# North Hagley RSA Bowling Club

PRK 1190 BLDG 008 EQ2

***Detailed Engineering Evaluation***

**Qualitative Assessment Report**

**North Hagley Park, Christchurch**

**Christchurch City Council**

# North Hagley RSA Bowling Club Detailed Engineering Evaluation Qualitative Assessment Report

## North Hagley Park, Christchurch
## Christchurch City Council

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North Hagley RSA Bowling Club
PRK 1190 BLDG 008 EQ2

Detailed Engineering Evaluation
Qualitative Report - SUMMARY
Final – Version 2

North Hagley Park, Christchurch

**Background**

This is a summary of the qualitative report for the building structure known as the North Hagley RSA Bowling Club and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on 17 March 2011, 27 June 2011, 14 December 2011, 21 January 2012 and 28 March 2012, available drawings and calculations.

**Key Damage Observed**

Key damage observed includes:-

- Step cracking in unreinforced masonry walls on three sides of the building
- Minor cracking to foundations
- Damage to non-structural elements was also observed.

**Critical Structural Weaknesses**

The following critical structural weaknesses have been identified:

a) The building is supported by unreinforced double skinned brick walls. Should these walls fail, the mechanism is likely to be brittle and may lead to partial collapse of the building.
b) At ground floor, the northern wall has a large number of openings and combined with the additional rigidity provided by the stairway in the northwest corner, this is likely to result in a torsional response to lateral loads applied in the east-west direction.
c) The first floor is relatively light weight construction but a significant seismic mass is applied at roof level by the heavy concrete tile roof. The supporting bolted trusses provide some load transfer but there is no ceiling diaphragm in place.
d) There are no ties between the lintel beams or concrete perimeter beam and the brick wall. While the perimeter beam appears to provide some restraint, tying the brick walls together in the structure, it does not appear to be connected to these walls in any way and will not provide any restraining action.

**Indicative Building Strength (from qualitative assessment)**

Based on the information available, and from undertaking a qualitative assessment, the building's original capacity has been assessed to be in the order of 11% NBS. The building is therefore classed as a potentially earthquake prone building.

**Recommendations**

It is recommended that:

a) It is recommended that the building not be occupied, given its structural weaknesses and the elevated level of seismic risk in Christchurch.

b) Due to the nature of the collapse mechanisms, a cordon should be placed around the full perimeter of the building urgently. This should be to a minimum of 1.5 times the maximum height of the building.

c) The unreinforced brick walls mean that a quantitative assessment would be of limited value. We recommend that a conceptual design for strengthening the building to 67%NBS is undertaken. This should be priced by a Quantity Surveyor alongside the costs for demolition and rebuild.

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# 1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council (CCC) to undertake a detailed seismic assessment following the M6.3 Christchurch earthquake on 22 February 2011 of the North Hagley RSA Bowling Club located in North Hagley Park.

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

The seismic assessment and reporting have been undertaken based on the qualitative and quantitative procedures detailed in the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) 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. Three relevant sections are:

**Section 29 – Information**

This section provides for the Chief Executive to obtain information on buildings from any person holding it. This section overrides legal professional privilege and means that this report and associated information may be demanded by CERA at any time.

**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 One 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 One: NZSEE Risk Classifications Extracted from Table 2.2 of the NZSEE 2006 AISPBE Guidelines**

Table One 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 One: %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[i] 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.

[i] 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 North Hagley Bowling Club was constructed in 1946 and continues to be used as a club facility by the Christchurch Petanque Club to support activities on the adjacent petanque lawns. It is a two storey building accessed from the northern side. The ground floor structure is double skinned unreinforced brick walls founded on a perimeter beam. The tongue and groove ground floor is supported by piles believed to be concrete. The tongue and groove first floor is supported off brick walls below. Light timber framed walls support timber trusses and a tiled roof and these are seated on a perimeter beam atop brick walls. It is accessed via an internal timber staircase.

The ground floor of the building is divided into a number of rooms and additional support is provided by a number of steel and timber beams and 125mm diameter circular steel columns. The first floor is a hall area.

The building is rectangular in shape with a 20m long street frontage adjacent to the the main carpark for the Botanic Gardens. It is 10m wide in the north-south direction. The roof slopes from a central ridge sloping toward the north and south.

## 4.2 Gravity Load Resisting System

At roof level, a heavy concrete tile roof is supported by bolted timber trusses. The lower two bolts on each truss are bolted through a steel plate however the connection between the base of the truss and the timber wall it is seated on requires additional, more intrusive investigation. The first floor is an open hall area with a kitchen and toilet areas on the western end. The walls are light timber framing with a tongue and groove timber floor.

At first floor level, two timber beams run in the east-west direction and provide additional gravity support to the long spans of T&G flooring. The beams are supported by 125mm diameter steel columns which are screwed to the underside of the beams. It is unlikely that these beams provide any lateral resistance.

