Whakahoa Village Units Block A

BU 2680-001 EQ2

Detailed Engineering Evaluation

Quantitative Report

Christchurch City Council

Christchurch City Council

Whakahoa Village Block A

Detailed Engineering Evaluation Quantitative Report

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Whakahoa Village Block A
BU 2680-001 EQ2

Detailed Engineering Evaluation
Quantitative Report - SUMMARY
Final

Gowerton Place, Richmond

Background

This is a summary of the quantitative report for the Whakahoa Village Block A building, and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on 21 and 25 March 2012, available drawings and calculations.

Key Damage Observed

The common area stairwell and associated level 2 landing/walkway has pulled away at the western building support. There were also obvious indications of ground movement around the building evidenced by fissuring and ejected silt associated with liquefaction. The building has been subjected to significant differential settlement.

Critical Structural Weaknesses

No critical structural weaknesses have been identified.

Indicative Building Strength

When subject to the current new building standard (NBS) seismic design forces, the reinforced masonry shear walls have a capacity greater than 100% NBS in the north-south direction. However, in the east-west direction, the reinforced masonry shear walls have a capacity of greater than 67% NBS.

Based on the information available, and from undertaking a quantitative assessment, the structural wall along grid AE and the adjacent timber framing bolted to the wall have been severely damaged due to out-of-plane displacements caused by liquefaction. This wall is sufficiently long and is stabilised at its top by a link slab hence it retains sufficient residual in-plane capacity to act as a shear wall, but it must be repaired as the attached block veneer is now unstable.

The building in its current state has been assessed to have a post-earthquake seismic capacity of greater than 67% NBS.

Recommendations

It is recommended that:

a) The upper level units 44-47 as well as ground floor unit 41 remain unoccupied.
b) The central stairs should be cordoned off on the both sides of the building out a radial distance of 6.0 metres. This will remove the vehicular access to unit 42. The end stairs should be cordoned off to prevent access.
c) The damaged masonry wall to grid AE at first floor level and connected timber structure is repaired.
d) Remove carpets and investigate the ground floor slab for cracking.
e) Two hand auger/scalas should be undertaken surrounding the west side of Block A to assess the bearing capacity of the underlying material.
f) Six Cone Penetrometer Tests to a depth of 25m be undertaken to enable a site wide liquefaction assessment.

g) If the site is assessed to be equivalent to the DBH Technical Category 3, in accordance with the interim guidance, a foundation re-level is likely for the affected units, particularly units 42 and 43, at Whakahoa Village. CCC will need to accept that more damage to the existing concrete slab foundations is likely in future seismic events. Further investigations will be required for detailed design.

Contents

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council (CCC) to undertake a detailed seismic assessment of the Whakahoa Village Block A, located at Gowerton Place, Richmond, following the M6.3 Christchurch earthquake on 22 February 2011.

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 quantitative procedures detailed in the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011.

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.

Any building with a capacity of less than 34% of new building standard (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67% as required by 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).

Section 115 – Change of Use

This section requires that the territorial authority (in this case Christchurch City Council (CCC)) 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 CCC as being 67% of the strength of an equivalent new building. 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.

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:

- 36% increase in the basic seismic design load for Christchurch (Z factor increased from 0.22 to 0.3);
- 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). It is noted that the current seismic risk in Christchurch results in a 6% risk of exceedance 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

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

they are made aware of our assessment. Based on information received from CERA to date, 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/Christchurch City Council 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.

4 Building Description

4.1 General

The Whakahoa Village Block A building is a double storey reinforced masonry building with precast concrete 1st floor and a pitched timber framed roof. The building sits on reinforced concrete strip footings.

Figure 2 – Location of Block A on site

The building comprises eight residential apartments and is 36.8m long in the east-west direction and 10.4m wide in the north-south direction. The first floor is approximately 3m from the ground while the apex of the roof is approximately 6.8m from the ground. The first floor structure consists of two large precast concrete floor diaphragms with a central staircase. The two floors are linked by an in-situ reinforced concrete walkway. The roof construction consists of two timber framed roof structures linked by a precast concrete roof over the central staircase.

The building construction was completed in 2007.

4.2 Gravity Load Resisting System

The ground floor construction is a 100mm thick in situ concrete ground bearing slab.

As noted on the plans, the suspended first floor is constructed from a 75mm Unispan precast concrete floor slab with a 75mm thick in situ reinforced concrete topping. The insitu topping has one layer of 665 mesh and typically has D10 reinforcing bars at 600mm centres into the walls. This floor is supported on solid filled reinforced concrete masonry walls which are supported on strip footing foundations.

The stairs are shown on the structural drawings to be precast concrete with a 150mm thick waist.

The roof is a timber framed roof clad in lightweight profiled metal roof sheeting, with a plaster board ceiling, supported on the reinforced concrete masonry walls.

4.3 Seismic Load Resisting System

Seismic loads in both principal directions are resisted by the reinforced concrete masonry walls. A number of these walls exist in both the east-west and the north-south directions at both levels. To distribute the lateral loads to the walls, the in situ reinforced concrete topping forms a rigid diaphragm at first floor, while the roof/ceiling provides a flexible timber/plasterboard diaphragm.

5 Survey

The building currently has a red placard on Unit 45, indicating that the unit was potentially severely damaged and is unsafe for occupancy, due to the visible cracking and rotation of the western wall to the stair core.

Opus has undertaken a level survey of both the ground floor and the first floor.

Copies of the following drawings were referred to as part of the assessment:

- Structural drawings by Powell Fenwick Consultants Limited titled “Gowerton Place Social Housing” (drawing numbers S1.1 – S1.7)
- Architectural drawings by City Solutions titles “Gowerton Place Social Housing” (drawing numbers WD01-01 – WD07-06)

No copies of the design calculations have been obtained for this building.

The drawings have been used to ascertain the structural systems, investigate potential critical structural weaknesses (CSW) wherever possible and identify details which required particular attention.

6 Damage Assessment

The western wall to the central stair core appears to have pulled away 50-70mm from the stair landing due to the settlement of one end of the building. Within the roof space on the internal face of this wall, the main timber roof rafter adjacent the wall has split where it was connected to the wall, which is compromising the gravity support of the roof structure in this location with some of the timber purlins not having adequate support. This damaged masonry wall at first floor level and the connected timber structure should therefore be repaired.

Typically, a masonry veneer wall exists to the external walls that supported on strip footings at ground level. In the central stair area, large cracks and displacements to the masonry veneer wall have been observed. In the same area some separation between the roof timbers and the ceiling plaster has occurred.

For the reasons outlined above, the upper level units 44 to 47 as well as the ground floor unit 41 should remain unoccupied and the central stairs should be cordoned off on the both sides of the building out a radial distance of 6.0 metres. The end stairs should be cordoned off to prevent access.

External to the building, obvious indications of ground movement exist such as fissuring and ejected silt associated with liquefaction. To properly assess any damage to the ground floor slab, carpets should be removed and the ground floor slab investigated for cracking.

Significant differential settlement exists throughout the entire building. The overall differential settlement is in the order of 130mm, while the maximum differential settlement over a 4m length is in the order of 65mm. This settlement exceeds the maximum allowable differential settlement of 25mm over 6m as specified in Clause B1 of the New Zealand Building Code. The building is fairly level to the east of the central stair, however there appears to be a significant fall to the west of the central stairs. The wall to the central stair core area appears to have been pulled towards the eastern end.

7 Detailed Seismic Assessment

7.1 Critical Structural Weaknesses

As outlined in the Critical Structural Weakness and Collapse Hazards draft briefing document, issued by the Structural Engineering Society (SESOC) on 7 May 2011, 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 the building.

No critical structural weaknesses have been identified.

7.2 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 D, clause 3.1.3 NZS 1170.5:2004;
- Site hazard factor, Z=0.3, B1/VM1 clause 2.2.14B;
- Return period factor R_u = 1.0 from Table 3.5, NZS 1170.5:2004, for an Importance Level 2 structure with a 50 year design life;
- μ_{max} = 2 and S_p = 0.7 (limited ductility) for flexure of reinforced concrete masonry walls.
- μ_{max} = 1.25 and S_p = 0.9 (nominal ductility) for shear in reinforced masonry walls.

7.3 Detailed Seismic Assessment Results

A summary of the structural performance of the building is 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 building may have significantly greater capacity when compared with the governing element.

Table 2: Summary of Seismic Performance

Structural Element/System	Failure mode and description of limiting criteria	% NBS based on calculated capacity
Walls in the east-west direction i.e. along the building	In-plane bending capacity of the reinforced masonry bracing walls	53-100% (>67% with redistribution)
Walls in the east-west direction i.e. along the building	In plane shear capacity of the reinforced masonry bracing walls	90%
Walls in the north-south direction i.e. across the building	In-plane bending capacity of the reinforced masonry bracing walls	>100%
Walls in the north-south direction i.e. across the building	In plane shear capacity of the reinforced masonry bracing walls	>100%
1st floor diaphragm	Capacity of the 75mm reinforced concrete topping to act as a diaphragm	>100%

7.4 Discussion of Results

The reinforced concrete masonry wall to grid AE has undergone 50-70mm of lateral displacement at roof level and the attached block veneer has been displaced resulting in reduced stability. The permanent deformation to this wall has occurred out-of-plane to the east, while the level survey shows the structure to the west of this wall having a significant fall away from the wall. This suggests that the structure to the east of the wall has settled, pulling the wall with it and subjecting the wall to significant out of plane bending forces beyond its design capacity. The wall is stable but there is a risk of further damage from liquefaction settlements at the site. This damage is not likely to affect the overall stability of the building, but does represent a local falling hazard.

When subject to an earthquake in the east-west direction, the walls in this same direction have a calculated minimum capacity of 53% NBS based on wall forces calculated with elastic stiffness. With allowance for some limited redistribution of loads between walls the actual capacity is estimated to be greater than 67%NBS. This capacity is based upon an assumption of limited ductility reinforced masonry walls. There is no evidence of any damage to the east-west walls due to in-plane seismic forces.

The area of walls in the north-south direction are significantly larger and has a calculated capacity greater than 100% NBS.

Although not specifically detailed, the plasterboard ceiling at first floor level could be expected to act satisfactorily as a diaphragm.

7.5 Limitations and Assumptions in Results

With the exception of the local assessment of the deformed wall to grid AE, our analysis and assessment is based on an assessment of the building in its undamaged state.

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:

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

8 Geotechnical Assessment

The full geotechnical assessment completed by Opus is included in this report as Appendix 3. A summary of the geotechnical report is as follows:

8.1 General

The Whakahoa Village Residential Housing Units are situated approximately 2km north-east of Christchurch City in the suburb of Richmond. It is a relatively flat site, approximately 220m north-west of the Avon River.

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

This Geotechnical Desk Study has been prepared in accordance with the Engineering Advisory Group's Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Revision 5, 19 July 2011. This Geotechnical Desk Study forms parts 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.

8.2 Expected Ground Conditions

A review of the Environmental Canterbury (ECan) Wells database showed three wells located within approximately 100m of the property boundary. Material logs available from these wells in addition to EQC CPT tests have been used to infer the ground conditions at the site, as shown in the table below.

Stratigraphy	Thickness (m)	Depth Encountered from (m) below ground
TOPSOIL	0.2-0.5	0
SAND (not present in M35/16568)	0-0.75	0.2-0.5
SILT	1.2-1.5	0.5-1.0
SAND	20.3-21.0	1.8-2.5
Clayey SILT	1.5	22.8
GRAVELS (Riccarton)	-	24.3

The groundwater level was recorded as 1.2m-2.5m bgl in the borehole records.

8.3 Liquefaction Hazard

Examination of post-earthquake aerial photos taken by New Zealand Aerial Mapping (refer Project Orbit) identified evidence of significant quantities of liquefied soils ejected at the ground surface of the site after the 22 February 2011 and 13 June 2011 events but not after the 4 September 2010 or 23 December 2011 events.

A preliminary CLiq analysis has been performed using the CPT-RCH-37 and CPT-RCH-50 data sets located 100m south east and 170m west of the site, respectively. A summary of the results of the analysis are presented in the table below.

CPT	Distance from site boundary (m)	Direction	Event	Inferred Liquefiable Layers (bgl)	Total Liquefaction Induced Subsidence (mm)
CPT-RCH-37	100	South East	ULS (0.35g)	-Ground Water Level to 4m -Thin lenses at 7m and 11m	60
CPT-RCH-50	170	West	ULS (0.35g)	-Ground Water level to 7m -8.5m to 13.5m	190

8.4 Site Walkover Inspection

A walkover inspection of the exterior of the buildings of Block A to E and surrounding land was carried out by an Opus Geotechnical Engineer on 14 May 2012. The following observations were made:

- Significant heave in the pavement up to 300mm is evident around the Whakahoa Village buildings.
- Liquefaction ejected soil is located in several gardens around the Whakahoa Village complex.
- A door frame in unit 54 (Block C) appears to be skewed by 10mm, likely due to differential settlement of foundations.
- The concrete driveways of unit 43 (west end of Block A) and unit 54 (south end of Block C) have cracked and have been offset vertically by up to 20mm (relative to the driveway) and laterally by 10mm directly outside of the garage doors.
- Cracking and settlement has occurred at several locations in the paved footpath in-between Blocks A, B and C.
- Gaps of up to 50mm wide have formed on the north and south sides of Block A.
- The footpath along the east side of Block B has settled by approximately 20mm.
- The units located on the west side of the stairs in Block A appear to have differentially settled compared to the units on the east side.
- No evidence of differential settlement or cracking was observed around Blocks D and E.

8.5 Level Survey

A summary of the level survey undertaken by Opus Surveyors on 14 May 2012 are given in the table below. Refer to the level survey results in the geotechnical report.

