R&R Sport

BU 2677-005 EQ2

Detailed Engineering Evaluation

Quantitative Assessment Report

Christchurch City Council

R&R Sport
Detailed Engineering Evaluation
Quantitative Assessment Report

645 - 647 Colombo Street, Christchurch
Christchurch City Council

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R& R Sport Building
BU 2677-005 EQ2

Detailed Engineering Evaluation
Quantitative Report - SUMMARY
Final

645-647 Colombo Street, Christchurch

Background

This is a summary of the quantitative report for the building structure at 645-647 Colombo Street (R & R Sport), and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on 15 December 2011 and 19 January 2012, available drawings and calculations.

Key Damage Observed

Key damage observed includes:-

- Cracking in the precast wall panels on the south elevation
- Diagonal shear cracking in the precast concrete columns along the store front exterior sides of the building, between the bottom of the roof wall panels and the steel framed canopy.
- Partial roof collapse at the south end of the building. The roof diaphragm has disconnected from the precast concrete wall panels and the roof has lost gravity support.
- The connection of the precast spandrel along gridline 3 to the wall panel along gridline E has heavy damage.
- Damage to non-structural elements was also observed.

Critical Structural Weaknesses

The following critical structural weaknesses have been identified:

a) The connections from the interior precast walls to the continuous footings have limited capacity and little or no ductility.
b) The precast panel connections to the precast columns along Lichfield and Colombo Streets (north and east faces, respectively) have limited capacity and little or no ductility. If these connections fail, the lateral load will have to be resisted by the wall panels, and the roof bracing is inadequate to deliver the lateral load through torsion.
c) The roof diaphragm bracing does not have a complete load path to adequately deliver the lateral load to the lateral force resisting elements.
d) The connection of the roof diaphragm bracing to the precast wall panels has insufficient lateral load carrying capacity.
e) There are insufficient wall ties for out of plane lateral support of the wall panels.
f) The mezzanine floor level has an insufficient amount of lateral force resisting elements. In some locations, the mezzanine has lateral resistance on only two sides. The rest of the mezzanine has lateral resistance on three sides, but the diaphragm lacks the rigidity to transfer the lateral load through torsion to the lateral load resisting elements.

Indicative Building Strength (from quantitative assessment)
Based on the information available, and from undertaking a quantitative assessment,, the building's original capacity has been assessed to be in the order of 12% NBS and post-earthquake capacity in the order of 12% NBS. The building is therefore classed as an earthquake prone building.

Recommendations
It is recommended that:

a) A strengthening works 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 be engaged to determine the costs for either strengthening the building or demolishing and rebuilding.

c) A cordon should be placed around the full perimeter 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.

Contents

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council (CCC) to undertake a detailed seismic assessment of the retail building, located at 645-647 Colombo Street, Christchurch following the M6.3 Christchurch earthquake on 22 February 2011.

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

The seismic assessment and reporting have been undertaken based on the 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. Two relevant sections are:

Section 38 – Works

This section outlines a process in which the chief executive can give notice that a building is to be demolished and if the owner does not carry out the demolition, the chief executive can commission the demolition and recover the costs from the owner or by placing a charge on the owners' land.

Section 51 – Requiring Structural Survey

This section enables the chief executive to require a building owner, insurer or mortgagee to carry out a full structural survey before the building is re-occupied.

We understand that CERA will 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). It is anticipated that CERA will adopt 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.

We anticipate that 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).

2.2.1 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 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.

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
Low Risk Building	A or B	Low	Above 67	Acceptable (improvement may be desirable)
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally. Improvement recommended
Risk Building	D or E	High	33 or lower	Unacceptable (Improvement required under Act)

Improvement of Structural Performance	
Legal Requirement	**NZSEE Recommendation**
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
	Not recommended. Acceptable only in exceptional circumstances
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.

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

Table 1: %NBS compared to relative risk of failure

4 Background Information

4.1 Building Description

The retail building is located at 645-647 Colombo Street at the intersection of Colombo Street and Lichfield Street.

The building is a single storey retail structure with a partial mezzanine floor constructed with precast concrete walls and frames and a timber roof. The north portion of the building was occupied by R&R Sport and the south portion was occupied by Penny Lane Records. The north and south portions are separated by interior precast concrete shear walls.

The building is roughly rectangular in shape, with Lichfield Street to the north and Colombo Street to the east. The building is 23m long in the east-west direction and 25m wide in the north-south direction. The roof levels vary through the building with a maximum height of approximately 7.5m and a minimum height 5.6m. The roof generally slopes from east to west, with one portion on the west end sloping from north to south.

The original drawing consent set is dated in December 1982, with amendments in January 1983 and March 1983. Site observations indicate that the extent of the mezzanine floor is greater than that shown on the consent drawing set, but drawings were not available for this addition, nor was there any information on the date of construction.

Ground floor and mezzanine floor plans have been included in Appendix 2 of this report.

4.2 Gravity Load Resisting System

The gravity load resisting system consists of a mixture of timber and steel floor framing supported by precast concrete wall panels and columns along the interior and exterior of the building. At the roof level, 0.5mm thick galvanised steel tray roofing is supported by boxed timber purlins at the roof level. The purlins in turn are supported by steel beams or the precast concrete wall panels. At the mezzanine floor level, the floor consists of 20mm thick particle board over 250x50mm timber joists. The joists are supported by steel beams which frame to interior steel tube columns and exterior precast concrete columns.

A steel framed canopy runs along the Lichfield and Colombo sides of the building (north and east respectively) framed with hollow steel tubes.

At the ground level there is a 100mm unreinforced concrete slab on grade over 150mm of compacted hardfill. Precast concrete columns are supported by reinforced concrete spread footings. Precast wall panels are supported by reinforced concrete continuous footings.

4.3 Lateral Load Resisting System

Lateral resistance is provided by precast reinforced concrete shear wall panels, as well as frame action between roof level precast panels and precast concrete columns. There are lines of lateral resistance around the entire perimeter of the building. The moment frames occur along the store front sides of the building, the north and east faces (gridlines A and 6), with the walls occurring on the south and west faces (gridlines G and 1). There are also interior walls along gridlines E and 3. The exterior concrete walls are doweled directly into their supporting foundations while at the interior walls, embed plates doweled into the walls and into the footings are welded together. There are no supplemental lateral force resisting elements at the mezzanine floor level. Portions of the mezzanine floor only have lateral resistance on two sides.

Diaphragm action at the roof level is provided by tension equal angle steel braces. The providing bracing arrangement is insufficient to adequately distribute the lateral load to the lateral force resisting elements.