The ground floor walls are unreinforced double brick. These walls follow the perimeter of the building and an additional wall runs north-south across the width of the building. These loadbearing walls are topped by a concrete perimeter beam however there is no evidence of any ties between the walls and beams. Reinforced concrete lintel beams across window and door frames at ground level are seated on the same walls – there is no evidence of these being tied into the walls in any way either. A steel lintel beam spans across the main entry at ground level. Further intrusive investigation is required to assess whether the floor is tied into the walls and beams as that has not been clear from investigations to date.

An open porch runs along the northern side of the building providing a covered walkway and entrance at ground level. This is supported by concrete columns and the floor of the porch is supported concrete ribs spanning between the columns and masonry wall.

No ties or reinforcing could be found between the two skins of brick.

Foundation details are unknown but the brick walls appear to be built directly off the perimeter footing. Sketches provided by the Petanque Club indicate 300mm x 300mm piles at approximately 900mm centres under the building, including the line supporting the interior brick wall.

## 4.3 Lateral Load Resisting System

At roof level, lateral load resistance is provided by bolted timber trusses. As previously noted the connection between the trusses and walls is not clear but it is assumed to be bolted to the timber framed walls. While there is a pinex ceiling in place, this follows the line of the trusses and is expected to provide very little lateral load resistance.

First floor light timber framed walls are lined with Pinex or similar lightweight board. Any lateral loads are expected to be taken by diagonal bracing within the timber framing rather than any sheeting fixed to the framing. The timber beams spanning between walls in the east-west direction are unlikely to transfer loads applied from the east or west. The result is no obvious load path to transfer the lateral loads at first floor level in the east-west direction.

In the north south direction, loads are taken in plane by the three unreinforced brick walls.

Tongue and groove floors at ground and first floor levels provide limited diaphragm action due to their flexibility but it is unclear how these are connected to the walls. More intrusive investigation is required to ascertain the floor-wall connections at both levels and the floor contribution to lateral load resistance.

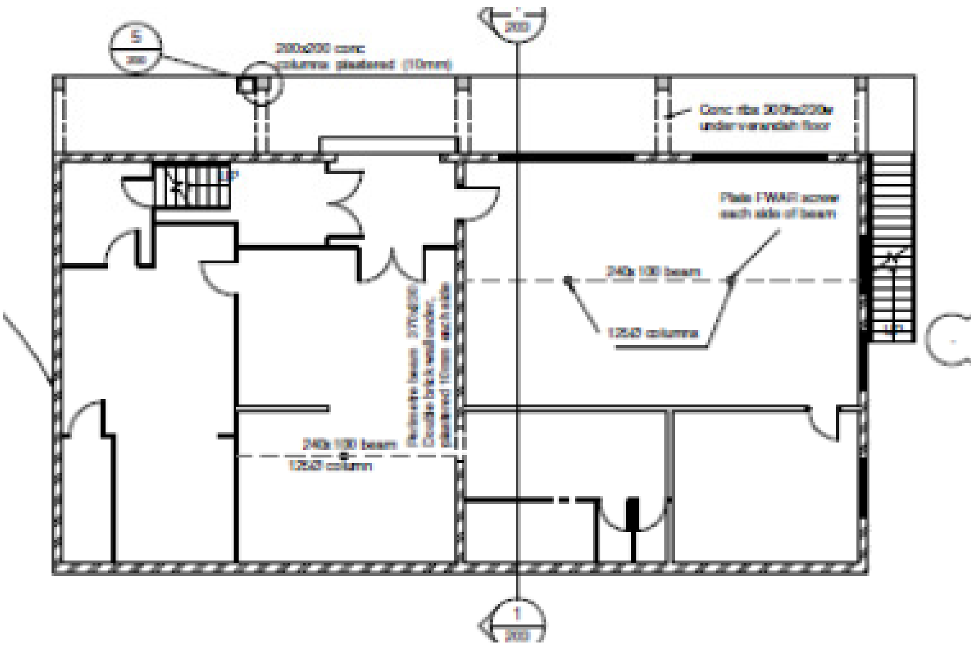


Figure Three: Ground Floor Plan

### 4.4 Survey

#### 4.4.1 Post 22 February 2011 Rapid Assessment

An engineer from Opus International Consultants undertook a Level 1 Assessment on this building on 17 and 18 March 2011 and recorded cracking in unreinforced masonry in the north-west and south-west corners of the building.

A structural (Level 2) assessment of the property was undertaken on 27 June 2011 by Opus International Consultants with damage to unreinforced masonry recorded.

#### 4.4.2 Further Inspections

Further inspections were undertaken by Opus International Consultants on 14 December 2011, 21 January 2012 and 28 March 2012. No further significant damage was recorded.

### 4.5 Original Documentation

No original documentation has been sourced but the cornerstone indicates the building was opened in 1946. Some drawings have been provided by a member of the Petanque Club (Architectural and Building Services Ltd) and these support our observations that some minor alterations have been made to the building which are unlikely to be structural. Any assessment of structural systems, critical structural weaknesses (CSW) and details which required particular attention has been based on visual observation, engineering judgement and the drawings provided.

