Hagley Park North Pavilion
PRK 1190 BLDG 010 EQ2
Detailed Engineering Evaluation

Quantitative Assessment Report

Christchurch City Council

Hagley Park North Pavilion

North Hagley Park, Christchurch

Detailed Engineering Evaluation

Quantitative Assessment Report

Opus International Consultants Limited
Christchurch Office
20 Moorhouse Avenue
PO Box 1482, Christchurch Mail Centre,
Christchurch 8140, New Zealand

Telephone: +64 3 363 5400
Facsimile: +64 3 365 7858

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Hagley Park North Pavilion
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Detailed Engineering Evaluation
Quantitative Report - SUMMARY
Final

North Hagley Park, Christchurch

Background

This is a summary of the quantitative report for the Hagley Park North Pavilion located at North Hagley Park, and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on 9 August 2011 and 26 March 2012. No structural or architectural drawing is available.

Key Damage Observed

- Some minor settlement and cracking of up to 3mm wide to the internal ground bearing slab at several locations;
- Moisture damage to timber post and beam at the west end of the hipped roof due to water leakage from the roof;
- Minor to moderate diagonal cracking to top corners of several window and door openings; and
- Minor cracking to external perimeter apron slab. There is also minor settlement and separation observed to the slab;

Critical Structural Weaknesses

No significant critical structural weakness were noted for this building. However, the lack of roof diaphragm/bracing could result in increased levels of damage to the longitudinal north and south elevation walls.

Indicative Building Strength

The overall %NBS for this building is 56%.

Recommendations

- Strengthen the building to at least 67% NBS with the installation of a roof diaphragm or cross bracing.
- Seal the weather gap due to cracking at the top corner of the window to the west of the front entrance.
- Replace the moisture damaged timber post & support beam near the west end of the hipped roof.
- Review and re-fix as necessary the nominal repairs made to the roof framing prior to the Canterbury earthquakes.
- Remove the prefabricated timber frames stored within the ceiling space.

Contents

1 Introduction

Opus International Consultants Limited (Opus) has been engaged by Christchurch City Council (CCC) to undertake a detailed seismic assessment of the Hagley Park North Pavilion building, located at North Hagley Park, Christchurch (43° 31' 44.52"S, 172° 37' 27.88" E) following the M6.3 Christchurch earthquake on 22 February 2011.

The report is a quantitative assessment of the building structure incorporating the key aspects of a qualitative assessment. The methodology is based on the qualitative and quantitative procedures detailed in the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011. By inspection of the initial survey, it was apparent that a quantitative assessment would be more appropriate.

This assessment involves a desktop review of existing structural and geotechnical information, including existing drawings and calculations (if available) and undertaking some non-intrusive and intrusive site investigation as necessary. The purpose of the assessment is to:

- determine the likely building performance and damage patterns;
- identify any potential critical structural weaknesses or collapse hazards;
- undertake an analysis of seismic capacity of the bracing systems for seismic loads in the transverse and longitudinal directions to determine the likely building strength in terms of percentage of new building standard (% NBS); and
- Provide recommendations and/or strengthening concepts for the structure if it is found to be less than 34% NBS.

At the time of this report, only a covermeter scan of the building structure has been carried out to detect the existence, and spacing of any reinforcement bars and to estimate the depth of concrete cover.

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 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:

5. 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
6. 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
7. 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
8. There is a risk that other property could collapse or otherwise cause injury or death; or
9. 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.

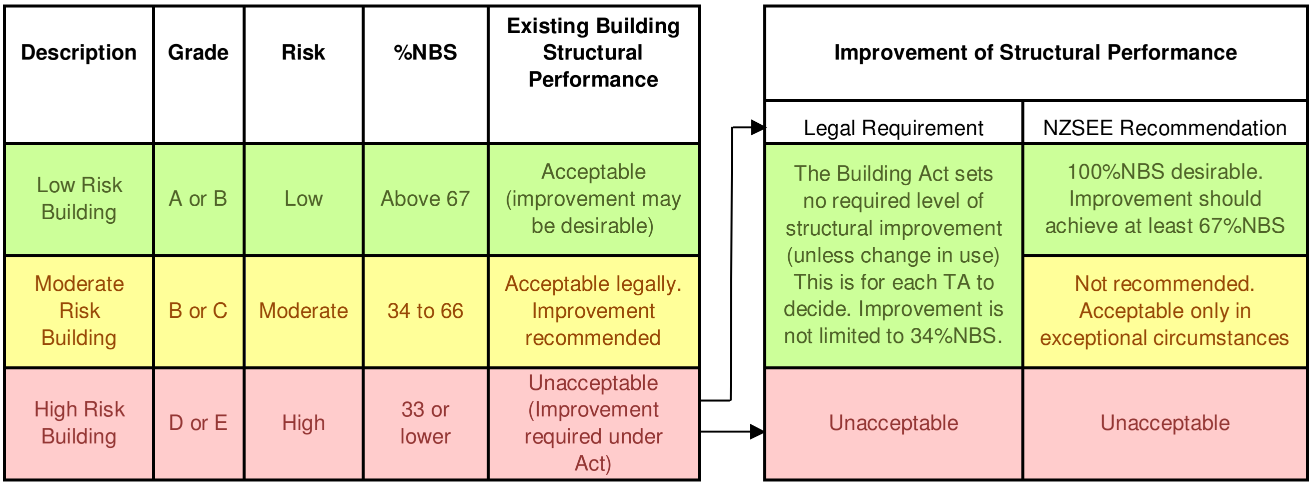
1.2 Ensuring that responsible steps are taken to minimise the risk of loss of life, injury or suffering which may result from your engineering activities, either directly or indirectly.

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 3-1 below.



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

Figure 3-1: NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE Guidelines

Table 3.1 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 3.1: %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, 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.

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[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 Hagley Park North Pavilion building is a simple rectangular building with perimeter reinforced concrete (RC) walls with openings for windows and doors, and a shallow hipped roof. It is likely to have been built circa 1914 based on the inscription in front of the building.

The building has an open plan layout with a minimal permanent partition at the west end of the building for toilet facilities. It is being used primarily as the storage facilities for CCC's events management team.

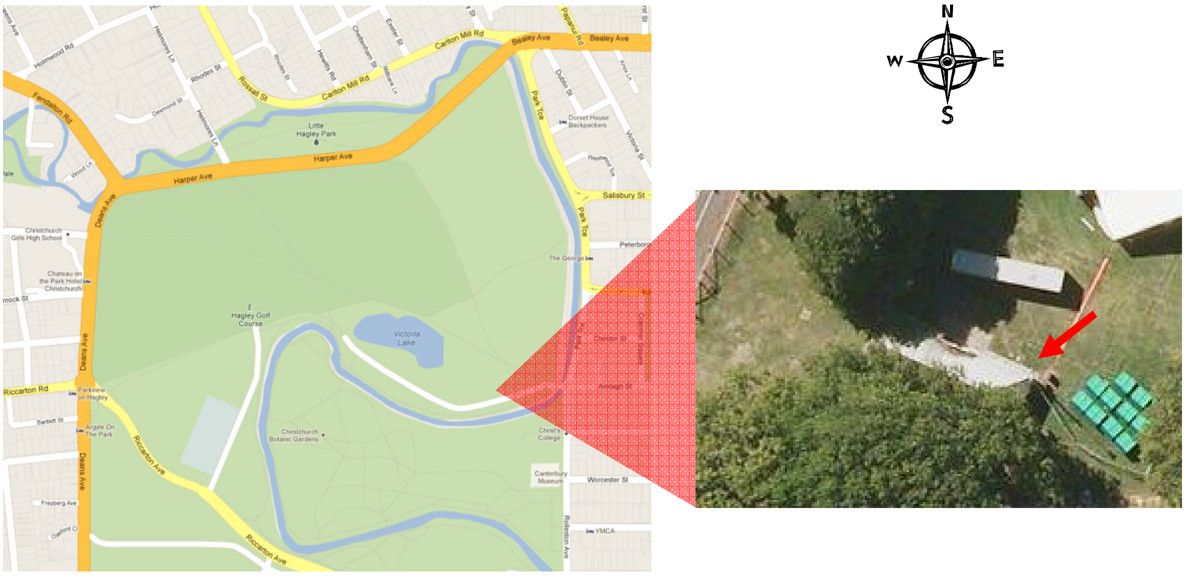

Figure 4.1 - Building Location within Hagley Park North

The building is north facing and is approximately 6.4m wide by 17m long. The roof apex is approximately 3.8m above ground level and the external wall is 2.8m high. The area of its footprint is approximately 109m^2. For the purpose of this report, we refer east to west as the longitudinal direction and north-south as the transverse direction.

4.2 Gravity Load Resisting System

The corrugated iron roof is supported on timber boarding on timber rafter trusses which span transversely across the building. The trusses are spaced approximately 900mm apart and are supported directly on the perimeter reinforced concrete wall.

There is also an internal lightweight ceiling suspended from the timber trusses.

4.3 Lateral Load Resisting System

The lateral load resisting system in both principal directions is the perimeter 102mm thick RC wall acting primarily as a shear wall. Also, since there is no roof diaphragm or cross bracing, the perimeter wall perpendicular to the direction of the lateral load direction will bend out-of-plane.

Although there are numerous openings on each of the wall elevations, there is unlikely to be any short column effects as all the piers are of the same height around the building.