Block	Unit	Differential Settlement [1,2]
A	40,41	50mm (centre)
A	42,43	120mm (west)
Notes: (1) Floor slab levels rounded to the nearest 10mm (2) Direction of fall indicated in brackets		

8.6 Discussion

Due to the ground motion during the seismic events, the lateral movement that Block A has undergone may have caused the soils to consolidate resulting in the gaps observed on both the north and south sides of Block A.

Due to the reinforced masonry block construction of the units, the structural form is not directly recognised in the DBH guidance document. Therefore, appropriate remedial solutions will be dependent on the integrity of the superstructure and liaison with the Structural Engineer.

No evidence of cracking in the floor slabs and perimeter footing were observed. Areas able to be inspected were limited due to the carpeted flooring and shrubbery.

The level survey results are consistent with the observations of differential settlement of the western units of Block A. Observations include pavement cracking of the driveway outside the western most unit (unit 43) separating from the stair well.

There is an open swale 10m east of Block A and B retained by a timber pole retaining wall. The depth of the swale invert relative to Whakahoa Village is approximately 1.5m. This open face represents a potential hazard for lateral spreading. The Avon River is located 150m south east of the Whakahoa Village. There has been no evidence of cracking on the site associated with lateral spreading.

The CLiq analysis based on the CPTs 170m west and 100m south east of the building indicated that there is possible total settlement of up to 190mm during an Ultimate Limit State seismic event. Liquefiable layers have been identified from the ground water level up to 13.0m bgl. The CPT results correlate with the observed differential settlement observed on site.

The differential settlement that appears to have occurred to Block A relative to the footpath may be attributed to a temporary loss of bearing capacity during the seismic shaking. Shallow investigations including hand augers and scalas should be undertaken to confirm the bearing capacity of the underlying material.

If the existing units are to be retained, a building consent will be necessary for remedial works. Remedial works will include re-levelling of Block A. Site specific investigations comprising of approximately 6 Cone Penetrometer Tests (CPT's) to a depth of 20m are recommended to be undertaken to enable a site wide liquefaction assessment and combined with shallow investigations to identify potential revelling/remedial solutions.

9 Remedial Options

The wall to grid AE and the damaged timber roof frame parallel to this wall must be repaired. The designer should assess the methodology and reinstatement of the wall's connection to the concrete roof and consider some form of articulation in view of the potential for future liquefaction.

Re-levelling of the two-storey Block A building should only be undertaken if necessary, as it could result in further damage to block walls and the first and ground floor slabs. An investigation should be undertaken of the condition of the ground floor slabs to determine the degree of cracking that may have occurred and is currently not evident due to coverings. Repair of cracks may be necessary.

10 Conclusions

(a) The building has a seismic capacity of greater than 67% NBS.

(b) The reinforced block wall with block veneer on grid AE and the adjacent timber roof framing are severely damaged due to liquefaction. The wall has undergone significant out-of-plane deformation and must be repaired.

(c) Further geotechnical investigations are required to assess liquefaction potential and ground bearing.

(d) The building has undergone significant differential settlement, with the settlement in several areas exceeding the maximum allowable differential settlement specified in the Building Code. The building could be re-levelled to within the Building Code limits.

11 Recommendations

(a) The upper level units 44 to 47 as well as ground floor unit 41 should remain unoccupied.

(b) The central stairs should be cordoned off on the both sides of the building out a radial distance of 6.0 metres. . This will remove the vehicular access to unit 42. The end stairs should be cordoned off to prevent access.

(c) The damaged masonry wall to grid AE at first floor level and connected timber structure is repaired.

(d) Remove carpets and investigate the ground floor slab for cracking.

(e) Two hand auger/scalas should be undertaken surrounding the west side of Block A to assess the bearing capacity of the underlying material.

(f) Six Cone Penetrometer Tests to a depth of 25m be undertaken to enable a site wide liquefaction assessment.

(g) If the site is assessed to be equivalent to the DBH Technical Category 3, in accordance with the interim guidance, a foundation re-level is likely for the affected units, particularly units 42 and 43, at Whakahoa Village. CCC will need to accept that more damage to the existing concrete slab foundations is likely in future seismic events. Further investigations will be required for detailed design.

12 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 CCC to assist with assessing remedial works required for council buildings and facilities. It is not intended for any other party or purpose.

(d) The geotechnical assessment has been prepared solely for the benefit of CCC as our client with respect to the particular brief given to us. Data or opinions in this report may not be used in other contexts by any other party or for any other purpose.

(e) It is recognised that the passage of time affects the information and assessment provided in this document. Opus's opinions are based upon information that existed at the time of the production of this report. It is understood that the Services provided allowed Opus to form no more than an opinion on the actual conditions of the site at the time the site was visited and cannot be used to assess the effect of any subsequent changes in the quality of the site, or its surroundings or any laws or regulations.

13 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, *Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes*, Structural Engineering Society of New Zealand, 21 December 2011.

Appendix 1 – Photographs

Photo 1: Deformation to masonry veneer in central stair area

Photo 2: Longitudinally split timber rafter adjacent wall to grid ae

Photo 3: Longitudinally split timber rafter plate to wall on grid ae

Photo 4: Evidence of liquefaction to external areas

Photo 5: Cracking to the external masonry veneer in the central stair area

Photo 6: Displaced masonry veneer in the central stair area

Photo 7: Separation between the timber roof members and the plaster ceiling

Appendix 2 – Geotechnical Report

21 June 2012

Michael Sheffield
Christchurch City Council
PO Box 2522
Addington
CHRISTCHURCH 8140

6-QUCCC.93

Dear Michael

Whakahoa Village - Geotechnical Desk Study

1. Introduction

The Christchurch City Council (CCC) has requested Opus International Consultants (Opus) provide a geotechnical desktop study and walkover inspection of the Whakahoa Village (Gowerton Place) Residential Housing Units following the Canterbury Earthquake Sequence initiated by the 4 September 2010 earthquake.

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

This Geotechnical Desk Study has been prepared in accordance with the Engineering Advisory Group's Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Revision 5, 19 July 2011.

This Geotechnical Desk Study forms parts 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 Whakahoa Village Residential Housing Units are situated approximately 2km north-east of Christchurch City in the suburb of Richmond. It is a relatively flat site, approximately 220m north-west of the Avon River.

The housing development was constructed in 2007 and comprises 10 units of a single storey configuration and 8 units in a two storey configuration. The units are predominantly constructed of reinforced concrete masonry blocks with timber veneer being used in some areas.

2.2 Available Building Drawings

Design drawings prepared by Powell Fenwick for Whakahoa Village were sourced from the CCC property file (refer to extract contained in Appendix C).

The drawings indicate the buildings foundations are reinforced concrete perimeter strip footings founded 525mm to 725mm below the finished floor slab level, with a 100mm thick reinforced concrete floor slab laid on compacted tailings or hard fill.

2.3 Regional Geology

The published geological map of the area, (Geology of the Christchurch Urban Area 1:25,000, Brown and Weeber, Map 1, 1992) indicates the site is at the boundary between two surficial geological units; that being sand of fixed and semi-fixed dunes and beaches belonging to the Christchurch Formation and alluvial gravel sand and silt overbank deposits belonging to the Yaldhurst member of the Springston Formation.

A groundwater table depth of approximately 1m has been shown on the published map by Brown and Weeber (1992).

2.4 Earthquake Commission Subsurface Investigations

Three Cone Penetrometer Tests (CPT's) have been completed within 170m of the site on behalf of the Earthquake Commission (EQC). The CPT's indicate the soils comprise silty SAND/sandy SILT layers to 1.6m depth, underlain by clayey SILT to 1.9m, before transitioning into clean and/or silty SAND to the end of the test holes at approximately 12m depth (Refer Appendix D). Note that CPT-RCH-38 refused on a possible shallow dense sand, gravel layer or obstruction at approximately 4.0m below ground level (bgl), which was not encountered in the remaining CPT's

In addition to the CPT's, two boreholes were completed on behalf of the Earthquake Commission within approximately 280m from the site. Due to their location and distance from the site, these EQC boreholes have not been used to infer the underlying geology.

2.5 Expected Ground Conditions

A review of the Environmental Canterbury (ECan) Wells database showed three wells located within approximately 100m of the property boundary (refer to Appendix D). Material logs available from these wells in addition to the EQC CPT tests have been used to infer the ground conditions at the site, as shown in table 1 below.

Table 1: Inferred Ground Conditions

Stratigraphy	Thickness (m)	Depth Encountered from (m) below ground
TOPSOIL	0.2-0.5	0
SAND (not present in M35/16568)	0-0.75	0.2-0.5
SILT	1.2-1.5	0.5-1.0
SAND	20.3-21.0	1.8-2.5
clayey SILT	1.5	22.8
GRAVELS (Riccarton)	-	24.3

The groundwater level was recorded as 1.2m-2.5m bgl in the borehole records.

2.6 Liquefaction Hazard

The 2004 Environment Canterbury Solid Facts Liquefaction Study indicates the site is in an area designated as having 'moderate liquefaction ground damage potential'. According to this study, based on a low groundwater table, ground damage from liquefaction is expected to be moderate and may be affected by 100mm to 300mm of ground subsidence.

Examination of post-earthquake aerial photos taken by New Zealand Aerial Mapping (refer Project Orbit) identified evidence of significant quantities of liquefied soils ejected at the ground surface of the site after the 22 February 2011 and 13 June 2011 events but not after the 4 September 2010 or 23 December 2011 events.

The Tonkin and Taylor Reconnaissance indicated evidence of liquefaction was observed at the site after the 22 February 2011 and 13 June 2011 events.

Following the recent strong earthquakes in Canterbury, the Canterbury Earthquake Recovery Authority (CERA, 2012) has zoned land in the greater Christchurch area according to its ground performance in future large earthquakes.

The residential properties from Vogel Street to the Avon River, 120m south east of the Whakahoa Village complex, are zoned “Red” which is evaluated as not being practical to rebuild, repair or reoccupy. Refer to the Land Recovery Zone Map in Appendix B.

The Department of Building and Housing has sub-divided the CERA “Green” residential land on the flat in Christchurch into technical categories. The three technical categories are summarised in Table 2 which has been adapted from the Department of Building and Housing guidance document (DBH, 2011).

Table 2: Technical Categories based on Expected Land Performance

Foundation Technical Category	Future land performance expected from liquefaction	Expected SLS land settlement	Expected ULS land settlement
TC 1	Negligible land deformations expected in a future small to medium sized earthquake and up to minor land deformations in a future to large earthquake.	0-15mm	0-25mm
TC 2	Minor land deformations possible in a future small to medium sized earthquake and up to moderate land deformations in a future moderate to large earthquake.	0-50mm	0-100mm
TC 3	Moderate land deformations possible in a future small to medium sized earthquake and significant land deformations in future moderate to large earthquake.	>50mm	>100mm

Whakahoa Village has been zoned as N/A-Urban Non-residential, as it is council owned land. The neighbouring residential properties have been zoned as Green-TC3 “blue zone”, which is determined to have a moderate to significant risk of land damage due to liquefaction in future significant earthquakes.

A preliminary CLiq analysis has been performed using the CPT-RCH-37 and CPT-RCH-50 data sets located 100m south east and 170m west of the site, respectively. A summary of the results of the analysis are presented in Table 3 below.

Table 3: Results from a brief CLiq analysis

CPT	Distance from site boundary (m)	Direction	Event	Inferred Liquefiable Layers (bgl)	Total Liquefaction Induced Subsidence (mm)
CPT-RCH-37	100	South East	ULS (0.35g)	-Ground Water Level to 4m -Thin lenses at 7m and 11m	60
CPT-RCH-50	170	West	ULS (0.35g)	-Ground Water level to 7m -8.5m to 13.5m	190

3. Site Walkover Inspection

A walkover inspection of the exterior of the buildings of Block A to E and surrounding land was carried out by an Opus Geotechnical Engineer on 14 May 2012. The following observations were made (refer to the Site Walkover Plan and Site Photographs attached to this report):

- Significant heave in the pavement up to 300mm is evident around the Whakahoa Village buildings (Figures 5, 6 and 10).
- Liquefaction ejected soil is located in several gardens around the Whakahoa Village complex (Figure 8).
- A door frame in unit 54 (Block C) appears to be skewed by 10mm, likely due to differential settlement of foundations (Figure 7).
- The concrete driveways of unit 43 (west end of Block A) and unit 54 (south end of Block C) have cracked and have been offset vertically by up to 20mm (relative to the driveway) and laterally by 10mm directly outside of the garage doors (Figure 3).
- Cracking and settlement has occurred at several locations in the paved footpath in-between Blocks A, B and C (Figure 11).
- Gaps of up to 50mm wide have formed on the north and south sides of Block A (Figures 4, 11 and 12).
- The footpath along the east side of Block B has settled by approximately 20mm (Figure 9).
- The units located on the west side of the stairs in Block A appear to have differentially settled compared to the units on the east side (Figure 14).
- No evidence of differential settlement or cracking was observed around Blocks D and E.

4. Level Survey

A summary of the level survey undertaken by Opus Surveyors on 14th May 2012 are given in Table 4. Refer to Level Survey results in Appendix F.

Table 4: Results from the Level Survey

Block	Unit	Differential Settlement [1,2]
A	40,41	50mm (centre)
A	42,43	120mm (west)
Notes: (1) Floor slab levels rounded to the nearest 10mm (2) Direction of fall indicated in brackets		

5. Discussion

As a result of the 4th September 2010 Canterbury Earthquake and the following aftershocks; cracking, heaving and settlement has occurred in Whakahoa Village.

