Heathcote Domain
Former Tennis Club Shed
Detailed Engineering Evaluation
Quantitative Report

Christchurch City Council

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Heathcote Domain Former Tennis Club Shed

Detailed Engineering Evaluation Quantitative Report

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Heathcote Domain Former Tennis Club Shed
PRK 1880 BLDG 007 EQ2

Detailed Engineering Evaluation
Quantitative Report - SUMMARY
Final

Heathcote, Christchurch

Background
This is a summary of the quantitative report for the Tennis Shed in Heathcote Domain. The summary is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group, visual inspections and measurements taken on 5 June 2012, and calculations.

Key Damage Observed
Key damage observed includes:-
- Cracks in the walls and ground slab/foundation
- Walls laterally displaced

Indicative Building Strength
Based on the information available, and from undertaking a quantitative assessment, the building's original capacity has been assessed to be 75%NBS across the building, and 7%NBS along the building. In its damaged state the building estimated capacity is 3%NBS

Recommendations
It is recommended that:

a) Options be developed for strengthening to at least 67%NBS, or demolishing the building.
b) If the building is to be retained, it is recommended that foundation conditions be verified by doing a floor level survey to ascertain differential settlement.

Contents

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council (CCC) to undertake a detailed seismic assessment of the former tennis club shed, located at the northern end of Heathcote Domain. This report has been commissioned following the M6.3 Christchurch earthquake on 22 February 2011.

The purpose of the assessment is to determine if the structure 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 procedure detailed in the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011 and its supplement Assessment and Improvement of Unreinforced Masonry Building for Earthquake Resistance.

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. It uses 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 owner's 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 requires 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 has 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) 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:

- in the ordinary course of events (excluding the occurrence of an earthquake), the building is likely to cause injury or death or damage to other property; or
- in the event of fire, injury or death to any persons in the building or on other property is likely because of fire hazard or the occupancy of the building; or
- there is a risk that the building could collapse or otherwise cause injury or death as a result of earthquake shaking that is less than a 'moderate earthquake' (refer to Section 122 below); or
- there is a risk that other property could collapse or otherwise cause injury or death; or
- a territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

Section 122 – Earthquake Prone Buildings (EPB)

This section defines a building as earthquake prone if its ultimate capacity would be exceeded in a 'moderate earthquake' and it would be likely to collapse causing injury or death, or damage to other property.

A moderate earthquake is defined by the building regulations as one that would generate loads 33% of those used to design an equivalent new building.

Section 124 – Powers of Territorial Authorities

This section gives the territorial authority the power to require strengthening work within specified timeframes or to close and prevent occupancy to any building defined as dangerous or earthquake prone.

Section 131 – Earthquake Prone Building Policy

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

2.3 Christchurch City Council Policy

Christchurch City Council adopted their Earthquake Prone, Dangerous and Insanitary Building Policy in 2006. This policy was amended immediately following the Darfield Earthquake on 4 September 2010.

The 2010 amendment includes the following:

1. a process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
2. a strengthening target level of 67% of a new building for buildings that are Earthquake Prone;
3. a timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and
4. repair works for buildings damaged by earthquakes will be required to comply with the above.

The council has stated their willingness to consider retrofit proposals on a case by case basis, considering the economic impact of such a retrofit.

If strengthening works are undertaken, a building consent will be required. A requirement of the consent will require upgrade of the building to comply 'as near as is reasonably practicable' with:

- The accessibility requirements of the Building Code.
- The fire requirements of the Building Code. This is likely to require a fire report to be submitted with the building consent application.

2.4 Building Code

The Building Code outlines performance standards for buildings and the Building Act requires that all new buildings comply with this code. Compliance documents published by The Department of Building and Housing can be used to demonstrate compliance with the Building Code.

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

- 36% increase in the basic seismic design load for Christchurch (Z factor increased from 0.22 to 0.3);
- Increased serviceability requirements.

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

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

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

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

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

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

3 Earthquake Resistance Standards

For this assessment, the building's earthquake resistance is compared with the current New Zealand Building Code requirements for a new building constructed on the site. This is expressed as a percentage of new building standard (% NBS). The loadings are in accordance with the current earthquake loading standard NZS1170.5 [1].

A generally accepted classification of earthquake risk for existing buildings in terms of % NBS that has been proposed by the NZSEE 2006 [2] is presented in Figure 1 below.

Description	Grade	Risk	%NBS	Existing Building Structural Performance	Improvement of Structural Performance	
					Legal Requirement	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	C	Moderate	34 to 66	Acceptable legally. Improvement recommended		Not recommended. Acceptable only in exceptional circumstances
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement required under Act)	Unacceptable	Unacceptable

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

Table 1 below compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% risk of exceedance in 50 years (i.e. 0.2% in the next year). It is noted that the current seismic risk in Christchurch results in a 6% risk of exceedance in the next year.

Table 1: %NBS compared to relative risk of failure

Percentage of New Building Standard (%NBS)	Relative Risk (Approximate)
>100	<1 time
80-100	1-2 times
67-80	2-5 times
33-67	5-10 times
20-33	10-25 times
<20	>25 times

3.1 Minimum and Recommended Standards

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

3.1.1 Occupancy

– The Canterbury Earthquake Order[1] in Council 16 September 2010, modified the meaning of 'dangerous building' to include buildings that were identified as being Earthquake Prone Buildings (EPB). 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 of it) 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.

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

4 Building Description

4.1 General

The building is situated on a flat section and is approximately 8.8m long in the east-west direction and 5.5m wide in the north-south direction. The roof is approximately 3m above the ground. There is a concrete masonry retaining wall approximately 2m in front of and below the building on the north side.

The tennis shed is a single storey concrete masonry walled building with a monoslope light timber framed roof. The north elevation is dominated by windows and contains the only door, leaving only a small masonry pier extending up to the roof at each end. The east and west walls are solid masonry and there are two windows in the rear (south) wall. There are no internal walls.

The building has a concrete slab floor. We have assumed that the foundations consist of the concrete ground bearing slab with edge thickenings beneath the masonry walls.

The full perimeter of the building is currently cordoned off with a temporary fence.

4.2 Gravity Load Resisting System

The roof is a timber framed with corrugated iron sheeting. The roof rafters span between the north and south walls.

The external walls are partially filled concrete masonry units with minimal reinforcing. No internal walls are present.

4.3 Seismic Load Resisting System

The main lateral support for the building in both principal directions is provided by the external masonry walls. The lack of walls extending up to the roof on the north elevation means that the building is irregular in plan under loads in the east-west direction. This means the building would be expected to show a torsional behaviour and twist under seismic loads in this direction, concentrating damage at either end of the north wall and applying additional loads to the east and west walls.

5 Survey

The building currently does not have a placard (one not issued as part of this inspection).

No copies of the design calculations or structural drawings have been obtained for this structure but we have measured the structure accurately and made calculations based on these figures.

The non-intrusive inspections have been used to confirm the structural systems, investigate potential critical structural weaknesses (CSW) wherever possible and to identify details which required particular attention.

6 Damage Assessment

The building shows a lot of damage to the masonry walls that appears to have been the result of the recent earthquake events. The following damage has been noted:

6.1 Masonry Cracks

The building has a large number of cracks that are the result of seismic actions. Diagonal shear cracks are evident in all of the walls. Some of the cracks have caused pieces of the masonry block face shells to break off.

6.2 Wall Displacement

Horizontal cracks have also formed allowing the upper part of the east and west walls (and roof) to shift laterally compared to the lower portion of the wall. This is particularly evident on the north elevation. The east side wall has also broken away from the rest of the building and is leaning out of plane. The west wall appears to have also moved outward relative to the building foundation.