4.4 Foundations

Structural drawings were not available and no exploratory excavations were undertaken. It is assumed that the external RC wall is founded on a perimeter shallow strip footing with limited flexural capacity at the base. The internal concrete slab appears to be ground bearing.

4.5 Survey

4.5.1 Post 22 February 2011 Rapid Assessment

Engineers from Opus undertook two structural (Level 2) assessments of the building on 28 June 2011 and 9 August 2011. The inspections included external and internal visual inspections of all structural elements above foundation level, and areas of damage to structural and non structural elements. No linings were removed.

The site was posted with a Green (G2) placard, indicating that the building access is not restricted.

4.5.2 Further Inspections

On 26 March 2012, an engineer from Opus re-inspected the site. The Profoscope covermeter was used to provide the closest approximation of the reinforcement and concrete cover within the RC wall, without the need for physical invasive investigation. Refer to Section 4.5.3 for the investigation findings.

4.5.3 Reinforcement Estimates and Material Properties

As no structural drawings were available, a Profoscope covermeter was used to scan the existing RC wall. This provided an approximation of the steel reinforcement within the concrete. While the Profoscope was able to detect the existence of steel reinforcement and its cover within the perimeter RC wall, it was not able to determine the size of the reinforcing bar at all locations. This could be due to numerous factors such as the cover depth and presence of interference from other surrounding metallic objects.

For the purposes of our analysis, we assumed that the reinforcement would be consistent throughout the entire perimeter RC wall. Based on the data collected, the vertical reinforcement is estimated to be a single layer of $^3/_8$ inch diameter bar at 12 inch spacing located at the wall central. This reinforcement is equivalent to a 9.6mm diameter bar at 305mm centres. Only minimal horizontal reinforcement was detected. This is not unexpected considering the construction practice of the period. The horizontal reinforcement detected is not likely to provide any shear reinforcement and therefore not considered in the shear capacity check.

The following material properties were used in the analysis:

Material	Nominal Strength
Reinforcing steel, f_y	250[2] MPa
Concrete, f_c	25[3] MPa

4.6 Original Documentation

No structural or architectural drawings were provided.

5 Damage Assessment

The following damage has been noted:

5.1 Floor Slab

Cracking up to 3mm wide is observed to the internal ground bearing slab at several locations. The cracking tends to be in the transverse direction which suggests that they could be originally due to shrinkage but aggravated by the earthquake shaking. See Photo 2 in Appendix 1.

5.2 Roofing

There does not appear to be any earthquake damage to the roof framing. However, it is observed that the base of the timber post near the west end of the hipped roof appears to have rotted due to water leakage from the roofing. The post's supporting beam is also severely damaged and does not appear to bear any roof load.

The current roof load is likely to be redistributed to other primary and secondary support members. There is also electrical cable running adjacent to the water damaged area. See Photo 3 in Appendix 1.

5.3 Perimeter RC Wall

There are minor diagonal cracks of up to 1mm wide which appear mostly at the top corners of window and door openings. However, there is a moderate crack of up to 5mm wide which extends diagonally from the top corner of the window to the west of the front entrance. The crack appears on both sides of the wall creating a weather gap. See Photo 4 in Appendix 1.

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[2] Clause 7.1.1 (e) NZSEE (June 2006) suggested 300MPa for structural grade reinforcement of the 1930 – 70 period. A lower grade of 250MPa is adopted as a prudent measure since the building was built circa 1914

[3] Based on guidance from Clause 7.1.1 (f) NZSEE (June 2006), a conservative nominal strength of 25MPa is adopted as the concrete strength.

5.4 Non Structural

The external perimeter ground bearing concrete apron slab appears to have cracked at several locations especially at the corners. There is also minor separation of the apron from the perimeter RC wall. See Photo 5 in Appendix 1.

6 General Observations

The building has sustained minor to moderate earthquake related damage which is consistent with the expected building performance. The damage is expected to be cost effective to repair.

There had been some repairs made to the roof timber framing prior to the Canterbury earthquake. It is noted that some of the repairs do not appear to be engineered solutions and will need to be reviewed, in particular the joining of bottom chords of the roof framing. See Photo 6 in Appendix 1.

There are also stacks of prefabricated timber frames stored within the roof space at the northeast corner of the ceiling. See Photo 7 in Appendix 1. The roof truss is unlikely to be designed to support such heavy loading. There is also a risk of the timber frames becoming fall hazard in the event of strong earthquake shaking.

7 Critical Structural Weaknesses

As outlined in the Critical Structural Weakness and Collapse Hazards draft briefing document issued by the Structural Engineering Society (SESOC) on 19 July 2011, the term 'Critical Structural Weakness' (CSW) refers to a component/s or structural feature/s of a building that could contribute to increased levels of damage or cause premature collapse of a building.

Although there are no significant CSW's for this building, the lack of roof diaphragm/bracing could result in increased levels of damage to the longitudinal north and south elevation walls. This is because seismic load in the transverse (north-south) direction cannot be transferred back to the eastern and western in-plane shear walls and must instead be resisted by out of plane bending of the north and south walls.

8 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.

This quantitative assessment is intended to initially assess the residual capacity of the building in its undamaged state and then to assess the efficacy of repairs and strengthening as necessary.

8.1 Seismic coefficient parameters

The seismic design parameters based on current design requirements from NZS1170:2002 and the NZBC clause B1 for this building are:

- Site soil class D, clause 3.1.3 NZS 1170:2002
- 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 [1], for an Importance Level 2 structure with a 50 year design life.

Based on these parameters, static and modal response spectrum analyses were carried out to establish the actions on the structural elements.

8.2 Expected ductility factor

Based on our assessment of the building structural layout and using guidance from the concrete structures standard NZS3101:2006, our estimates for the expected maximum structural ductility factor for the primary seismic resisting systems are as follows:

Direction	**Element**	μ_{max}
Transverse	Perimeter RC wall	1.0
Longitudinal	Perimeter RC wall	1.0

8.3 Limitations and Assumptions in Results

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 drawings, 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.

8.4 Quantitative Analysis Methodology

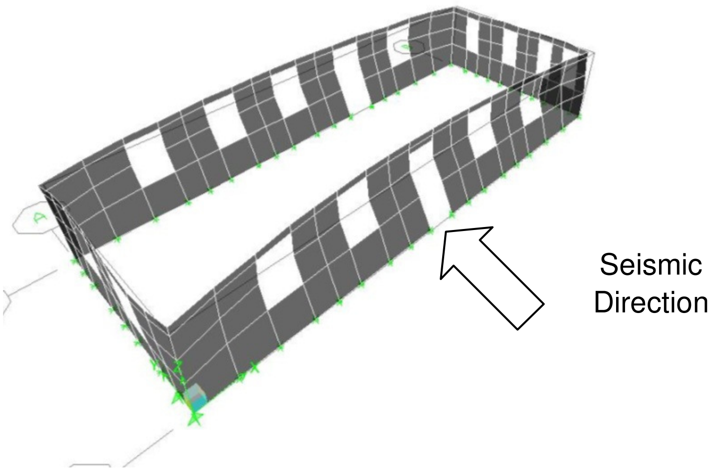
Since the roof is made of timber truss framing without any cross bracing, there is no diaphragm action to distribute the lateral loads to the perimeter RC walls parallel to the direction of the force.

A 3D model of the building was created and analysed in ETABS, which is a finite element structural analysis programme. The perimeter wall is assumed to be pin supported at the top and fixed at the bottom.

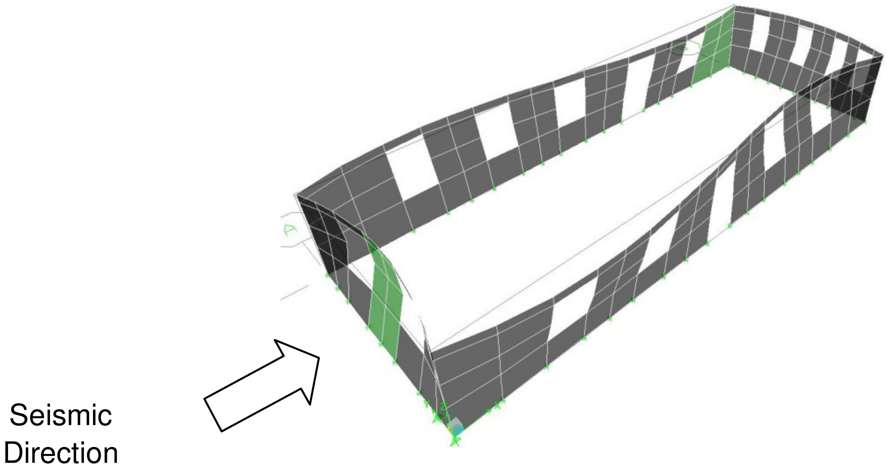
Key Components Analysed

Based on the results of the ETABS analysis, the walls with most critical design actions in shear and flexure (both in-plane and out-of-plane) in both the transverse and longitudinal seismic directions are checked against their respective capacities.

