & clay | li? |
| - 72.9m | Yellow sand & clay | li? |
| - 78.9m | Brown sand | li? |
| - 82.6m | Brown sand & clay | he? |
| - 86.3m | Scoria | he? |
| - 89.6m | Yellow sand & clay | he? |
| - 90.2m | Scoria | he? |
| - 96.6m | Yellow sand & clay | he? |
| - 99.1m | Blue sand & clay | bu? |
| - 99.4m | Scoria | bu? |
| - 104.3m | Yellow sand & clay | sh? |
| - 105.7m | Rock | vo |

# Borelog for well M36/1104

Gridref: M36:887-390 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 1.8 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Driven Pipe
Drill Depth : -56m Drill Date : 1/10/1942

Environment Canterbury Regional Council

Water Level: Artesian

| Depth(m) | Full Drillers Description | Formation Code |
|---|---|---|
| - 23.7m | Blue sand | ch |
| - 25.2m | Blue sand & gravel | ri |
| - 27.1m | Yellow clay | ri |
| - 34.4m | Blue clay | ri |
| - 45.7m | Blue sand | br |
| - 50.2m | Scoria | br |
| - 51.8m | Blue sand | br |
| - 52.7m | Brown sand | br |
| - 53.3m | Yellow clay | br |
| - 54.2m | Brown sand | br |
| - 56.6m | Brown gravel | li-1 |

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

# Borelog for well M36/1128 page 1 of 2

Gridref: M36:887-390 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 1.8 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -139.2m Drill Date : 30/09/1910

Environment Canterbury Regional Council

Water Level: Artesian

| Depth(m) | Full Drillers Description | Formation Code |
|---|---|---|
| - 23.1m | Blue sand | ch |
| - 47.5m | Blue sand & clay | ri-br |
| - 51.8m | Yellow sand & clay | br |
| - 57.6m | Blue sand & clay & small rocks | li |
| - 58.2m | Yellow sand & clay | li |
| - 58.8m | Brown gravel small flow | li |
| - 61.5m | Yellow sand & clay, small show of water | li |
| - 69.6m | Yellow sand, gravel & shells | li |

# Borelog for well M36/1128 page 2 of 2

Gridref: M36:887-390 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 1.8 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -139.2m Drill Date : 30/09/1910

Environment Canterbury Regional Council

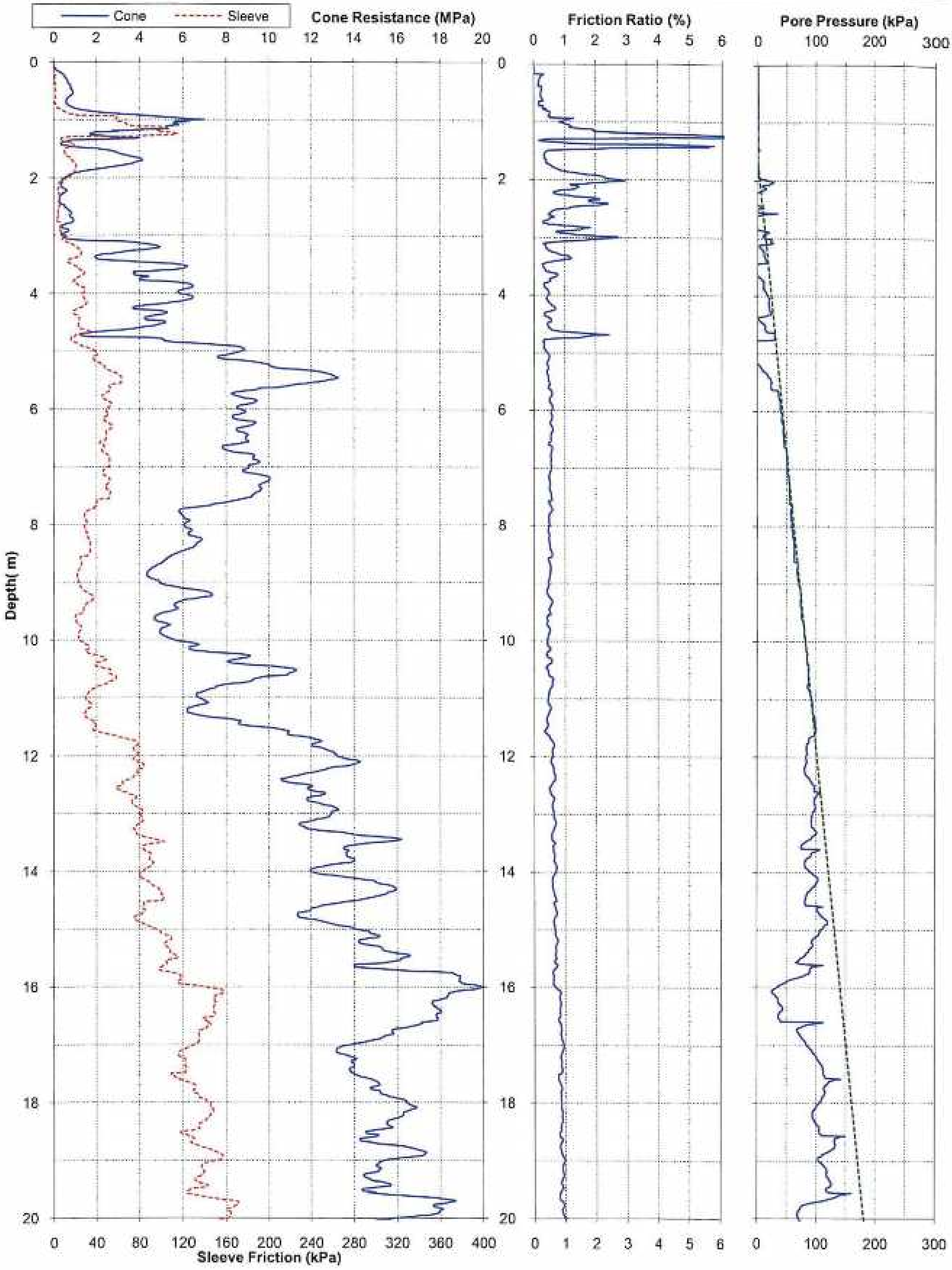
Water Level: Artesian

| Depth(m) | Full Drillers Description | Formation Code |
|---|---|---|
| - 78.0m | Yellow sand, gravel & shells | li |
| - 81.0m | Fine Brown gravel | li-3 |
| - 82.6m | Yellow sand & clay | he |
| - 90.2m | Grey sand & clay | he |
| - 91.4m | Rotten rock | he |
| - 111.5m | Blue clay | he-sh |
| - 111.8m | Clay & peat | sh |
| - 117.6m | Blue clay | sh |
| - 118.5m | Yellow clay | sh |
| - 122.2m | Yellow clay & rock | sh |
| - 126.0m | Yellow clay & sand | sh |
| - 126.7m | Yellow sand & gravel, rise 3.3m 104.8m3/d | wa |
| - 131.9m | Scoria | vo |
| - 132.8m | Black rock | vo |
| - 134.4m | Scoria | vo |
| - 136.2m | Hard rock | vo |
| - 136.8m | Scoria | vo |
| - 139.4m | Very hard rock | vo |