6.3 Ground Slab/Foundation Cracking

Cracks have formed in the ground slab of the building, potentially caused by ground settlements related to movement of the adjacent retaining wall.

7 General Observations

Overall the building has performed poorly, as can be expected of partially filled masonry buildings with minimal reinforcing. The building has suffered extensive cracking due to seismic actions and poor building layout.

Due to the non-intrusive nature of the original survey, many connection details could not be ascertained.

8 Detailed Seismic Assessment

8.1 Seismic Coefficient Parameters

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

- Site soil class C, clause 3.1.3 NZS 1170.5:2004
- Site hazard factor, Z=0.3, B1/VM1 clause 2.2.14B
- Return period factor R_u = 1.0 from Table 3.5, NZS 1170.5:2004, for an Importance Level 2 structure with a 50 year design life
- Ductility factor μ_{max} = 1.0 for a partially filled masonry block structure with minimal reinforcement

8.2 Detailed Seismic Assessment Results

A summary of the structural performance of the building is shown in the following table. Note that the values given represent the worst performing elements in the building, as these effectively define the building's capacity. Other elements within the building may have significantly greater capacity when compared with the governing element.

Table 2: Summary of Seismic Performance

Structural Element/System	Failure mode and description of limiting criteria	% NBS based on calculated capacity
Walls in the north south direction (i.e. across the building)	In-plane bracing capacity of the walls across the building	75%
Walls in the east west direction (i.e. along the building)	In-plane bracing capacity of the walls along the building	8%

8.3 Discussion of Results

The structure has a calculated capacity of approximately 6%NBS, with the capacity being limited by the bracing capacity of the south walls. This is below the threshold limit for structures classified as 'Earthquake Prone' which is one third (34%) of the seismic performance specified in the current loading standard for new structures. This building is therefore classed as earthquake prone and has a high risk profile due to the very low level of reinforcement in the walls.

The above calculated capacity is based on the building in an undamaged condition; in its damaged state the building's estimated capacity is around 3%NBS.

The two small piers at either end of the north side of the building would have failed quickly in the earthquake, leading to a torsional response from the building. This in turn would have increased the out-of-plane effects on the walls on the east and western sides leading to the lateral displacements seen in the building.

Due to the building's close proximity to the epicentre of the 22 February 2012 earthquake it experienced shaking greater than that of a design level earthquake which contributed to the large amount of damage. Another moderate seismic event could cause this building to collapse completely. The roof framing and friction between the masonry blocks are what is preventing partial collapse of the wall leaning out of plane.

8.4 Limitations and Assumptions in Results

Our analysis and assessment is based on an assessment of the building in its undamaged state. Therefore the current capacity of the building will be lower than that stated.

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

- 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.

9 Geotechnical Assessment

This section is a summary of the Geotechnical Desktop Study in Appendix C of this report.

9.1 Site Description

The Heathcote Domain Former Tennis Club is located off Port Hills Road on the north side of the domain. The shed is surrounded by the western portal of the Lyttelton rail tunnel to the east, areas of on-going development (formerly demolished buildings damaged by the earthquake to the north and east and the park playground to the west

The ground profile gently slopes down from the domain which is south of the shed and then falls steeply to the east towards the railway and north to the on-going area of development.

9.2 Regional Geology

The 1:250,000 Geological Map of Christchurch[2] indicates the site is underlain by Banks Peninsula Loess which generally comprises wind- blown silt deposits up to 3m thick. These are underlain by basaltic and trachytic lava flows of the Lyttelton Volcanic Group.

9.3 Expected Ground Conditions

From the eroded steep slope east of the court area, the loess is estimated to be about 3m to 4m overlying basalt.

According to Environment Canterbury Regional Council records, the closest groundwater monitoring well (M36/1159) is in Scruttons Road, Ferrymead 2km north east of the site and groundwater level is anticipated to be approximately 2.0m below ground level.

9.4 Liquefaction Hazard

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 and there is no surface evidence of liquefaction around the Heathcote Domain.

The Christchurch Earthquake Recovery Authority (CERA) last updated 11 December,2011 has classified the area surrounding the Heathcote Domain former tennis club shed under

[2] GNS 1:100,000 Geological Map 3 - Geology of Banks Peninsula.

Green Zone, indicating the repair and rebuilding process can begin with appropriate advice from local authority.

9.5 Site Observations

A site walkover inspection was carried out by an Opus Geotechnical Engineer on 26 June 2012.

The following observations were made (refer to the Geotechnical Desktop Study attached to this report for photographs):

- Hairline and web-like cracks observed in some places around the tennis court.
- The slopes east of the tennis club are very steep and fenced with waratahs and aluminium wire.
- The concrete masonry retaining wall sustained approximately 2 to 3mm wide cracks on the face running from bottom to top of the wall.
- Timber storage shed founded on severely cracked concrete slab.
- Light post leaning approximately 5 degrees north with cracks radiating from the bottom of post. Vertical settlement of more than 10mm separated the concrete slab from the perimeter wall (north elevation of the shed).
- Lateral displacement of approximately 20mm between the concrete masonry retaining wall and the concrete slab.

9.6 Conclusions and Discussion

The ground conditions at this site remain unknown. Based on local knowledge, we anticipate the ground conditions to consist of up to 4m of loess deposits overlying basalt. No evidence of liquefaction at the site has been observed.

The horizontal peak ground acceleration recorded at Heathcote Valley Primary School, 130m east of the building, was 1.5g. It was these very high accelerations which caused the lateral movement of the ground evidently observed by the leaning of the retaining wall and the light post towards north of the site.

Dependent on the location of the epicentre, further ground shaking damage could be experienced at this site. It is expected that the probability of occurrence is likely to decrease with time, following periods of reduced seismic activity.

10 Remedial Options

Any remedial options for increasing the seismic capacity to at least 67% NBS would need to address the large cracks in the walls throughout the building.

11 Conclusions

(a) The building has a seismic capacity of 3% NBS in its current condition, and is therefore classified as earthquake prone.

(b) In its current state the building is a collapse risk.

(c) The seismic capacity of the building is governed by the in-plane shear capacity of the southern masonry walls.

(d) The foundation conditions of the structure are currently unknown.

12 Recommendations

(a) It is recommended that the CCC consider their options for repairing and strengthening or demolishing the building.

(b) If the building is to be retained, it is recommended that foundation conditions be verified by doing a floor level survey to ascertain differential settlement.

13 Limitations

(a) This report is based on an inspection of the structure with a focus on the damage sustained from the 22 February 2011 Canterbury Earthquake and aftershocks only.