Transverse direction



Longitudinal direction



8.5 Quantitative Assessment Results

The results of the analysis are reported in the following table as %NBS, where for the component:

$$\% NBS = \frac{\text{Reliable Strength}}{\text{New Building Standard force}}$$

Structural Element/System	**Failure mode or description of limiting criteria based on elastic capacity of critical element**	**Critical Structural Weakness and Collapse Hazard**	**% NBS based on calculated capacity**
Primary Components (those that are required parts of the lateral resisting system)			
Transverse (N-S) Direction In-situ concrete wall on east and west elevation	Concrete shear wall resists lateral load in north-south direction loading. The failure mode is in-plane flexure.	No	>100% NBS
In-situ concrete wall on north and south elevation	Central section of wall cantilevering from fixed base support to resist lateral load in north-south direction loading. The failure mode is out of plane flexure.	No	56% NBS
Longitudinal (E-W) Direction In-situ concrete wall on north and south elevation	Concrete shear wall resists lateral load in east-west direction loading. The failure mode is in-plane flexure.	No	>100% NBS
In-situ concrete wall on east and west elevation	Central section of wall cantilevering from fixed base support to resist lateral load in east-west direction loading. The failure mode is out of plane flexure.	No	>100% NBS

8.6 Evaluation of Results

Based on the results of the analysis, the building has a seismic capacity of around 56% NBS as limited by the out of plane flexure capacity of the central section of the northern and southern wall. The building is therefore not considered as earthquake prone and has a moderate earthquake risk as defined in the NZSEE guidelines. We do not believe that occupancy needs to be restricted for this building.

Strengthening of the building to at least 67% NBS is recommended, and this could be achieved by the addition of a diaphragm or cross bracing at ceiling level.

9 Summary of Geotechnical Appraisal

9.1 Local 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 the Yaldhurst member of the Springston Formation with dominantly alluvial sand and silt overbank deposits.

9.2 Liquefaction Hazard

A liquefaction hazard study was conducted by the Canterbury Regional Council (ECan) in 2004 to identify areas of Christchurch susceptible to liquefaction during an earthquake. This Hagley Park site is located in an area identified as 'moderate ground damage potential may be expected' for a low groundwater scenario. According to this study, the ground damage potential is moderate indicating the ground may be affected by 100mm to 300mm of subsidence.

Tonkin and Taylor Ltd (T&T Ltd) have been engaged as the Earthquake Commission's (EQC) geotechnical consultants and have prepared maps showing areas of liquefaction interpreted from high resolution aerial photos for the 4th September earthquake, and the aftershocks of February 2011, June 2011 and December 2011. There is no surface evidence of liquefaction at the North Pavilion. However significant surface rupture of liquefaction is recorded throughout the grassed area 160m north of the building.

9.3 Summary

Based on current external evidence, the existing foundations are considered appropriate for the building with the client's acceptance that the potential for differential settlement may occur in future seismic events. Refer to Appendix 2 for the detailed geotechnical appraisal.

9.4 Further Work

It is recommended that:

- Based on the past performance in recent earthquakes, the existing foundations should be acceptable in terms of future ULS and SLS loadings, although CCC may have to accept the risk for potential differential settlement in the order of 0 to 50mm in a future seismic event;
- If CCC wishes to further evaluate and quantify the liquefaction potential at this site, additional site specific testing with CPT's and associated analysis would be necessary.

10 Conclusions

a) The overall seismic capacity for this building is around 56% NBS, and therefore the building is not considered as earthquake prone. However, it is recommended that the building be strengthened to at least 67% NBS.

b) The building structure has performed relatively well under the Canterbury earthquakes.

c) The building foundations appear to have performed satisfactorily with no observed earthquake damage.

d) Pre-earthquake damage within the roof timber framing needs to be re-assessed. In particular, the rotted timber post and beam and the repair made to the bottom chord of the truss should be assessed by a structural engineer.

e) The roof framing is not designed to support any significant ceiling load such as the prefabricated timber frames currently stored within the ceiling space.

f) The building has a moderate earthquake risk, and we do not believe that occupancy needs to be restricted.

11 Recommendations

a) Strengthen the building to at least 67% NBS with the installation of a roof diaphragm or cross bracing.

b) Seal the weather gap due to cracking at the top corner of the window to the west of the front entrance.

c) Replace the moisture damaged timber post and support beam near the west end of the hipped roof.

d) Review and fix as necessary the repairs made to the roof framing prior to the Canterbury earthquakes.

e) Remove the prefabricated timber frames stored within the ceiling space.

12 Limitations

a) This report is based on an inspection of the structure of the building and focuses on the structural damage resulting from the 22 February 2011 Canterbury Earthquake and aftershocks only. Some non-structural damage is described but this is not intended to be a complete list of damage to 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 this time.

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

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

No.	**Item description**	**Photo**
1.	General building elevations North elevation Southwest elevation	 
2.	Cracking to ground bearing slab at several locations	

<table>
<tr><td></td><td></td><td></td></tr>
<tr><td>3.</td><td>Damage to roof timber post and beam due to water leakage</td><td>
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4.	Diagonal cracking to window opening west of front entrance Crack appears to both sides of wall	  
5.	Cracking to perimeter concrete apron	

		
6.	Pre-earthquake nominal repairs to bottom chords of roof trusses	
7.	Prefabricated timber frames stored within roof space	

Appendix 2 – Geotechnical Appraisal

18 April 2012

Christchurch City Council
C/O:- Michael Sheffield
Property Asset Manager

6-QUCCC.46

Dear Michael

Geotechnical Desk Study – North Hagley Park Rugby Memorial Building

1. Introduction

Christchurch City Council (CCC) has commissioned Opus International Consultants (Opus) to undertake a Geotechnical Desk Study and site walkover of the North Hagley Park Rugby Memorial Building (North Pavilion), Christchurch. The purpose of this study is to collate existing subsoil information and undertake an appraisal of the potential geotechnical hazards at this site and to determine whether further investigations are required. The site walkover was completed by Opus on 1 November 2011.

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.

The Geotechnical Desk Study forms part of a Detailed Engineering Evaluation prepared by Opus. A level survey has not been undertaken. The Geotechnical Desk Study 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 North Pavilion is located in the north eastern quarter of Hagley Park, adjacent to the Hagley Park North RSA Bowling Club, tennis courts and opposite Armagh Street. The Avon River, at its closest point, is approximately 45m south of the building. Refer to the Site Location Plan in Appendix B.

The North Pavilion building is a one storey masonry building. Refer to the Opus Qualitative Structural Assessment Report for more detailed description of the building.

No Geotechnical Reports or site specific investigations were available from the CCC Property file.

The ground profile is relatively flat and level with the adjacent pavement and grassed areas.

2.2 Structural Drawings

Extracts from the Structural Drawings illustrating details of the foundation have not been available for review from CCC property files. It is assumed that the building is founded on perimeter strip footings and a concrete floor slab.

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 the Yaldhurst member of the Springston Formation with dominantly alluvial sand and silt overbank deposits.

2.4 Expected Ground Conditions

A review of the Environmental Canterbury (ECan) wells database showed four wells located within approximately 330m of the property (refer to Site Location Plan in Appendix B). The locations of Boreholes and CPT's by the Earthquake Commission have been reviewed. The nearest Borehole is located 270m east of the site. Material logs available from these wells 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)
Sandy SILT	2-2.4m	Surface
Sandy GRAVEL	11.5-12.8m	2-2.4m
SILT and Clay	0.5-4.3m	13.5-15.2m
SAND	7.75m	14m
GRAVEL (Riccarton Formation)	-	19.5-21.75m

A groundwater depth of approximately 1.5m to 2.5m below ground level has been estimated from groundwater depth contour maps (Environment Canterbury (2003) and Elder et al. (1991)).

2.5 Liquefaction Hazard

A liquefaction hazard study was conducted by the Canterbury Regional Council (ECan) in 2004 to identify areas of Christchurch susceptible to liquefaction during an earthquake. This Hagley Park site is located in an area identified as 'moderate ground damage potential may be expected' for a low groundwater scenario. According to this study, the ground damage potential is moderate indicating the ground may be affected by 100mm to 300mm of subsidence.

Tonkin and Taylor Ltd (T&T Ltd) have been engaged as the Earthquake Commission's (EQC) geotechnical consultants and have prepared maps showing areas of liquefaction interpreted from high resolution aerial photos for the 4th September earthquake, and the aftershocks of February 2011, June 2011 and December 2011. There is no surface evidence of liquefaction at the North Pavilion. However significant surface rupture of liquefaction is recorded throughout the grassed area 160m north of the building.

3. Site Walkover Inspection

A walkover inspection of the exterior, interior, and adjacent areas was carried out by an Opus Geologist on 2 November 2011. The following observations were made (refer to the Site Photos attached to this report):

- The floor slab appears to have settled 5mm in (Photo 4).
- The concrete footpath surrounding the building has cracked and settled by up to 20mm (Photo 2).
- During the internal inspection it was apparent that the building had suffered some shaking induced damage with minor cracking. The floor slab is cracked in two locations (Photo 5), one to the east which was deemed to be oldest. The western crack is approximately 2mm wide and appears to have been earthquake induced.
- The roadway adjacent to the building appears undamaged by the earthquakes but the adjacent paved tennis courts located 25m north west of the building have suffered cracking and shaking induced ground deformation.
- A service trench located north of the building has settled by approximately 50mm (Photo 3).

4. Discussion

Minor land damage has occurred to the North Pavilion building due to the Canterbury Earthquake Sequence following the 4 September 2010 earthquake. Surface rupture of liquefaction has occurred to the grassed areas 160m north of the site.

Some minor cracking of the floor has been noted, which is inferred to be seismic shaking induced rather than liquefaction induced settlement.