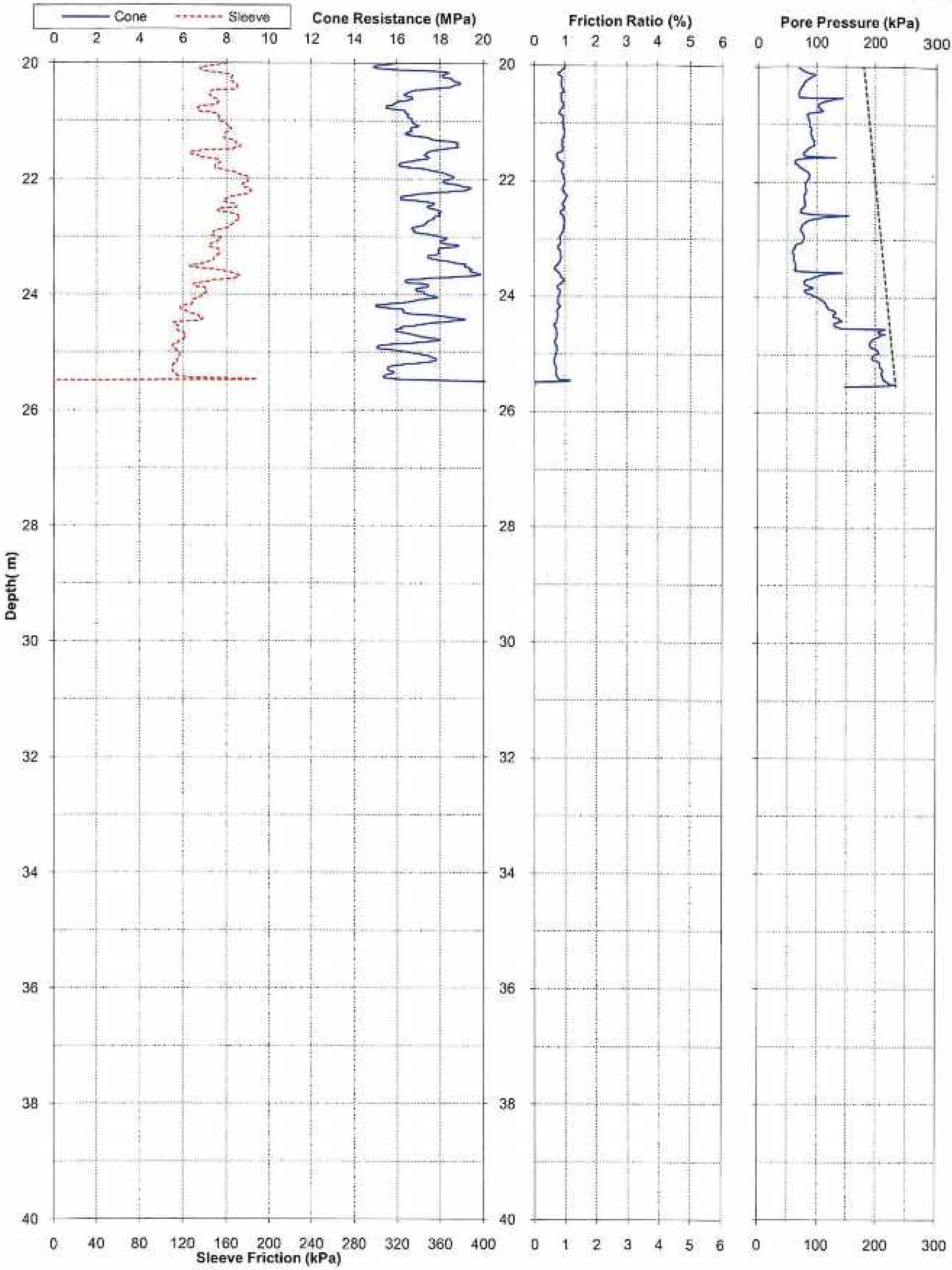
Scale(m): -70, -80, -90, -100, -110, -120, -130

**Appendix C:**
**CPT-RCL-02 and CPTu-01 to 04**

| Project: | Darfield 2010 Earthquake - EQC Ground Investigations | | | | Page: 1 of 2 | CPT-RCL-02 |
|---|---|---|---|---|---|---|
| Test Date: | 6-Dec-2010 | Location: | Redcliffs | Operator: | Opus | |
| Pre-Drill: | 1.2m | Assumed GWL: | 1.6mBGL | Located By: | Survey GPS | |
| Position: | 2488857.6mE | 5738882mN | 1.72mRL | Coord. System: | NZMG & MSL | |
| Other Tests: | | | | Comments: | | |



| Project: | Darfield 2010 Earthquake - EQC Ground Investigations | | | Page: 2 of 2 | CPT-RCL-02 |
|---|---|---|---|---|---|
| Test Date: | 6-Dec-2010 | Location: | Redcliffs | Operator: | Opus |
| Pre-Drill: | 1.2m | Assumed GWL: | 1.6mBGL | Located By: | Survey GPS |
| Position: | 2488857.6mE | 5738882mN | 1.72mRL | Coord. System: | NZMG & MSL |
| Other Tests: | | | | Comments: | |




T-T Ref: 51731.001

Printed: 14/01/2011 11:11 a.m.

Template: CPT Graph Template v0.41.xls

## CPT ANALYSIS NOTES

### Soil Type

Interpretation using chart of Robertson & Campanella (1983). This is a simple but well proven interpretation using cone tip resistance ($q_C$) and friction ratio ($f_R$) only. No normalisation for overburden stress is applied. Cone tip resistance measured with the piezocone is corrected with measured pore pressure ($u_C$).

- sand (and gravel)
- silt-sand
- silt
- clay-silt
- clay
- peat

### Liquefaction Screening

The purpose of the screening is to highlight susceptible soils, that is sand and silt-sand in a relatively loose condition. This is not a full liquefaction risk assessment which requires knowledge of the particular earthquake risk at a site and additional analysis. The screening is based on the chart of Shibata and Teparaksa (1988).

- high susceptibility
- medium susceptibility
- low susceptibility

High susceptibility is here defined as requiring a shear stress ratio of 0.2 to cause liquefaction with $D_{50}$ for sands assumed to be 0.25 mm and for silty sands to be 0.05 mm.

Medium susceptibility is here defined as requiring a shear stress ratio of 0.4 to cause liquefaction with $D_{50}$ for sands assumed to be 0.25 mm and for silty sands to be 0.05 mm.

Low susceptibility is all other cases.

### Relative Density ($D_R$)

Based on the method of Baldi et. al. (1986) from data on normally consolidated sand.

### Undrained Shear Strength ($S_U$)

Derived from the bearing capacity equation using $S_U = (q_C - \sigma_{VO})/15$.

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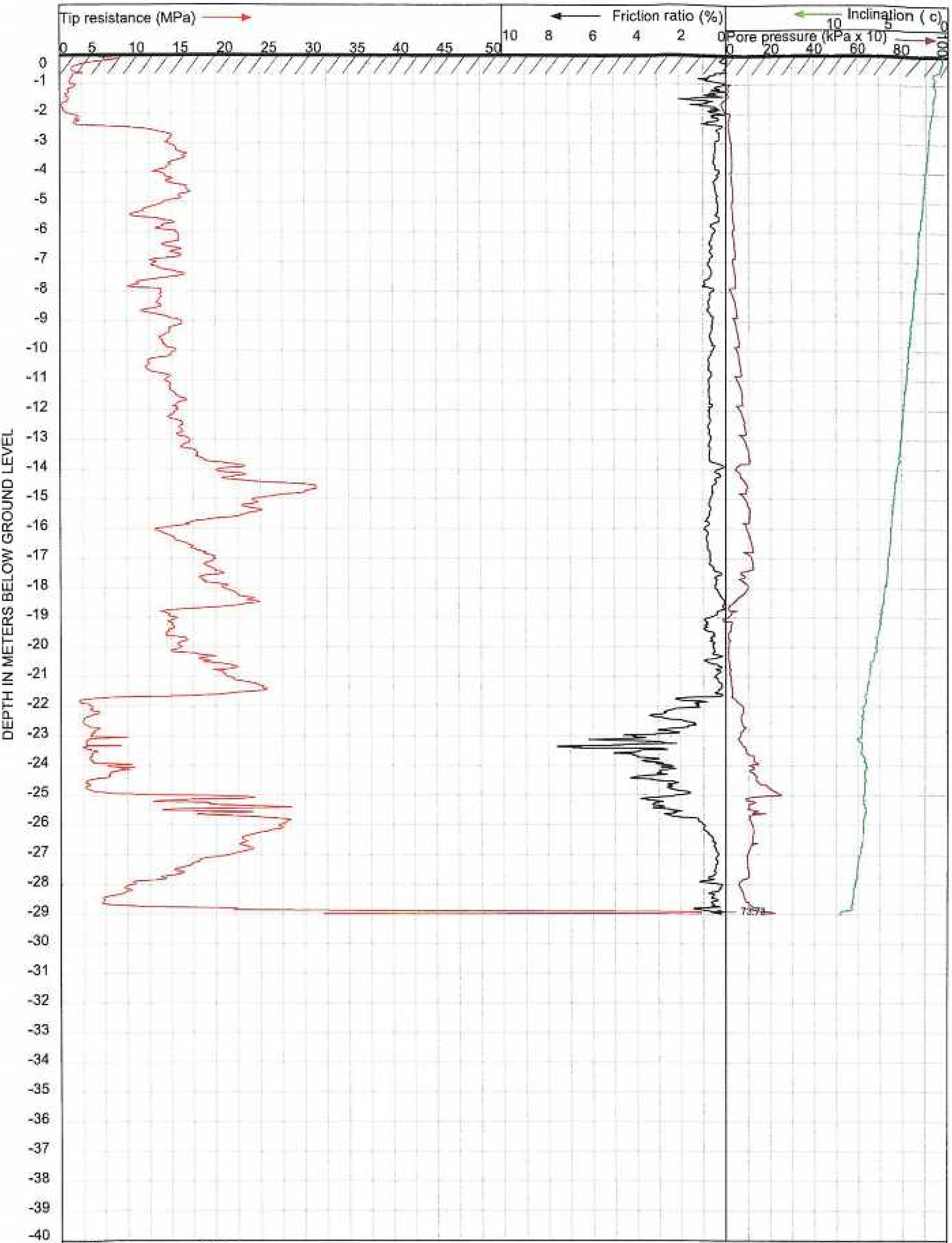