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

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

14 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 A – Photographs

Photo 1: View of the north wall of the building

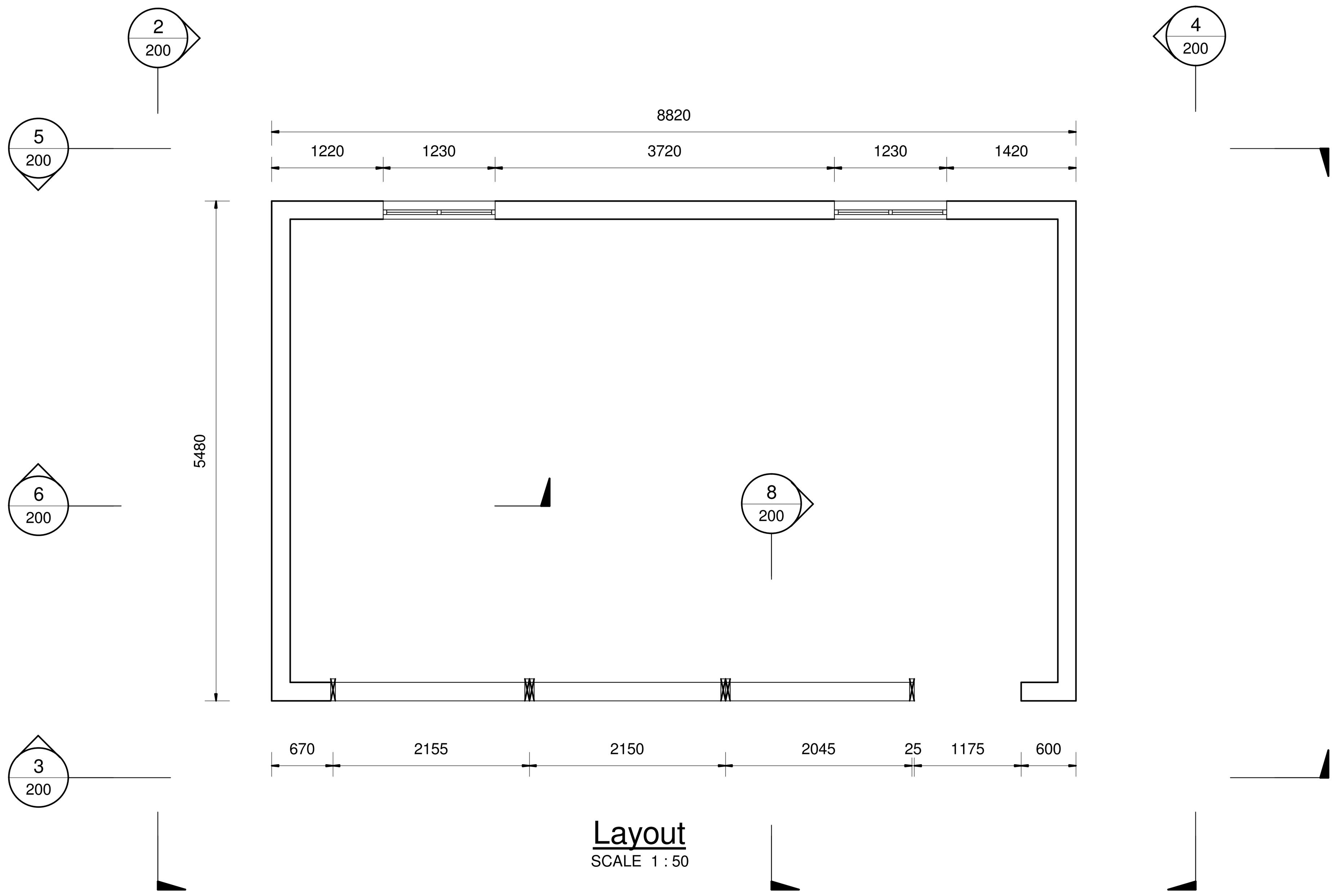
Photo 2: View of the large crack on the east wall

Photo 3: View of the wall lateral displacement near the top of the east wall

Photo 4: View of the south wall showing the crack caused by the east wall leaning out of plane

Photo 5: View of the west wall displacing laterally at the foundations

Appendix B – Building Plan



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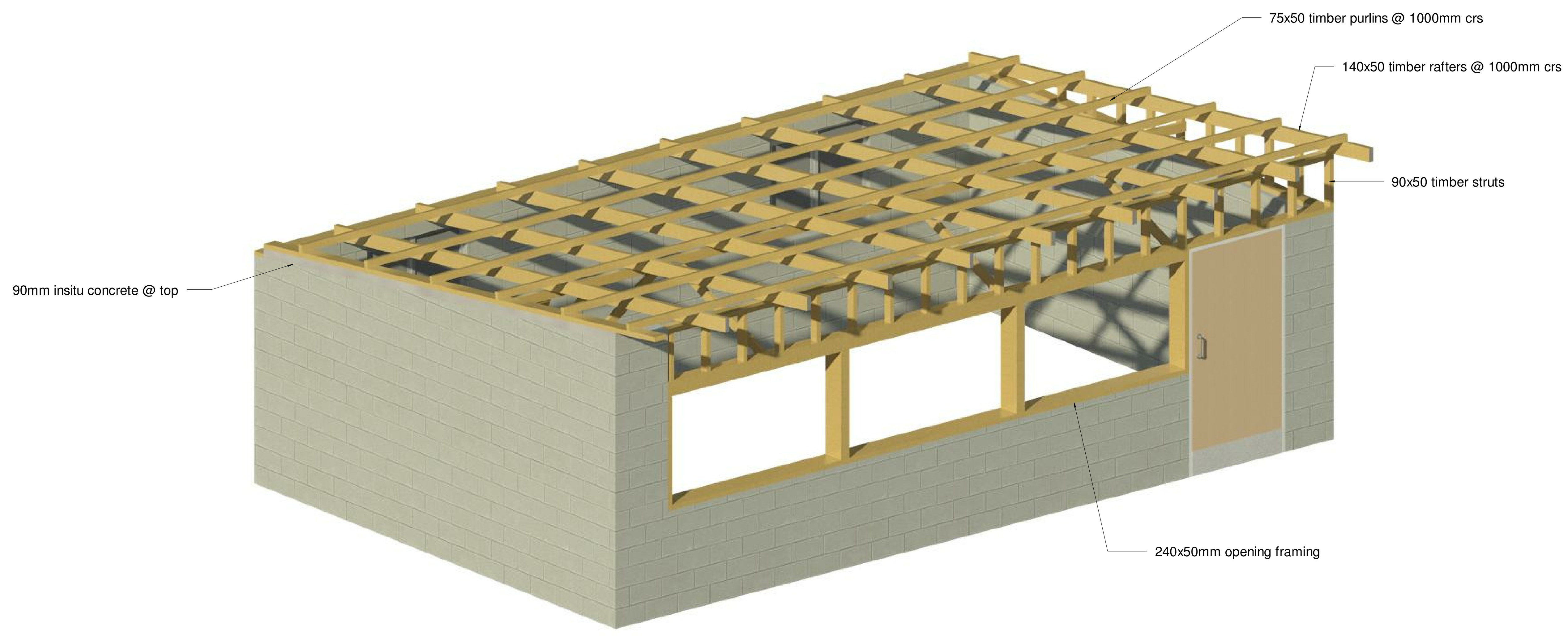