Settlement (varying from 10mm to 20mm) in the concrete paving around the building and 5mm of settlement has occurred in the floor slab.

ECan well logs indicate the building is likely to be founded on a thin layer of silt and sand overlying 11m to 13m thick gravel layer. We would expect some liquefaction resistance, which is reflected in the relatively good performance of the foundations.

There is no evidence that the retaining structures around the edge of Victoria Lake have moved, which would indicate that there has not been any significant lateral spreading and ground deformation around the lake.

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 15% 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.

No level survey or site investigations have been undertaken as part of this Desk Study.

Based on current external evidence, the existing foundations are considered appropriate for the building with the client's acceptance that the potential for differential settlement may occur in future seismic events.

If the CCC wish to quantify the potential liquefaction induced settlement following an ULS seismic event, further ground information would be required.

5. Recommendations

It is recommended that:

- Based on the past performance in recent earthquakes, the existing foundations should be acceptable in terms of future ULS and SLS loadings, although CCC may have to accept the risk for potential differential settlement in the order of 0 to 50mm in a future seismic event;
- If CCC wishes to further evaluate and quantify the liquefaction potential at this site, additional site specific testing with CPT's and associated analysis would be necessary.

6. 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.

7. References:

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 Soild 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 2 April 2012.

Appendices:

Appendix A: Site Photos
Appendix B: Site Location Plan
Appendix C: ECan and EQC Borehole Logs

APPENDIX A:

Site Photos

Photo 1: North elevation of the North Pavilion.

Photo 2: The concrete footpath on the east elevation has settled approximately 20mm.

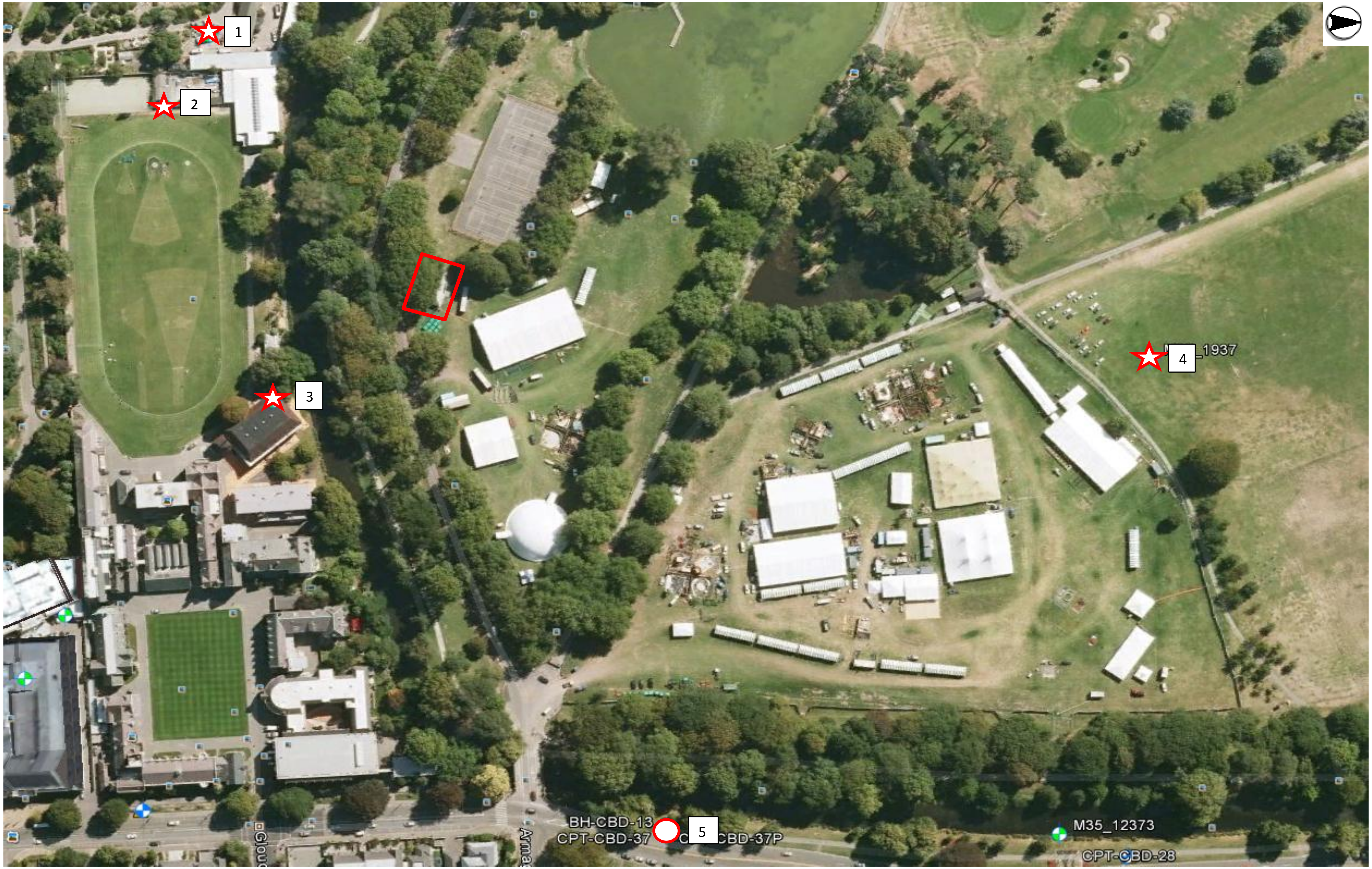
Photo 3: The ground surrounding the water pipe has settled by approximately 50mm.

Photo 4 : The floor slab has settled approximately 5mm.

Photo 5: Internal floor crack that is approximately 2mm wide.

APPENDIX B:

Site Location Plan



☆ ECan Borehole Location

○ EQC Borehole Location

BH ref	ECan ref
1	M35/10619
2	M35/1936
3	M35/7631
4	M35/1937
5	BH-CBD-13

0 100m

Approximate Scale 1:2300 at A3

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: North Hagley Park Rugby Memorial Building
Geotechnical Desk Study

Project No.: 6-QUCCC.46

Client: Christchurch City Council

Site Location Plan

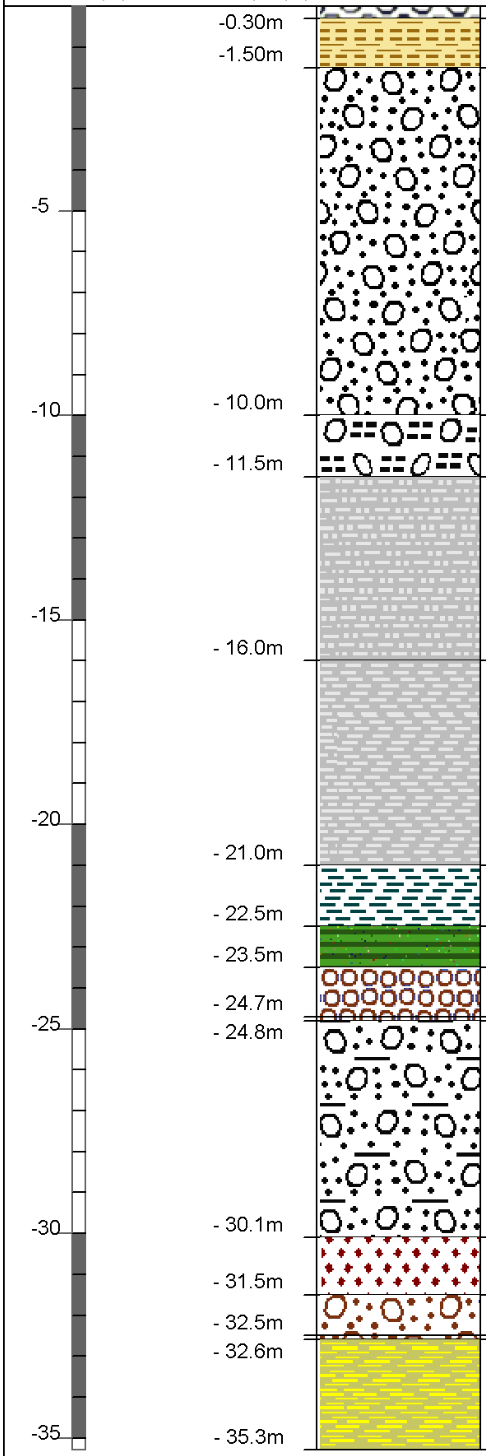
Drawn: Opus Geotechnical Engineer

Date: 28-Mar-12

APPENDIX C:
Environment Canterbury Borehole Logs

Borelog for well M35/10619 page 1 of 3

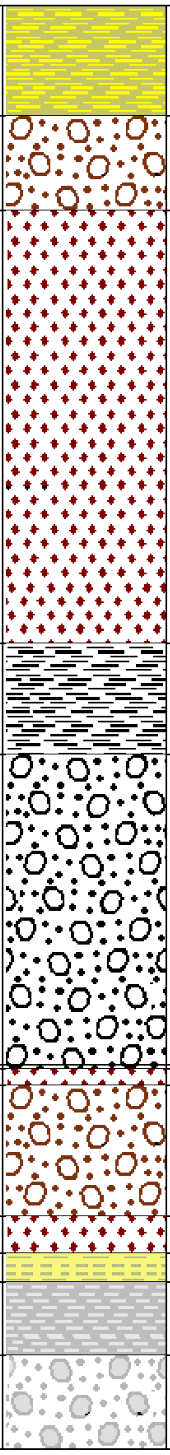
Gridref: M35:7952-4188 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 7.5 +MSD
Driller : Clemence Drilling Contractors
Drill Method : Rotary/Percussion
Drill Depth : -105.9m Drill Date : 6/10/2006