CLIENT : SCIRT
LOCATION : Beachville Rd & Celia St, Christchurch
DATE : 22-3-2012
OPERATOR : N.Barnes
REMARK 1 : 10824 - CPTu - 01
REMARK 2 : Effective Refusal

JOB # : 10343
TEST # : 1

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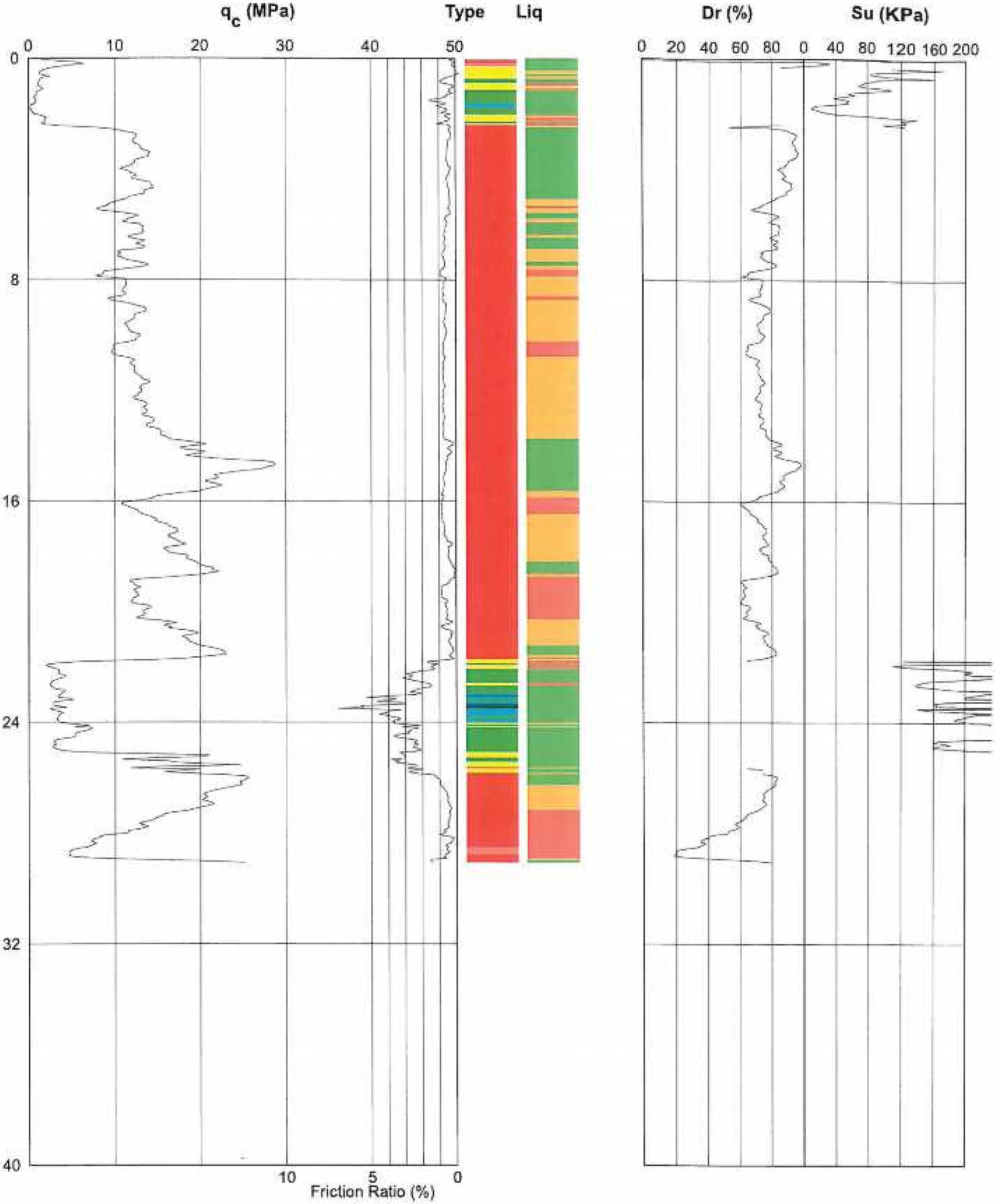
www.drilling.co.nz

# PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT




Job No: 10343

CPT No: 10824 - CPTu - 01

Project: SCIRT

Location: Beachville Rd & Celia St, Christchurch

Date: 22-3-2012

Operator: N Barnes

Remark: Effective Refusal

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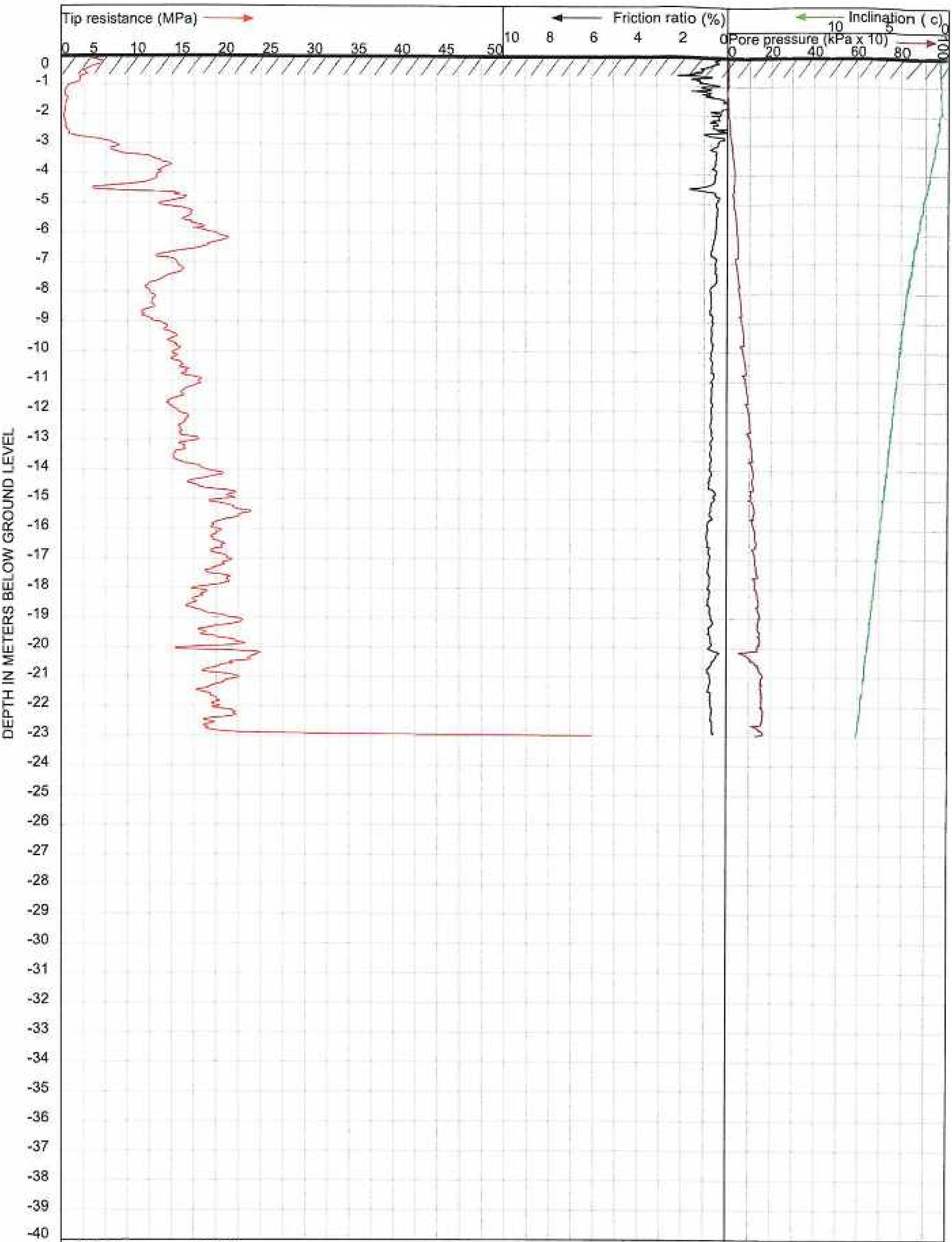