Liquefaction has occurred in the Whakahoa Village complex and the wider vicinity in both the February 2011 and June 2011 earthquakes. This is evident due to the large amounts of ground heave in paved areas, liquefaction induced settlement, and liquefaction observed from aerial photographs.

The apparent settlement of the footpath along the east side of Block B appears to be due to liquefaction subsidence of the underlying soils. Up to 300mm of ground heave has occurred around the village, which is inferred to result from ejected soils accumulating under an impermeable surface, such as asphalt.

Due to the ground motion during the seismic events, the lateral movement that Block A has undergone may have caused the soils to consolidate resulting in the gaps observed on both the north and south sides of Block A.

It is unknown whether up to 10mm of vertical skew of the door in Unit 54 (Block C) is attributed to settlement or structural damage due to the shaking. A level survey is recommended of Block C to determine whether differential settlement has occurred.

A level survey has been undertaken on the ground floor and first floor of Block A, as it had sustained the majority of the damage. The results have been assessed by separating Block A into two areas divided by the staircase in the centre of the building. The results from the eastern units (40 and 41) showed differential settlement to up to 50mm, with a low point in the centre of Unit 40. Whereas, the units on the western side (42 and 43) showed differential settlement of up to 120mm, with the direction of fall towards the west.

The buildings at the site are two storey reinforced concrete masonry block. The Department of Building and Housing New Zealand guidance documents for repairing and rebuilding foundations in Technical Category 3 (DBH, 2012) is likely to be applicable for the buildings at this site. The guidance indicates that for concrete floor slab on grade (type C2) which is out of level between 50mm to 150mm, with cracks in the floor slab less than 3mm width; a foundation re-level is required.

Due to the reinforced masonry block construction of the units, the structural form is not directly recognised in the DBH guidance document. Therefore, appropriate remedial solutions will be dependent on the integrity of the super structure and liaison with the Structural Engineer.

No evidence of cracking in the floor slabs and perimeter footing were observed. Areas able to be inspected were limited due to the carpeted flooring and shrubbery.

The level survey results are consistent with the observations of differential settlement of the western units of Block A. Observations include pavement cracking of the driveway outside the western most unit (unit 43) separating from the stair well.

There is an open swale 10m east of Black A and B retained by a timber pole retaining wall. The depth of the swale invert relative to Whakahoa Village is approximately 1.5m. This open face represents a potential hazard for lateral spreading. The Avon River is located 150m south east of the Whakahoa Village. There has been no evidence of cracking on the site associated with lateral spreading.

The CLiq analysis based on the CPTs 170m west and 100m south east of the building indicated that there is possible total settlement of up to 190mm during an Ultimate Limit State seismic event. Liquefiable layers have been identified from the ground water level up to 13.0m bgl. The CPT results correlate with the observed differential settlement observed on site.

The peak ground accelerations (PGA) applied for the Ultimate Limit State (ULS) and Serviceability Limit State (SLS) seismic events at the site are based upon extensive probabilistic modelling by GNS Science and observations of land and building damage caused during the Canterbury Earthquake Sequence. The values used are recommended in Appendix C of the Department of Building and Housing guidance document (DBH, April 2012). The PGA are based on a Class D soil type (deep or soft soils), importance level 2 (IL2), and a design life of 50 years for the structure.

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 (Geonet) indicates there is currently a 14% probability of another Magnitude 6 or greater earthquake occurring in the next 12 months in the Canterbury region. Ground damage similar to what has been observed is anticipated in such an event, dependent 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.

The differential settlement that appears to have occurred to Block A relative to the footpath may be attributed to a temporary loss of bearing capacity during the seismic shaking. Shallow investigations including Hand Augers and Scalas should be undertaken to confirm the bearing capacity of the underlying material.

If the existing units are to be retained, a building consent will be necessary for remedial works. Remedial works will include re-levelling of Block A Site specific investigations comprising of approximately 6 Cone Penetrometer Tests (CPT's) to a depth of 20m are recommended to be undertaken to enable a site wide liquefaction assessment (refer to Appendix G) and combined with shallow investigations to identify potential revelling/remedial solutions.

6. Recommendations

It is recommended that:

- A level survey should be undertaken in Block C to confirm whether differential settlement has occurred.

- Two hand auger/scalas are undertaken surrounding the west side of Block A to assess the bearing capacity of the underlying material.

- Six (6) Cone Penetrometer Tests to a depth of 25m be undertaken to enable a site wide liquefaction assessment.

- If the site is assessed to be equivalent to the DBH Technical Category 3, in accordance with the interim guidance, a foundation re-level is likely for the affected units at Whakahoa Village. CCC will need to accept that more damage to the existing concrete slab foundations is likely in future seismic events. Further investigations will be required for detailed design.

7. Limitation

This report has been prepared solely for the benefit of Christchurch City Council as our client with respect to the particular brief given to us. Data or opinions in this desk study may not be used in other contexts, by any other party or for any other purpose.

It is recognised that the passage of time affects the information and assessment provided in this document. Opus's opinions are based upon information that existed at the time of the production of this Desk Study. It is understood that the Services provided allowed Opus to form no more than an opinion on the actual conditions of the site at the time the site was visited and cannot be used to assess the effect of any subsequent changes in the quality of the site, or its surroundings or any laws or regulations.

8. Reference

Brown, LJ; Webber, JH 1992: Geology of the Christchurch Urban Area. Scale 1:25,000. Institute of Geological and Nuclear Sciences geological map, 1 sheet + 104p.

Environment Canterbury, Canterbury Regional Council (ECan) website:

ECan Well Card
http://ecan.govt.nz/services/online-services/tools-calculators/Pages/well-card.aspx

ECan 2004: The Solid Facts on Christchurch Liquefaction. Canterbury Regional Council, Christchurch, 1 sheet.

Project Orbit, 2011: Interagency/organisation collaboration portal for Christchurch recovery effort. https://canterburyrecovery.projectorbit.com/SitePages/Home.aspx

GNS Science reporting on Geonet Website: http://www.geonet.org.nz/canterbury-quakes/aftershocks/ updated on 28 May 2012.

‘Interim recommendations for PGA values for geotechnical design in Canterbury’: Department of Building and Housing New Zealand (2012) *Appendix C: Interim guidance for repairing and rebuilding foundations in Technical Category 3.*

<u>Appendices:</u>

Appendix A: Site Photographs
Appendix B: Land Recovery Zones, Site Location and Walkover Plans
Appendix C: Available Structural Drawings
Appendix D: Earthquake Commission Subsurface Investigations
Appendix E: Environment Canterbury Borehole Logs
Appendix F: Level Survey
Appendix G: Site Investigation Location Plan

Appendix A:
Site Photographs

Figure 1: South elevation of Block A.

Figure 2: East elevation of Block B, C and D.

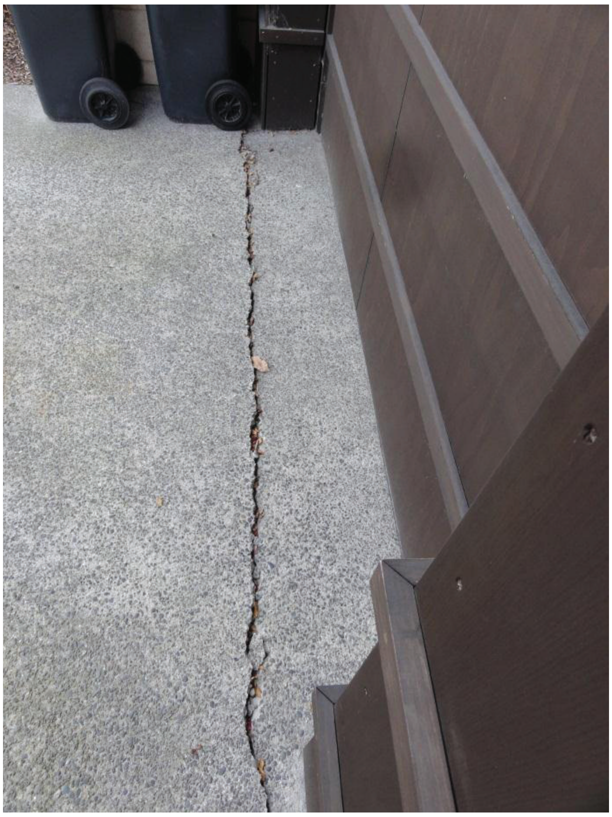


Figure 3: Up to 10mm lateral and 20mm of vertical movement.

Figure 4: Up to 30mm of lateral movement.

Figure 5: Up to 200m of heave in pavement south west of Block A.

Figure 6: Up to 300mm of heave in the pavement along the western boundary.

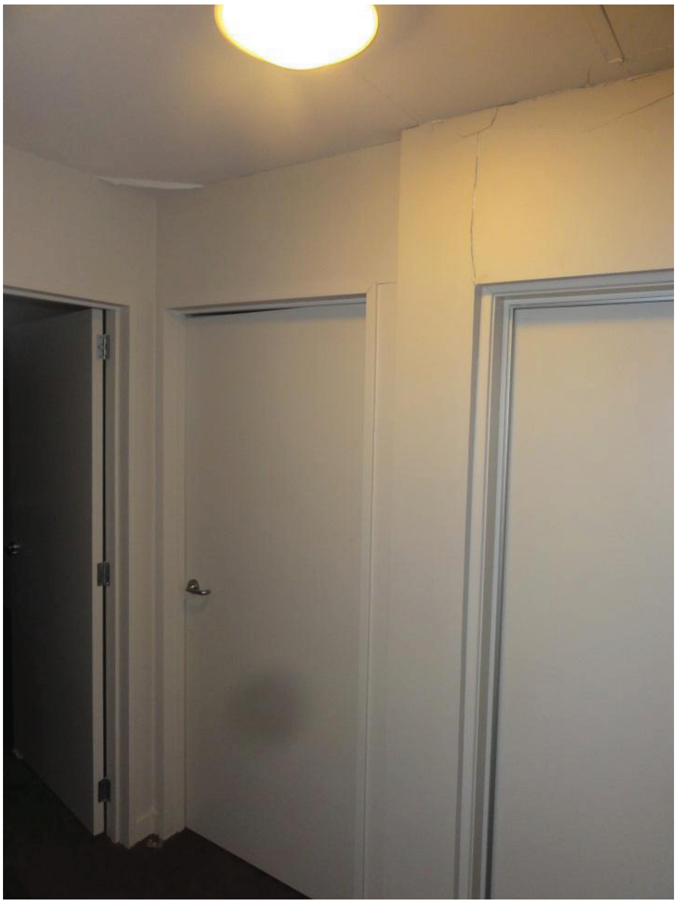


Figure 7: The bedroom door has skewed by up to 10mm.

Figure 8: Liquefaction ejected material is evident in gardens around the site.

Figure 9: Approximately 20mm of vertical settlement in footpath outside Block B.

Figure 10: Heave in the pavement along the western boundary.

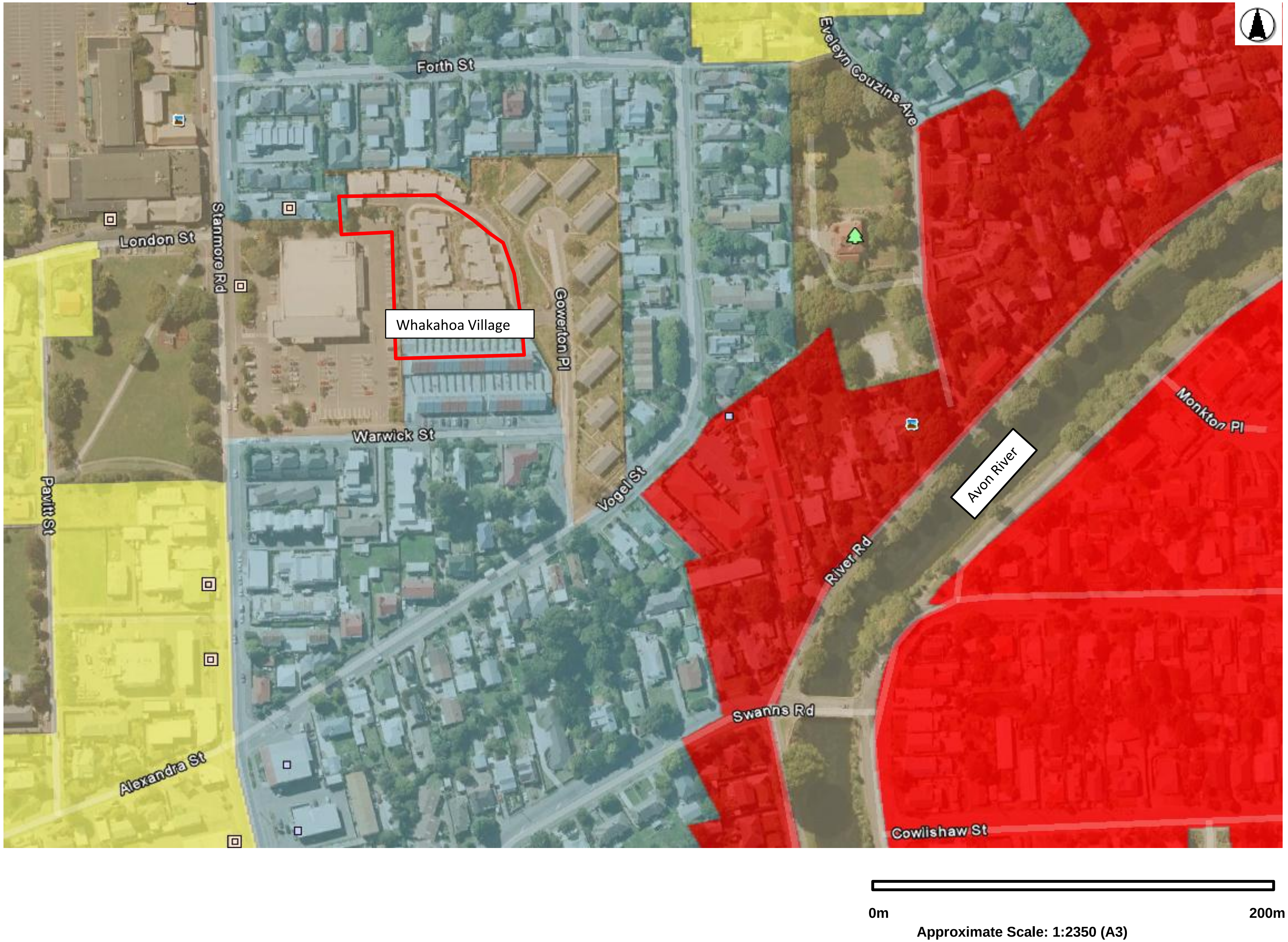
Figure 11: A gap up to 50mm is evident between the asphalt and concrete patio north of Block A.