Water Level	Depth(m)	Full Drillers Description	Formation Code
	-0.30m	topsoil	sp
	-1.50m	silty brown clay	sp
	- 10.0m	gravel and sand	sp
	- 11.5m	grey pug and gravel	sp
	- 16.0m	puggy grey sand	ch?
	- 21.0m	soft silty grey pug	ch?
	- 22.5m	hard blue/green pug	ch?
	- 23.5m	soft puggy peat	ch
	- 24.7m	tight brown gravel	ri
	- 24.8m	yellow clay seam	ri
	- 30.1m	tight sandy brown gravel (traces of clay)	ri
	- 31.5m	brown sand	ri
	- 32.5m	loose brown sandy gravel	ri
	- 32.6m	yellow clay seam	ri
	- 35.3m	hard sticky yellow clay	ri

Borelog for well M35/10619 page 2 of 3

Gridref: M35:7952-4188 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 7.5 +MSD
Driller : Clemence Drilling Contractors
Drill Method : Rotary/Percussion
Drill Depth : -105.9m Drill Date : 6/10/2006



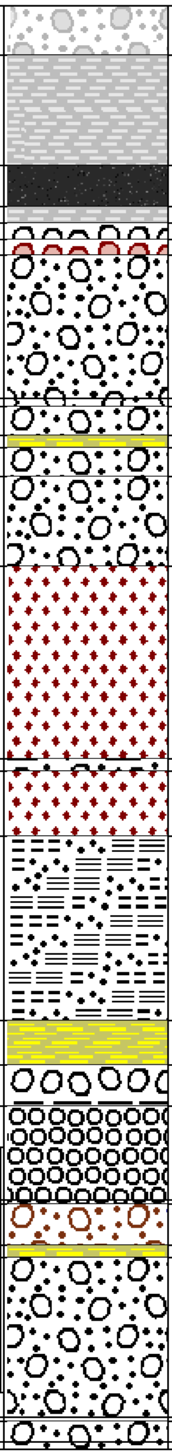
Scale(m)	Water Level	Depth(m)	Full Drillers Description	Formation Code
		- 38.0m	hard sticky yellow clay	ri
-40		- 40.3m	small brown gravel - progressively sandier	br
-45 / -50		- 50.9m	brown sand	br
		- 53.6m	hard sticky yellow/orange clay	br
-55 / -60		- 61.2m	sandy grey/brown gravel	li-1
		- 61.3m	clay seam	li-2
		- 61.7m	brown sand (rusty water)	li-2
		- 64.9m	tight brown stained gravel - sandy	li-2
-65		- 65.8m	brown sand (traces gravel)	li-2
		- 66.5m	hard silty yellow clay	li-2
		- 68.3m	silty grey pug (traces peat)	li-2
-70		- 70.6m	loose grey sandy gravel	li-2

Borelog for well M35/10619 page 3 of 3

Gridref: M35:7952-4188 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 7.5 +MSD
Driller : Clemence Drilling Contractors
Drill Method : Rotary/Percussion
Drill Depth : -105.9m Drill Date : 6/10/2006

Environment Canterbury Regional Council

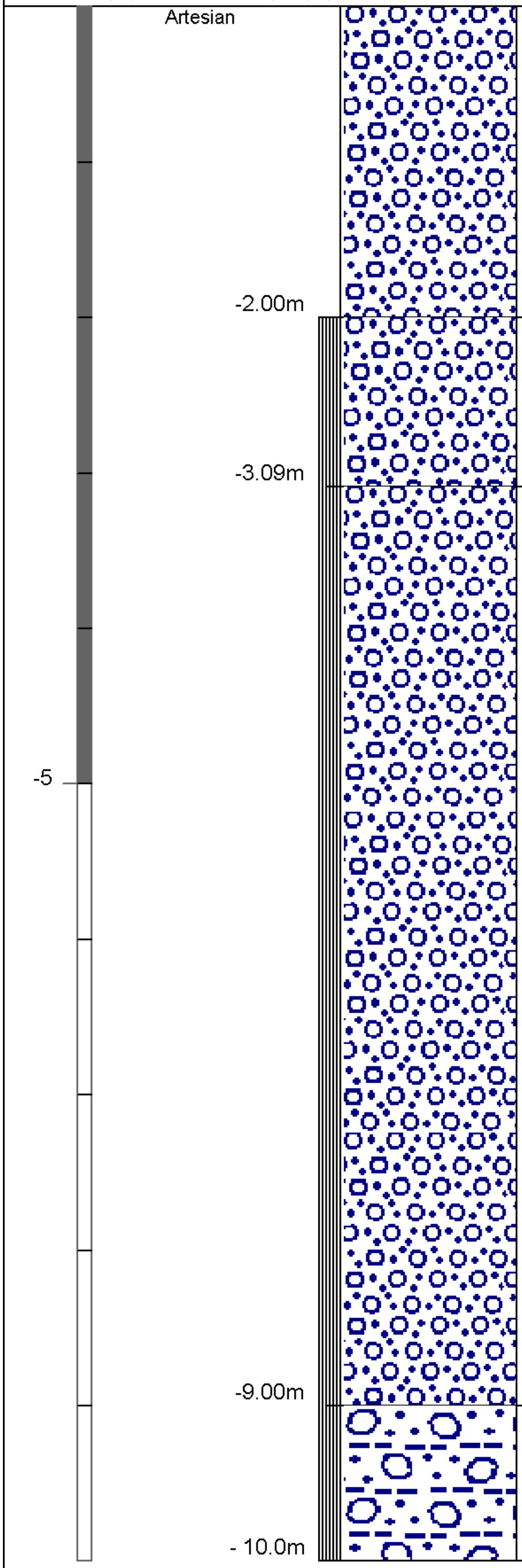
Scale(m)
-75
-80
-85
-90
-95
-100
-105



Depth(m)	Full Drillers Description	Formation Code
- 71.8m	loose grey sandy gravel	li-2
- 74.5m	soft sticky grey pug (traces peat)	li-2
- 75.5m	peat (some timber)	li-2
- 75.9m	hard sticky grey pug	li-2
- 76.3m	grey/blue clay bound gravel	li-3
- 76.7m	brown clay bound gravel	li-3
- 80.2m	loose very sandy heavily stained gravel	li-3
- 80.4m	hard sticky yellow clay	li-3
- 81.1m	tight sandy stained gravel	li-3
- 81.4m	hard yellow clay	li-3
- 82.1m	tight lightly stained sandy gravel	li-3
- 84.3m	tight lightly stained very sandy gravel	li-3
- 89.0m	brown sand (traces gravel)	he
- 89.3m	small sandy brown gravel (traces clay)	he
- 90.9m	brown sand (traces gravel)	he
- 95.4m	hard silty/sandy yellow/brown clay	he
- 96.5m	hard sticky yellow clay	he
- 97.5m	claybound gravel	bu
- 99.8m	loose grey/brown gravel	bu
- 99.9m	yellow clay seam	bu
- 100.9m	loose sandy brown gravel	bu
- 101.2m	hard yellow clay	bu
- 105.1m	very loose sandy grey/brown gravel	bu
- 105.2m	yellow clay seam	bu
- 105.7m	large loose stained sandy gravel (some heavily stained)	bu?
- 107.5m		

Borelog for well M35/7631

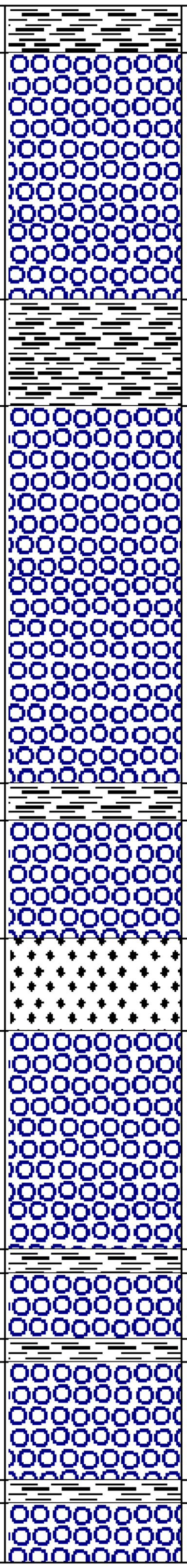
Gridref: M35:797-419 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 6.7 +MSD
Driller : McMillan Water Wells Ltd
Drill Method : Rotary Rig
Drill Depth : -10m Drill Date : 21/03/1994



Water Level	Depth(m)	Full Drillers Description	Formation Code
Artesian	-2.00m	fine-med sand and gravel (to 50 mm) slight tph odour. grey	sp
	-3.09m	silty gravel sand (fine med) and gravel (to 60mm) slight tph odour grey	sp
	-9.00m	silty gravel (to 60mm) with minor sand(fine-med)wet tph odour in upper part of unit	sp
	- 10.0m	Gravel (to 40mm) Brown stained sand (med-coarse) trace silt no tph odour	sp