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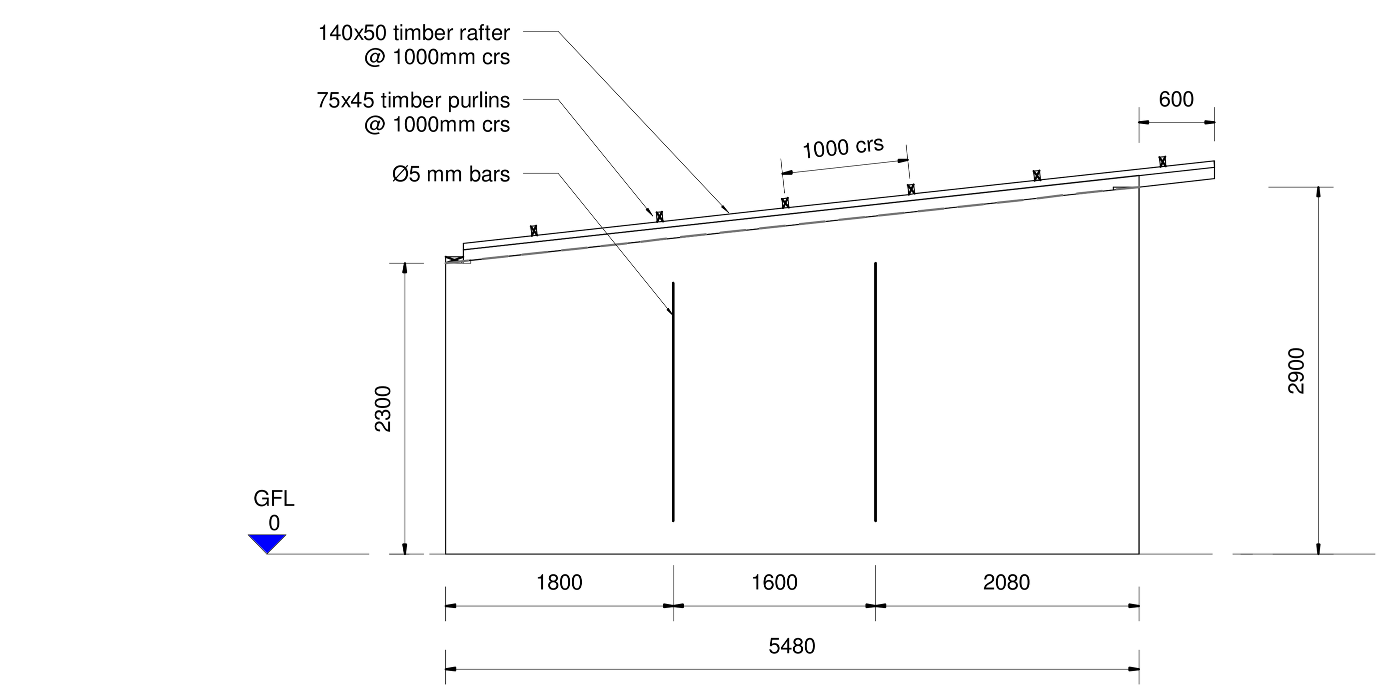

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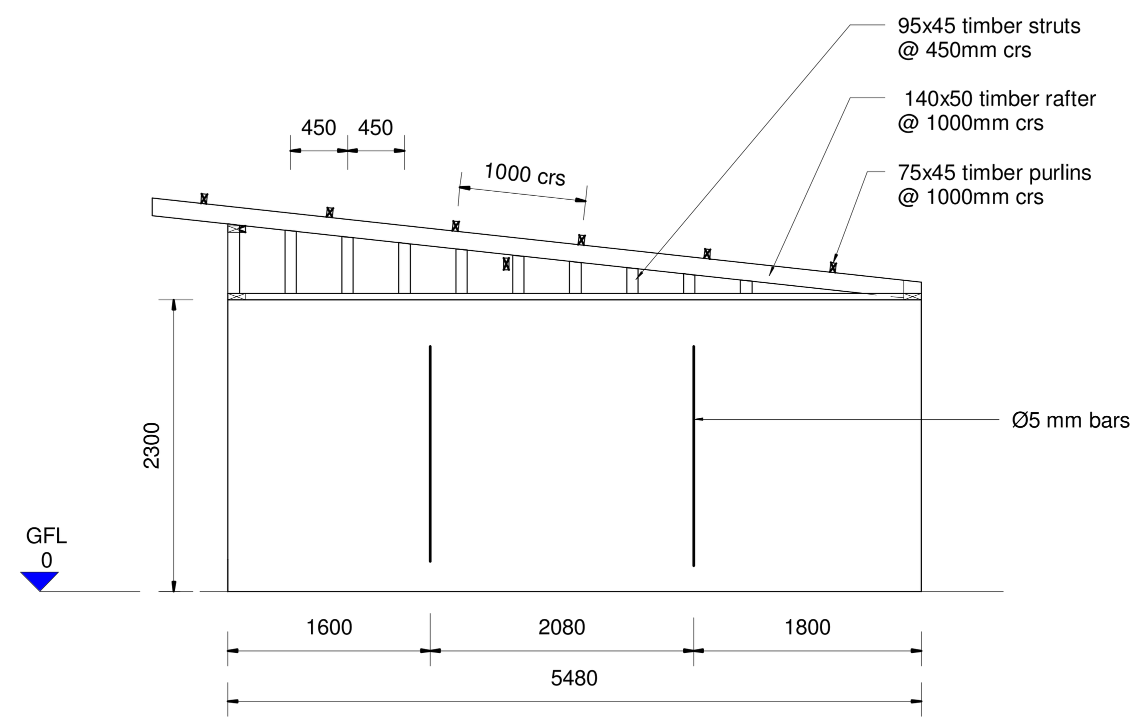

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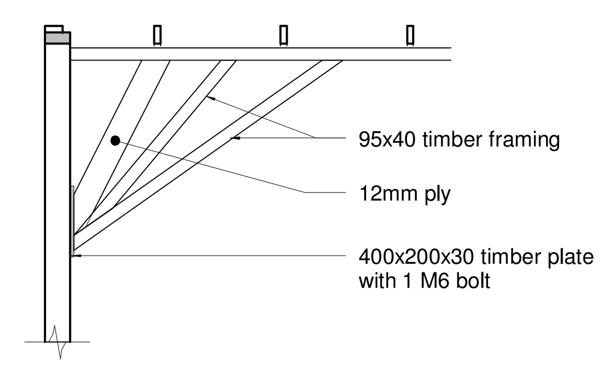

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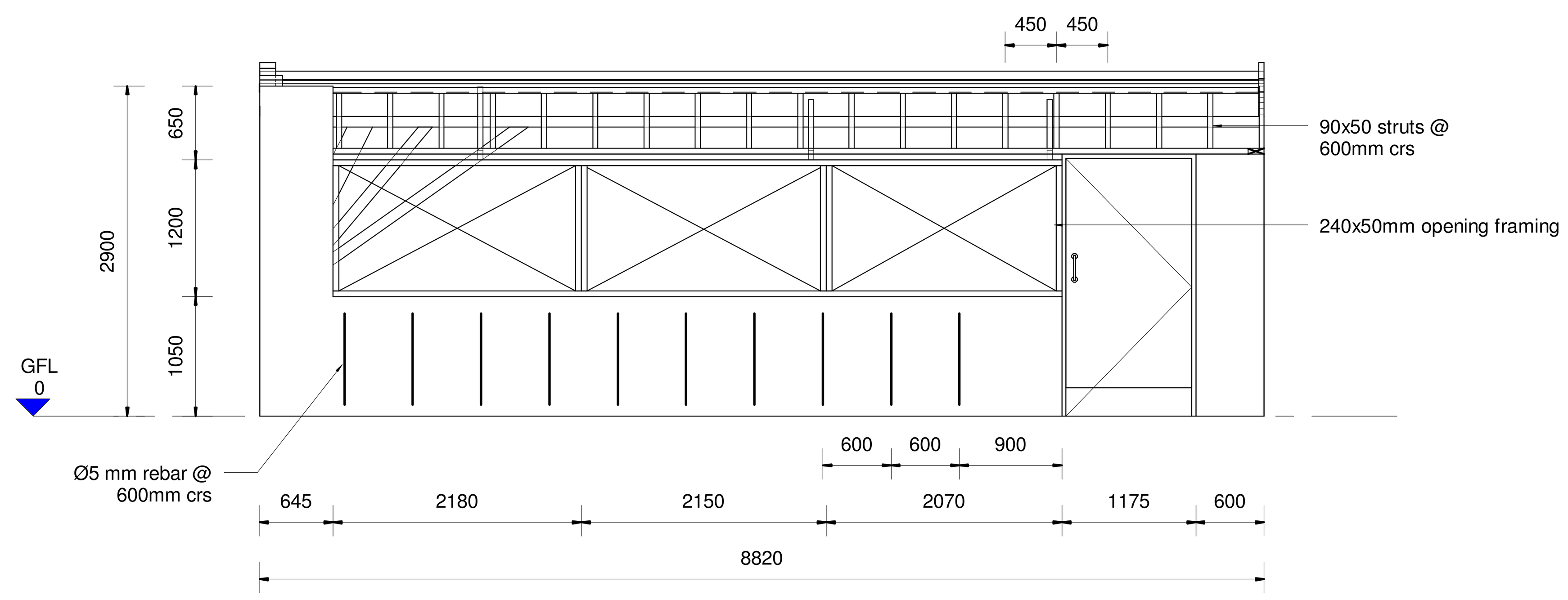