## 5 General Observations

### 5.1 Surrounding Buildings

This building stands alone on the northern side of the Botanic Gardens carpark area. The only adjacent building is a concrete block toilet block approximately 20 metres from the eastern entrance to the bowling club grounds.

### 5.2 Foundations

Minimal ground settlement was observed on this site (<10mm) and no damage has been observed that could be attributed to ground settlement. There are cracks in the foundations on the eastern and western sides of the building however no intrusive investigation has been undertaken at this stage. The attached geotechnical report notes that pavement to the north of the building has settled but anecdotal evidence suggests this pre-dates September 2010.

## 6 Damage Assessment

There is little noticeable damage to the surrounding land and we do not believe there has been any earthquake related settlement of the building.

At ground level, there are a number of diagonal cracks in both skins of brick. On the north-west, south-west and south-east corners, there are cracks which extend the full height of the wall from foundation to first floor level. These are mirrored in the interior where the plaster has cracked and fallen away in some areas. There are a number of wide (up to 5mm) cracks in the foundations along the western side of the building. The interior brick wall to the building is not exhibiting any signs of damage through the 10mm plaster coating on both sides of the wall.

The interior timber stairs show some separation at ground and first floor from the adjacent walls. At first floor level, there is minor damage to the interior walls. The linings have separated in a number of locations. Cracks appear in the concrete balcony slab coincident with the edge of the supporting beams.

A small section spalled concrete on the northern edge of the first floor balcony is not earthquake related damage.

While some non-structural damage has been described, this report is not intended to include a complete list of damage to non-structural items.

# 7 Detailed Seismic Assessment

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

## 7.1 Critical Structural Weaknesses

The term Critical Structural Weakness (CSW) refers to a component of a building that could contribute to increased levels of damage or cause premature collapse of a building. The following potential CSWs were identified and considered during the analysis of the building:

- The building is supported by unreinforced double skinned brick walls. The recent Canterbury earthquake sequence has illustrated poor performance of this type of construction and the damage to the exterior walls is indicative of that. Should these walls fail, the mechanism is likely to be brittle and may lead to partial collapse of the building.

- At ground floor, the northern wall has a large number of openings and combined with the additional rigidity provided by the stairway in the northwest corner, this is likely to result in a torsional response to lateral loads applied in the east-west direction.

- The first floor is relatively light weight construction but a significant seismic mass is applied at roof level by the heavy concrete tile roof. The supporting bolted trusses provide some load transfer but there is no ceiling diaphragm in place.

- There are no ties between the lintel beams or concrete perimeter beam and the brick wall. While the perimeter beam appears to provide some restraint, tying the brick walls together in the structure, it does not appear to be connected to these walls in any way and will not provide any restraining action.

## 7.2 Seismic 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 Soft Soil (Clause 3.1.3 NZS1170.5:2004)
- Site Hazard Factor, z=0.3 (SESOC Christchurch Seismic Design Load Levels Interim Advice, Building Code B1/VM4 amendment, August 2011)
- Importance Level 2 structure with 50 year design life
- Return period factor, Ru = 1.0 (Table 3.5 NZS1170.5:2004)

Based on investigations of the building and the performance of similar structures, our initial estimate for the expected minimum structural ductility for the main lateral load resisting systems in both directions is $\mu_{max}$ = 1.00

## 7.3 Qualitative Assessment

The results of the qualitative assessment are summarised below in Table Two. The qualitative assessment was undertaken by completing the DEEP IEP spreadsheet, a copy of which is contained in Appendix 3 of this report.

**Table Two: Assessed %NBS based on the Initial Evaluation Process**

| Seismic Resisting System | Assumed Ductility factor, μ | Assumed fundamental period, t | PAR x Baseline (%NBS) | Overall Minimum %NBS | Overall Earthquake Risk Category |
|---|---|---|---|---|---|
| Longitudinal Direction (East-West) | 1.0 | 0.4 | 11% | 11% | 11% ≤ 33%<br>= potentially earthquake Prone |
| Transverse Direction (North-South) | 1.0 | 0.4 | 14% | | |

## 7.4 Discussion of Results

The building has suffered damage in the recent earthquake swarm and is currently closed for use. Based on preliminary analysis and inspection, the building appears to have a number of critical structure weaknesses and these limit its capacity to resist lateral loads.

The seismic capacity of the building is likely to be governed by the capacity of the URM walls at ground level. In its undamaged state, we assess the building to have a capacity of

11%NBS and the building is therefore defined as being potentially earthquake prone in accordance with the Building Act 2004. Given the existing damage to the ground floor walls and potential for a brittle failure mechanism causing partial collapse of the building, the area should remain cordoned. It should be noted that the building is immediately adjacent to the carpark area for the Botanic Gardens which has high usage.

A more detailed quantitative assessment to confirm the seismic capacity, and preparation of a strengthening scheme is recommended.

# 8 Summary of Geotechnical Appraisal

## 8.1 General

The site is located in the north eastern quarter of Hagley Park, adjacent to the Hagley Golf Club and opposite Armagh Street. The Avon River runs parallel 60m south of the building. A large sealed carpark separates the site from the river. Victoria Lake, which was previously used by remote control water craft enthusiasts, is approximately 40m north of the building. The building is located to the south of the site behind a large area of loose aggregate used for Petanque.