Borelog for well M35/7410

Gridref: M35:7923-4221 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 7.9 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -80.5m Drill Date : 1/12/1906

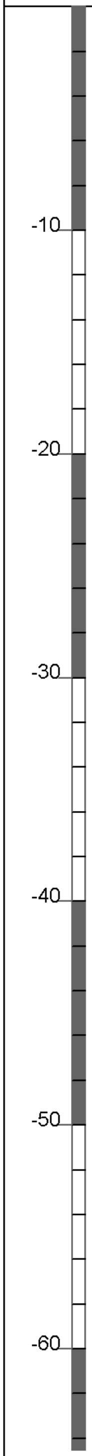


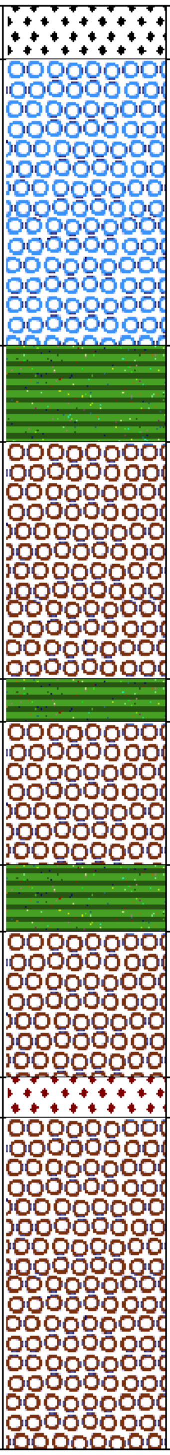
Water Level	Depth(m)	Full Drillers Description	Formation Code
Artesian	-2.40m	Surface soil & Yellow clay	sp?
	- 15.2m	Brown shingle	sp?
	- 20.7m	Blue clay	ch
	- 40.2m	Brown shingle,water 1.2m from surface	ri
	- 42.1m	Blue clay	br
	- 48.2m	Brown shingle	br
	- 53.0m	Brown sand	br
	- 64.3m	Brown shingle,water rise 1.37m above surface	li
	- 65.5m	Blue clay	li
	- 68.9m	Brown shingle	li
	- 70.1m	Yellow clay	li
	- 76.2m	Brown shingle	li
	- 77.4m	Yellow clay	li
	- 80.5m	Brown shingle,water rise 2.44m & flow 1.5 l/s at surface	li

Scale(m): -10, -20, -30, -40, -50, -60, -70, -80

Borelog for well M35/1937 page 1 of 2

Gridref: M35:797-423 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 6.9 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -129.1m Drill Date : 24/01/1906

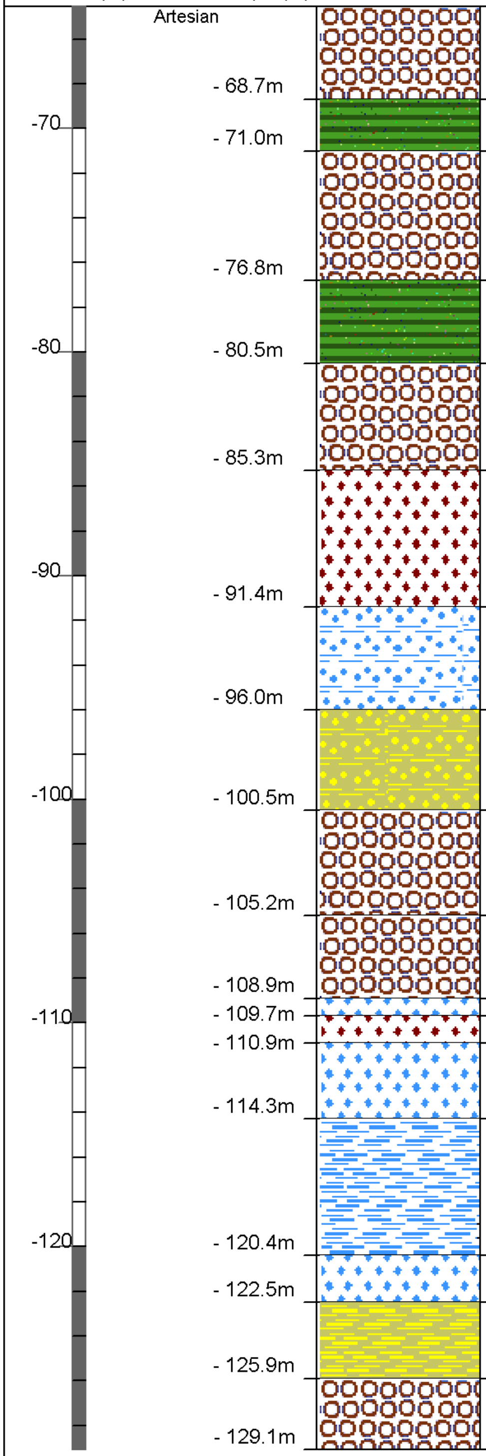




Water Level	Depth(m)	Full Drillers Description	Formation Code
Artesian	-2.40m	Sand	sp?
	- 15.2m	Gravel, blue	sp?
	- 19.5m	Blue clay (Peat at 15.8m)	ch
	- 30.1m	Brown gravel	ri
	- 32.0m	Blue clay & peat	ri
	- 38.4m	Brown gravel	ri
	- 41.4m	Blue clay (Peat at 39.6m)	br
	- 47.9m	Brown gravel	br
	- 49.7m	Brown sand	br
	- 64.6m	Brown gravel	li-1

Borelog for well M35/1937 page 2 of 2

Gridref: M35:797-423 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 6.9 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -129.1m Drill Date : 24/01/1906



Water Level	Depth(m)	Full Drillers Description	Formation Code
Artesian	- 68.7m	Brown gravel	li-1
	- 71.0m	Blue clay (Peat at 70.1m)	li-2
	- 76.8m	Brown gravel	li-2
	- 80.5m	Blue clay (Peat at 77.7m)	li-2
	- 85.3m	Brown gravel, water level +3.0m, flow 262m3/d	li-3
	- 91.4m	Brown sand	he
	- 96.0m	Blue sandy clay	he
	- 100.5m	Yellow clay & sand	he
	- 105.2m	Brown gravel, water level +4.6m, flow 524m3/d	bu
	- 108.9m	Brown gravel	bu
	- 109.7m	Blue sand	sh
	- 110.9m	Brown sand	sh
	- 114.3m	Blue sand	sh
	- 120.4m	Blue clay	sh
	- 122.5m	Blue sand	sh
	- 125.9m	Yellow clay	sh
	- 129.1m	Brown gravel, water level +8.8m	wa

Borelog for well M35/1936 page 1 of 2

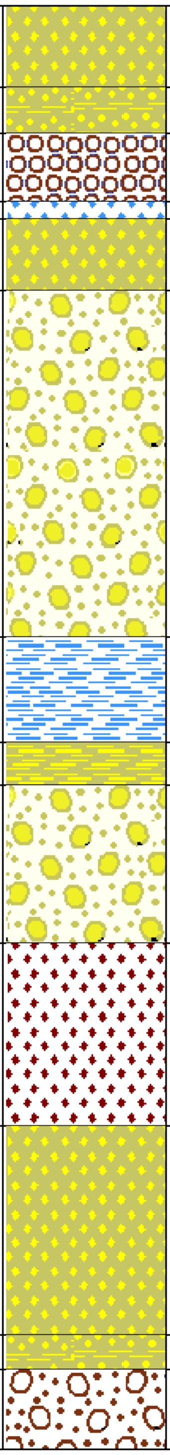
Gridref: M35:79554-41858 Accuracy : 2 (1=high, 5=low)
Ground Level Altitude : 7.6 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -100.9m Drill Date : 2/07/1898

Environment Canterbury
Regional Council

Water Level	Depth(m)	Full Drillers Description	Formation Code
Artesian	-2.40m	Surface soil & Yellow sand	sp?
	- 14.6m	Hard Blue shingle	sp?
	- 19.8m	Soft Blue sands	ch
	- 22.2m	Clay and peat	ch
	- 37.5m	Brown shingle 1st strata water .9m below surface	ri
	- 41.8m	Clay & peat	br
	- 44.8m	Hard Brown sand & gravel	br
	- 50.5m	Soft Yellow sand	br

Borelog for well M35/1936 page 2 of 2

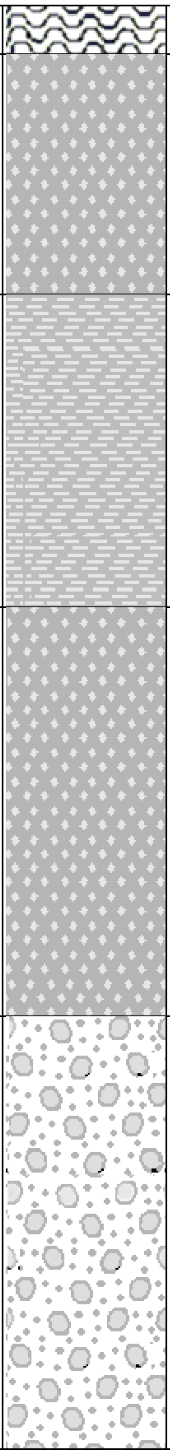
Gridref: M35:79554-41858 Accuracy : 2 (1=high, 5=low)
Ground Level Altitude : 7.6 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -100.9m Drill Date : 2/07/1898