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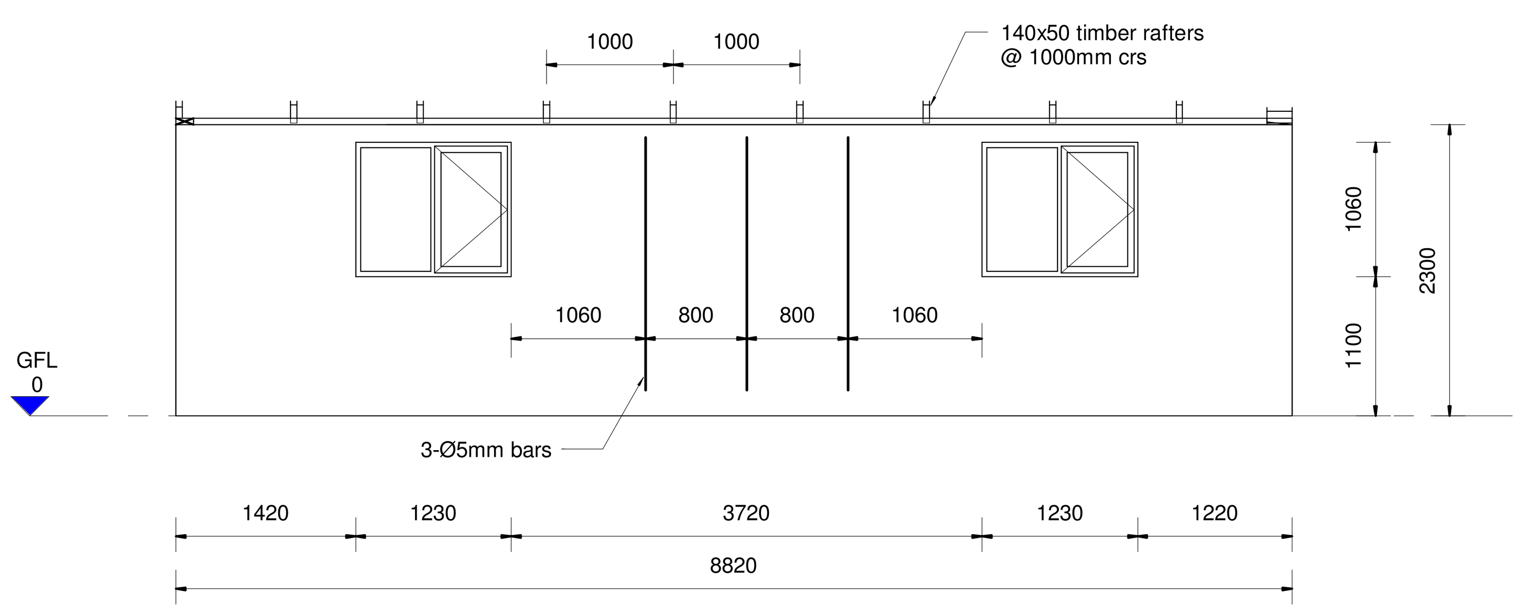

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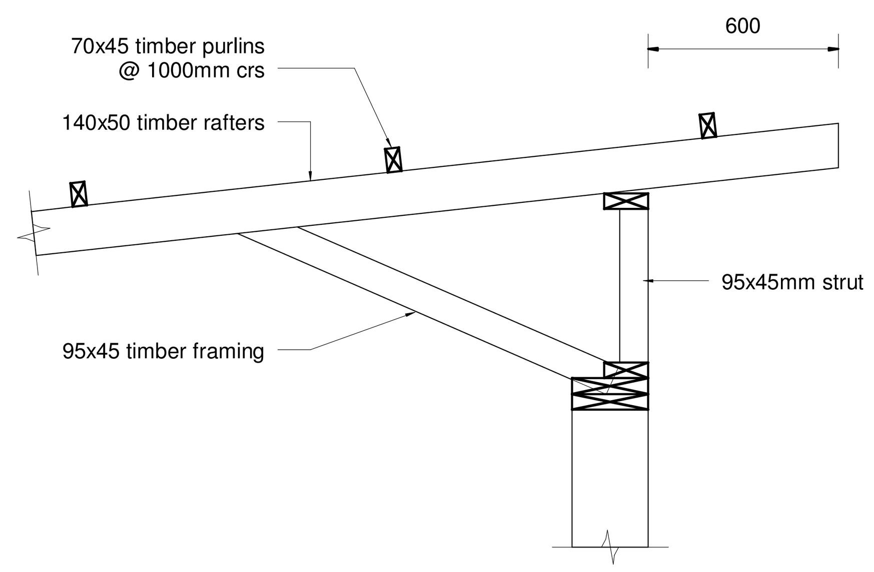

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Construction Details

Revision	Amendment	Approved	Revision Date

CHRISTCHURCH
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Title
Construction Drawings

Drawn	Designed	Approved	Revision Date
J.R.W			06.07.2012

Project No.	Scale	Drawing No.	Sheet No.	Revision
6-QUCC1.11	As indicated	6/1366/277/ 8602	200	R0

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Appendix C – Geotechnical Appraisal

15 August 2012

Christchurch City Council
C/O:- Michael Sheffield

Dear Michael

6-QUCC1.11

Geotechnical Desktop Study – Heathcote Domain – Former Tennis Club Shed

1. Introduction

Christchurch City Council (CCC) has commissioned Opus International Consultants (Opus) to undertake a brief geotechnical desktop study of the Heathcote Domain – Former Tennis Club Shed, 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.

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 Desk Study forms part of a Detailed Engineering Evaluation prepared by Opus and has been undertaken without the benefit of any site specific investigations and is therefore preliminary in its nature.

A floor level survey has not been undertaken by Opus.

2. Desktop Study

2.1 Site Description

The Heathcote Domain – Former Tennis Club Shed is located off Port Hills Road on the north side of the Domain. The site is bounded by the western portal of the Lyttleton rail tunnel to the east, areas of on-going development (formerly demolished buildings damaged by the earthquake to the north and east and the park playground area to the west. Refer to the Site Location Plan in Appendix A.

There were no as-built drawings available. From a visual external inspection, the shed foundations appear to be concrete slab on grade supporting perimeter masonry walls.

The ground profile gently slopes down from the Heathcote Domain which is south of the shed, and then falls steeply to the east towards the railway line and north to the on-going area of development.

A hard court area is located to the immediate north of the Club Shed whilst a concrete masonry retaining wall approximately 1.4m high is situated about 2m north of the Club Shed.

2.2 Regional Geology

The 1:250,000 Geological Map of Christchurch[1] indicates the site is underlain by Banks Peninsula Loess which generally comprises wind blown silt deposits up to 3m thick. These are underlain by basaltic and trachytic lava flows of the Lyttleton Volcanic Group. (Refer to Appendix B - Geological Map)

2.3 Expected Ground Conditions

A review of the Environmental Canterbury (ECan) wells database showed no well or boreholes available within a 2km radius from the site. The nearest borehole is located 2.8km north east at McCormacks Bay Road, Mount Pleasant.

From the photograph taken on the eroded steep slope east of the hard court area (Photo 2 of Site Walkover Plan), the loess is estimated to be approximately 3m to 4m overlying basalt.