| | |
|---|---|
| CLIENT | : SCIRT |
| LOCATION | : Beachville Rd & Celia St, Christchurch |
| DATE | : 23-3-2012 |
| OPERATOR | : N.Barnes |
| REMARK 1 | : 10824 - CPTu - 02 |
| REMARK 2 | : Effective Refusal |

JOB # : 10343
TEST # : 2

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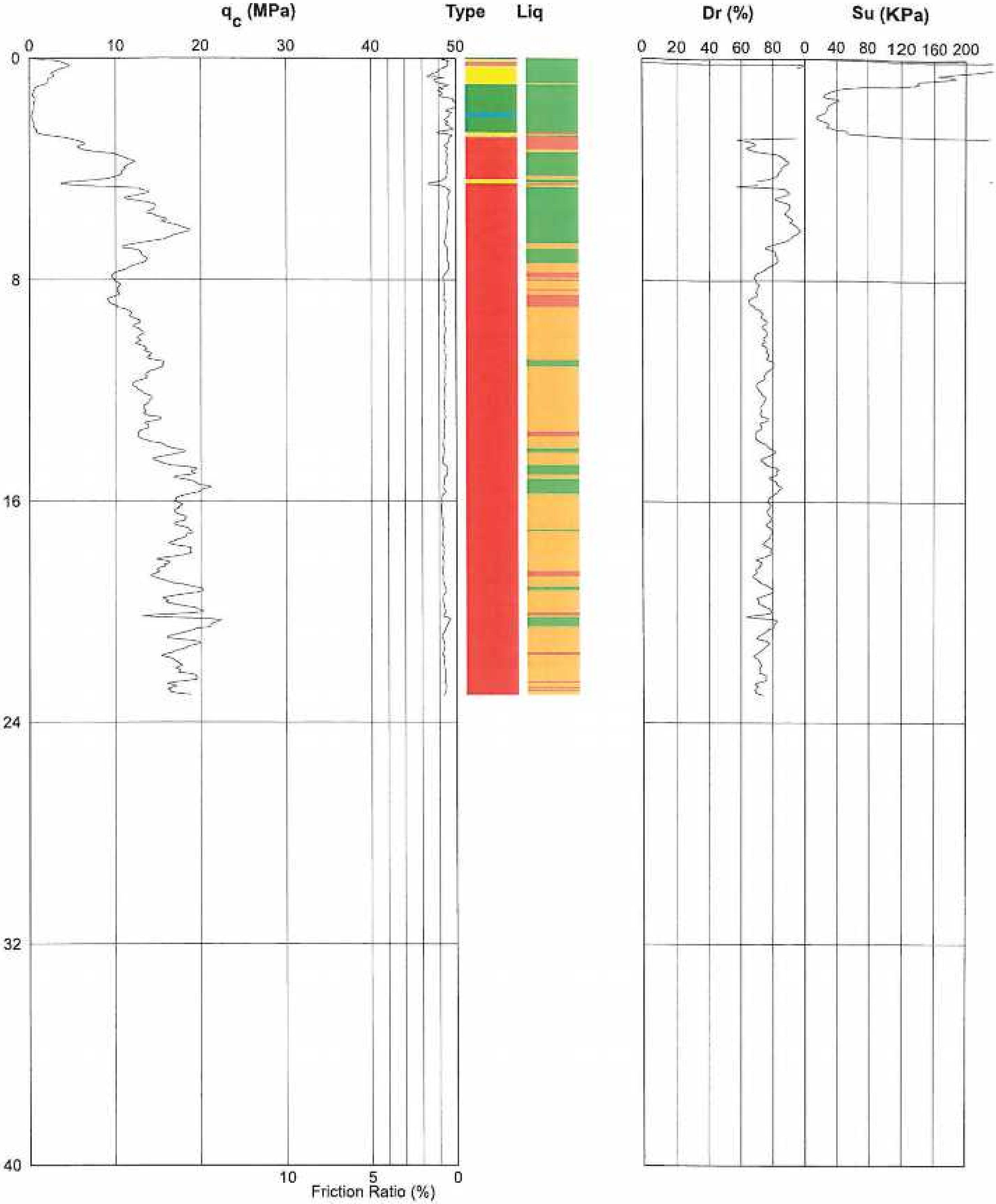
## PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT

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|---|---|---|---|
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| CPT No: | 10824 - CPTu - 02 | Operator: | N Barnes |
| Project: | SCIRT | Remark: | Effective Refusal |
| Location: | Beachville Rd & Celia St, Christchurch | | |

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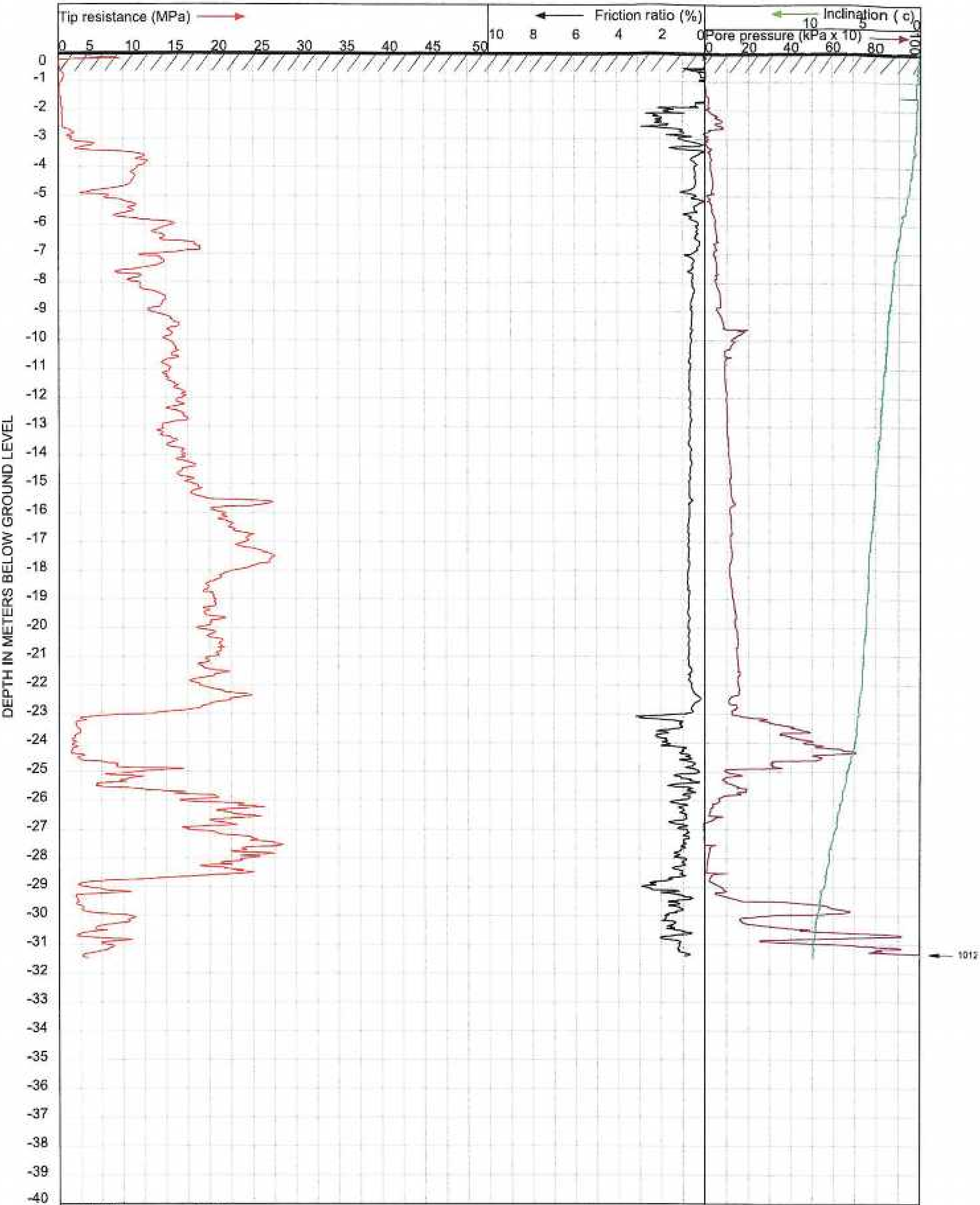