Scale(m)	Water Level	Depth(m)	Full Drillers Description	Formation Code
	Artesian	- 53.3m	Soft Yellow sand	br
		- 54.9m	Soft Yellow clay & sand mixed	br
		- 57.3m	Hard Brown shingle	li-1
		- 57.9m	Soft Blue sand	li-2
-60		- 60.4m	Soft Yellow sand	li-2
-70		- 72.5m	Hard Yellow sand & gravel water rise +0.91m flow 6 gpm	li-2
		- 76.2m	Soft Blue clay	li-2
		- 77.7m	Soft Yellow clay	li-2
-80		- 83.2m	Hard Yellow sand & gravel. Water rise +2.7m & flow 30gpm 0.61m high	li-3
		- 89.6m	Soft Brown sand	he
-90		- 96.9m	Soft Yellow sand	he
		- 98.1m	Soft Yellow sand with clay	he
-100		- 100.9m	Brown gravel & sand. Water rise +4.3m flow 45gpm 1.1m high	bu

Borelog for well M35/12373

Gridref: M35:79930-42247 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 7.92 +MSD
Well name : CCC BorelogID 448
Drill Method : Not Recorded
Drill Depth : -6m Drill Date :



Water Level	Depth(m)	Full Drillers Description	Formation Code
	-0.20m	black loose moist topsoil	
	-1.20m	grey loose moist sand	
	-2.50m	grey wet silt	
	-4.20m	grey saturated running sand	
	-6.00m	grey saturated gravel and running sand	

TONKIN & TAYLOR LTD

BOREHOLE LOG

BOREHOLE No: CBD 13
Hole Location: Opposite 18 Park Tce
SHEET 1 OF 6

PROJECT: CHRISTCHURCH CITY 2011 EARTHQUAKE
LOCATION: CENTRAL CITY
JOB No: 52000.3400

CO-ORDINATES 5742068.55 mN
2479919.32 mE
R.L. 6.87 m
DATUM NZMG

DRILL TYPE: Rotary
DRILL METHOD: OB/TT
DRILL FLUID: Mud

HOLE STARTED: 3/8/11
HOLE FINISHED: 5/8/11
DRILLED BY: Pro-Drill
LOGGED BY: CP
CHECKED: BMcD

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa)	COMPRESSIVE STRENGTH (MPa)	DEFECT SPACING (mm)	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.
HAND DIG FILL. (Potholed for services check and backfilled.)			0	PRE-DUG				6.5 6.0 5.5	0.5 1.0 1.5							FILL: Borehole drilled through pre-dug and backfilled pothole.
YALDHURST MEMBER OF THE SPRINGSTON FORMATION (ALLUVIAL)				SPT		1/1/1/0/1/1 N=3		5.0		ML	M	S				Sandy SILT with rare fibrous rootlets, brown. Soft, moist, low plasticity. 1.7 to 2.0m no recovery
			95	OB				4.5 4.0	2.0 2.5 3.0	GW	M	D				Sandy, fine to coarse GRAVEL with rare cobbles, grey. Dense, moist. Gravel is subrounded to subangular. Sand is medium to coarse.
				SPT		2/3/6/7/9/9 N=31		3.5	3.5							
			100	OB				3.0 2.5	4.0 4.5							
				SPT		6/19/13/ 10/8/11 N=42		2.0	5							

TONKIN & TAYLOR LTD

BOREHOLE LOG

BOREHOLE No: CBD 13
Hole Location: Opposite 18 Park Tce

SHEET 2 OF 6

PROJECT: CHRISTCHURCH CITY 2011 EARTHQUAKE
LOCATION: CENTRAL CITY
JOB No: 52000.3400

CO-ORDINATES 5742068.55 mN
2479919.32 mE
R.L. 6.87 m
DATUM NZMG

DRILL TYPE: Rotary
DRILL METHOD: OB/TT
DRILL FLUID: Mud

HOLE STARTED: 3/8/11
HOLE FINISHED: 5/8/11
DRILLED BY: Pro-Drill
LOGGED BY: CP
CHECKED: BMcD

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

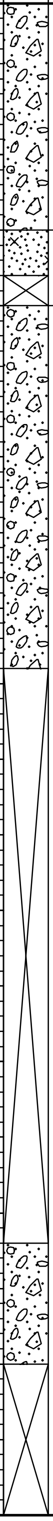
YALDHURST MEMBER OF THE SPRINGSTON FORMATION (ALLUVIAL)

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.

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)	CORE RECOVERY (%)	METHOD	TESTS	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SOIL DESCRIPTION
5.0 – 5.75	90	OB		GW	M	D	Sandy, fine to coarse GRAVEL with rare cobbles, grey. Dense, moist. Gravel is subrounded to subangular. Sand is medium to coarse.
5.75 – 5.9				SW	M	D	Fine to coarse SAND with trace silt, grey motled orange brown. Dense, moist.
5.9 – 6.0							5.9 to 6.0m no recovery
6.0 – 6.45		SPT	4/9/12/ 18/10/10 N=50	GW	M	VD	Sandy, fine to coarse GRAVEL with rare cobbles, grey. Very dense, moist. Gravel is subrounded to subangular. Sand is medium to coarse.
6.45 – 7.5	71	OB					
7.2 – 9.1							7.2 to 9.1m no recovery
7.5 – 7.8		SPT	12/15/ 20/30 for 75mm N>50				
7.8 – 9.5	26	OB					
9.1 – 9.5							- contains trace cobbles. Fines washed away during drilling process.
9.5 – 9.65		SPT	25/25 for 75mm N>50				9.5 to 10.55m no recovery

R.L. (m)	DEPTH (m)
1.5	
	5.5
1.0	
	6.0
0.5	
	6.5
0.0	
	7.0
-0.5	
	7.5
-1.0	
	8.0
-1.5	
	8.5
-2.0	
	9.0
-2.5	
	9.5
-3.0	
	10

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BOREHOLE LOG

BOREHOLE No: CBD 13
Hole Location: Opposite 18 Park Tce

SHEET 3 OF 6

PROJECT: CHRISTCHURCH CITY 2011 EARTHQUAKE
LOCATION: CENTRAL CITY
JOB No: 52000.3400

CO-ORDINATES 5742068.55 mN
2479919.32 mE
R.L. 6.87 m
DATUM NZMG

DRILL TYPE: Rotary
DRILL METHOD: OB/TT
DRILL FLUID: Mud

HOLE STARTED: 3/8/11
HOLE FINISHED: 5/8/11
DRILLED BY: Pro-Drill
LOGGED BY: CP
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.

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	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	R.L. (m)	DEPTH (m)	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SOIL DESCRIPTION
YALDHURST MEMBER OF THE SPRINGSTON FORMATION (ALLUVIAL)			43	HQTT			-3.5					
								10.5	SW		MD	Fine to medium SAND with trace silt. Medium dense, moist.
							-4.0	11.0				
				SPT		8/7/6/5/7/7 N=25	-4.5					
			24	HQTT				11.5				11.45 to 12.25m no recovery
							-5.0	12.0				
												- fines washed during drilling process
							-5.5	12.5	SW	M	MD	Fine to medium SAND with trace gravel, brown. Medium dense, moist. Gravel is medium to coarse, subrounded to subangular.
				SPT		5/2/3/3/3/7 N=16	-6.0					
			81	HQTT				13.0				12.95 to 13.05m no recovery - contains minor gravel
CHRISTCHURCH FORMATION (MARINE AND ESTUARINE)							-6.5		ML	M	VSt	SILT with trace sand, grey. Very stiff, moist, low plasticity. Sand is fine.
								13.5				- contains minor sand
							-7.0					13.9 to 14.0m no recovery
				SPT		2/9/13/14/16/7 for 35mm N>50		14.0	SW	M	VD	Fine to medium SAND with some silt and trace organics, grey. Very dense, moist.
							-7.5	14.5				- contains minor silt
			100	HQTT			-8.0	15				

BOREHOLE LOG

BOREHOLE No: CBD 13
Hole Location: Opposite 18 Park Tce
SHEET 4 OF 6

PROJECT: CHRISTCHURCH CITY 2011 EARTHQUAKE
LOCATION: CENTRAL CITY
JOB No: 52000.3400

CO-ORDINATES 5742068.55 mN
2479919.32 mE
R.L. 6.87 m
DATUM NZMG

DRILL TYPE: Rotary
DRILL METHOD: OB/TT
DRILL FLUID: Mud

HOLE STARTED: 3/8/11
HOLE FINISHED: 5/8/11
DRILLED BY: Pro-Drill
LOGGED BY: CP
CHECKED: BMcD

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	CORE RECOVERY (%)	METHOD	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	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 AND ESTUARINE)							SW	M	VD	Fine to medium SAND with minor silt and trace organics, grey. Very dense, moist. - contains some very closely spaced silt laminae.
					-8.5					
		SPT	1/2/3/4/9/9 N=25			15.5			MD	- becoming medium dense. - contains minor silt. Becoming brown.
					-9.0					
	76	HQTT	✱FC	B		16.0				
					-9.5					
						16.5				16.75 to 17.0m no recovery
					-10.0					
		SPT	2/2/7/11/ 13/17 N=48			17.0			D	- becoming dense
					-10.5					
	100	HQTT				17.5				
					-11.0					
			✱FC	B		18.0				
					-11.5					
		SPT	1/5/9/ 13/17/11 for 50mm N>50			18.5			VD	- becoming very dense
					-12.0					
	86	HQTT				19.0				
					-12.5					
						19.5				- interbedded silt and traces of shells. Becoming grey.
					-13.0					19.85 to 20.0m no recovery
						20				