The ground profile is relatively flat and level with the adjacent carpark and grassed areas. Victoria Lake has an approximate depth of 1m.

Drawings provided by the Petanque Club indicate that the timber floor is supported by concrete piles (assumed) and an unreinforced perimeter strip footing with unknown dimensions.

## 8.2 Liquefaction Potential

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 100 to 300mm of subsidence.

Based on the maps prepared by Tonkin and Taylor Ltd for Earthquake Commission (EQC) there is no surface evidence of liquefaction at this site. Significant surface rupture of liquefaction is recorded throughout the golf course area north of Victoria Lake and sand boils were located 150m north of the building.

## 8.4 Summary

It is our assessment that the magnitude of seismically induced settlement which has occurred on site is minor (<10mm) and is not considered to have caused damage to the building. Buildings are typically designed to allow for up to 50mm of land settlement in a serviceability limit state (SLS) event, or up to 100mm in an ultimate limit state event (ULS).

Liquefaction appears to have been minor on the site. Settlement in the concrete paving and minor uplift of the manhole are both consistent with liquefaction induced local damage. The step cracking on the west wall of the building may indicate some differential settlement of the building.

Damage to the east foundation is likely to be superficial. 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. This site specific value is lower that the values given through the ECan study

### 8.5 Further Work

Based on the building performance in recent earthquakes, the existing foundations should be acceptable in terms of future ULS and SLS loadings. However, the Christchurch City Council may have to accept the risk for potential differential settlement of up to 50mm.

If Christchurch City Council wishes to further estimate the risk of damage from differential settlement in future seismic events, consideration could be given to Undertaking ground investigations and a more detailed liquefaction assessment to more accurately estimate the potential differential settlement from liquefaction. This would require collection and analysis of CPT data from the site as the nearest existing CPT is 700m east of the site which does not provide any useful information.

## 9 Conclusions

a) The seismic performance of the building is governed by the capacity of the unreinforced double skin brick walls. The result is a seismic capacity of 11%NBS. The building is therefore considered to be potentially earthquake prone in accordance with the Building Act 2004.

b) Torsional action of the building will have an effect on the loads that are transferred to the walls of the building and is likely to exacerbate the poor performance of the masonry.

c) The building contains a number of critical structural weaknesses which include unreinforced masonry, torsionality, a large seismic mass at roof level and lack of load path to transfer lateral loads at first floor level in the east-west direction.

d) Liquefaction hazard for the site is considered moderate with a maximum expected differential settlement under SLS conditions being 50mm.

e) The building contains a number of brittle failure mechanisms which could lead to partial collapse of the building. We recommend that the building remain unoccupied and the existing cordon around the building remain in place and consideration be given to widening the cordon area to 1.5 times the building height in all directions.

f) A quantitative assessment should be undertaken to assess the building capacity in more detail and a strengthening scheme produced.

# 10 Recommendations

a) A quantitative analysis of this building would be of limited value in isolation due to the nature of construction. We recommend that a conceptual strengthening scheme be developed to increase the seismic capacity of the building to at least 67% NBS. This will need to consider compliance with accessibility and fire requirements.

b) A quantity surveyor should be engaged to determine the costs for either strengthening the building or demolishing and rebuilding.

c) Due to the nature of the collapse mechanisms, a cordon should be placed around the full perimeter of the building urgently. This should be to a minimum of 1.5 times the maximum height of the building.

d) It is recommended that the building not be occupied, given its structural weaknesses and the elevated level of seismic risk in Christchurch.

# 11 Limitations

a) This report is based on an inspection of the structure of the buildings and focuses on the structural damage resulting from the 22 February 2011 Canterbury Earthquake and aftershocks only.

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.

# 12 References

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

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

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

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

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

# Appendix 1: Photographs

**North Elevation**

**East Elevation**

**Western Side of Building**

**South Elevation**

**Step cracking on north west corner**

**Foundation Damage**

**Spalled concrete on First Floor balcony**

# Appendix 2: Geotechnical Appraisal

21 May 2012

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

Dear Michael

**Geotechnical Desktop Study – Hagley Park North RSA Bowling Club**

## *1. Introduction*

Christchurch City Council (CCC) has commissioned Opus International Consultants (Opus) to undertake a geotechnical desktop study and site walkover of the Hagley Park North RSA Bowling Club, North Hagley Park, 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 27 January 2012.

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 has been undertaken without the benefit of any site specific site investigations and is therefore preliminary in nature.

## *2. Desktop Study*

### *2.1 Site Description*

The Hagley Park North RSA Bowling Club is located in the north eastern quarter of Hagley Park, adjacent to the Hagley Golf Club and opposite Armagh Street. The Avon River runs parallel 60m south of the building. A large sealed carpark separates the site from the river. Victoria Lake, which was previously used by remote control water craft enthusiasts, is approximately 40m north of the building. The building is located to the south of the site behind a large area of loose aggregate used for Petanque.