Figure 12: A gap up to 50mm is evident between a garden and concrete patio north of Block A.

Figure 13: Liquefaction induced settlement of the park benches east of the site.

Figure 14: Separation between west side of Block A and the stair well.

Appendix B:
Land Recovery Zones, Site Location and Walkover plans



SOURCE: canterburyrecovery.projectorbit.com (Accessed on 5/06/12)

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:	Whakahoa Village Geotechnical Desktop Study
Project No.:	6-QUCCC.93
Client:	Christchurch City Council

Land Recovery Zones

Drawn: Opus Geotechnical Engineer

Date: 5/06/2012

0 100m

Approximate Scale: 1:1850 (A3)

Key:

- EQC CPT
- EQC Borehole
- Ecan Borehole

BH	ECan Ref
1	M35/16806
2	M35/16805
3	M35/16568
4	M35/1893

BH/CPT	EQC Ref
5	CPT-RCH-37
6	CPT-RCH-38
7	BH-RCH-09
8	BH-AVS-01
9	CPT-RCH-50

SOURCE: 1) canterburyrecovery.projectorbit.com (Accessed on 19/04/12)
2) http://arcims.ecan.govt.nz/ecanmapping/ (Accessed on 19/04/12)

OPUS

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Tel: +64 3 363 5400 Fax: +64 3 365 7857

Project: Whakahoa Village
Geotechnical Desktop Study

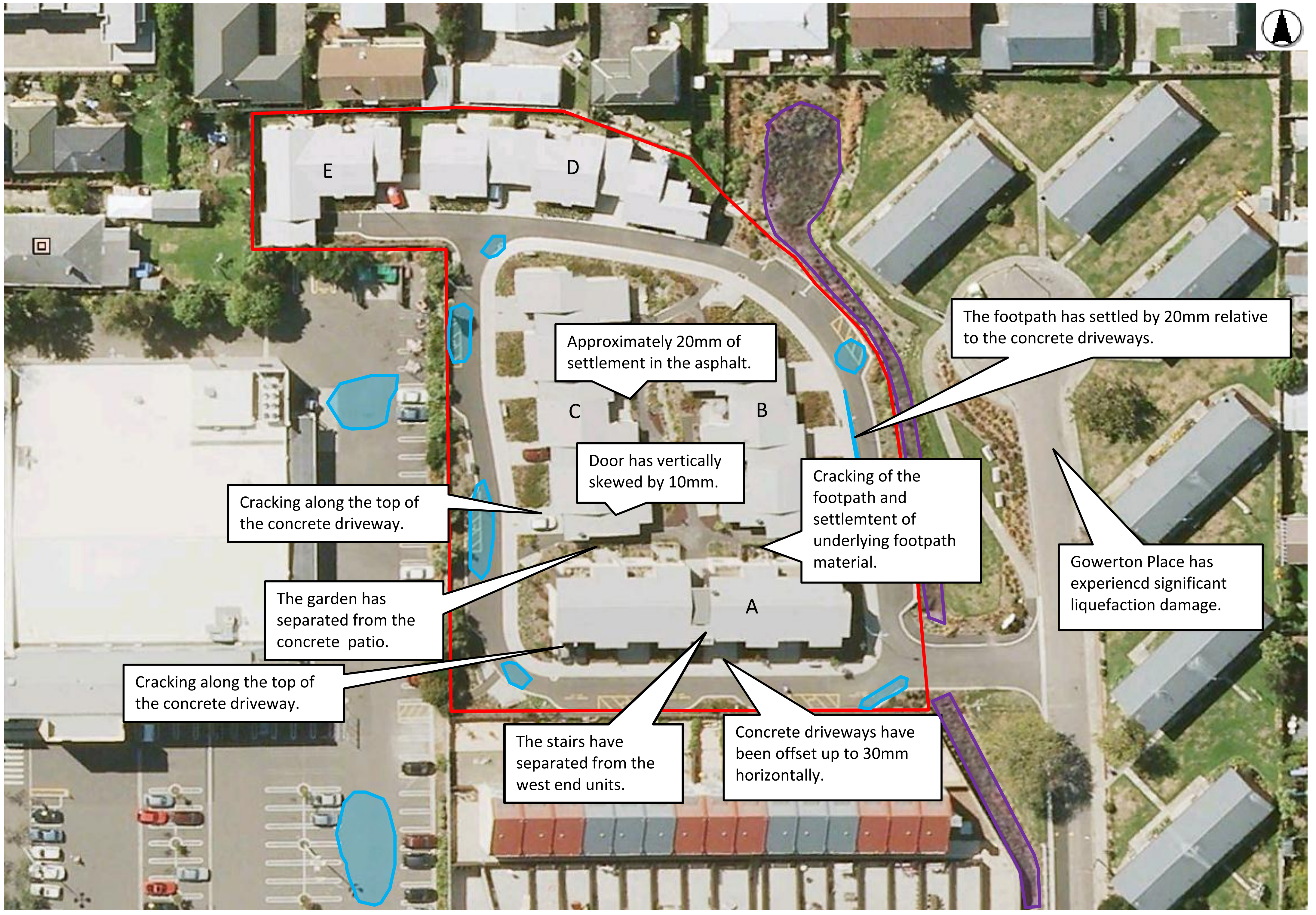
Project No: 6-QUCCC.93

Client: Christchurch City Council

Site Location Plan

Drawn: Opus Geotechnical Engineer

Date: 27/04/2012



Observed Ground Heave

Swale

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PO Box 1482
Christchurch, New Zealand
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Project: Whakahoa Village Geotechnical Desktop Study

Project No: 6-QUCCC.93

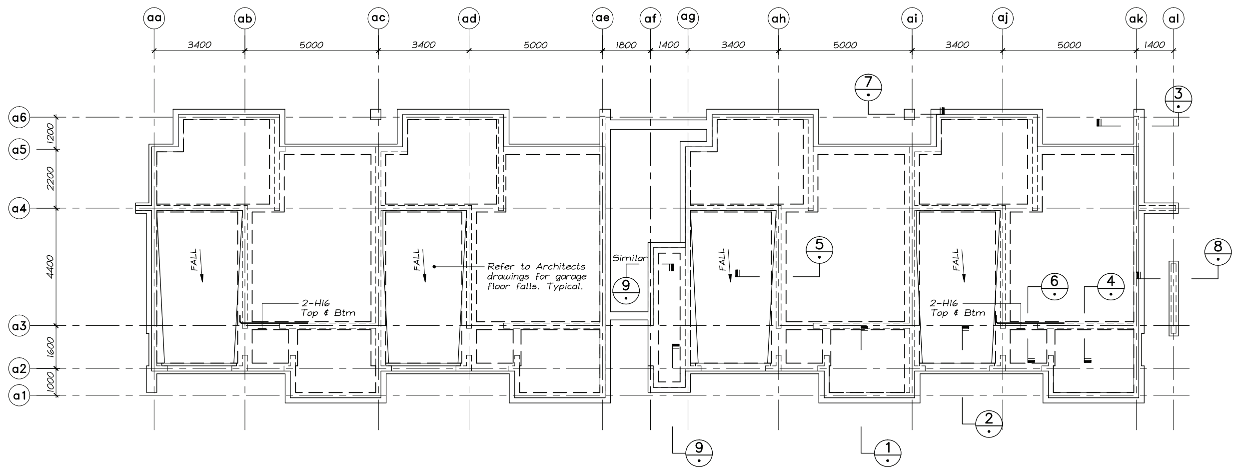
Client: Christchurch City Council

Site Walkover Plan

Drawn: Opus Geotechnical Engineer

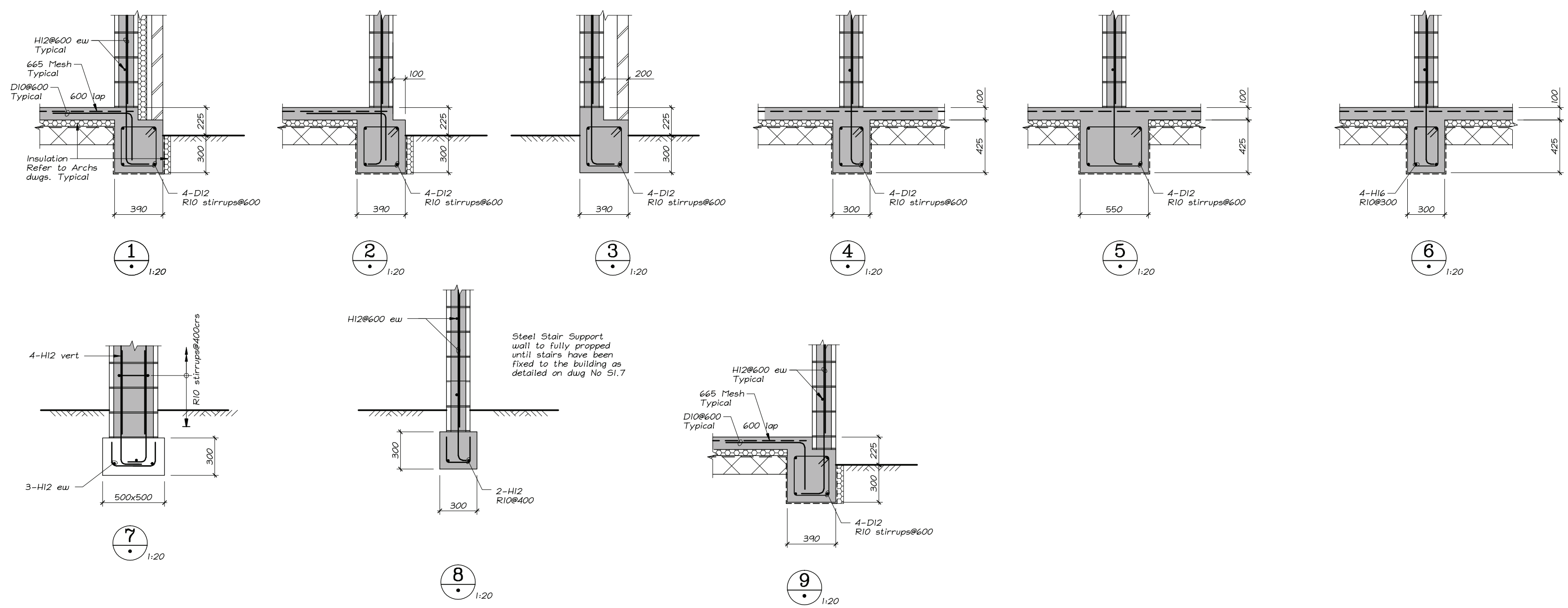
Date: 14/05/2012

Appendix C:
Available Structural Drawings



FOUNDATION PLAN - GROUP A

1:100



ISSUE	DATE	AMENDMENT	CHECKED
A	01.12.06	Construction Issue	
2	15.09.06	Tender Issue	
1	28.06.06	Consent Issue	

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35 Warwick Street, Christchurch