According to Environment Canterbury Regional Council records, the closest groundwater monitoring well (M36/1159) is in Scruttons Road, Ferrymead 2km north east of the site. The groundwater level is expected to be at depths greater than 2.0m below ground level at the Former Tennis Club Shed.

2.4 Ground Damage

Specific details of observed ground damage are recorded in Section 3 of this desk study.

No evidence of liquefaction was observed in aerial photographs taken after the 4 September 2010, 24 February 2011 or 13 June 2011 earthquakes.

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. The site is classified as 'Port Hills and Banks Peninsula", which indicates that land damage from liquefaction is not anticipated.

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 and June 2011. There is no surface evidence of liquefaction around the Heathcote Domain.

The Christchurch Earthquake Recovery Authority (CERA) last updated 11 December,2011 has classified the area surrounding the Heathcote Domain Former Tennis Club Shed under Green Zone, indicating the repair and rebuilding process can begin with appropriate advice from local authority.

Port Hills Geotechnical Group (PHGG) have indicated there are no known mass landslide features and there is no known rockfall potential affecting this site.

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[1] GNS 1:100,000 Geological Map 3 - Geology of Banks Peninsula.

3. Site Walkover Inspection

A site walkover inspection was carried out by an Opus Geotechnical Engineer on 26 June 2012. Only exterior observations were made at the time of the visit due the limited access as the Club Shed has been fenced.

The following observations were made, refer to the Appendix C - Site Walkover Plan and Appendix D - Site Photographs attached to this report:

- Hairline and web-like cracks observed in some places around the tennis court (Photograph 1, Appendix C).
- Corner wall of the concrete retaining structure leaning north by approximately 6 degrees (Photograph 3, Appendix C).
- Concrete masonry retaining wall has sustained approximately 2mm to 3mm wide cracks on the face running from bottom to top of the wall (Photograph 4, Appendix C).
- Timber storage shed founded on severely cracked concrete slab (Photograph 5, Appendix C).
- Light post leaning approximately 5 degrees north, with cracks radiating from the bottom of post (Photograph 6, Appendix C).
- Vertical settlement of more than 10mm observed between the concrete block work and concrete slab (north elevation of the shed) (Photograph 7, Appendix C).
- Lateral displacement of approximately 20mm between the concrete masonry retaining wall and the concrete slab (Photograph 4, Appendix D).

4. Discussion

The ground conditions at this site remain unknown. Based on local knowledge, we anticipate the ground conditions to consist of up to 4m of loess deposit overlying Basalt. No evidence of liquefaction at the site has been observed.

There was limited access around the site and no internal access to the Club Shed due to structural safety hazard. Visual observations were carried out a safe distance from the structure. We were not able to observe if there are hairline cracks or subsidence in the foundations of the building.

The light post and the concrete masonry retaining wall at the Former Tennis Club Shed have rotated north in the direction of down slope movement.

GNS have established a network of strong motion monitoring stations across New Zealand for the purpose of measuring the strengths of seismic events. The closest monitoring station to the school which recorded the peak ground acceleration (PGA) for the February 2011 earthquake is Heathcote Valley Primary School located 130m east of the shed just across the railway.

The horizontal PGA recorded at Heathcote Valley Primary School was 1.5g. It was these very high PGAs which has caused the lateral movement of the ground as observed by the leaning retaining wall and the light post towards the north.

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 14% probability of another Magnitude 6 or greater earthquake occurring in the next 12 months in the Canterbury region. Dependent on the location of the epicentre, further ground shaking damage could be experienced at this site. It is expected that the probability of occurrence is likely to decrease with time, following periods of reduced seismic activity.

5. Recommendations

If the building is deemed to be structurally repairable, the following actions are recommended:

1. Undertake a floor level survey to ascertain if differential settlement has occurred to the foundations.
2. Set up monitoring of the crack widths and tilt is recommended.
3. Undertake repairs to concrete masonry wall.

6. Limitations

This report has been prepared solely for the benefit of CCC as our client with respect to the particular brief given to us. Data or opinions in this 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:

Sewell,R.J.; Weaver, S.D.; Reay,M.B. 1992: Geology of Banks Peninsula. Scale 1:100,000. Institute of Geological and Nuclear Sciences geological map 3, sheet 1.

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 24 February 2012.

Appendices:
Appendix A: Site Plan
Appendix B: Geological Map
Appendix C: Site Walkover Plan
Appendix D: Site Photographs

APPENDIX A:
Site Location Plan

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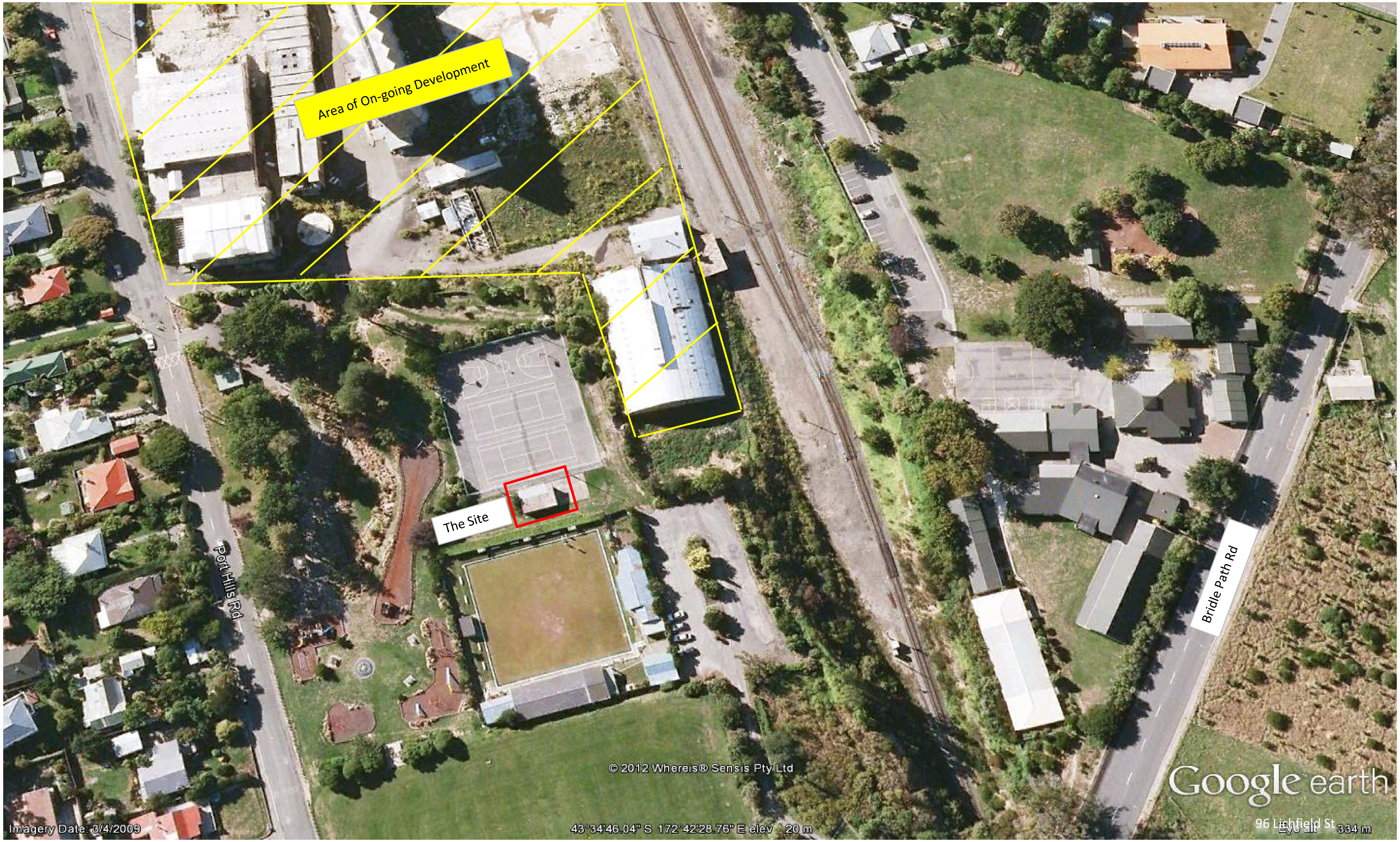