4.4 Original Documentation

Copies of the following construction drawings were provided by CCC:

- Retail Development, Corner of Colombo & Lichfield Streets, Structural Drawings, stamped 23 December 1982. The drawings were prepared by Holmes Wood Poole and Johnstone Ltd.

5 Survey

5.1 Post 22 February 2011 Rapid Assessment

A structural (Level 2) assessment of the above buildings/property was undertaken on 24 March 2011 by Raj Unka of Opus International Consultants.

5.2 Further Inspections

A further inspection was undertaken by Diana Barr of Opus International Consultants on 21 December 2011.

These inspections included external and internal visual inspections of all structural elements above foundation level, and areas of damage to structural and non-structural elements.

6 Damage Assessment

The following damage has been noted:

6.1 Primary Seismic Structure

a) Cracking in the precast wall panels on the south elevation

b) Diagonal shear cracking in the precast concrete columns along the store front exterior sides of the building, between the bottom of the roof wall panels and the steel framed canopy.

c) Partial roof collapse at the south end of the building. The roof diaphragm has disconnected from the precast concrete wall panels and the roof has lost gravity support.

d) The connection of the precast spandrel along gridline 3 to the wall panel along gridline E has heavy damage.

6.2 Non Structural Elements

a) Damage to non-structural partitions throughout the building.

b) Damage to ceiling tiles throughout the building.

c) Cracking in the glass in the canopy along both the Lichfield and Colombo sides of the building.

7 General Observations

Both structures appear to have generally performed adequately during the earthquake.

The building has sustained moderate to severe damage to structural elements, as well as some moderate damage to non-structural elements. The observed damage is consistent with the expected building performance, following a review of the structural drawings and site investigations.

8 Detailed Seismic Assessment

The detailed seismic assessment has been based on the NZSEE 2006 [2] guidelines for the "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" together with the Detailed Engineering Evaluation Procedure [3] (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011.

8.1 Critical Structural Weaknesses

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

a) The connections from the interior precast walls to the continuous footings have limited capacity and little or no ductility.

b) The precast panel connections to the precast columns along Lichfield and Colombo Streets (north and east faces, respectively) have limited capacity and little or no ductility. If these connections fail, the lateral load will have to be resisted by the wall panels, and the roof bracing is inadequate to deliver the lateral load through torsion.

c) The roof diaphragm bracing does not have a complete load path to adequately deliver the lateral load to the lateral force resisting elements.

d) The connection of the roof diaphragm bracing to the precast wall panels has insufficient lateral load carrying capacity.

e) There are insufficient wall ties for out of plane lateral support of the wall panels.

f) The mezzanine floor level has an insufficient amount of lateral force resisting elements. In some locations, the mezzanine has lateral resistance on only two sides. The rest of the mezzanine has lateral resistance on three sides, but the diaphragm lacks the rigidity to transfer the lateral load through torsion to the lateral load resisting elements.

8.2 Quantitative Assessment Methodology

The assessment assumptions and methodology have been included in Appendix 4 of the report due to the technical nature of the content. A brief summary follows:

A 3D model of the building was created in SAP2000, a finite element structural analysis programme.

Static and modal response spectrum analyses were carried out using the spectral values established from NZS1170.5, with an updated Z factor of 0.3 (B1/VM1). These analyses were used to establish the actions on the structural elements. Based on the actions determined from the analyses, an assessment of the building capacities was made.

A two tier check was performed. First, it was assumed both the frames and the walls contributed to the lateral load resistance in this direction. The frames were determined to have insufficient lateral load carrying capacity. Secondly, it was then assumed the frames had failed so the rest of the structure was checked for the full lateral load.

8.3 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.

8.4 Quantitative Assessment

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.

Table 2: Summary of Seismic Performance

Structural Element/System	Failure mode and description of limiting criteria	Critical Structural Weakness and Collapse Hazard	% NBS based on calculated capacity
Interior precast wall connection to foundation	Brittle concrete blowout failure in the embed plate at the panel edge which connects to the foundation embed plate. The embed plate has very little edge distance, so while there is additional capacity in the steel elements, the concrete limits the capacity of the connection. Once the connection fails, the panel can no longer resist shear and overturning load demand, and the load is shed to the exterior walls.	Yes	12%
Precast panel connection to precast concrete column	Brittle concrete blowout failure in the anchor at the panel edge which connects to a steel angle clipped to the column. The anchor has very little edge distance, so while there is additional capacity in the steel elements, the concrete limits the capacity of the connection. Once the connection fails, the panel can no longer resist moment demand, and the load is shed to the walls	Yes	16%
Incomplete load path in roof bracing	The roof bracing has an incomplete load path. Once the panel connections at the interior walls and in the frames along the north and east faces of the building begin to fail, it is imperative that there be a complete load path to deliver lateral load to the remaining lateral force resisting elements	Yes	<34%
Connection of roof bracing to wall panels	Brittle concrete blowout failure in the anchor at the panel edge which connects to a steel angle which connects to the roof bracing. The anchors have very little edge distance, so while there is additional capacity in the steel elements, the concrete limits the capacity of the connection. Once the connection fails, the diaphragm force can no longer be delivered to the panel	Yes	28%
Insufficient wall ties	Out of plane lateral load is resisted through cross grain bending in the timber runner bolted along the wall panel. This is a weak failure mode for out of plane lateral resistance in the panel.	Yes	<34%

Structural Element/System	Failure mode and description of limiting criteria	Critical Structural Weakness and Collapse Hazard	% NBS based on calculated capacity
Mezzanine floor lateral resistance	The mezzanine floor lacks a sufficient amount of lateral load resisting elements. Some places have lateral load resisting elements on only two sides.	Yes	<34%

8.5 Discussion

The seismic capacity of the building is governed by the capacity of the connection from the interior precast concrete walls to the foundation, with this connection having a capacity of 12% NBS. As highlighted in Table 2 above a number of other elements also have seismic capacities less than 34% NBS, and the building is therefore defined as being earthquake prone in accordance with the Building Act 2004.

It is considered that the brittle failure mechanisms of the structural elements in the above table could lead to a partial collapse of the building in a large aftershock and it is therefore recommended that the full perimeter of the building be cordoned off.

9 Summary of Geotechnical Appraisal

A copy of the geotechnical appraisal is attached as Appendix 3. A summary of this appraisal is as follows:

9.1 General

The site is located on the relatively flat lying plains of Christchurch's city centre and is located approximately 270m east of the Avon River.