FOUNDATION PLAN & DETAILS GROUP A

CONTRACTOR MUST VERIFY ALL DIMENSIONS ON SITE

SCALES:	DESIGNED	JK	06/06
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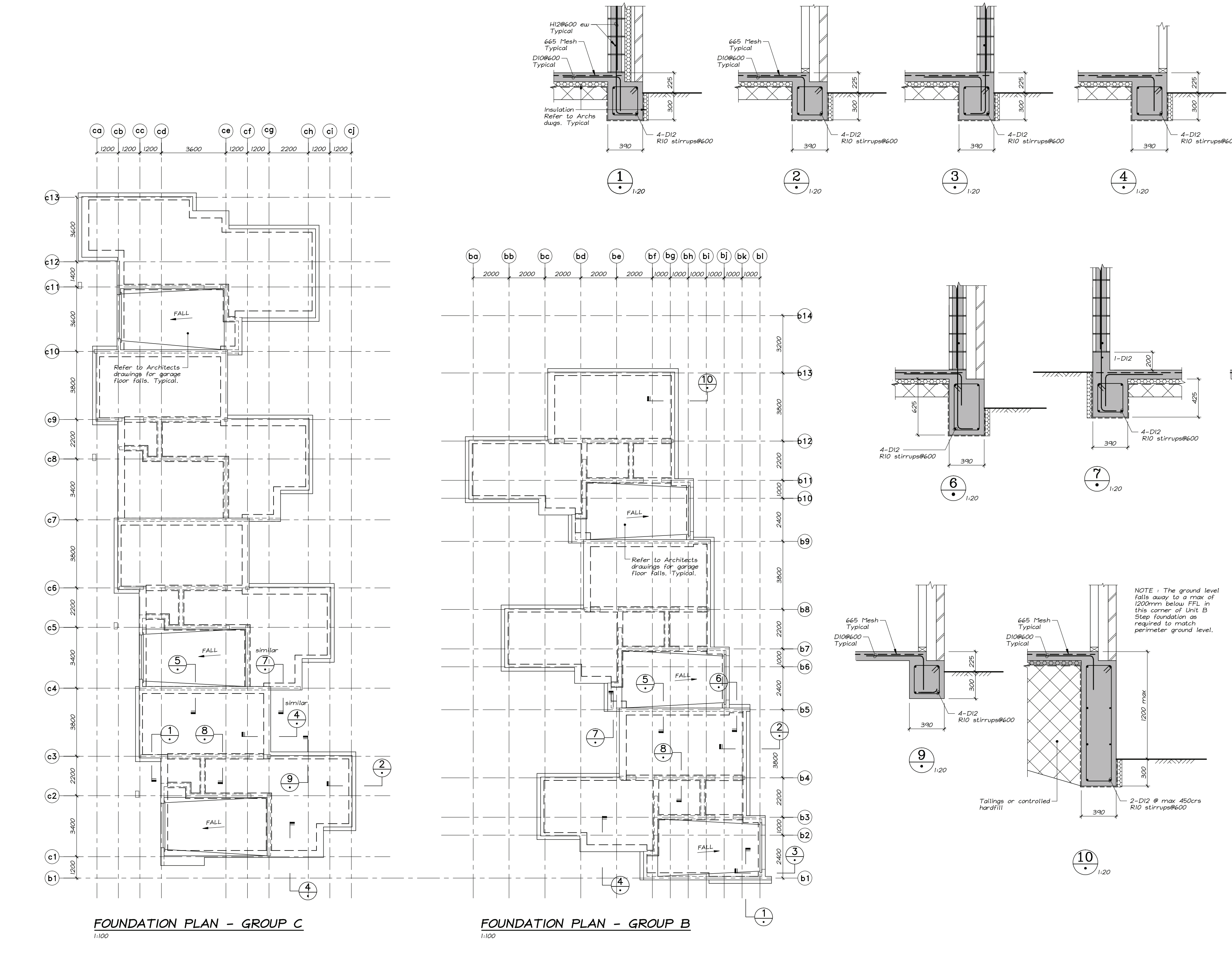

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FOUNDATION PLAN & DETAILS GROUP B & C

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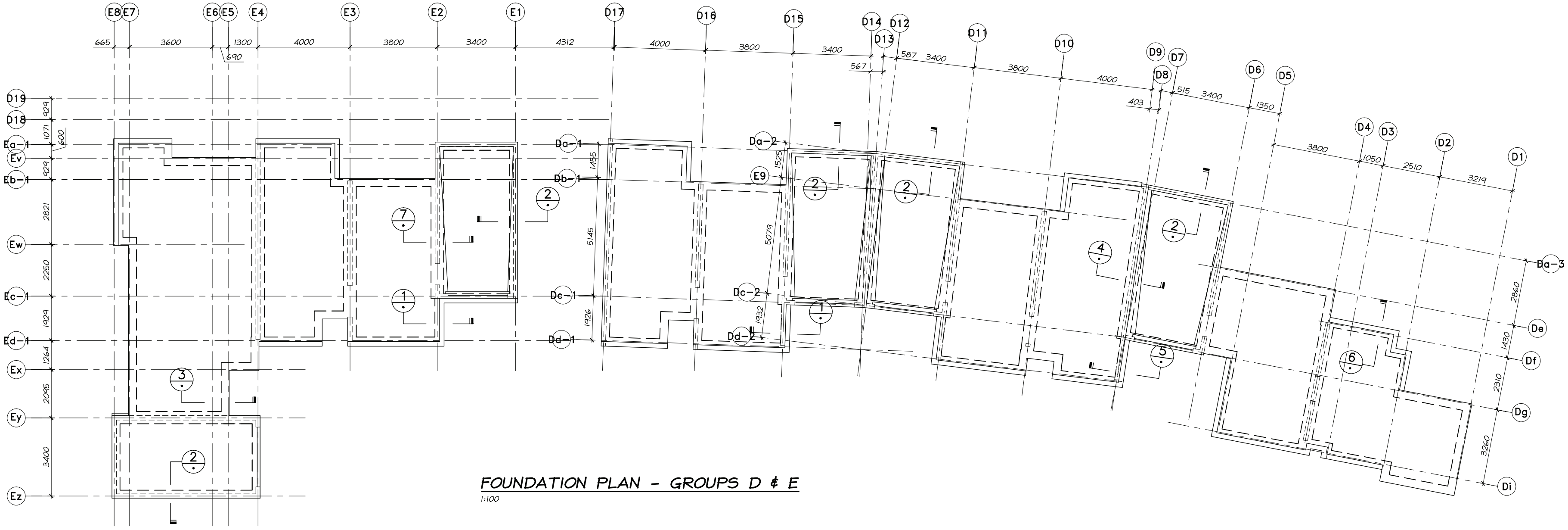
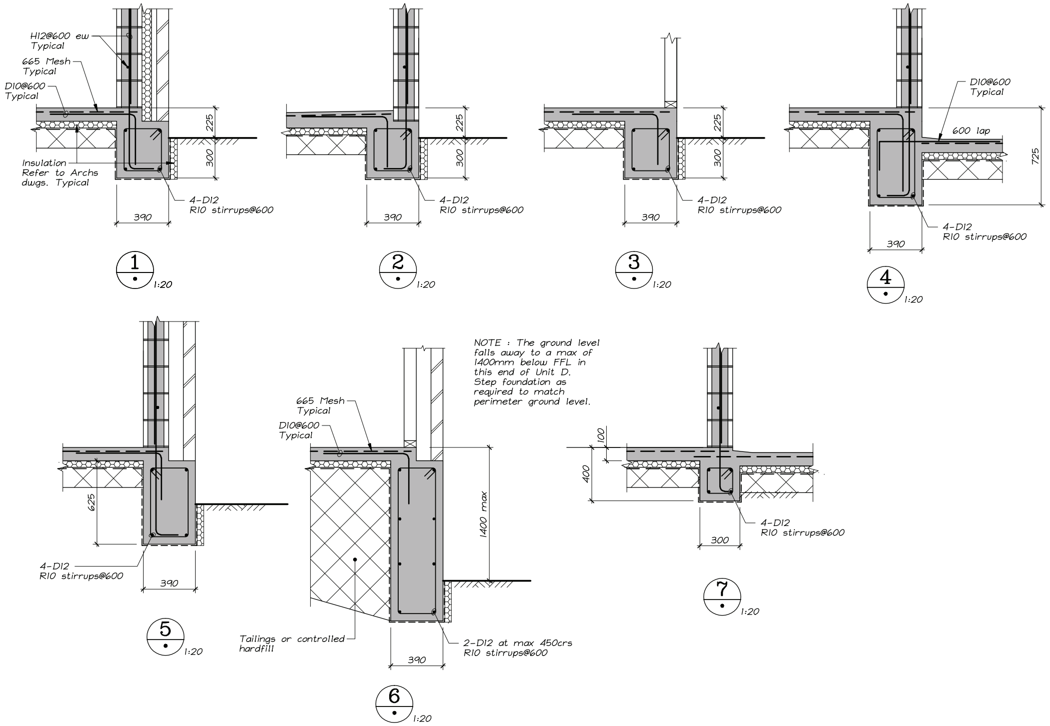


FOUNDATION PLAN - GROUPS D & E

1:100



ISSUE	DATE	AMENDMENT	CHECKED
A	01.12.06	Construction Issue	
2	15.09.06	Tender Issue	
1	28.06.06	Consent Issue	

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FOUNDATION PLANS GROUPS D&E

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DATE PRINTED:

Appendix D:
Earthquake Commissions Subsurface Investigations

TONKIN & TAYLOR LTD

BOREHOLE LOG

BOREHOLE No: RCH 09
Hole Location: On reserve opposite 33 Pavit St

PROJECT: CHRISTCHURCH 2011 EARTHQUAKE	LOCATION: RICHMOND	JOB No: 52000.3200
CO-ORDINATES 5742886.62 mN 2482155.38 mE	DRILL TYPE: Direct Push	HOLE STARTED: 23/9/11
R.L. 4.46 m	DRILL METHOD: Sonic Vibration	HOLE FINISHED: 24/9/11
DATUM NZMG	DRILL FLUID: N/A	DRILLED BY: DCN; LOGGED BY: TH; CHECKED: BMcD

GEOLOGICAL: GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.

ENGINEERING DESCRIPTION: SOIL DESCRIPTION — Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION — Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.

Columns: FLUID LOSS, WATER, CORE RECOVERY (%), METHOD, CASING, TESTS, SAMPLES, R.L. (m), DEPTH (m), GRAPHIC LOG, CLASSIFICATION SYMBOL, MOISTURE CONDITION / WEATHERING, STRENGTH/DENSITY CLASSIFICATION, SHEAR STRENGTH (kPa) 10 25 50 100 200, COMPRESSIVE STRENGTH (MPa) 1 5 20 50 100 250, DEFECT SPACING (mm) 50 250 1000 2000

Depth (m)	Geological	Core Recovery (%)	Method	Tests	Samples	Class. Symbol	Moisture	Strength/Density	Description
0.0–0.9	HAND DIG FILL. (Pothole for services check and backfilled.)	0	PRE-DUG						FILL: Borehole drilled through pre-dug and backfilled pothole.
0.9–1.5	YALDHURST MEMBER OF THE SPRINGSTON FORMATION (ALLUVIAL)	100	SONIC VIBR.			SP	M	MD	Silty, fine SAND with trace rootlets, grey mottled orange brown. Medium dense, moist.
1.5–1.95			SPT	2/4/6 N=10					1.5 to 1.95m no recovery
2.0–2.35		100	SONIC VIBRATION			ML	M	St	SILT with trace sand, brownish grey mottled dark brown. Stiff, wet, low plasticity. Sand is fine.
2.35–2.7						SW	W	L	Fine to medium SAND with some silt, grey. Loose, wet. - becoming silty, fine sand - contains trace silt. Sand becoming fine to medium.
2.7–3.0						ML	W	F	SILT, grey. Firm, wet, low plasticity.
3.0–3.2			SPT	1/1/2 N=3		SW	W	VL	Fine to medium SAND, dark grey. Very loose, wet.
3.2–4.5		86	SONIC VIBRATION	✱FC	B	ML	W	S	SILT with some sand, grey. Soft, wet, low plasticity. Sand is fine. 4.85 to 4.5m no recovery
4.25								St	- becoming stiff
4.5–5.0			SPT	3/6/5 N=11					

R.L. (m): -4.0 at 0.5; -3.5 at 1.0; -3.0 at 1.5; -2.5 at 2.0; -2.0 at 2.5; -1.5 at 3.0; -1.0 at 3.5; -0.5 at 4.0; -0.0 at 4.5; -0.5 at 5

TONKIN & TAYLOR LTD

BOREHOLE LOG

BOREHOLE No: RCH 09
Hole Location: On reserve opposite 33 Pavit St
SHEET 2 OF 4

PROJECT: CHRISTCHURCH 2011 EARTHQUAKE	LOCATION: RICHMOND	JOB No: 52000.3200
CO-ORDINATES 5742886.62 mN 2482155.38 mE	DRILL TYPE: Direct Push	HOLE STARTED: 23/9/11
	DRILL METHOD: Sonic Vibration	HOLE FINISHED: 24/9/11
R.L. 4.46 m		DRILLED BY: DCN
DATUM NZMG	DRILL FLUID: N/A	LOGGED BY: TH CHECKED: BMcD

GEOLOGICAL: GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.

ENGINEERING DESCRIPTION: SOIL DESCRIPTION — Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION — Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.

Columns: FLUID LOSS | WATER | CORE RECOVERY (%) | METHOD | CASING | TESTS | SAMPLES | R.L. (m) | DEPTH (m) | GRAPHIC LOG | CLASSIFICATION SYMBOL | MOISTURE CONDITION / WEATHERING | STRENGTH/DENSITY CLASSIFICATION | SHEAR STRENGTH (kPa) 10 25 50 100 200 | COMPRESSIVE STRENGTH (MPa) 1 5 20 50 100 250 | DEFECT SPACING (mm) 50 250 1000 2000

Geological unit: YALDHURST MEMBER OF THE SPRINGSTON FORMATION (ALLUVIAL)

Depth (m)	R.L. (m)	Core recovery (%)	Method	Tests	Samples	Class. symbol	Moisture	Strength/density	Description
5.0–5.1		100	SONIC VIBRATION			ML	W	St	
5.1–7.4						SW	W	MD	Fine to medium SAND, dark grey. Medium dense, wet.
5.5	-1.0								- thin lense of non-plastic silt
5.9	-1.5			*FC	B				
6.0–6.45			SPT	6/12/20 N=22					
6.5	-2.0	100	SONIC VIBRATION						- sand becoming fine to coarse
7.0	-2.5								
7.4–7.5	-3.0			*FC	B	ML	W	VSt	SILT with some sand, grey. Very stiff, wet, non plastic. Sand is fine.
7.5–8.9			SPT	6/12/15 N=27		SW	W	MD	Fine to medium SAND, dark grey. Medium dense, wet.
8.0	-3.5	100	SONIC VIBRATION						
8.5	-4.0								
8.9–10.0	-4.5		SPT	5/6/7 N=13		SW	W	MD	Gravelly, fine to medium SAND, dark grey. Medium dense, wet. Gravel is fine to coarse, rounded to subrounded.
9.5	-5.0	100	SONIC VIBR.						
10	-5.5								

TONKIN & TAYLOR LTD

BOREHOLE LOG

BOREHOLE No: RCH 09
Hole Location: On reserve opposite 33 Pavit St
SHEET 3 OF 4

PROJECT: CHRISTCHURCH 2011 EARTHQUAKE		LOCATION: RICHMOND	JOB No: 52000.3200
CO-ORDINATES	5742886.62 mN 2482155.38 mE	DRILL TYPE: Direct Push	HOLE STARTED: 23/9/11
		DRILL METHOD: Sonic Vibration	HOLE FINISHED: 24/9/11
R.L.	4.46 m		DRILLED BY: DCN
DATUM	NZMG	DRILL FLUID: N/A	LOGGED BY: TH CHECKED: BMcD

GEOLOGICAL — GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.