The building is two storey with the first storey predominantly masonry with the second storey timber. Refer to the qualitative structural assessment report for a more detailed description.

The ground profile is relatively flat and level with the adjacent carpark and grassed areas. Victoria Lake has an approximate depth of 1m.

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### *2.2 Structural Drawings*

Extracts from the Structural drawings illustrating a cross-section of the foundation have been available for review. The drawings indicate that the timber floor is supported by timber piles (assumed) and an unreinforced perimeter strip footing with unknown dimensions.

### *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 five wells located within approximately 300m of the property (refer to site location plan in Appendix B). The locations of Boreholes and CPT's by Earthquake Commission have been reviewed. The nearest CPT is located 700m east of the site, therefore has been excluded from this study. Material logs available from the three closest 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) |
|---|---|---|
| Topsoil/silty brown clay | 1.5-2.4m | Surface |
| Sandy GRAVEL | 8.5-12.8m | 1.5-2.4m |
| Blue CLAY | 4.3-5.5m | 15.2m |
| GRAVEL (Riccarton Formation) | - | 19.5-23.5m |

A groundwater depth of approximately 1m to 2m below ground level has been extracted 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 100 to 300mm of subsidence.

Tonkin and Taylor Ltd (T&T Ltd) have been engaged as the Earthquake Commision's (EQC) geotechnical consultants and have prepared maps showing areas of liquefaction interpreted from high resolution aerial photos for the 4$^{th}$ September earthquake, and the aftershocks of February 2011 and June 2011. There is no surface evidence of liquefaction at the Hagley Park North RSA Bowling Club building. However significant surface rupture of liquefaction is recorded throughout the golf course area north of Victoria Lake. The sand boils were located from 150m north of the building.

### *3. Site Walkover Inspection*

A walkover inspection of the exterior, interior, and adjacent areas was carried out by an Opus Geotechnical Engineer on 27 January 2012. The following observations were made (refer to the Walkover Inspection Plan and Site Photos attached to this report):

- Observations of the building indicate that there has been no differential settlement or rotation.

- An area of approximately $1m^2$ located 20m north of the building was affected by surface rupture liquefaction. Refer to Photo 8 and site walkover plan for location.

- The manhole at the north of the building appears to have risen by 10mm to 20mm causing the corner of concrete path to crack. Refer to Photo 3.

- Concrete paving on the northern side of the building has 10mm horizontal and vertical displacements. Refer to Photo's 3 and 4. Appears to have settled by 10mm to 50mm as indicated on site walkover drawing in Appendix B. Refer to Photo 3.

- Minor crack in concrete foundations steps up the masonry wall on the west elevation of the building. Refer to Photo 5.

- A 10mm wide crack across the footpath at the bottom of the stairs on the east elevation of the building. Refer to Photo 6.

- Lateral crack in foundation on east elevation. Refer to Photo 7.

- The pavement north east of the building looks as though it has settled, but anecdotal evidence from a Club Member suggests the settlement pre dates the Canterbury Earthquake sequence.

- Victoria Lake has been emptied and a new clay liner is currently being installed. It would suggest that the lakes previous liner had ruptured as a result of the Earthquakes. Refer to Photo 10.

### *4. Discussion*

Very minor land damage has occurred to the Hagley Park North RSA Bowling Club due to the Canterbury Earthquake Sequence following the 4 September 2010 earthquake.

There appears to have been minor movement (0 to 10mm) of the ground illustrated by the lateral cracks that have formed on the east footpath and west foundation.

Liquefaction appears to have been relatively minor at the site and the close vicinity. Settlement (varying from 10mm to 50mm) in the concrete paving to the north and minor uplift of the manhole is consistent with liquefaction induced local damage.

The crack stepping up the western exterior wall may also indicate differential settlement of the building has occurred.

Longitudinal cracking on the east foundation looks to be superficial, and is likely to be decorative plaster breaking off.

ECan well logs indicate the building is probably founded on a thin layer of silt and sand overlying an 8m 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.

Buildings are typically designed to allow for up to 50mm of land settlement in a serviceability limit state (SLS) event, or up to 100mm in an ultimate limit state event (ULS).

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 a 20% probability of another Magnitude 6 or greater earthquake occurring in the next 12 months in the Canterbury region. It is expected that the probability of occurrence is likely to decrease with time, following periods of reduced seismic activity.

Based on current 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 CCC wish to quantify the risk of damage from differential settlement in future seismic events, consideration could be given to undertaking ground investigations to more accurately estimate the potential differential settlement from liquefaction. Allowance for predrilling through shallow gravels will need to be included in the scope of a site investigation.

## *5. Recommendations*

- 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 CCC as our client with respect to the brief. The reliance by other parties on the information or opinions contained in the report shall, without our prior review and agreement in writing, be at such parties' sole risk.