The foundations consist of a 100mm thick unreinforced concrete slab supported on hardfill and demolition bricks. Internal columns are supported on shallow concrete.

9.2 Liquefaction Potential

The 2004 Environment Canterbury (ECan) Solid Facts Liquefaction Study indicates the site is approximate within an area designated as 'low liquefaction ground damage potential'. According to this study, based on a low groundwater table, ground damage is expected to be minor and may be affected by up to 100mm of ground subsidence.

9.3 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).

The existing foundations have performed satisfactorily and do not appear to have sustained significant damage. The existing foundations are considered appropriate for the building, however it must be noted that minor settlement, similar to what has already occurred, may occur in future seismic events.

9.4 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. An existing CPT exists 30m to the east of the site but does not extend through the shallow gravel layer. We recommend an additional CPT close to the site that extends to a depth of ~ 15 to 20m with pre drilling of gravel layers in order to assess the liquefaction potential of sand layers below the shallow gravel.

- Founding the building on deeper, more competent soils by installing piles or installing a reinforced raft type foundation.

10 Remedial Options

The building requires repair and strengthening, with a target of increasing the seismic performance to as near as practicable to 100%NBS, and at least 67%NBS. Our concept strengthening scheme to achieve this would include:

1) Upgrade of the panel connections to the foundation at the base of the interior walls.

2) Upgrade of the precast panel connections to the precast concrete columns along Lichfield and Colombo Streets:

3) Upgrade of the roof diaphragm and mezzanine bracing to provide a complete lateral load path.

4) Upgrade of the roof bracing connections to the precast wall panels.

5) Upgrade of the out of plane support system for wall panels.

6) Repair of all current earthquake induced damage to the building.

We believe that it will not be economically feasible to strengthen the building, with a target of increasing the seismic to as near as practicable to 100%NBS, or at least 67%NBS. There are too many identified deficiencies in the lateral force resisting system for a feasible strengthening scheme to be cost effective. This would need to be confirmed by a quantity surveyor.

Any strengthening scheme will also need to allow for assessing and potentially upgrading the building to meet current Building Code accessibility and fire requirements.

11 Conclusions

a) The seismic performance of the building is governed by the shear capacity of the precast wall panel embed plates along gridline 3 and E due to concrete breakout of the embed plate anchors, which have an expected strength of 12% NBS. The building is therefore considered to be earthquake prone in accordance with the Building Act 2004.

b) Also of concern are the precast panel connections to the precast concrete columns at the roof level along the building frontage. The shear capacity of the mechanical anchors into the panels have an expected strength of 20%NBS, due to concrete breakout of the anchors in shear.

c) The building contains a number of critical structural weaknesses, including precast concrete wall connections with limited capacity and little or no ductility, an incomplete load path for the roof diaphragm bracing, insufficient wall ties for out of plane lateral support of the wall panels and an insufficient amount of lateral load resisting elements for the mezzanine floor.

d) The liquefaction hazard for the site is considered low.

e) The building contains a number of brittle failure mechanisms which could lead to a partial collapse of the building in a large aftershock, and it is recommended that the full perimeter of the building be cordoned off.

f) At this stage it is thought that it is possible to strengthen the building to at least 67% NBS, however it is expected due to the number of structural deficiencies identified that it will not be economically feasible to strengthen the building.

12 Recommendations

a) A strengthening works 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 be engaged to determine the costs for either strengthening the building or demolishing and rebuilding.

c) A cordon should be placed around the full perimeter of the building.

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

13 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. Some non-structural damage is described but this is not intended to be a complete list of damage to non-structural items.

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

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

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.

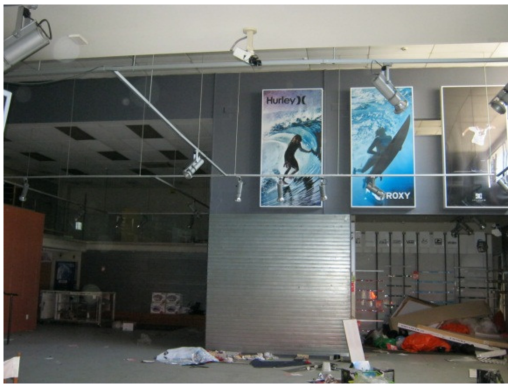
[3] Engineering Advisory Group, *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure*, Draft Prepared by the Engineering Advisory Group, Revision 5, 19 July 2011.

Appendix 1 - Photographs

Manchester Street Car Park Building		
No.	**Item description**	**Photo**
1.	View of the northeast corner at the intersection of Colombo and Lichfield	
2.	North face	
3.	Canopy on east face	