ENGINEERING DESCRIPTION — SOIL DESCRIPTION: Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION: Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.

Columns: FLUID LOSS, WATER, CORE RECOVERY (%), METHOD, CASING, TESTS, SAMPLES, R.L. (m), DEPTH (m), GRAPHIC LOG, CLASSIFICATION SYMBOL, MOISTURE CONDITION / WEATHERING, STRENGTH/DENSITY CLASSIFICATION, SHEAR STRENGTH (kPa) 10 25 50 100 200, COMPRESSIVE STRENGTH (MPa) 1 5 20 50 100 250, DEFECT SPACING (mm) 50 250 1000 2000

Depth (m)	R.L. (m)	Geological unit	Core recovery (%)	Method	Tests	Samples	Classification symbol	Moisture	Strength/Density	Description
10.0–10.5		YALDHURST MEMBER OF THE SPRINGSTON FORMATION (ALLUVIAL)	100	SONIC VIBRATION	*PSD WS	B	SW	W	MD	Gravelly, fine to medium SAND, dark grey. Medium dense, wet. Gravel is fine to coarse, rounded to subrounded.
10.5–10.95	-6.0			SPT	4/6/17 N=28					
10.95–12.0	-6.5, -7.0		100	SONIC VIBRATION						
12.0–12.45	-7.5			SPT	5/3/4 N=7					12.0 to 12.45m no recovery
12.5–13.5	-8.0, -8.5	CHRISTCHURCH FORMATION (MARINE & ESTUARINE)	100	SONIC VIBRATION			SW	W	L	Fine to coarse SAND, grey. Loose, wet.
13.5–13.95	-9.0			SPT	4/4/10 N=14		ML	S	St	SILT with some gravel, sand and shells, grey. Stiff, saturated, low plasticity. Gravel is fine, subrounded. Sand is fine to coarse. 13.8 to 13.95m no recovery
14.0–15.0	-9.5, -10.0, -10.5		100	SONIC VIBRATION			SW	W	MD	Fine to coarse SAND with trace shells, dark grey. Medium dense, wet. 14.5 to 14.65m contains some very closely spaced silt lenses

TONKIN & TAYLOR LTD

BOREHOLE LOG

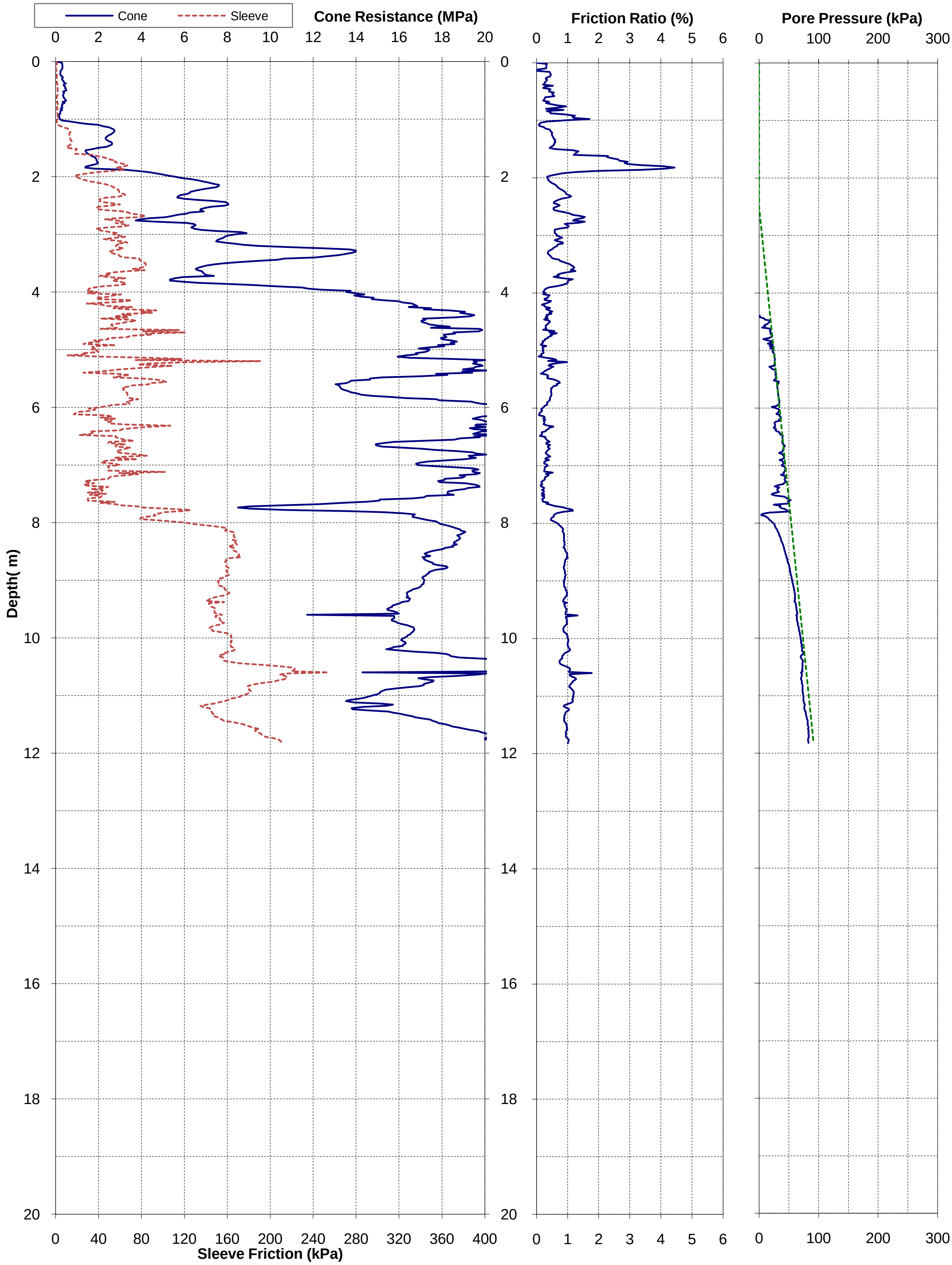
BOREHOLE No: RCH 09
Hole Location: On reserve opposite 33 Pavit St
SHEET 4 OF 4

PROJECT: CHRISTCHURCH 2011 EARTHQUAKE	LOCATION: RICHMOND	JOB No: 52000.3200
CO-ORDINATES 5742886.62 mN 2482155.38 mE	DRILL TYPE: Direct Push	HOLE STARTED: 23/9/11
R.L. 4.46 m	DRILL METHOD: Sonic Vibration	HOLE FINISHED: 24/9/11
		DRILLED BY: DCN
DATUM NZMG	DRILL FLUID: N/A	LOGGED BY: TH CHECKED: BMcD

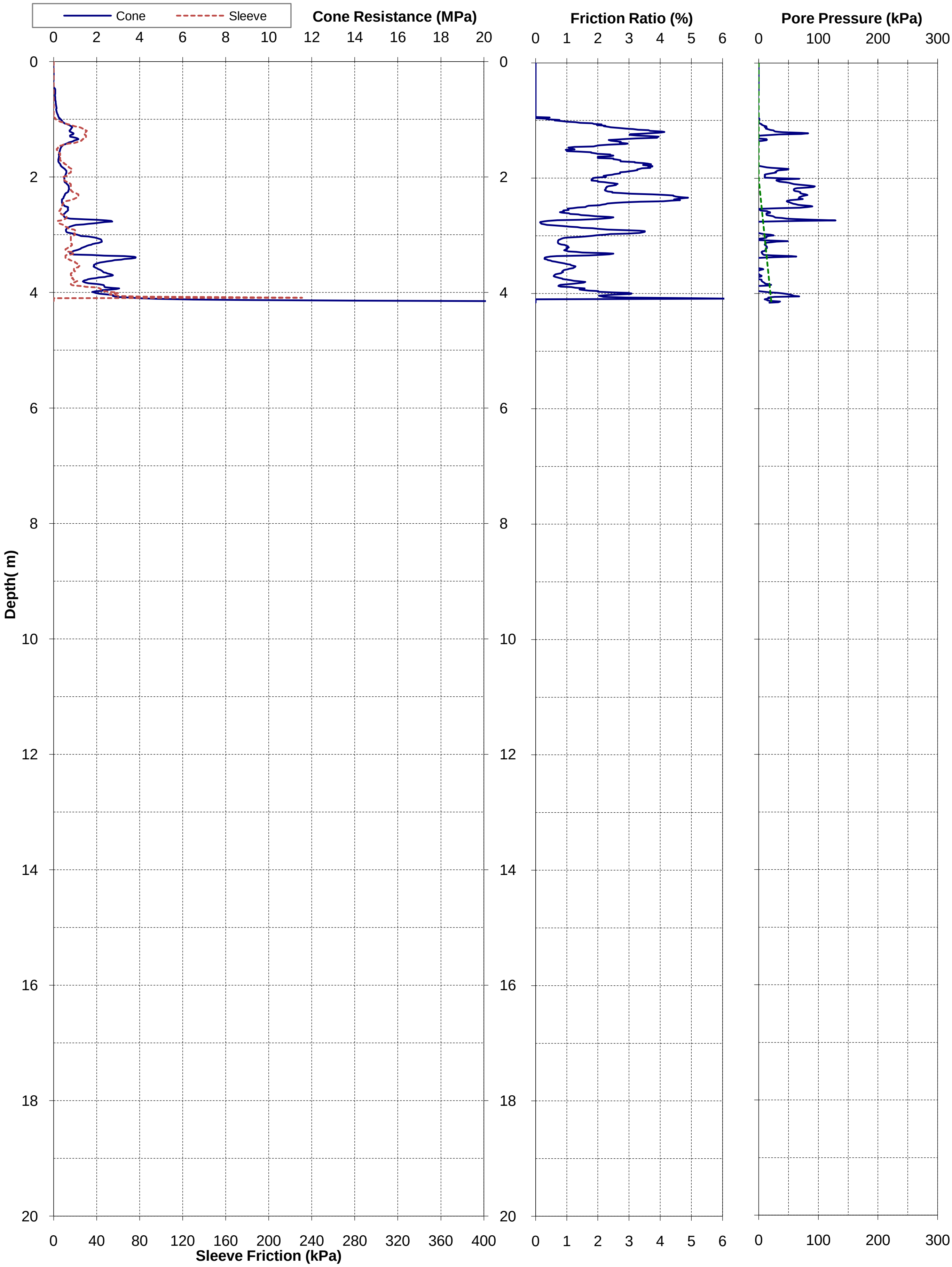
GEOLOGICAL | ENGINEERING DESCRIPTION

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa) 10 25 50 100 200	COMPRESSIVE STRENGTH (MPa) 1 5 20 50 100 250	DEFECT SPACING (mm) 50 250 1000 2000	SOIL DESCRIPTION Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.
CHRISTCHURCH FORMATION (MARINE & ESTUARINE)				SPT		8/17/28 N=45					SW	W	D				14.9 to 15.0m contains some very closely spaced silt laminae - becoming dense
								-11.0	15.5								
			100	SONIC VIBRATION													- thin bed of shells and gravel. Gravel is fine, subrounded.
								-11.5	16.0								
								-12.0	16.5								
				SPT		12/16/18 N=34											
								-12.5	17.0								
			100	SONIC VIBRATION													
								-13.0	17.5								
								-13.5	18.0				VD				- becoming very dense
				SPT		9/27/23 for 95mm N>50											
								-14.0	18.5								
			87	SONIC VIBRATION													
								-14.5	19.0								
						✱FC	B										19.35 to 19.5m no recovery
								-15.0	19.5								
				SPT		13/20/30/ for 135mm N>50											**End of borehole at 19.9mbgl.** Open standpipe piezometer installed. Please see attached diagram in Appendix C.
								-15.5	20								