## *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 16 December 2011.

Appendices:
Appendix A: Site Photos
Appendix B: Site Walkover Plan
Appendix C: Environment Canterbury Borehole Logs

# APPENDIX A:
## Site Photos

**Photo 1: East elevation of the building.**

**Photo 2: North elevation of the building.**

**Photo 3: Localised settlement of footpath (10mm-50mm) and manhole uplift (10mm to 20mm).**

**Photo 4: Upto 10mm of vertical and horizontal displacements of concrete paving.**

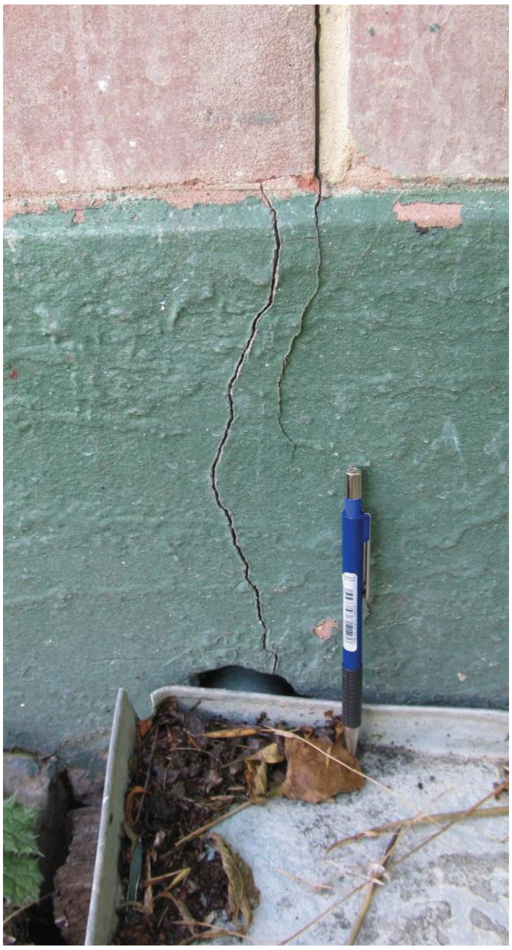

**Photo 5: Minor cracking in the west elevation foundation. Start of “step cracking” up wall.**

**Photo 6: 10mm crack across footpath at east elevation.**

**Photo 7: Possible superficial damage to east foundation.**

**Photo 8: Approximately 1m$^2$ of liquefaction.**

**Photo 9: No evidence of cracking and heaving due to the earthquakes.**

**Photo 10: New liner in Victoria Lake.**

# APPENDIX B:
## Site Walkover Plan


ECan Borehole Location

| BH ref | ECan ref |
|---|---|
| 1 | M35/7410 |
| 2 | M35/10619 |
| 3 | M35/1936 |
| 4 | M35/7631 |
| 5 | M35/1937 |

0 100m

Approximate Scale 1:2500 at A3

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**Project:** Hagley Park RSA Bowling Club
Geotechnical Desk Study