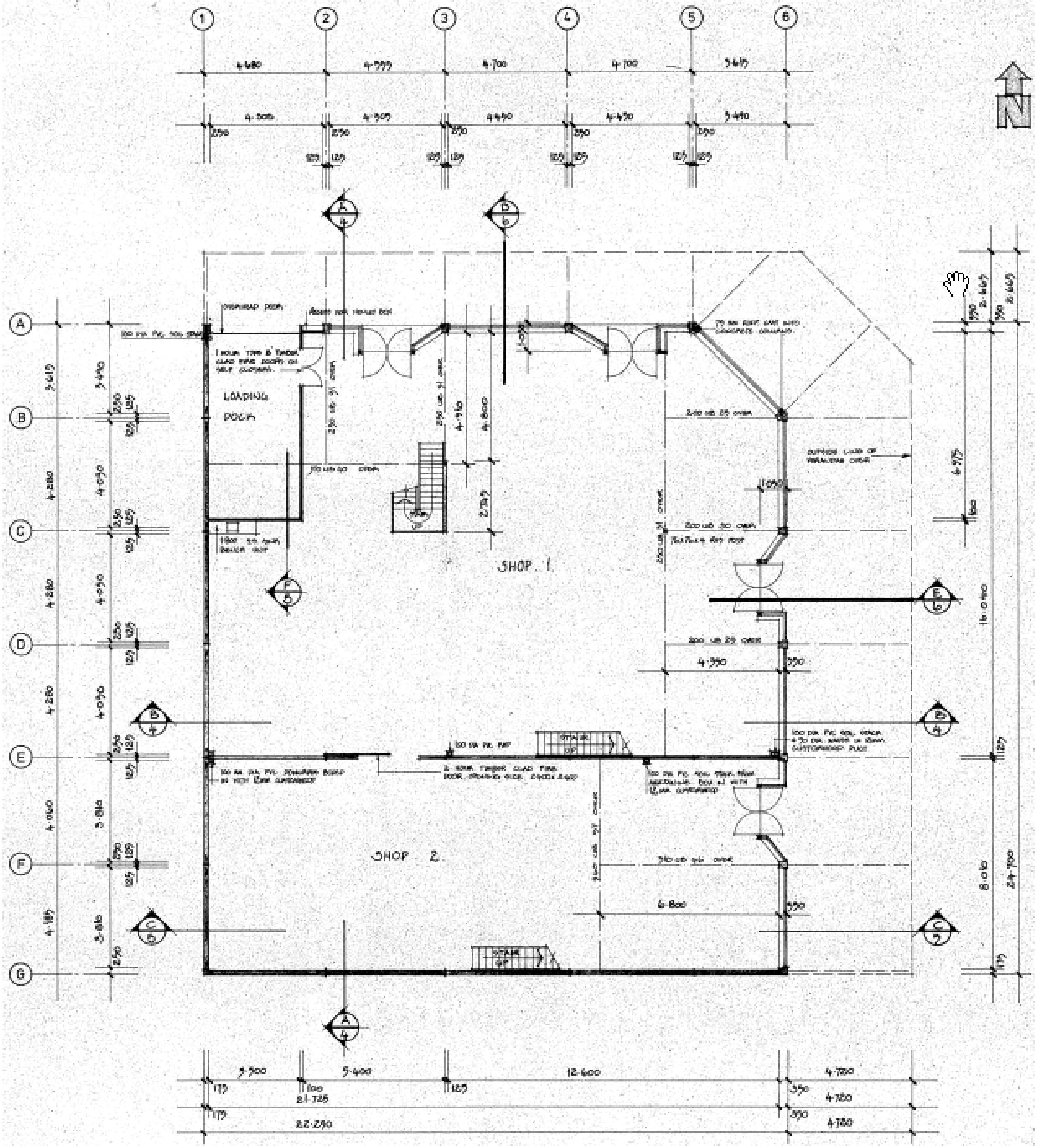
4.	South face	
5.	Gap between adjacent building on west face	
6.	Cracking in Panel P4	
7.	Southeast corner	

8.	Cracking in precast column on east face between canopy and roof panel	
9.	Interior of R&R Sport	
10.	Boxed plywood roof purlin	
11.	Roof bracing connection to precast column	

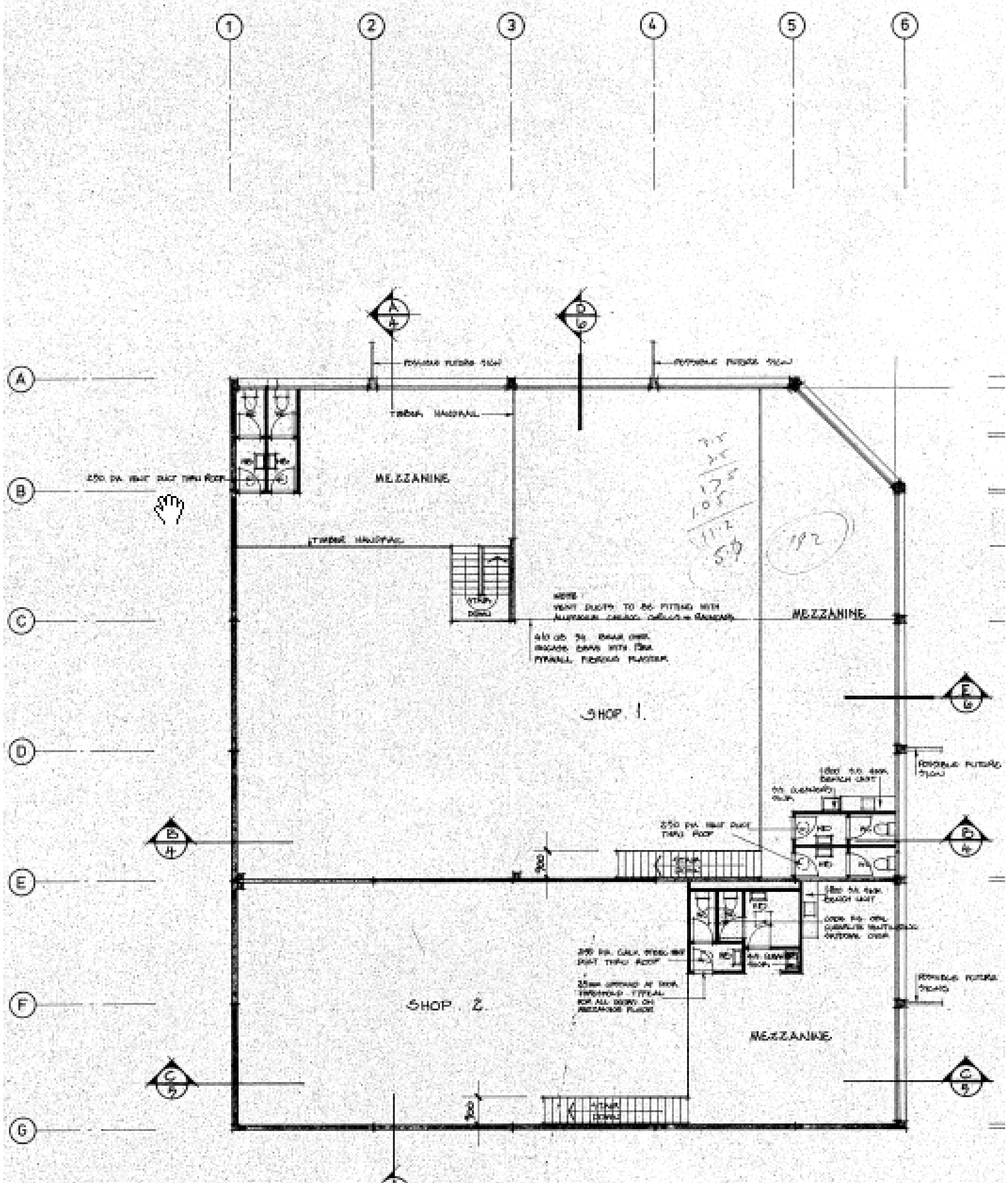
12.	Roof bracing connection to Panels P30 and P14	
13.	Cracking at Panel P30 and P14 interface	
14.	Interior Panel P31	

15.	Roof collapse along south face in Penny Lane Records	
16.	Penny Lane records interior	

Appendix 2 – Floor Plans



Ground Floor Plan



Mezzanine Floor Plan

Appendix 3 – Quantitative Assessment Methodology and Assumptions

A. Referenced Documents

- AS/NZS 1170.0:2002, *Structural design actions, Part 0: General principles,* Standards New Zealand.
- AS/NZS 1170.1:2002, *Structural design actions, Part 1: Permanent, imposed and other actions,* Standards New Zealand.
- NZS 1170.5:2004, *Structural design actions, Part 5: Earthquake actions – New Zealand,* Standards New Zealand.
- NZS 3101: Part 1: 2006, *Concrete Structures Standard, The Design of Concrete Structures,* Standards New Zealand.
- NZS 3101: Part 2: 2006, *Concrete Structures Standard, Commentary on the Design of Concrete Structures,* Standards New Zealand.
- NZBC, *Clause B1 Structure, Verification Method B1/VM1,* Department of Building and Housing.
- NZSEE: 2006, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes,* New Zealand Society for Earthquake Engineering.
- 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.