SOURCE: canterburyrecovery.projectorbit.com (Accessed on 26/06/12)

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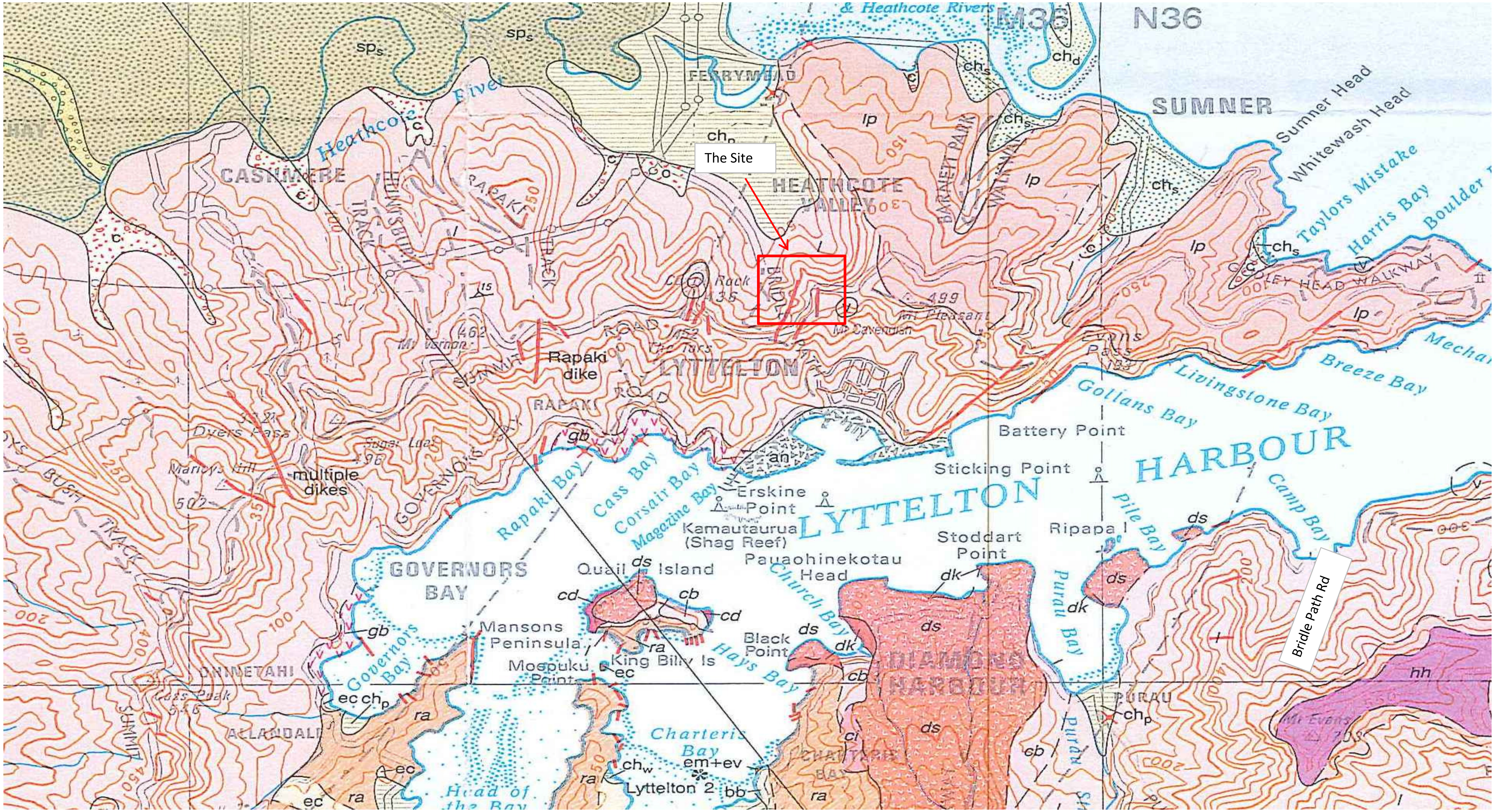
Project:	CCC- Heathcote Domain - Former Tennis Club Shed Geotechnical Desktop Study
Project No.:	6-QUCC1.11
Client:	Christchurch City Council

Site Location Plan

Date: 26/06/2012

APPENDIX B:
Geological Map



Geological Legend **Description** **Formation / Member** **Group**

SOURCE: Sewell,R.J,et.al.,Geology of Banks Peninsula.Scale 1:100,000,GNS Ltd, 1993.

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Geological Map

Date: 26/06/2012

APPENDIX C:
Site Walkover Plan

Hairline and web -like cracks observed on the ground in some places around the tennis court.


Photograph 1

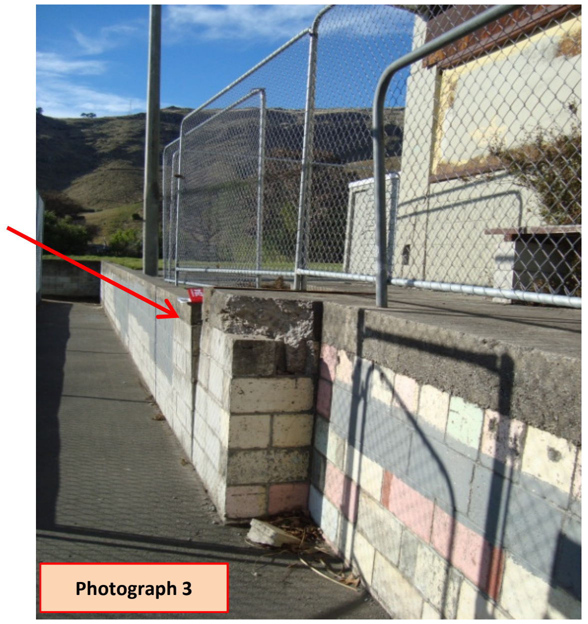

Photograph 2

N

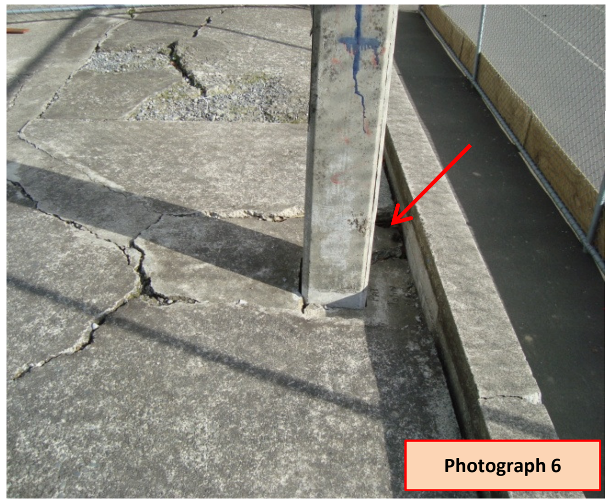
Very steep slopes east of the tennis club fenced with warratahs and aluminum wire at the top


Photograph 5

Timber storage shed on severely cracked concrete slab and apron area.