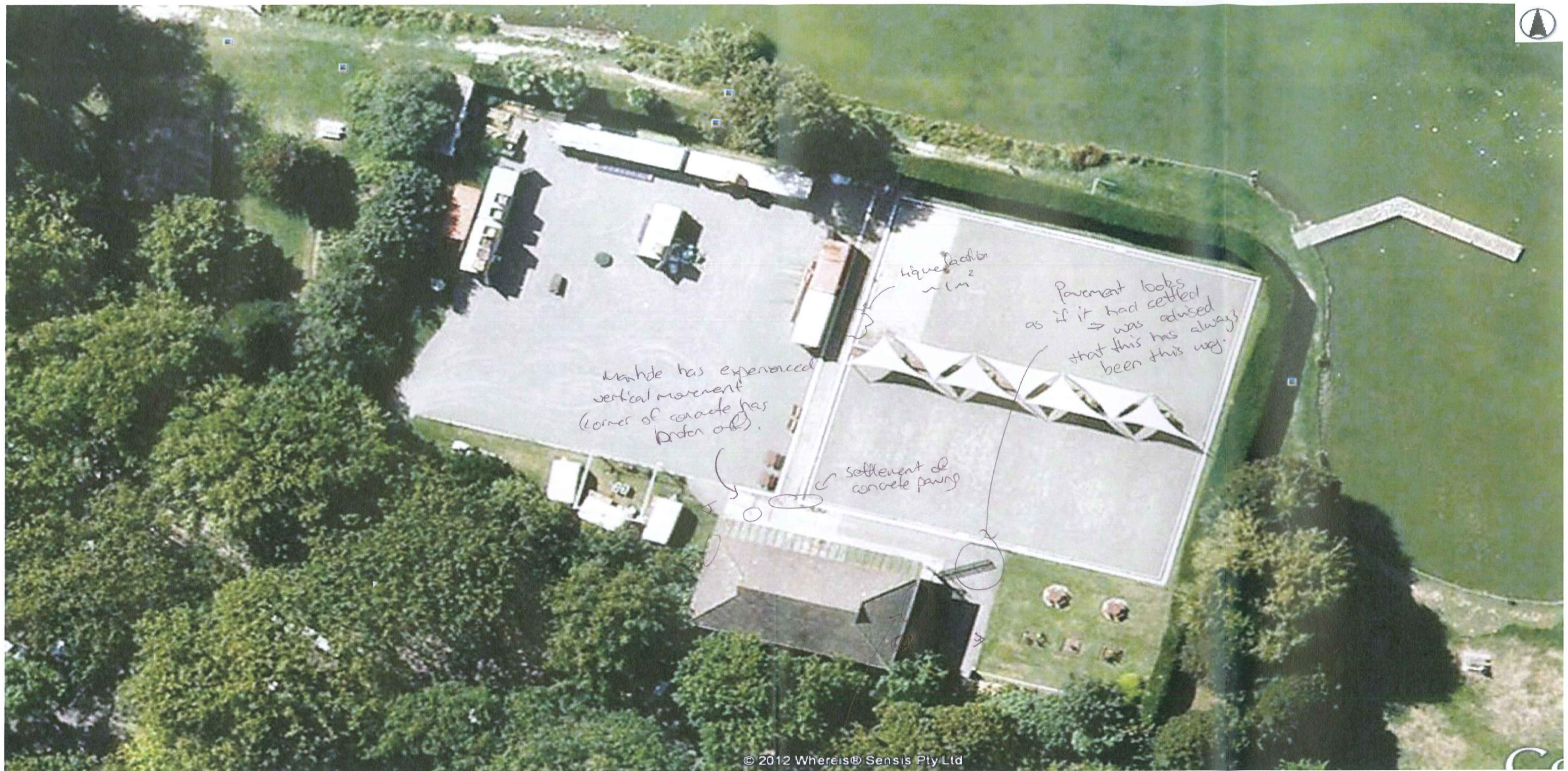
**Project No.:** 6-QUCCC.56 0055SC

**Client:** Christchurch City Council

# Site Location Plan

**Drawn:** Opus Geotechnical Engineer

**Date:** 25-Jan-12




1mm vertical crack in foundation
that proceeds up the wall at 45°

transverse crack in footpath
– possible uplift.

~0.4mm crack along foundation
⇒ possibly a piece of plaster breaking off

| | | |
|---|---|---|
| OPUS<br>Opus International Consultants Ltd<br>Christchurch Office<br>20 Moorhouse Ave<br>PO Box 1482<br>Christchurch, New Zealand<br>Tel: +64 3 363 5400 Fax: +64 3 365 7857 | **Project:** Hagley Park RSA Bowling Club<br>Geotechnical Desk Study<br>**Project No.:** 6-QUCCC.56 0055SC<br>**Client:** Christchurch City Council | **Site Walkover Plan**<br>**Drawn:** Opus Geotechnical Engineer<br>**Date:** 25-Jan-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

Environment Canterbury Regional Council

| 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 | |
| - 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 | |
| - 35.3m | hard sticky yellow clay | ri |

Scale(m): -5, -10, -15, -20, -25, -30, -35

# 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

Environment Canterbury Regional Council

| Depth(m) | Full Drillers Description | Formation Code |
|---|---|---|
| - 38.0m | hard sticky yellow clay | ri |
| - 40.3m | small brown gravel - progressively sandier | br |
| - 50.9m | brown sand | br |
| - 53.6m | hard sticky yellow/orange clay | br |
| - 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 |
| - 65.8m | brown sand (traces gravel) | li-2 |
| - 66.5m | hard silty yellow clay | li-2 |
| - 68.3m | silty grey pug (traces peat) | li-2 |
| - 70.6m | 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

| 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 | | |

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

# 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

| Scale(m) | Water Level | Depth(m) | Full Drillers Description | Formation Code |
|---|---|---|---|---|
| | Artesian | -2.40m | Surface soil & Yellow sand | sp? |
| -10 | | - 14.6m | Hard Blue shingle | sp? |
| | | - 19.8m | Soft Blue sands | ch |
| -20 | | - 22.2m | Clay and peat | ch |
| -30 | | - 37.5m | Brown shingle 1st strata water .9m below surface | ri |
| -40 | | - 41.8m | Clay & peat | br |
| | | - 44.8m | Hard Brown sand & gravel | br |
| -50 | | - 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): -60, -70, -80, -90, -100

Water Level: Artesian

| Depth(m) | Full Drillers Description | Formation Code |
|---|---|---|
| - 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 |
| - 60.4m | Soft Yellow sand | li-2 |
| - 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 |
| - 83.2m | Hard Yellow sand & gravel. Water rise +2.7m & flow 30gpm 0.61m high | li-3 |
| - 89.6m | Soft Brown sand | he |
| - 96.9m | Soft Yellow sand | he |
| - 98.1m | Soft Yellow sand with clay | he |
| - 100.9m | Brown gravel & sand. Water rise +4.3m flow 45gpm 1.1m high | bu |

# Borelog for well M35/1937 

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: Artesian

| Depth(m) | Full Drillers Description | Formation Code |
|---|---|---|
| -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

Environment Canterbury Regional Council

| 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 |

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

# 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: Artesian

| Depth(m) | Full Drillers Description | Formation Code |
|---|---|---|
| -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 |

# 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

| Scale(m) | 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 |
| -5 | | -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 |