B. Analysis Parameters

The following parameters were used for the seismic analysis:

- Site soil category — Cl. 3.1.3, NZS1170.5
  D (deep or soft soil)
- Seismic hazard factor — Cl. 2.2.14$_B$, B1/VM1
  $Z = 0.30$
- Return period factor — Table 3.5, NZS1170.5
  $R_u = 1.0$ (Importance Level 2 structure, 50 year design life)
- Ductility factor — Cl. 2.6.1.2, NZS3101:2006
  $\mu = 1.25$
- Structural performance factor — Cl. 2.6.2.2, NZS3101:2006
  $S_p = 0.925$

- Material properties

Table A2: Analysis Material Properties

Concrete, compressive strength, f'_c (MPa)	30
Mild reinforcing, yield strength, f_y (MPa)	280
Rolled shapes, yield strength, f_y (MPa)	250

- Effective section properties

Table A3: Effective section properties from NZS3101:2006

Table C6.6 – Effective section properties, I_e

Type of member	Ultimate limit state		Serviceability limit state		
	f_y = 300 MPa	f_y = 500 MPa	μ = 1.25	μ = 3	μ = 6
1 Beams					
(a) Rectangular ¶	0.40 I_g (use with E_{40}) §	0.32 I_g (use with E_{40}) §	I_g	0.7 I_g	0.40 I_g (use with E_{40})§
(b) T and L beams ¶	0.35 I_g (use with E_{40}) §	0.27 I_g (use with E_{40}) §	I_g	0.6 I_g	0.35 I_g (use with E_{40})§
2 Columns					
(a) $N^*/A_g f'_c > 0.5$	0.80 I_g (1.0 I_g) ‡	0.80 I_g (1.0 I_g) ‡	I_g	1.0 I_g	As for the ultimate limit state values in brackets
(b) $N^*/A_g f'_c = 0.2$	0.55 I_g (0.66 I_g) ‡	0.50 I_g (0.66 I_g) ‡	I_g	0.8 I_g	
(c) $N^*/A_g f'_c = 0.0$	0.40 I_g (0.45 I_g) ‡	0.30 I_g (0.35 I_g) ‡	I_g	0.7 I_g	
3 Walls ¶					
(a) $N^*/A_g f'_c = 0.2$	0.48 I_g	0.42 I_g	I_g	0.7 I_g	As for the ultimate limit state values
(b) $N^*/A_g f'_c = 0.1$	0.40 I_g	0.33 I_g	I_g	0.6 I_g	
(c) $N^*/A_g f'_c = 0.0$	0.32 I_g	0.25 I_g	I_g	0.5 I_g	
4 Diagonally reinforced coupling beams	0.6I_g for flexure Shear area, A_{shear}, as in text		I_g 1.5 A_{shear} for ULS	0.75 I_g 1.25 A_{shear} for ULS	As for ultimate limit state

NOTES –
(§) With these values the E value should be the elastic modulus for concrete with a strength of 40 MPa regardless of the actual concrete strength.
(‡) The values in brackets apply to columns which have a high level of protection against plastic hinge formation in the ultimate limit state.
(¶) For additional flexibility, within joint zones and for conventionally reinforced coupling beams refer to the text.

- Earthquake load combination — Cl. 4.2.2, AS/NZS1170.0
  $G + E_u + \Psi_E Q$

- Mezzanine live loading — Table 3.1 Part G, AS/NZS1170.1
  Q = 4.0 kPa

- Earthquake combination factor — Table 4.1, AS/NZS1170.0
  $\Psi_E = 0.3$

- Building seismic weight — Cl. 4.2, NZS1170.5
  $W_t = G + \Psi_E Q$
  W_t =3,500 kN

C. Assessment Methodology

Static & Modal Spectrum Analysis

The seismic assessment was undertaken by completing a static and modal response spectrum (MRS) analysis for the building in accordance with NZS 1170.5:2004.

A 3D model was set up using the structural analysis program SAP2000, and effective section properties for structural members were taken from Table A2 above.