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BOREHOLE LOG

BOREHOLE No: CBD 13
Hole Location: Opposite 18 Park Tce
SHEET 5 OF 6

PROJECT: CHRISTCHURCH CITY 2011 EARTHQUAKE
LOCATION: CENTRAL CITY
JOB No: 52000.3400

CO-ORDINATES	5742068.55 mN 2479919.32 mE	DRILL TYPE: Rotary	HOLE STARTED: 3/8/11
		DRILL METHOD: OB/TT	HOLE FINISHED: 5/8/11
R.L.	6.87 m		DRILLED BY: Pro-Drill
DATUM	NZMG	DRILL FLUID: Mud	LOGGED BY: CP CHECKED: BMcD

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 AND ESTUARINE)				SPT		6/11/11/8/ 5/4 N=28		-13.5			SW	M	MD				Fine to medium SAND with trace interbedded silt and organics, grey. Very dense, moist.
			71	HQTT				-14.0	20.5		SW	M	MD				Silty, fine to medium SAND with minor organics (wood), grey. Medium dense, moist.
									21.0								- becoming brown
								-14.5									21.2 to 21.8 no recovery
RICCARTON GRAVELS				S. SPT		12/18/ 23/25 for 55mm N>50			21.5								
			100	HQTT				-15.0 -15.5 -16.0	22.0 22.5		GW	M	VD				Medium to coarse GRAVEL, grey. Very dense, moist. Gravel is subrounded to subangular. Fines washed away during drilling process.
				SPT		18/32 for 75mm N>50			23.0								23.05 to 24.05m no recovery
			33	HQTT				-16.5 -17.0	23.5 24.0								
								-17.5			GW	M	VD				Sandy, fine to coarse GRAVEL with trace silt, brown. Very dense, moist. Gravel is subrounded to subangular. Sand is fine to coarse.
				SPT		5/11/13/ 15/12/10 for 45mm N>50		-18.0	24.5 25		GW	M	VD				Medium to coarse GRAVEL, grey. Very dense, moist. Gravel is subrounded to subangular. Fines washed away during drilling process.

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BOREHOLE LOG

BOREHOLE No: CBD 13
Hole Location: Opposite 18 Park Tce
SHEET 6 OF 6

PROJECT: CHRISTCHURCH CITY 2011 EARTHQUAKE
LOCATION: CENTRAL CITY
JOB No: 52000.3400

CO-ORDINATES 5742068.55 mN
2479919.32 mE
R.L. 6.87 m
DATUM NZMG

DRILL TYPE: Rotary
DRILL METHOD: OB/TT
DRILL FLUID: Mud

HOLE STARTED: 3/8/11
HOLE FINISHED: 5/8/11
DRILLED BY: Pro-Drill
LOGGED BY: CP
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.

RICCARTON GRAVELS

Depth (m)	R.L. (m)	Core recovery (%)	Method	Tests	Strength/Density	Description
25.0–26.0	-18.5 (25.5)	29	HQTT		D	25.0 to 25.7m no recovery - becoming dense
26.0–26.4	-19.0 (26.0)		S. SPT	10/10/12/ 13/15/10 for 40mm N>50		26.0 to 27.2m no recovery
26.4–27.5	-19.5 (26.5), -20.0 (27.0)	28	HQTT			
27.5–27.65	-20.5 (27.5)		S. SPT	25/25 for75mm N>50		27.5 to 28.75m no recovery
27.65–29.0	-21.0 (28.0), -21.5 (28.5), -22.0 (29.0)	19	HQTT			
29.0–29.15			SPT	23/27 for70mm N>50		29.0 to 29.15m no recovery

End of borehole at 29.15mbgl. Open standpipe piezometer installed. Please see attached diagram in Appendix F.

R.L. scale: -22.5, -23.0; Depth scale: 29.5, 30

Appendix 3 – CERA DEE Spreadsheet

Detailed Engineering Evaluation Summary Data

V1.11

Location

Building Name: Hagley Park North Pavilion
Unit No: Street
Building Address: North Hagley Park
Legal Description:

Degrees Min Sec
GPS south: 43 31 44.52
GPS east: 172 37 27.88

Building Unique Identifier (CCC): PRK_1190_BLDG_010

Reviewer: Alistair Boyce
CPEng No: 209860
Company: Opus International Consultants
Company project number: 6QUCCC.46
Company phone number: 03-3635400

Date of submission: 4-Oct-12
Inspection Date: 9 Aug 11 and 26Mar12
Revision: Final
Is there a full report with this summary? yes

Site

Site slope: flat
Soil type: mixed
Site Class (to NZS1170.5): D
Proximity to waterway (m, if <100m): 45
Proximity to clifftop (m, if < 100m):
Proximity to cliff base (m,if <100m):

Max retaining height (m):
Soil Profile (if available):
If Ground improvement on site, describe:
Approx site elevation (m):

Building

No. of storeys above ground: 1 single storey = 1
Ground floor split? no
Storeys below ground 1
Foundation type: strip footings
Building height (m): 3.80
Floor footprint area (approx): 109
Age of Building (years): 98

Ground floor elevation (Absolute) (m): 2.80
Ground floor elevation above ground (m): 0.10
if Foundation type is other, describe:
height from ground to level of uppermost seismic mass (for IEP only) (m): 3.1
Date of design: Pre 1935

Strengthening present? no
Use (ground floor): other (specify)
Use (upper floors):
Use notes (if required): Storage & office & kitchen
Importance level (to NZS1170.5): IL2

If so, when (year)?
And what load level (%g)?
Brief strengthening description:

Gravity Structure

Gravity System: load bearing walls
Roof: timber truss
Floors: other (note)
Beams: none
Columns:
Walls: load bearing concrete

truss depth, purlin type and cladding 100x50
describe sytem concrete ground bearing slab
overall depth x width (mm x mm)
#N/A

Lateral load resisting structure

Lateral system along: concrete shear wall
Ductility assumed, μ: 1.00
Period along: 0.20
Total deflection (ULS) (mm): 3
maximum interstorey deflection (ULS) (mm): 3

Note: Define along and across in detailed report!
0.01 from parameters in sheet

note total length of wall at ground (m): 33
wall thickness (m): 0.102
estimate or calculation? calculated
estimate or calculation? calculated
estimate or calculation? calculated

Lateral system across: concrete shear wall
Ductility assumed, μ: 1.00
Period across: 0.20
Total deflection (ULS) (mm): 8
maximum interstorey deflection (ULS) (mm): 8

0.05 from parameters in sheet

note total length of wall at ground (m): 12
wall thickness (m): 0.102
estimate or calculation? calculated
estimate or calculation? calculated
estimate or calculation? calculated

Separations:

north (mm):
east (mm):
south (mm):
west (mm):

leave blank if not relevant

Non-structural elements

Stairs:
Wall cladding:
Roof Cladding: Metal
Glazing: steel frames
Ceilings: light tiles
Services(list):

describe corrugated iron

Available documentation

Architectural none
Structural none
Mechanical none
Electrical none
Geotech report partial

original designer name/date
original designer name/date
original designer name/date
original designer name/date
original designer name/date Opus / Apr 2012

Damage

Site:
(refer DEE Table 4-2)

Site performance:
Settlement: 0-25mm
Differential settlement: none observed
Liquefaction: none apparent
Lateral Spread: none apparent
Differential lateral spread: none apparent
Ground cracks: 0-20mm/20m
Damage to area: slight

Describe damage:
notes (if applicable):
notes (if applicable):
notes (if applicable):
notes (if applicable):
notes (if applicable):
notes (if applicable): Minor cracking to building concrete apron
notes (if applicable):

Building:

Current Placard Status: green

Along Damage ratio: 0%
Describe (summary):

Describe how damage ratio arrived at: Minimal observed damage

Across Damage ratio: 0%
Describe (summary):

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

Diaphragms Damage?: no Describe:
CSWs: Damage?: no Describe:
Pounding: Damage?: no Describe:
Non-structural: Damage?: yes Describe: Minor cracking

Recommendations

Level of repair/strengthening required: minor non-structural
Building Consent required: no
Interim occupancy recommendations: full occupancy

Describe: strengthening recommended
Describe:
Describe:

Along Assessed %NBS before: 100% ##### %NBS from IEP below
Assessed %NBS after: 100%

Across Assessed %NBS before: 56% ##### %NBS from IEP below
Assessed %NBS after: 56%

IEP

Period of design of building (from above): Pre 1935
h_n from above: 3.1m

Seismic Zone, if designed between 1965 and 1992:
not required for this age of building
not required for this age of building

	along	across
Period (from above):	0.2	0.2
(%NBS)nom from Fig 3.3:		

Note:1 for buildings designed prior to 1976 as public buildings, to code at time, use 1.25: 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:**

2.5 Ductility Scaling Factor

	along	across
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**:	0.00	0.00

2.6 Structural Performance Scaling Factor:

Sp:	1.000	1.000
Structural Performance Scaling Factor **Factor E**:	1	1

2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom} \times A \times B \times C \times D \times E$

$\%NBS_b$: **#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

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max value =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!**

OPUS