# Appendix 3: DEEP Data Sheet

**Detailed Engineering Evaluation Summary Data** V1.11

**Location**

Building Name: North Hagley RSA Bowling Club
Unit No: Street
Building Address: North Hagley Park
Legal Description:
Degrees Min Sec
GPS south:
GPS east:
Building Unique Identifier (CCC): PRK 1190 BLDG 008 EQ2

Reviewer: Jan Stanway
CPEng No: 222291
Company: Opus International Consultants
Company project number: 6QUCCC56
Company phone number: 03 3635400
Date of submission: 6-Sep-12
Inspection Date:
Revision: Final V2
Is there a full report with this summary? yes

**Site**

Site slope: flat
Soil type: silty sand
Site Class (to NZS1170.5): D
Proximity to waterway (m, if <100m): 50
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): 20.00

**Building**

No. of storeys above ground: 2 — single storey = 1
Ground floor split? no
Storeys below ground 0
Foundation type: strip footings
Building height (m):
Floor footprint area (approx): 200
Age of Building (years): 66

Ground floor elevation (Absolute) (m):
Ground floor elevation above ground (m):
if Foundation type is other, describe:
height from ground to level of uppermost seismic mass (for IEP only) (m):
Date of design: 1935-1965

Strengthening present? no
Use (ground floor): other (specify)
Use (upper floors): other (specify)
Use notes (if required): Sports Clubrooms
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 — truss depth, purlin type and cladding
Floors: other (note) — describe sytem T&G on joists & bearers
Beams: timber — type
Columns: brick masonry — typical dimensions (mm x mm)
Walls: load bearing brick — #N/A

Lateral load resisting structure

Lateral system along: unreinforced masonry bearing wall - brick — **Note: Define along and across in detailed report!** — note wall thickness and cavity
Ductility assumed, μ: 1.00
Period along: 0.40 — 0.40 from parameters in sheet — estimate or calculation?
Total deflection (ULS) (mm): — estimate or calculation?
maximum interstorey deflection (ULS) (mm): — estimate or calculation?

Lateral system across: unreinforced masonry bearing wall - brick — note wall thickness and cavity
Ductility assumed, μ: 1.00
Period across: 0.40 — 0.00 — estimate or calculation?
Total deflection (ULS) (mm): — estimate or calculation?
maximum interstorey deflection (ULS) (mm): — estimate or calculation?

Separations:

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

Non-structural elements

Stairs: timber — describe supports
Wall cladding:
Roof Cladding: Heavy tiles — describe
Glazing: timber frames
Ceilings: fibrous plaster, fixed
Services(list):

**Available documentation**

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

**Damage**

Site: (refer DEE Table 4-2)
Site performance: Good — Describe damage:
Settlement: none observed — notes (if applicable):
Differential settlement: none observed — notes (if applicable):
Liquefaction: none apparent — notes (if applicable):
Lateral Spread: none apparent — notes (if applicable):
Differential lateral spread: none apparent — notes (if applicable):
Ground cracks: none apparent — notes (if applicable):
Damage to area: none apparent — notes (if applicable):

Building:
Current Placard Status: yellow

Along — Damage ratio: — Describe how damage ratio arrived at:
Describe (summary): Cracking in URM

Across — Damage ratio:
Describe (summary): Carcking in URM

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

Diaphragms — Damage?: no — Describe:
CSWs: — Damage?: — Describe:
Pounding: — Damage?: no — Describe:
Non-structural: — Damage?: no — Describe:

**Recommendations**

Level of repair/strengthening required: — Describe:
Building Consent required: — Describe:
Interim occupancy recommendations: — Describe:

Along — Assessed %NBS before: — 14% %NBS from IEP below
Assessed %NBS after:

Across — Assessed %NBS before: — 11% %NBS from IEP below
Assessed %NBS after:

**IEP**

Period of design of building (from above): 1935-1965 — $h_n$ from above: m

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): | 0.4 | 0.4 |
| (%NBS)nom from Fig 3.3: | 3.0% | 3.0% |

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 $(\%NBS)_{nom}$:** | **3%** | **3%** |

**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: 0.30
$Z_{1992}$, from NZS4203:1992
Hazard scaling factor, **Factor B:** 3.333333333

**2.4 Return Period Scaling Factor**

Building Importance level (from above): 2
Return Period Scaling factor from Table 3.1, **Factor C**: 1.00

**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: | 1.00 | 1.00 |
| 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, $(NBS\%)_b = (\%NBS)_{nom} \times A \times B \times C \times D \times E$** — $\%NBS_b$: **10%** **10%**

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

**3.1. Plan Irregularity, factor A:** significant 0.7
**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 | 2.0 | 1.5 |

Rationale for choice of F factor, if not 1: Perimeter beam at top of ground floor walls, light timber framing on first floor (incr along due to reduced torsionality in fa

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)** — **1.40** **1.05**

**4.3 PAR x (%NBS)b:** — PAR x Baselline %NBS: **14%** **11%**

**4.4 Percentage New Building Standard (%NBS), (before)** — **11%**

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