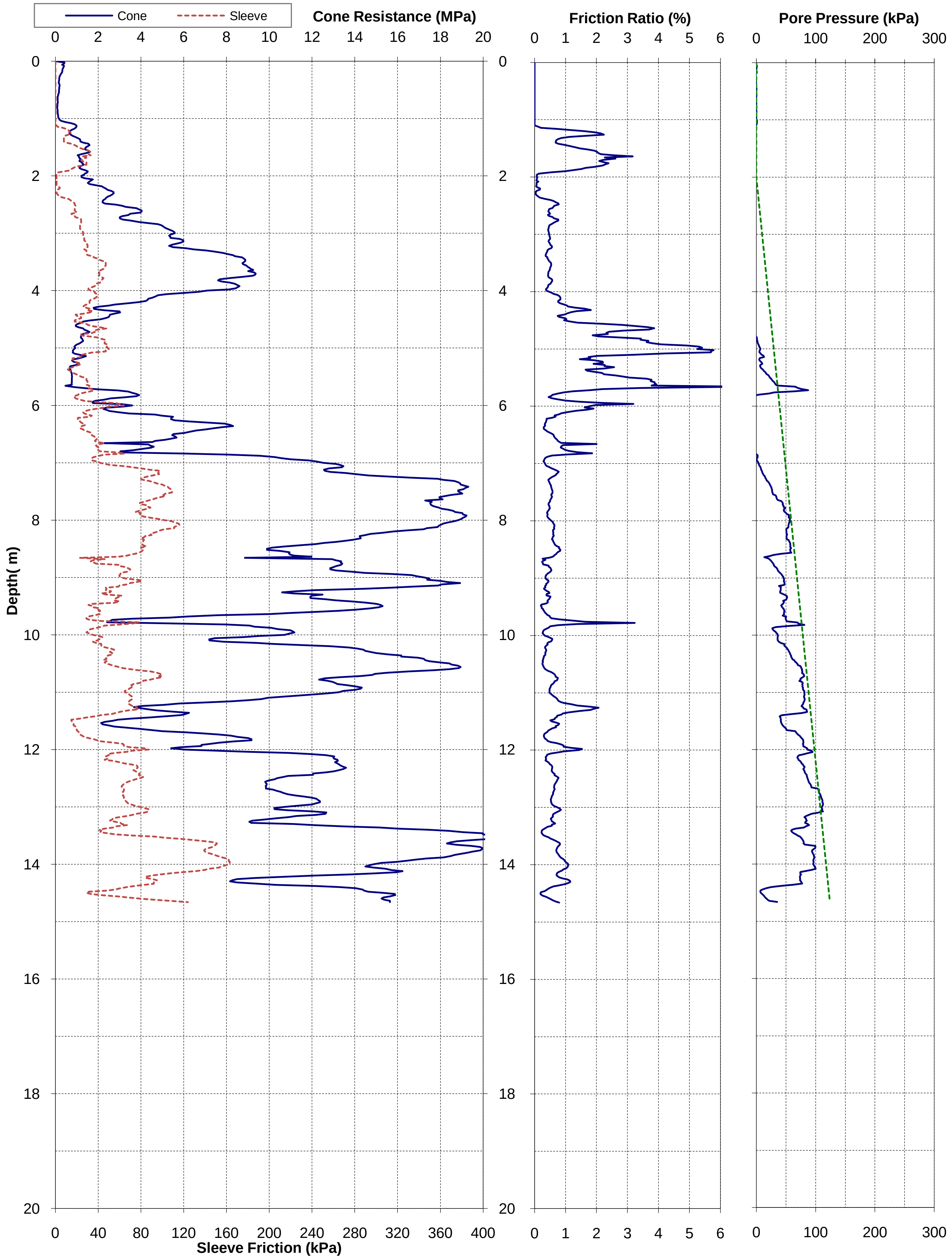
Project:	Christchurch 2011 Earthquake - EQC Ground Investigations			Page:	1 of 1	CPT-RCH-37
Test Date:	30-May-2011	Location:	Richmond	Operator:	Geotech	
Pre-Drill:	1.2m	Assumed GWL:	2.5mBGL	Located By:	Survey GPS	
Position:	2482472.1mE	5742941.2mN	3.87mRL	Coord. System:	NZMG & MSL	
Other Tests:				Comments:		



Project:	Christchurch 2011 Earthquake - EQC Ground Investigations			Page:	1 of 1	CPT-RCH-38
Test Date:	31-May-2011	Location:	Richmond	Operator:	Opus	
Pre-Drill:	1.2m	Assumed GWL:	2mBGL	Located By:	Survey GPS	
Position:	2482415.4mE	5742846.8mN	3.24mRL	Coord. System:	NZMG & MSL	
Other Tests:	Seismic downhole			Comments:		



Project:	Christchurch 2011 Earthquake - EQC Ground Investigations			Page: 1 of 1		CPT-RCH-50
Test Date:	31-May-2011	Location:	Richmond	Operator:	Geotech	
Pre-Drill:	1.2m	Assumed GWL:	2mBGL	Located By:	Survey GPS	
Position:	2482150.1mE	5743002.7mN	4.44mRL	Coord. System:	NZMG & MSL	
Other Tests:				Comments:		



Appendix E:
Environment Canterbury Borehole Logs

Unknown No:	M35/16568
Well Name:	CCC BorelogID 6129
Owner:	CCC borelog

Environment Canterbury
Your regional council

Street of Well:	Warwick Street	**File No:**	
Locality:	Richmond	**Allocation Zone:**	Christchurch/West Melton
NZGM Grid Reference:	M35:82292-42865 QAR 3		
NZGM X-Y:	2482292 - 5742865		
Location Description:		**Uses:**	Foundation/Investigation Bore
ECan Monitoring:			
Well Status:	Filled in		

Drill Date:	17 Jul 2006	**Water Level Count:**	0
Well Depth:	2.00m -GL	**Strata Layers:**	3
Initial Water Depth:	-1.20m -MP	**Aquifer Tests:**	0
Diameter:		**Isotope Data:**	0
		Yield/Drawdown Tests:	0
Measuring Point Ait:	7.43m MSD QAR 4	**Highest GW Level:**	
GL Around Well:	0.00m -MP	**Lowest GW Level:**	
MP Description:		**First Reading:**	
		Last Reading:	
Driller:		**Calc. Min. GWL:**	
Drilling Method:		**Last Updated:**	05 Dec 2008
Casing Material:		**Last Field Check:**	
Pump Type:	None Installed		
Yield:		**Screens:**	
Drawdown:		**Screen Type:**	
Specific Capacity:		**Top GL:**	
		Bottom GL:	
Aquifer Type:	Water Table		
Aquifer Name:	Christchurch Formation		

Borelog for well M35/16568

Gridref: M35:82292-42865 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 7.43 +MSD
Well name : CCC BorelogID 6129
Drill Method : Not Recorded
Drill Depth : -2m Drill Date : 17/07/2006

Scale(m)	Water Level	Depth(m)	Full Drillers Description	Formation Code
-0.2, -0.4		-0.50m	topsoil	
-0.6, -0.8, -1, -1.2, -1.4, -1.6		-1.80m	grey / brown silty clay	
-1.8, -2		-2.00m	grey sand	

Unknown No:	M35/16805		
Well Name:	CCC BorelogID 6486		
Owner:	CCC borelog		

Street of Well:	Stanmore Road	**File No:**	
Locality:	Richmond	**Allocation Zone:**	Christchurch/West Melton
NZGM Grid Reference:	M35:82276-43042 QAR 3		
NZGM X-Y:	2482276 - 5743042		
Location Description:		**Uses:**	Foundation/Investigation Bore
ECan Monitoring:			
Well Status:	Filled in		

Drill Date:	15 Nov 2006	**Water Level Count:**	0
Well Depth:	3.10m -GL	**Strata Layers:**	7
Initial Water Depth:	-2.50m -MP	**Aquifer Tests:**	0
Diameter:		**Isotope Data:**	0
		Yield/Drawdown Tests:	0
Measuring Point Ait:	7.47m MSD QAR 4	**Highest GW Level:**	
GL Around Well:	0.00m -MP	**Lowest GW Level:**	
MP Description:		**First Reading:**	
		Last Reading:	
Driller:		**Calc. Min. GWL:**	
Drilling Method:		**Last Updated:**	01 Sep 2009
Casing Material:		**Last Field Check:**	
Pump Type:			
Yield:		**Screens:**	
Drawdown:		**Screen Type:**	
Specific Capacity:		**Top GL:**	
		Bottom GL:	
Aquifer Type:	Water Table		
Aquifer Name:	Christchurch Formation		

Borelog for well M35/16805

Gridref: M35:82276-43042 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 7.47 +MSD
Well name : CCC BorelogID 6486
Drill Method : Not Recorded
Drill Depth : -3.1m Drill Date : 15/11/2006

Depth(m)	Full Drillers Description	Formation Code
-0.25m	topsoil	
-0.70m	damp sand	
-1.00m	mottled wet silty sand	
-2.10m	mottled wet silt	
-2.50m	blue mottled wet silt	
-2.80m	blue wet sand	
-3.10m	blue wet silty sand	

Unknown No: M35/16806
Well Name: CCC BorelogID 6487
Owner: CCC borelog

Environment Canterbury
Your regional council

Street of Well:	Stanmore Road	**File No:**	
Locality:	Richmond	**Allocation Zone:**	Christchurch/West Melton
NZGM Grid Reference:	M35:82262-43060 QAR 3		
NZGM X-Y:	2482262 - 5743060		
Location Description:		**Uses:**	Foundation/Investigation Bore
ECan Monitoring:			
Well Status:	Filled in		

Drill Date:	15 Nov 2006	**Water Level Count:**	0
Well Depth:	3.15m -GL	**Strata Layers:**	11
Initial Water Depth:	-1.70m -MP	**Aquifer Tests:**	0
Diameter:		**Isotope Data:**	0
		Yield/Drawdown Tests:	0
Measuring Point Ait:	7.48m MSD QAR 4	**Highest GW Level:**	
GL Around Well:	0.00m -MP	**Lowest GW Level:**	
MP Description:		**First Reading:**	
		Last Reading:	
Driller:		**Calc. Min. GWL:**	
Drilling Method:		**Last Updated:**	01 Sep 2009
Casing Material:		**Last Field Check:**	
Pump Type:			
Yield:		**Screens:**	
Drawdown:		**Screen Type:**	
Specific Capacity:		**Top GL:**	
		Bottom GL:	
Aquifer Type:	Water Table		
Aquifer Name:	Christchurch Formation		

Borelog for well M35/16806

Gridref: M35:82262-43060 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 7.48 +MSD
Well name : CCC BorelogID 6487
Drill Method : Not Recorded
Drill Depth : -3.15m Drill Date : 15/11/2006

Environment Canterbury Regional Council

Depth(m)	Full Drillers Description	Formation Code
-0.20m	topsoil	
-0.90m	mottled damp sand	
-1.10m	mottled damp silt	
-1.20m	mottled wet sand and silt	
-1.60m	mottled damp silt	
-1.70m	mottled wet sand	
-2.10m	mottled wet silt	
-2.70m	grey wet sand	
-2.90m	dark grey wet sand	
-3.10m	blue wet sand	
-3.18m	running sand	

Scale(m): -1, -2, -3 (ticks at -0.2, -0.4, -0.6, -0.8, -1, -1.2, -1.4, -1.6, -1.8, -2, -2.2, -2.4, -2.6, -2.8, -3)

Water Level

Bore or Well No:	M35/1893
Well Name:	
Owner:	RICHMOND SCHOOL

Environment Canterbury
Your regional council

Street of Well:	STANMORE ROAD	**File No:**	
Locality:	RICHMOND	**Allocation Zone:**	Christchurch/West Melton
NZGM Grid Reference:	M35:822-429 QAR 4		
NZGM X-Y:	2482200 - 5742900		
Location Description:	OLD SCHOOL SITE,RECREATION RESERVE	**Uses:**	
ECan Monitoring:			
Well Status:	Not Used		

Drill Date:	30 Jan 1993	**Water Level Count:**	0
Well Depth:	82.20m -GL	**Strata Layers:**	13
Initial Water Depth:	6.70m -MP	**Aquifer Tests:**	0
Diameter:	76mm	**Isotope Data:**	0
		Yield/Drawdown Tests:	0
Measuring Point Ait:	4.60m MSD QAR 3	**Highest GW Level:**	
GL Around Well:	0.00m -MP	**Lowest GW Level:**	
MP Description:		**First Reading:**	
		Last Reading:	
Driller:	Job Osborne (& Co/Ltd)	**Calc. Min. GWL:**	3.10m -MP
Drilling Method:	Hydraulic/Percussion	**Last Updated:**	21 Sep 2006
Casing Material:		**Last Field Check:**	
Pump Type:	Unknown		
Yield:	0 l/s	**Screens:**	
Drawdown:	0 m	**Screen Type:**	
Specific Capacity:		**Top GL:**	
		Bottom GL:	
Aquifer Type:	Flowing Artesian		
Aquifer Name:	Linwood Gravel		