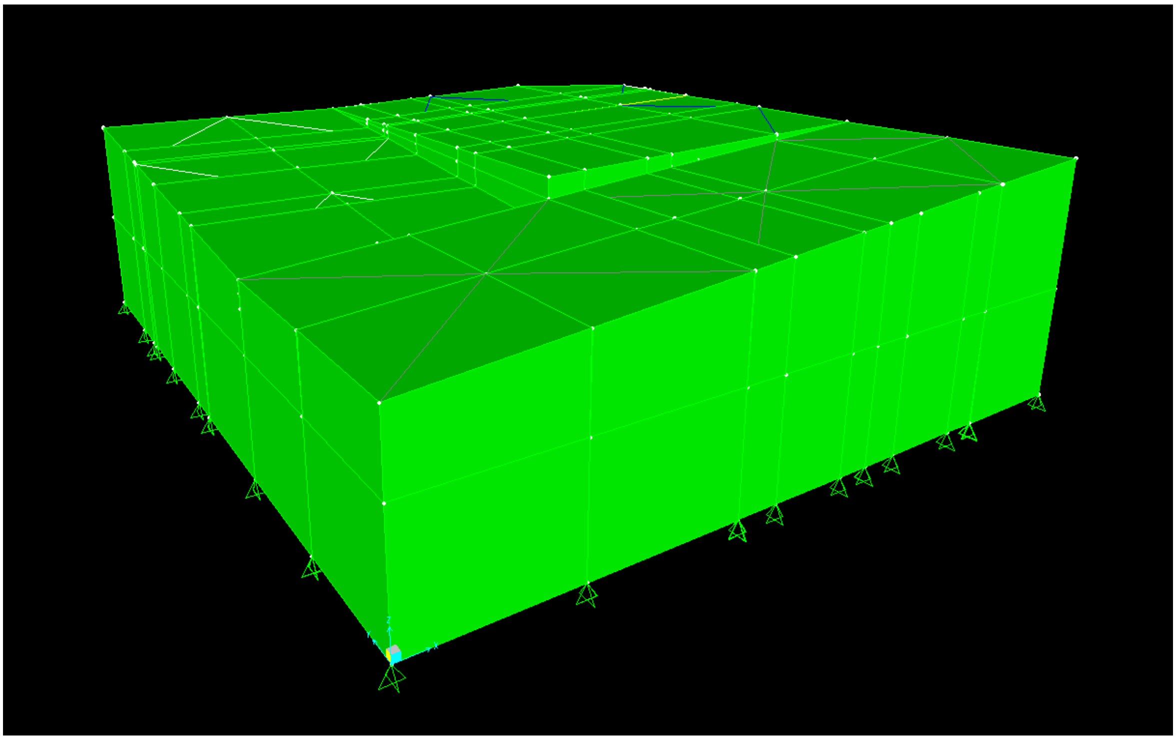


Figure A2: SAP2000 model of the building

The fundamental building periods output from SAP2000 were:

T_1 = 0.40 sec (N/S direction)

T_1 = 0.27 sec (E/W direction)

It should be noted that both primary modes of vibration were highly coupled due to the torsional sensitivity of the building.

The structural irregularity features of Clause 4.5 were checked, and the building was found to have a torsional plan irregularity. Thus, the modal response spectrum analysis was scaled to 100% of the equivalent static base shear (Cl. 5.2.2.2).

The building was assessed as being nominally ductile (μ = 1.25) in both directions. The lateral system is a mixture of precast concrete shear walls and precast concrete frames. Both types of lateral elements occur in both directions. The frames occur on the store front sides of the building, along Colombo and Lichfield Streets, Gridlines 6 and A, respectively. The walls panels occur on the west and south faces of the building, along Gridlines 1 and G, respectively. Additionally there are interior walls along Gridlines 3 and E. Because of the stiffness of the wall panels compared to the frames, and the distribution of the wall panels, the building is torsionally sensitive. A two tier check was performed on the building. First, it was assumed both the frames and the walls contributed to the lateral load resistance in both primary directions. The frame connections were determined to have insufficient lateral load carrying capacity. Because the frame connections to the columns have a brittle failure mechanism, once the design capacity was exceeded the frames lose all lateral load resistance. So, the second check assumed the frames no longer contributed to the lateral resistance in both directions, only the walls resist lateral load. This check assessed the ability of the roof bracing to carry the lateral load, formerly carried by the frames, into the walls. The torsional effects on the building were also assessed. Allowance was made for accidental eccentricity in the application of actions, as required by Clause 5.3.2.

Analysis for P-delta effects was not included in this analysis as it was determined to not be a plausible primary mode of failure.

MRS analyses on the original building were carried out for 100% of current code requirements to determine the design actions on the building.

An equivalent static analysis was carried out as a consistency check of the MRS analysis outputs. Based on the fundamental building periods and assumed ductility capacities, the following equivalent static seismic coefficients were calculated from NZS1170.5, Clause 5.2:

- C_d = 0.728 N/S direction
- C_d = 0.728 E/W direction

Appendix 4 – Geotechnical Appraisal

7 February 2012

Lindsay Fleming
Christchurch City Council
53 Hereford Street
PO Box 237
Christchurch
8140

6-QUCC.54

Geotechnical Desk Study, 645 – 647 Colombo Street

1 Introduction

The following letter summarises the findings of a Geotechnical Desk Study and Site Walkover completed on 24 January 2012. This study covers the building located at 645 – 647 Colombo Street. The purpose of this work is to assess the current ground conditions and the potential geotechnical hazards that may be present at the site. This information will be used to determine whether further subsurface geotechnical investigations are necessary.

It is our understanding that this is the first inspection of this property by a Geotechnical Professional since the initial 7.1 Darfield earthquake and subsequent aftershocks. This geotechnical desk study is being completed in conjunction with a structural quantitative assessment.

2 Desk Study

2.1 Site Description

The site is located at the intersection of Lichfield and Colombo Streets (Figure 1, Appendix A) and includes the buildings that contained the following businesses:

1) Penny Lane – A record music store located on Colombo Street
2) R and R Sports – An outdoor sports gear and clothing store fronting on both Lichfield and Colombo St.

The site is located on the relatively flat lying plains of Christchurch's city centre and is located approximately 270m east of the Avon River.

2.2 Structural Drawings

Structural drawings for the foundations of the building are available and extracts are included in Appendix B. The building is a two storey structure.

The foundations consist of a 100mm thick unreinforced concrete slab supported on 150mm of compacted hardfill. Internal columns and walls are supported on shallow concrete footings typically 250mm thick.

2.3 Regional Geology

The published geological map of the area, (Geology of the Christchurch Urban Area 1:25,000, Brown and Weeber, 1992) indicates the site is underlain predominantly by alluvial sand and silt overbank deposits belonging to the Yaldhurst member of the Springston Formation.

2.4 Expected Ground Conditions

A review of the Environmental Canterbury (Ecan) Wells database showed eight wells within approximately 150m of the property that had relevant data (Figure 2, Appendix A). The Christchurch City Council (CCC) has also released a Geological Interpretative Report and associated subsurface investigation data completed by Tonkin and Taylor in 2011. CPT-CBD-68 is located 30m east of the site and terminated in shallow gravels at a depth of approximately 5m. Logs for wells and CPTs are attached in Appendix B.

Review of the above information and structural drawings has been used to infer approximate ground conditions beneath the site.

Unit	Thickness (m)	Depth to Unit (m below ground surface)
FILL (brick and other compacted hardfill)	1 - 1.5	0
Interbeded layers of sandy SILT and silty SAND	2.5 - 4.5	1.0 - 1.5
sandy GRAVEL	5.5 - 6.0	3.5 – 6.0
SAND medium dense to dense	10 - 12	9.0 – 12.0
Sandy Gravel (Riccarton Formation)	-	20.7 – 23.8

A groundwater table depth of approximately 1m to 1.5m is likely beneath the site.

2.5 Liquefaction Hazard

Examination of post-earthquake aerial photos dated 24 February 2011 identified some evidence of liquefied soils ejected at the ground surface.