Photograph 3

Corner wall leaning north 100mm away from the Club Shed structure


Photograph 6

Light post slightly leaning north by approximately 5 degrees with respect to the vertical. Cracks radiating from the bottom of post.


Photograph 4

The concrete masonry retaining wall sustained cracks (≅ 2 to 3mm wide) on the face running from bottom to top of wall.

Vertical settlement of more than 10mm observed between the concrete block work and concrete slab (North elevation of the shed)


Photograph 7

SOURCE: canterburyrecovery.projectorbit.com
(Accessed on 30/04/12)

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Project:	CCC- Heathcote Domain - Former Tennis Club Shed Geotechnical Desktop Study
Project No.:	6-QUCC1.03
Client:	Christchurch City Council

Site Walkover Plan

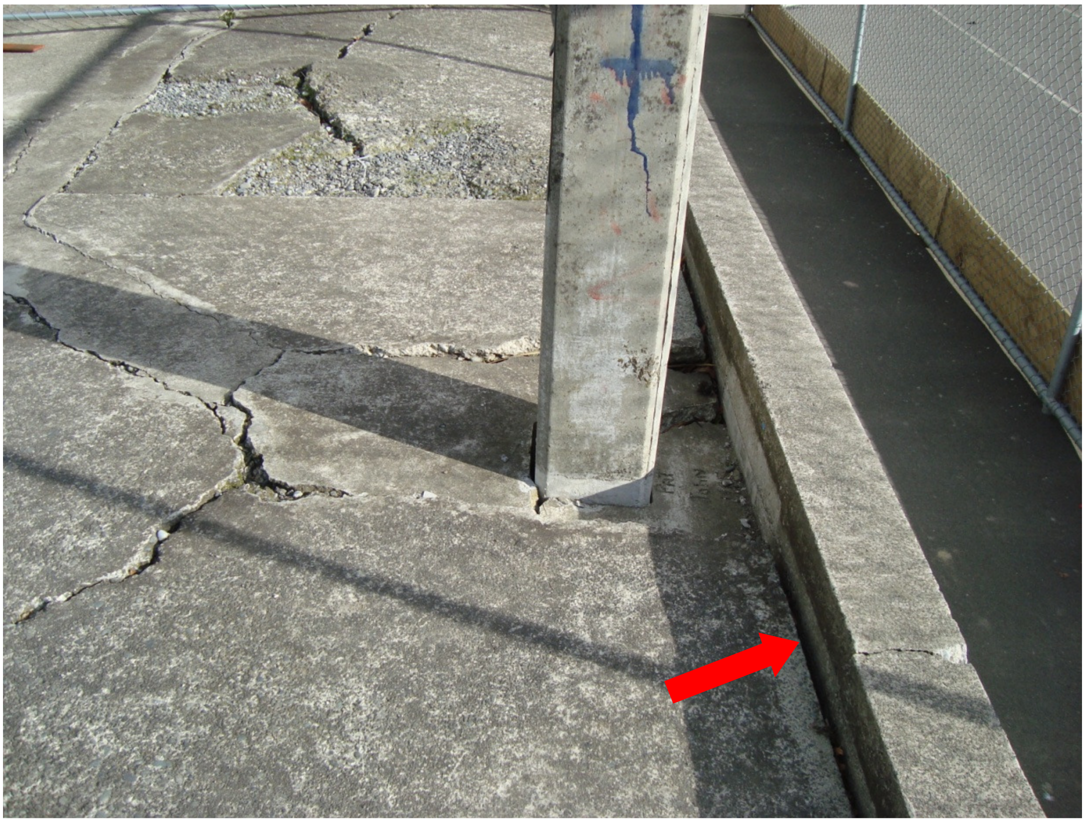
Date: 26/06/2012

APPENDIX D:
Site Photographs

Photograph 1: General view of the north facing side of the shed.

Photograph 2: View of the 10mm wide separation between the perimeter wall and the concrete slab.

Photograph 3: View of the 20mm subsidence of the concrete slab from perimeter wall. (Located far west corner of the north elevation of the shed)



Photograph 4: View of the 20mm lateral movement of the retaining wall.

Photograph 5: General view of the east and south elevations of the shed.

Appendix D – CERA DEEP Data Sheet

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Heathcote Domain Former Tennis Club Shed		Reviewer:	Dave Dekker
	Unit	No: Street	CPEng No:	1003026
Building Address:		40 Port Hills Road	Company:	Opus International Consultants
Legal Description:			Company project number:	6QUCCC.11
			Company phone number:	03 363 5400
	Degrees	Min Sec		
GPS south:			Date of submission:	27/08/2012
GPS east:			Inspection Date:	6-Jun-12
			Revision:	Final
Building Unique Identifier (CCC):	PRK 1880 BLDG 007 EQ2		Is there a full report with this summary?	yes

Site

Site slope:	slope < 1in 10	Max retaining height (m):	
Soil type:	silt	Soil Profile (if available):	
Site Class (to NZS1170.5):	C		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	10.00

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	
Ground floor split?	no		Ground floor elevation above ground (m):	
Storeys below ground	0			
Foundation type:	raft slab		if Foundation type is other, describe:	
Building height (m):	2.50		height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	48			
Age of Building (years):	18		Date of design:	
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	public		Brief strengthening description:	
Use (upper floors):				
Use notes (if required):				
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	
Floors:	concrete flat slab	slab thickness (mm)	
Beams:	none	overall depth x width (mm x mm)	
Columns:	other (note)	typical dimensions (mm x mm)	None
Walls:	partially filled concrete masonry	thickness (mm)	200

Lateral load resisting structure

Lateral system along:	partially filled CMU	**Note: Define along and across in**	note total length of wall at ground (m):	
Ductility assumed, μ:	1.00	**detailed report!**	wall thickness (m):	
Period along:	0.40	##### enter height above at H31	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	partially filled CMU		note total length of wall at ground (m):	
Ductility assumed, μ:	1.00		wall thickness (m):	
Period across:	0.40	##### enter height above at H31	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:		
Wall cladding:		
Roof Cladding:	Metal	describe
Glazing:	timber frames	
Ceilings:		
Services(list):		

Available documentation

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

Damage

Site:
(refer DEE Table 4-2)

Site performance:		Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	0-1:350	notes (if applicable):	
Liquefaction:	0-2 m²/100m³	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	slight	notes (if applicable):	

Building:

	Current Placard Status:	yellow		
Along	Damage ratio:	57%	Describe how damage ratio arrived at:	
	Describe (summary):			
Across	Damage ratio:	96%		
	Describe (summary):			
Diaphragms	Damage?:		Describe:	
CSWs:	Damage?:		Describe:	
Pounding:	Damage?:	yes	Describe:	
Non-structural:	Damage?:		Describe:	

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

Recommendations

	Level of repair/strengthening required:	significant structural and strengthening	Describe:	
	Building Consent required:	yes	Describe:	
	Interim occupancy recommendations:	do not occupy	Describe:	
Along	Assessed %NBS before:	7% ##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Quantitative
	Assessed %NBS after:	3%		
Across	Assessed %NBS before:	75% ##### %NBS from IEP below		
	Assessed %NBS after:	3%		

IEP

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

Period of design of building (from above): 0 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:	22.5%	22.5%

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

	along	across
Final $(\%NBS)_{nom}$:	**23%**	**23%**

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	0.8
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)		
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:		
Structural Performance Scaling Factor **Factor E**:	#DIV/0!	#DIV/0!

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:	insignificant	1
3.2. Vertical irregularity, Factor B:	insignificant	1
3.3. Short columns, Factor C:	insignificant	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	insignificant	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 valule =1.5, no minimum		
Rationale for choice of F factor, if not 1		

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)
List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

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

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