Borelog for well M35/1893

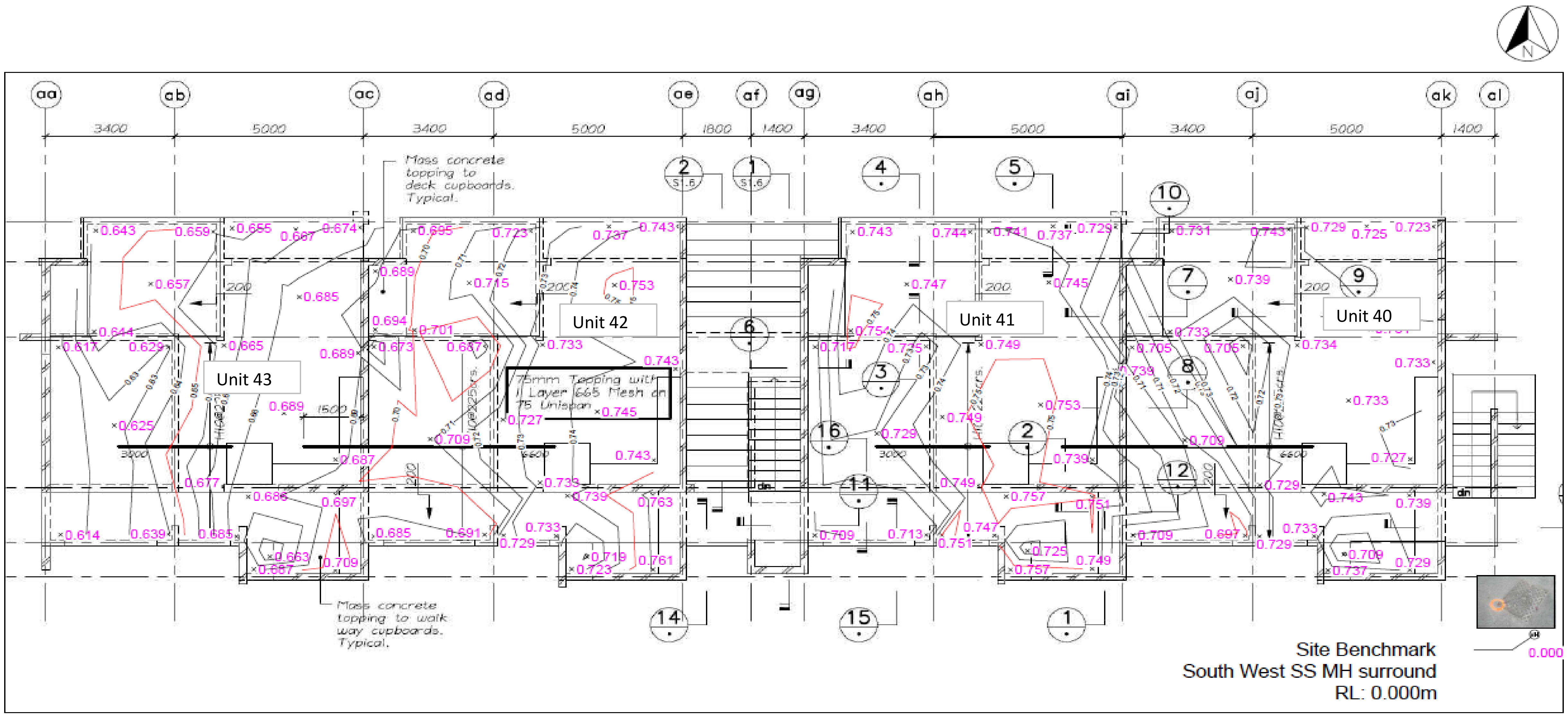
Gridref: M35:822-429 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 4.6 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -82.19m Drill Date : 30/01/1993

Environment Canterbury Regional Council

Water Level: Artesian

Depth(m)	Full Drillers Description	Formation Code
-3.59m	Clay	sp?
-7.90m	Gravel	sp?
- 22.8m	Sand	ch
- 24.3m	Clay	ch
- 32.0m	Gravel	ri
- 32.6m	Clay	ri
- 42.6m	Gravel	ri
- 54.8m	Sand	br
- 61.5m	Gravel	li-1
- 69.1m	Clay	li-2
- 69.7m	Peat	li-2
- 78.0m	Clay	li-2
- 82.2m	Gravel, water level +6.7m	li-3

Appendix F:
Level Survey



Ground Floor Block A

Vertical Datum: Assumed
Site Benchmark: South West Corner MH Surround (Gowerton Place)
RL: 0.000m

OPUS Christchurch Office

Whakahoa Village
Ground Floor Level Survey

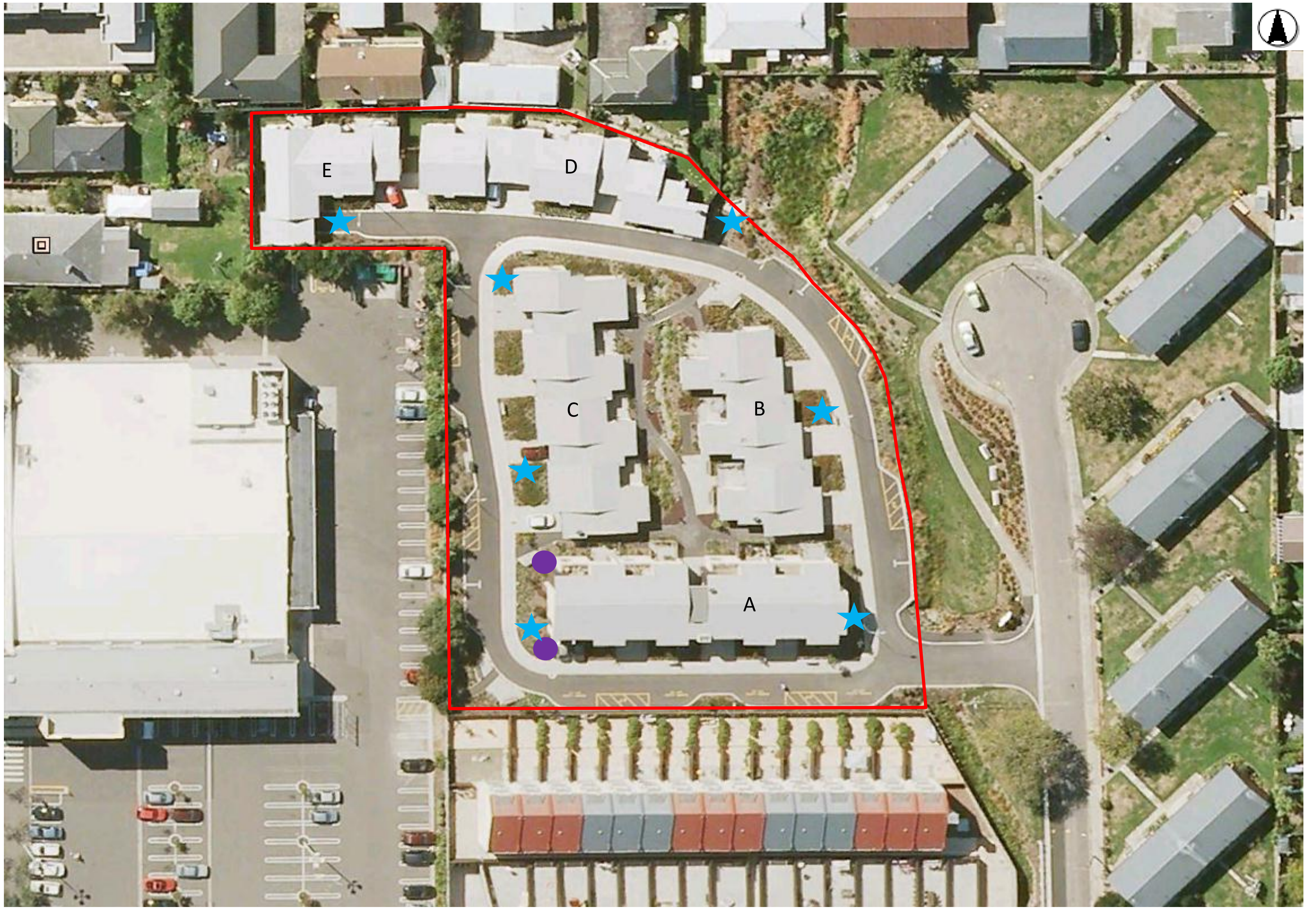
Not to Scale

6/1366/252/2604/R0

1/2

R0

Appendix G:
Site Investigation Location Plan



★ Cone Penetrometer Test

● Hand Auger/Scala

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:	Whakahoa Village Geotechnical Desktop Study
Project No:	6-QUCCC.93
Client:	Christchurch City Council

Site Investigation Plan

Drawn: Opus Geotechnical Engineer

Date: 14/05/2012

Appendix 3 – CERA DEE Data Sheet

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Whakahoa Village Block A	Reviewer:	Robert Davey
	Unit No: Street	CPEng No:	17912
Building Address:	Block A / Gowerton Place, Richmond	Company:	Opus International Consultants
Legal Description:		Company project number:	6-QUCCC.93
		Company phone number:	03 363 5400
	Degrees / Min / Sec		
GPS south:	43 / 31 / 13.00	Date of submission:	10-Sep-12
GPS east:	172 / 39 / 28.00	Inspection Date:	
		Revision:	Final
Building Unique Identifier (CCC):	BU 2080-001 EQ2	Is there a full report with this summary?	Yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	silty sand	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:	2 (single storey = 1)	Ground floor elevation (Absolute) (m):	14.20
Ground floor split?	no	Ground floor elevation above ground (m):	0.00
Storeys below ground	0		
Foundation type:	strip footings	if Foundation type is other, describe:	
Building height (m):	6.79	height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	346		
Age of Building (years):	5	Date of design:	2004-
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	multi-unit residential	Brief strengthening description:	
Use (upper floors):	multi-unit residential		
Use notes (if required):			
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	140x45 H3.1 Rafters @ 1200 centres, 70x45 Purlins @ 1000 centres
Floors:	precast concrete with topping	unit type and depth (mm), topping	75mm Unispan, 75mm topping
Beams:	none	overall depth x width (mm x mm)	
Columns:	load bearing walls	typical dimensions (mm x mm)	
Walls:	fully filled concrete masonry	#N/A	

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	fully filled CMU			
Ductility assumed, μ:	2.00		note total length of wall at ground (m):	
Period along:	0.25	##### enter height above at H31	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	estimated
Lateral system across:	fully filled CMU			
Ductility assumed, μ:	2.00		note total length of wall at ground (m):	
Period across:	0.25	##### enter height above at H31	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	estimated

Separations:

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

Non-structural elements

Stairs:	cast insitu	notes	
Wall cladding:	brick or tile	describe (note cavity if exists)	10 Series coloured masonry
Roof Cladding:	Metal	describe	Profiled metal roofing
Glazing:	aluminium frames		
Ceilings:	plaster, fixed		
Services(list):			

Available documentation

Architectural	full	original designer name/date	City Solutions, 24/11/2006
Structural	full	original designer name/date	Powell Fenwick Consultants Ltd, 01/12/2006
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:		Describe damage:	
Settlement:		notes (if applicable):	
Differential settlement:		notes (if applicable):	
Liquefaction:		notes (if applicable):	
Lateral Spread:		notes (if applicable):	
Differential lateral spread:		notes (if applicable):	
Ground cracks:		notes (if applicable):	
Damage to area:		notes (if applicable):	

Building:

Current Placard Status: red

Along — Damage ratio: (blank) — Describe how damage ratio arrived at:

Describe (summary): The shear wall to grid ae has limited capacity for future loading and additional bracing is required in the east-west direction

Across — Damage ratio: #DIV/0!

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

Describe (summary): The shear wall to grid ae has limited capacity for future loading

Diaphragms	Damage?: no	Describe:	
CSWs:	Damage?: yes	Describe:	Wall to grid ae has laterally displaced to the point it has e
Pounding:	Damage?: no	Describe:	
Non-structural:	Damage?: no	Describe:	

Recommendations

Level of repair/strengthening required:	minor structural	Describe:	Wall and connected roof structure to grid ae should be re
Building Consent required:	yes	Describe:	
Interim occupancy recommendations:	do not occupy	Describe:	

Along	Assessed %NBS before e'quakes:		##### %NBS from IEP below	If IEP not used, please detail assessment methodology:
	Assessed %NBS after e'quakes:	67%		
Across	Assessed %NBS before e'quakes:		##### %NBS from IEP below	
	Assessed %NBS after e'quakes:	100%		

IEP **Use of this method is not mandatory - more detailed analysis may give a different answer, which would take precedence. Do not fill in fields if not using IEP.**

Period of design of building (from above): 2004- — h_n from above: m

Seismic Zone, if designed between 1965 and 1992: — Design Soil type from NZS1170.5:2004, cl 3.1.3:

not required for this age of building

	along	across
Period (from above):	0.249	0.249
(%NBS)nom from Fig 3.3:		

Note:1 for specifically design public buildings, to the code of the day: pre-1965 = 1.25; 1965-1976, Zone A =1.33; 1965-1976, Zone B = 1.2; all else 1.0	1.00
Note 2: for RC buildings designed between 1976-1984, use 1.2	1.0
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)	1.0

	along	across
Final (%NBS)nom:	**0%**	**0%**

2.2 Near Fault Scaling Factor

Near Fault scaling factor, from NZS1170.5, cl 3.1.6: 1.00

	along	across
Near Fault scaling factor (1/N(T,D), **Factor A**:	1	1

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:

Z_{1992}, from NZS4203:1992

Hazard scaling factor, **Factor B:** #DIV/0!

2.4 Return Period Scaling Factor

Building Importance level (from above): 2

Return Period Scaling factor from Table 3.1, **Factor C**:

	along	across
2.5 Ductility Scaling Factor Assessed ductility (less than max in Table 3.2)	1.00	1.00
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	1.00	1.00
2.6 Structural Performance Scaling Factor: Sp:	1.000	1.000
Structural Performance Scaling Factor **Factor E**:	1	1
2.7 Baseline %NBS, (NBS%)b = (%NBS)nom x A x B x C x D x E %NBSb:	**#DIV/0!**	**#DIV/0!**

Global Critical Structural Weaknesses: (refer to NZSEE IEP Table 3.4)

3.1. Plan Irregularity, factor A: 1

3.2. Vertical irregularity, Factor B: 1

3.3. Short columns, Factor C: 1

3.4. Pounding potential

Pounding effect D1, from Table to right 1.0

Height Difference effect D2, from Table to right 1.0

Therefore, Factor D: 1

3.5. Site Characteristics 1

Table for selection of D1	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Alignment of floors within 20% of H	**0.7**	**0.8**	**1**
Alignment of floors not within 20% of H	**0.4**	**0.7**	**0.8**

Table for Selection of D2	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Height difference > 4 storeys	0.4	0.7	1
Height difference 2 to 4 storeys	0.7	0.9	1
Height difference < 2 storeys	1	1	1

	Along	Across
3.6. Other factors, Factor F For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum		
Rationale for choice of F factor, if not 1		

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)

List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

3.7. Overall Performance Achievement ratio (PAR)	**0.00**	**0.00**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	**#DIV/0!**	**#DIV/0!**
4.4 Percentage New Building Standard (%NBS), (before)		**#DIV/0!**

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