CLIENT : SCIRT
LOCATION : Beachville Rd & Celia St, Christchurch
DATE : 23-3-2012
OPERATOR : N.Barnes
REMARK 1 : 10824 - CPTu - 03
REMARK 2 : Effective Refusal

JOB # : 10343
TEST # : 3

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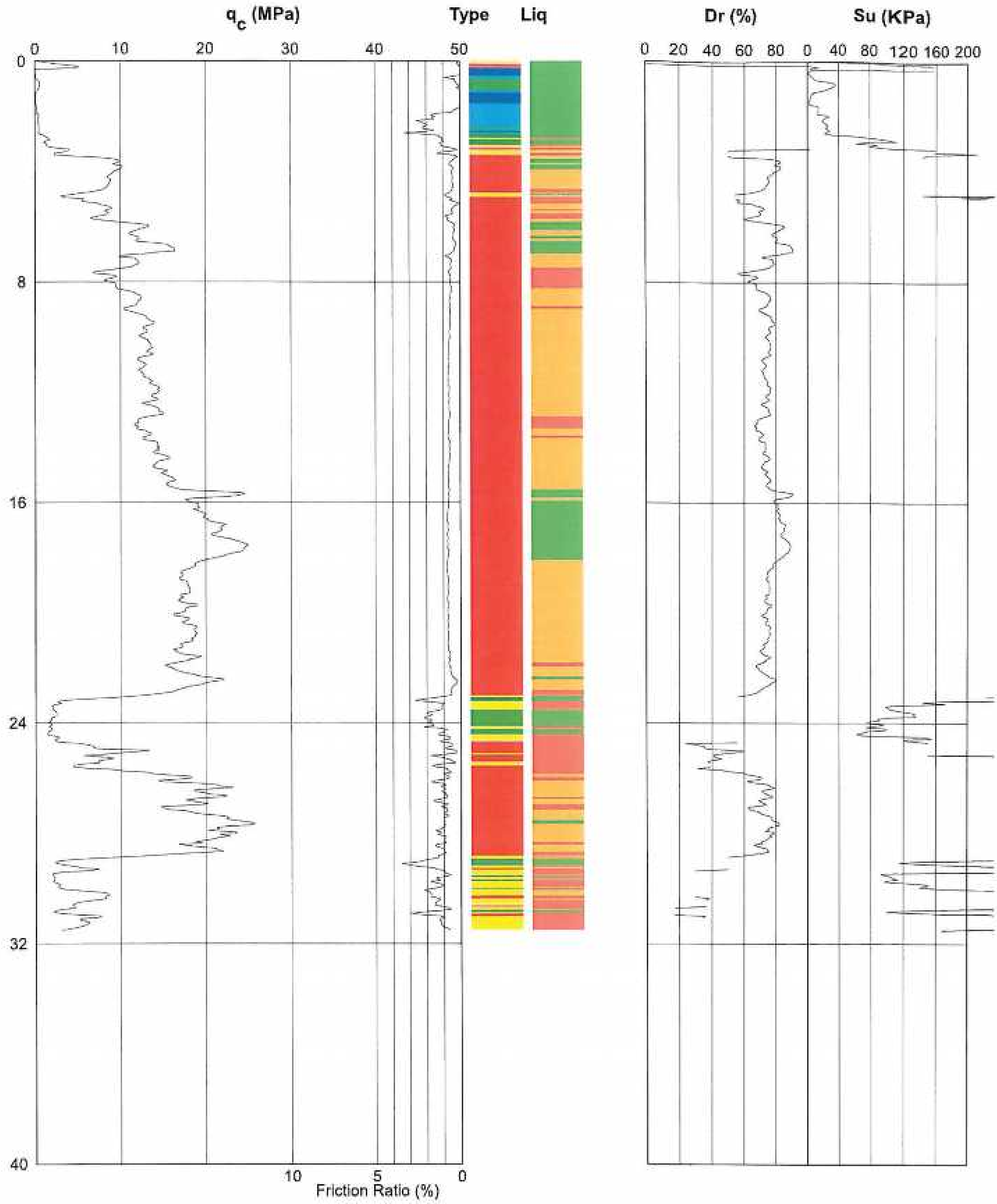
# PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT




| | | | |
|---|---|---|---|
| Job No: | 10343 | Date: | 23-3-2012 |
| CPT No: | 10824 - CPTu - 03 | Operator: | N Barnes |
| Project: | SCIRT | Remark: | Effective Refusal |
| Location: | Beachville Rd & Celia St, Christchurch | | |

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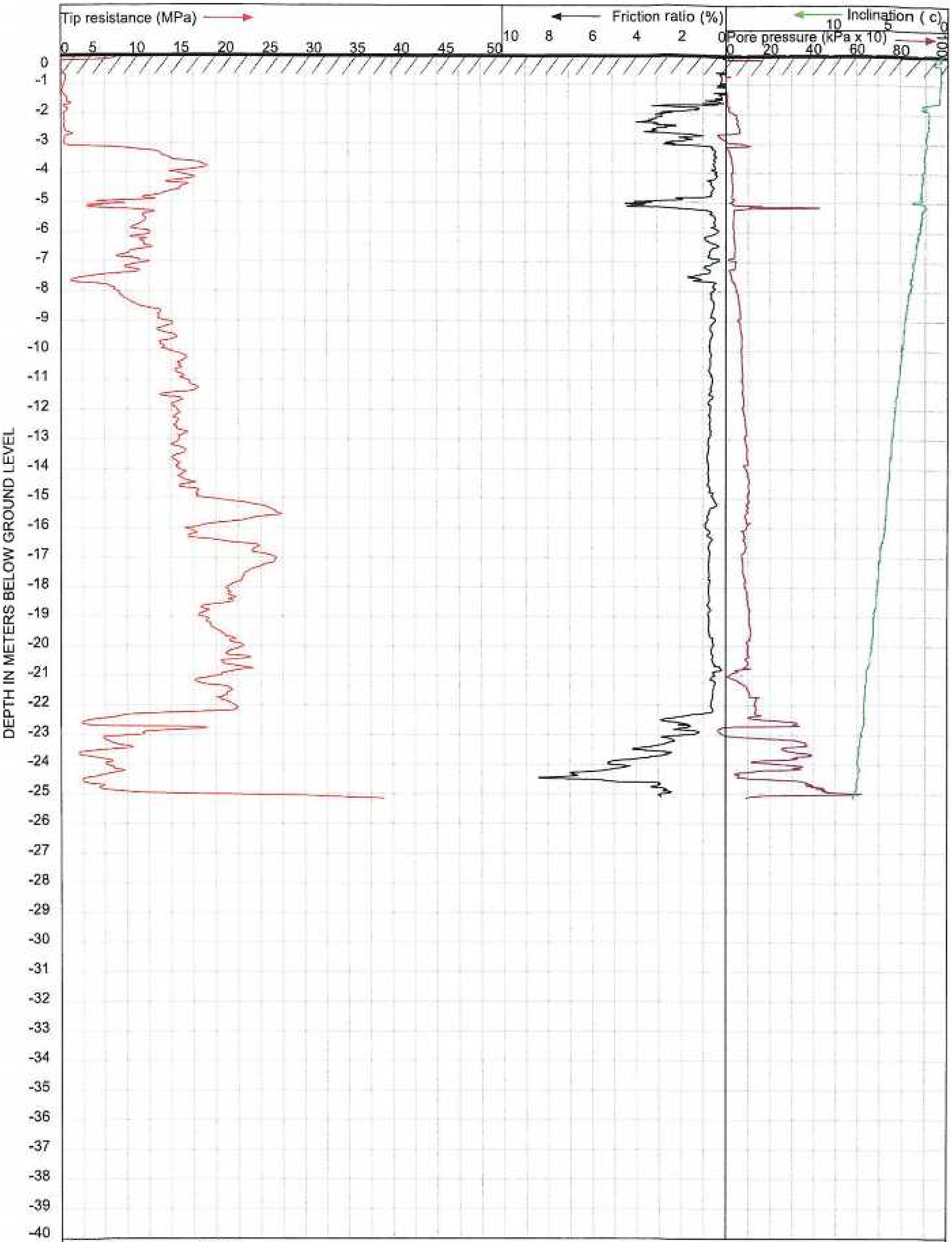


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LOCATION : Beachville Rd & Celia St, Christchurch
DATE : 23-3-2012
OPERATOR : N.Barnes
REMARK 1 : 10824 - CPTu - 04
REMARK 2 : Effective Refusal

JOB # : 10343
TEST # : 4

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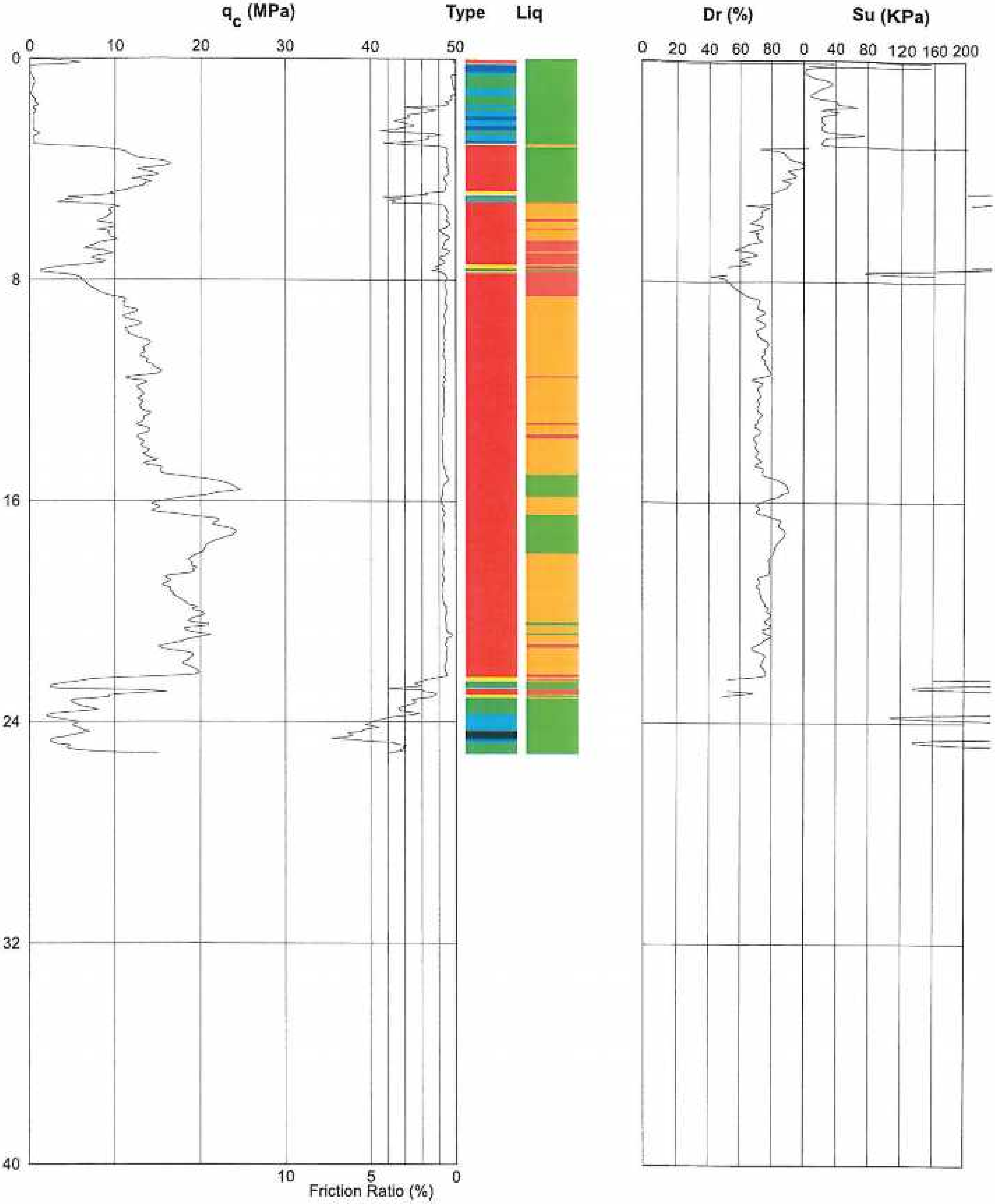
# PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT

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|---|---|---|---|
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| CPT No: | 10824 - CPTu - 04 | Operator: | N Barnes |
| Project: | SCIRT | Remark: | Effective Refusal |
| Location: | Beachville Rd & Celia St, Christchurch | | |

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## CIVIL CONSTRUCTION OVERVIEW

- 5 x Piling Rigs (20 to 80 tonne);
- 4 x Tieback/Micro-Piling Rigs (0.5 to 20 tonne);
- Sheet Piling & Injection Grouting;
- Dewatering;
- 26 x Drilling Rigs Company wide.

### A NEW ZEALAND FIRST METHOD – INTRODUCED TO THE MARKET BY MCMILLAN'S:

**Provisionally Patented Vibration Free Stone Column Method:**

- Can be used next to sensitive buildings;
- No mess (dry);
- Cost effective (minimal setup times);
- Further savings possible for building construction – i.e. ground beams, deep rafts, pile starters, boxing to piles;
- No corrosion issues, all natural materials;
- Reliance on individual piles, and the risk of differential settlement is reduced.

**Fully Instrumented Continuous Flight Auger / Displacement Auger Piling:**

- Cost effective;
- Sizes 350mm to 900mm and 19m depth;
- Fast (150m of 600mm diameter reinforced concrete pile can be installed per day);
- Lateral load capacity of RC piles exceed some other piling methods;
- Quiet & vibration free;
- Fully reinforced concrete piles, with no corrosion issues.

### McMILLAN'S ALSO OFFER THE FOLLOWING SERVICES:

- Screw Piles;
- Conventional Bored Concrete Piles;
- Mini & Micro Piles;
- Retaining Walls;
- Sheet Piling;
- Anchors & Tiebacks.

Please contact us to find out more information or visit our website www.drilling.co.nz

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**Appendix D:**
**Crack Monitoring Photos and Results**


Monitoring Station 1

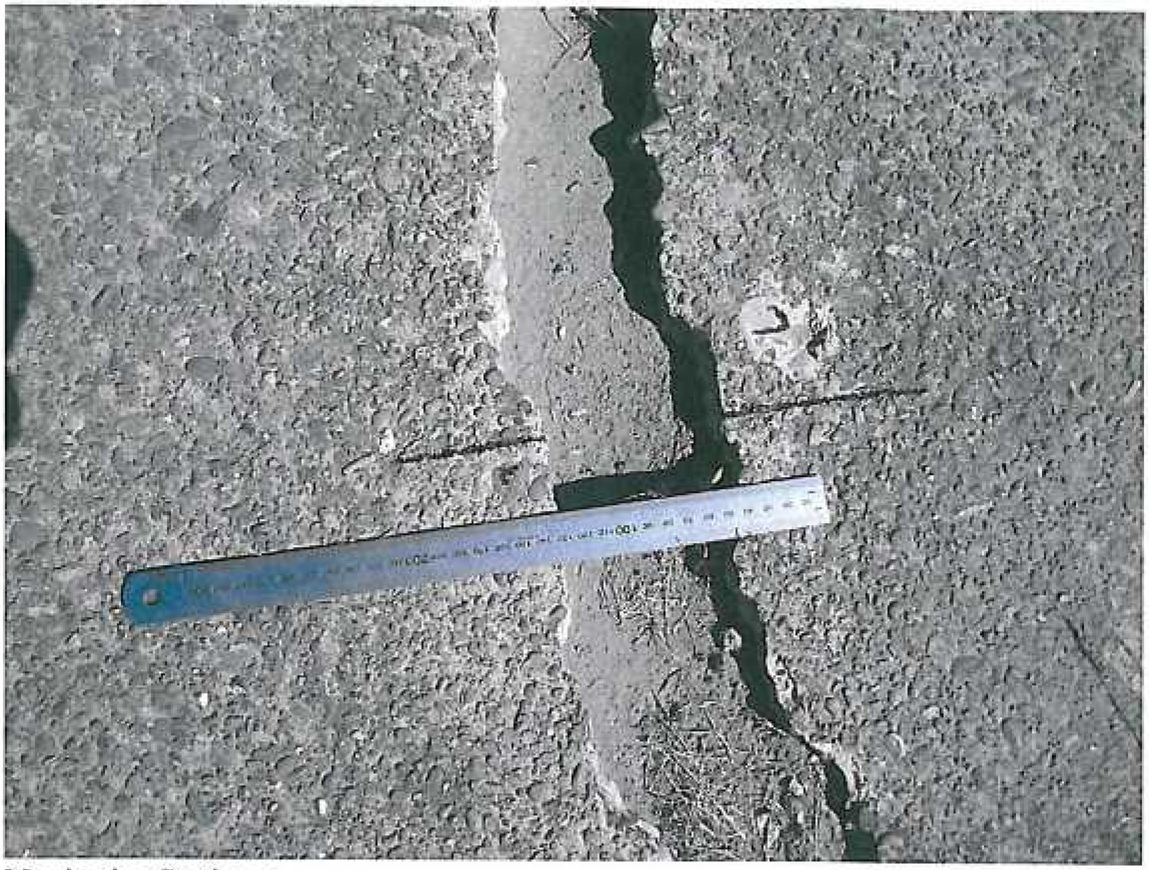

Monitoring Station 2

Monitoring Station 3

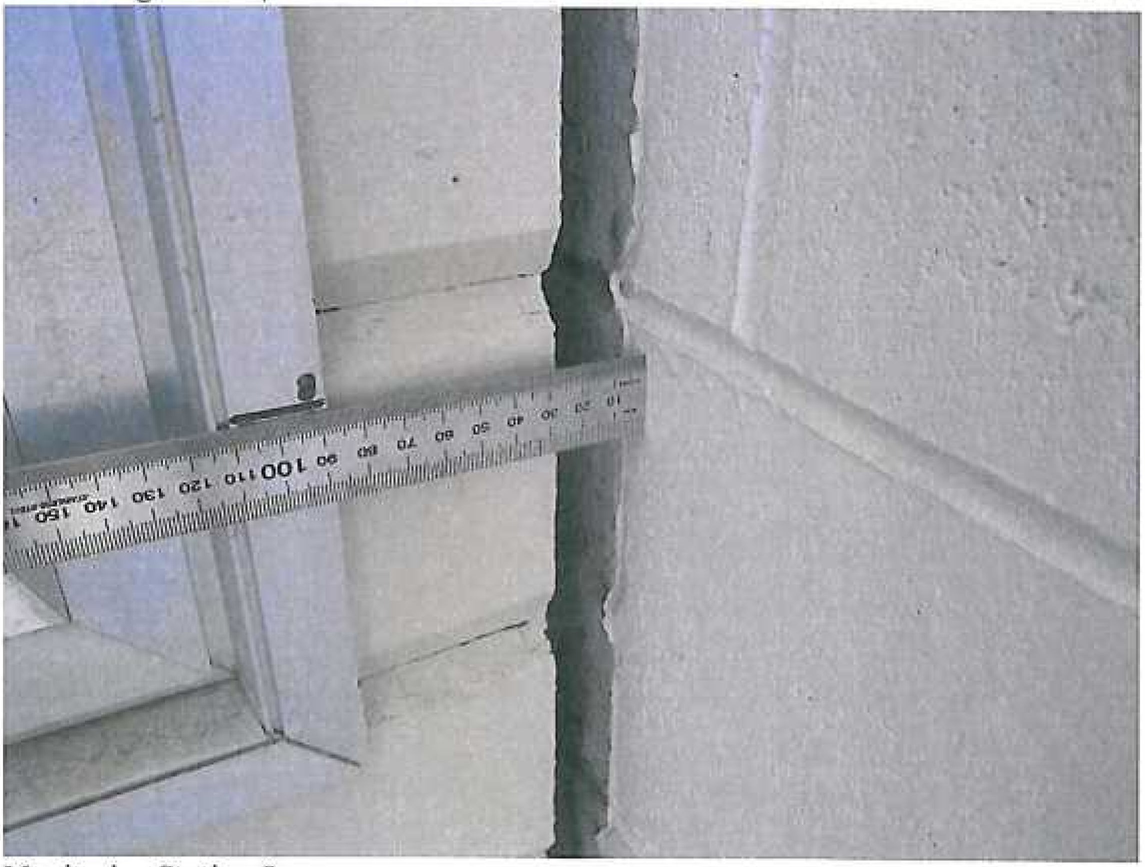
Monitoring Station 4

Monitoring Station 5

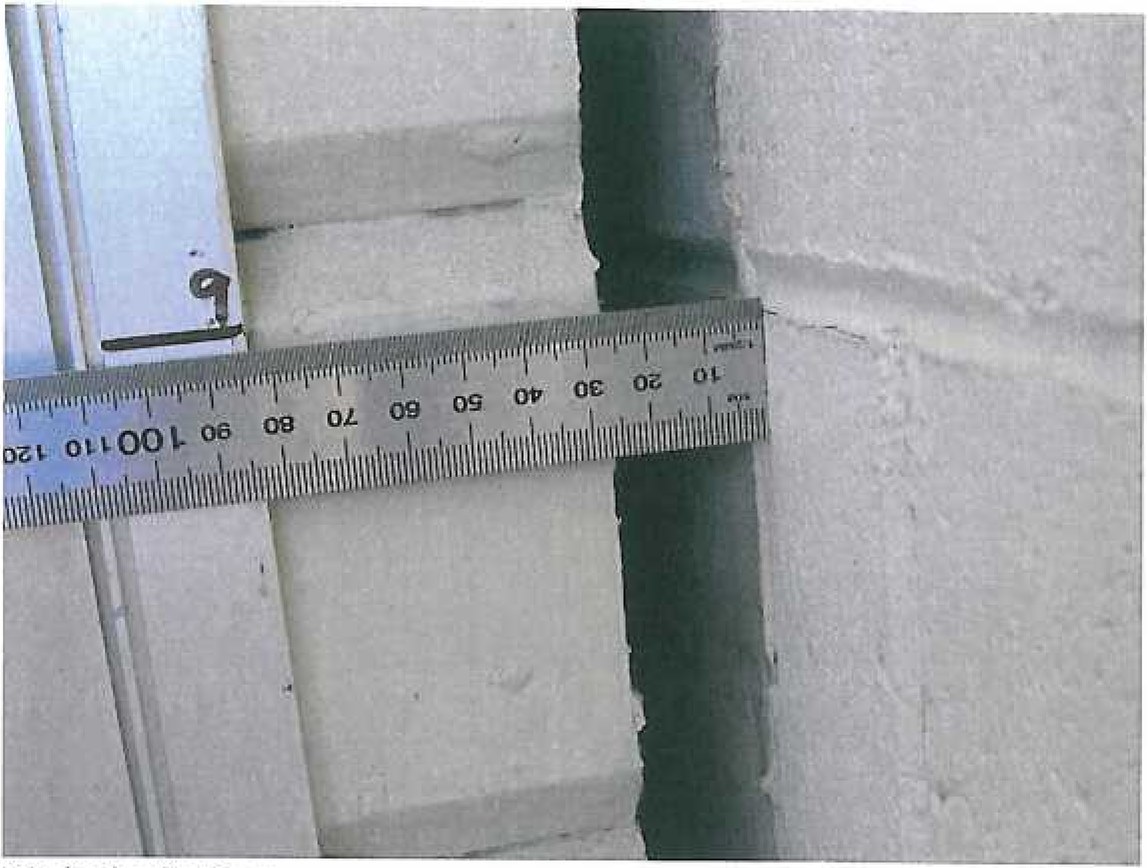
Monitoring Station 6



Monitoring Station 7



Monitoring Station 8


Monitoring Station 9

Noname

# Avonheath Court, 68A Beachville Road

Monitoring Surveys

| MARK | | DATE / SURVEY No | 3/09/2011 | 10/09/2011 | 15/09/2011 | 24/09/2011 | 1/07/2011 | 11/07/2011 | 18/07/2011 | 16/04/2011 | 20/04/2012 | 30/10/2012 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Station 1 N - S offset | mm | | 0.015 | 0.015 | 0.021 | 0.021 | 0.021 | 0.021 | 0.021 | 0.021 | 0.023 | 0.023 |
| Station 1 E - W Offset | mm | | 0.016 | 0.016 | 0.020 | 0.019 | 0.019 | 0.020 | 0.020 | 0.020 | 0.020 | 0.020 |
| Station 2 N - S | mm | | 0.007 | 0.007 | 0.011 | 0.011 | 0.011 | 0.011 | 0.011 | 0.012 | 0.011 | 0.011 |
| Station 3 N - S peg (reestablished 30/10/2012) | mm | | 0.008 | 0.009 | 0.050 | 0.051 | 0.050 | 0.050 | 0.050 | 0.048 | NA | 0.034 |
| Station 4 conc path | mm | | 0.061 | 0.060 | 0.065 | 0.064 | 0.063 | 0.063 | 0.063 | 0.063 | 0.062 | 0.062 |
| Station 5 peg (reestablished 30/10/2012) | mm | | 0.840 | 0.841 | 0.873 | 0.874 | 0.873 | 0.873 | 0.873 | 0.831 | NA | 0.888 |
| Station 6 peg | mm | | 0.704 | 0.705 | 0.731 | 0.736 | 0.732 | 0.730 | 0.732 | 0.730 | 0.721 | 0.721 |
| Station 7 path (reestablished 2/9/11) | mm | | 0.073 | 0.074 | N/A | 0.090 | 0.091 | 0.091 | 0.092 | 0.092 | 0.093 | 0.093 |
| Station 8 wall unit 4 | mm | | 0.016 | 0.016 | 0.025 | 0.024 | 0.023 | 0.023 | 0.023 | 0.023 | 0.023 | 0.023 |
| Station 9 unit 3 | mm | | 0.021 | 0.021 | 0.026 | 0.026 | 0.024 | 0.025 | 0.025 | 0.025 | 0.025 | 0.025 |

Movement since Previous Survey (mm)

| MARK | 30/08/2011 | 13/09/2011 | 15/09/2011 | 24/09/2011 | 1/07/2011 | 11/07/2011 | 18/07/2011 | 16/04/2011 | 20/04/2012 | 30/10/2012 |
|---|---|---|---|---|---|---|---|---|---|---|
| Station 1 N - S offset | BASE | 0.0 | 6.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 2.0 | 0.0 |
| Station 1 E - W Offset | BASE | 0.0 | 4.0 | -1.0 | 0.0 | 1.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| Station 2 N - S | BASE | 0.0 | 3.5 | 0.5 | 0.0 | 0.0 | 0.0 | 1.0 | -1.0 | 0.0 |
| Station 3 N - S peg (reestablished 30/10/2012) | BASE | 1.0 | 41.0 | 1.0 | -1.0 | 0.0 | 0.0 | -2.0 | | |
| Station 4 conc path | BASE | -1.0 | 15.0 | -1.0 | -1.0 | 0.0 | 0.0 | 0.0 | -1.0 | 0.0 |
| Station 5 peg (reestablished 30/10/2012) | BASE | 1.0 | 32.0 | 1.0 | -1.0 | 0.0 | 0.0 | -42.0 | | |
| Station 6 peg | BASE | 1.0 | 26.0 | 5.0 | -4.0 | -2.0 | 2.0 | -2.0 | -9.0 | 0.0 |
| Station 7 path (reestablished 2/9/11) | BASE | 1.0 | | | -1.0 | 0.0 | 1.0 | 0.0 | -2.0 | 0.0 |
| Station 8 wall unit 4 | BASE | 0.0 | 9.0 | -1.0 | -1.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| Station 9 unit 3 | BASE | 0.0 | 5.0 | 0.0 | -2.0 | 1.0 | 0.0 | 0.0 | 0.0 | 0.0 |

Movement since Base Survey (mm)

| MARK | 30/08/2011 | 10/09/2011 | 15/09/2011 | 24/09/2011 | 1/07/2011 | 11/07/2011 | 18/07/2011 | 16/04/2011 | 20/03/2012 | 30/10/2012 |
|---|---|---|---|---|---|---|---|---|---|---|
| Station 1 N - S offset | BASE | 0.0 | 6.0 | 6.0 | 6.0 | 6.0 | 6.0 | 6.0 | 8.0 | 8.0 |
| Station 1 E - W Offset | BASE | 0.0 | 4.0 | 3.0 | 3.0 | 4.0 | 4.0 | | | |
| Station 2 N - S | BASE | 0.0 | 3.5 | 4.0 | 4.0 | 4.0 | 4.0 | 5.0 | 4.0 | 3.0 |
| Station 3 N - S peg (reestablished 30/10/2012) | BASE | 1.0 | 42.0 | 43.0 | 42.0 | 42.0 | 42.0 | 40.0 | | |
| Station 4 conc path | BASE | -1.0 | 14.0 | 13.0 | 12.0 | 12.0 | 12.0 | 12.0 | 11.0 | 11.0 |
| Station 5 peg (reestablished 30/10/2012) | BASE | 1.0 | 33.0 | 34.0 | 33.0 | 33.0 | 33.0 | 21.0 | | |
| Station 6 peg | BASE | 1.0 | 27.0 | 32.0 | 28.0 | 26.0 | 28.0 | 26.0 | 17.0 | 17.0 |
| Station 7 path (reestablished 2/9/11) | BASE | 1.0 | | | 18.0 | 18.0 | 19.0 | 19.0 | 17.0 | 17.0 |
| Station 8 wall unit 4 | BASE | 0.0 | 9.0 | 8.0 | 7.0 | 7.0 | 7.0 | 7.0 | 7.0 | 7.0 |
| Station 9 unit 3 | BASE | 0.0 | 5.0 | 5.0 | 3.0 | 4.0 | 4.0 | 4.0 | 4.0 | 4.0 |

Data by: C.Brown Checked By: G.Brown Date: 1907

Date Printed 22 Nov 2012

OPUS

**Appendix E:**
**Proposed Ground Investigation Plan**


| | | | |
|---|---|---|---|
| OPUS Opus International Consultants Ltd, Christchurch Office, 20 Moorhouse Ave, PO Box 1482, Christchurch, New Zealand, Tel: +64 3 363 5400 Fax: +64 3 365 7857 | **Project:** | Avonheath Courts, 86A Beachville Road<br>Geotechnical Desk Study | **Proposed Site Investigation Plan - NTS** |
| | **Project No.:** | 6-QUCC2.02 | **Drawn:** Geotechnical Engineer |
| | **Client:** | Christchurch CC | **Date:** 2nd November2012 |

**Opus International Consultants Ltd**
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PO Box 1482, Christchurch Mail Centre,
Christchurch 8140
New Zealand

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f: +64 3 365 7858
w: www.opus.co.nz