The 2004 Environment Canterbury (ECan) Solid Facts Liquefaction Study indicates the site is within an area designated as ‘low liquefaction ground damage potential’. According to this study, based on a low groundwater table, ground damage is expected to be minor and may be affected by up to 100mm of ground subsidence.

3 Site Walkover Inspection

A walkover inspection of the exterior of the building and internal ground floor level was carried out by Shane Greene, Opus Engineering Geologist on 24 January 2012. Relevant observations are summarised below with a walkover inspection plan and photographs presented in Appendix A:

- Minor settlement (<10mm) and movement of the footpath flagstones in isolated locations along the north side of Lichfield Street and the Eastern side of Colombo Street (Photograph 1,Photograph 5,Photograph 6).

- Minor cracking of the pavement was observed on both Lichfield and Colombo Street. The predominant orientation of cracking was north – south (Photograph 2).

- Minor accumulation of ejected sand adjacent to a service duct on the north side of the R&R building (Photograph 4).

- Pavement repairs south of Penny Lane. It was unclear if this was related to liquefaction or construction of the new power pole in the area (Photograph 3).

- Internal inspection of the ground floor of the building did not show evidence of substantial differential settlement.

- Piling of sand from ejected sand on the eastern side of Colombo Street which is visible in the 24 February aerial photograph (Photograph 7).

- An area of $2m^2$ affected by ground heave of 50 – 100mm north of the site.

4 Discussion

Minor damage has occurred to the building at 465 – 467 Colombo Street due to the Canterbury Earthquake and aftershock sequence following the 4 September 2010 earthquake.

No evidence of lateral spreading has been observed in the vicinity of the site.

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).

The existing foundations have performed satisfactorily and do not appear to have sustained significant damage. The existing foundations are considered appropriate for the building, however it must be noted that minor settlement, similar to what has already occurred, may occur in future seismic events.

GNS Science indicates an elevated risk of seismic activity is expected in the Canterbury region as a result of the earthquake sequence following the 4 September 2010 earthquake. Recent advice[1] indicates there is a 20% probability of another Magnitude 6 or greater earthquake occurring in the next 12 months in the Canterbury region. Therefore there is currently still a significant risk of liquefaction and differential settlements occurring. It is expected that the probability of occurrence is likely to decrease with time, following periods of reduced seismic activity.

[1] GNS Science reporting on Geonet Website: http://www.geonet.org.nz/canterbury-quakes/aftershocks/ updated on 16 December 2011.

5 Recommendations

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 wish 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. An existing CPT exists 30m to the east of the site but does not extend through the shallow gravel layer. We recommend an additional CPT close to the site that extends to a depth of ~ 15 to 20m with pre drilling of gravel layers in order to assess the liquefaction potential of sand layers below the shallow gravel.

- Founding the building on deeper, more competent soils by installing piles or installing a reinforced raft type foundation.

6 Limitation of Liability

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

Prepared By:

Shane Greene
Engineering Geologist

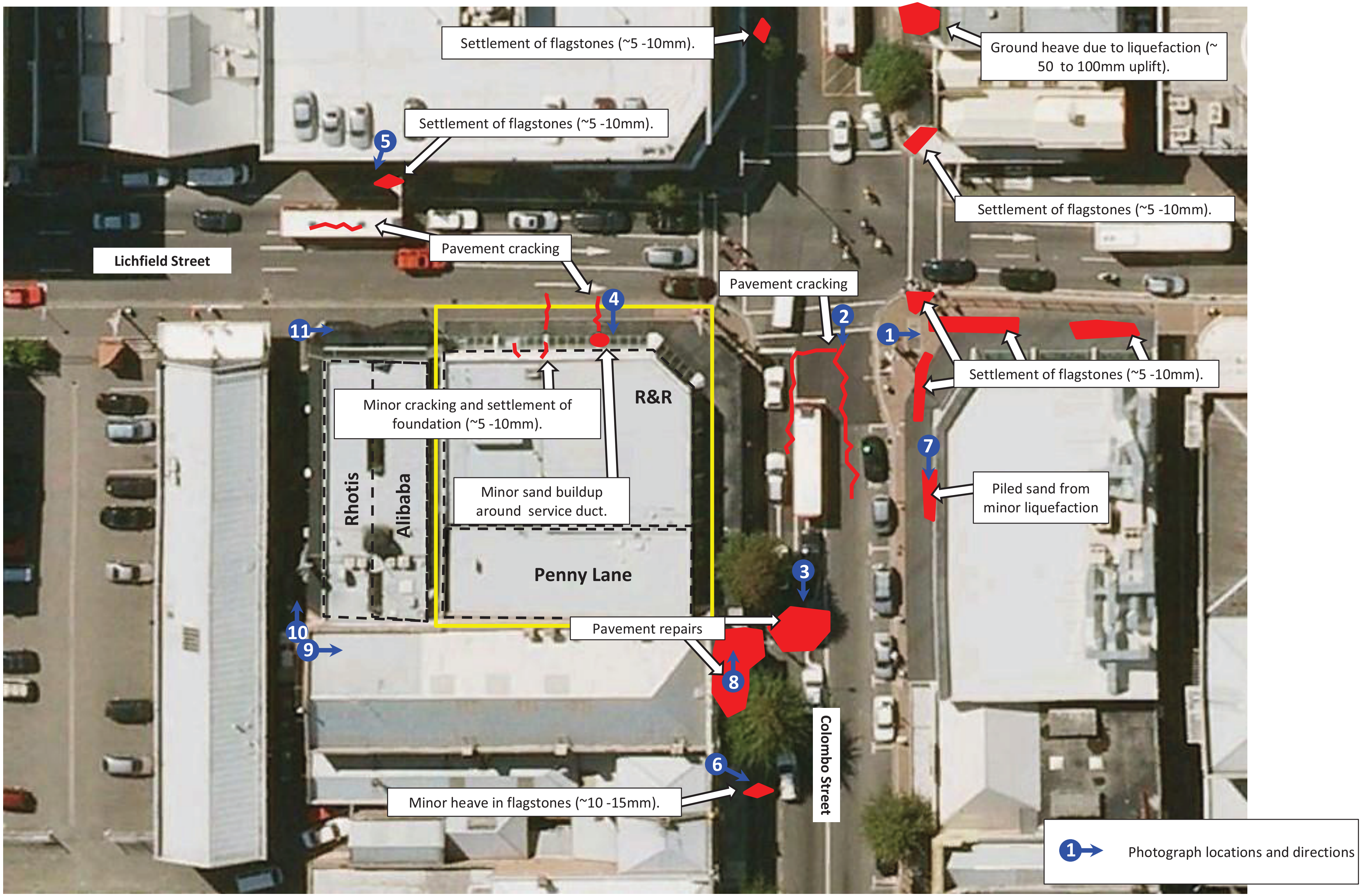
Reviewed By:

Graham Brown
Senior Geotechnical Engineer

Appendices:
Appendix A – Figures and Photographs
Appendix B – Structural Drawings
Appendix C – Boreholes and CPT logs

Appendix A:

Figures and Photographs

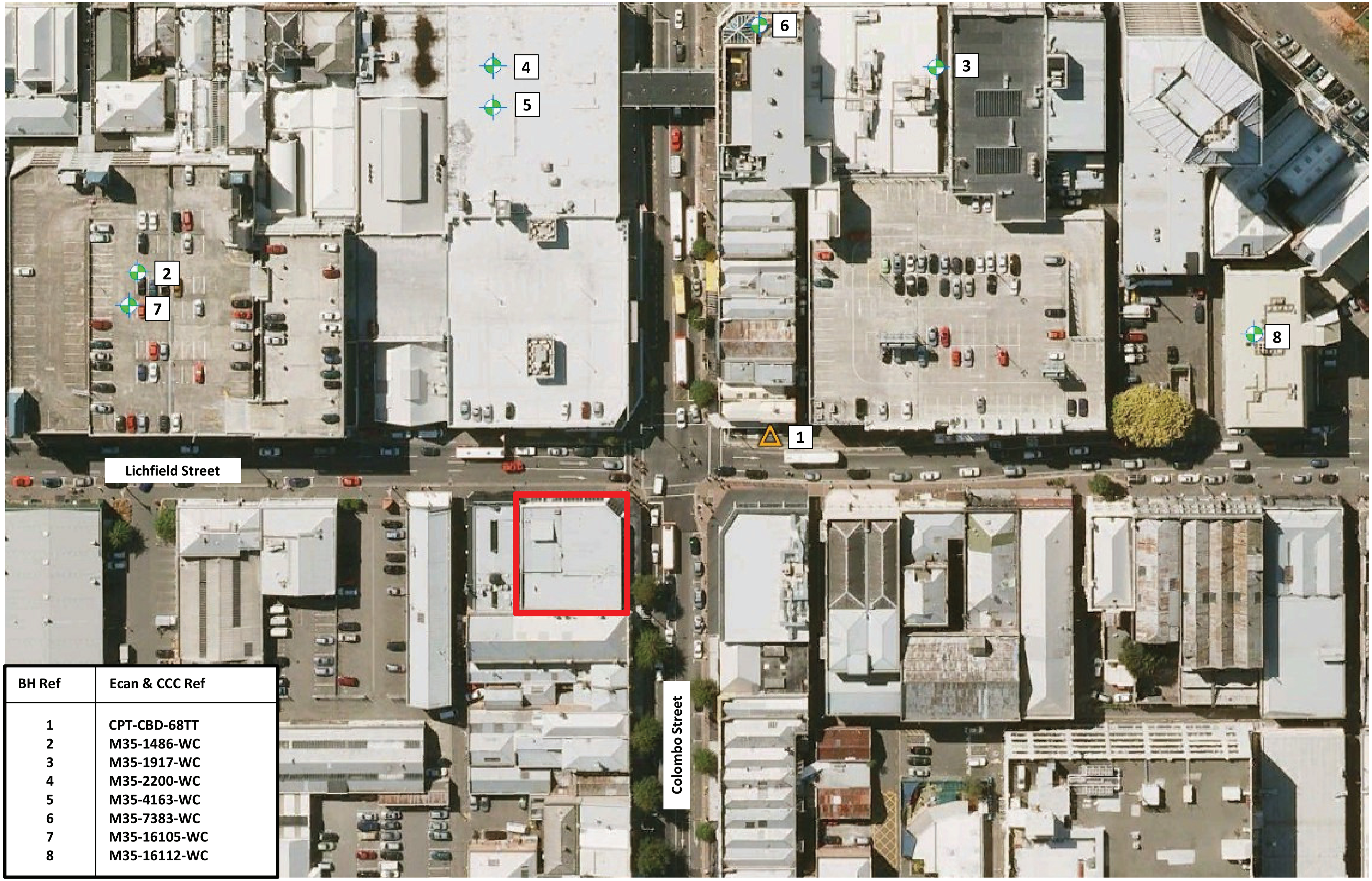


OPUS
Opus International Consultants Ltd
Christchurch Office
20 Moorhouse Ave
PO Box 1482
Christchurch, New Zealand
Tel: +64 3 363 5400 Fax: +64 3 365 7857

Project:	645 - 647 Colombo Street Geotechnical Desktop Study
Project No.:	6-QUCCC.54
Client:	Christchurch City Council

Figure 1 - Walkover Inspection Plan

Completed by:	Shane Greene - 24 January 2012 Engineering Geologist
Date Drawn:	24/01/2012



BH Ref	Ecan & CCC Ref
1	CPT-CBD-68TT
2	M35-1486-WC
3	M35-1917-WC
4	M35-2200-WC
5	M35-4163-WC
6	M35-7383-WC
7	M35-16105-WC
8	M35-16112-WC

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Project: 645 - 647 Colombo Street
Geotechnical Desktop Study

Project No.: 6-QUCCC.54

Client: Christchurch City Council

Figure 2 - Site Location Plan

Completed by: Shane Greene - 24 January 2012
Engineering Geologist

Date Drawn: 24/01/2012

Photograph 1 – View East along Lichfield Street from east Corner with Colombo (24 January 2012).

Photograph 2 - View south down Colombo from intersection with Lichfield; pavement cracking (24 January 2012).

Photograph 3 - View south down Colombo from outside Penny Lane; pavement repair (24 January 2012).

Photograph 4 – Minor sand ejection around service duct; north side of R and R building (24 January 2012).

Photograph 5 – Minor settlement in footpath flagstones on the North side of Lichfield Street across from Rhotis restaurant (24 January 2012).

Photograph 6 – Minor heave in footpath flagstones on the west side of Colombo Street (24 January 2012).

Photograph 7 – **Piling of sand ejected by minor liquefaction on the east side of Colombo Street across from R&R (24 January 2012).**

Photograph 8 – **General view looking north along Colombo Street from ~ 10m south of "Penny Lane" (24 January 2012).**

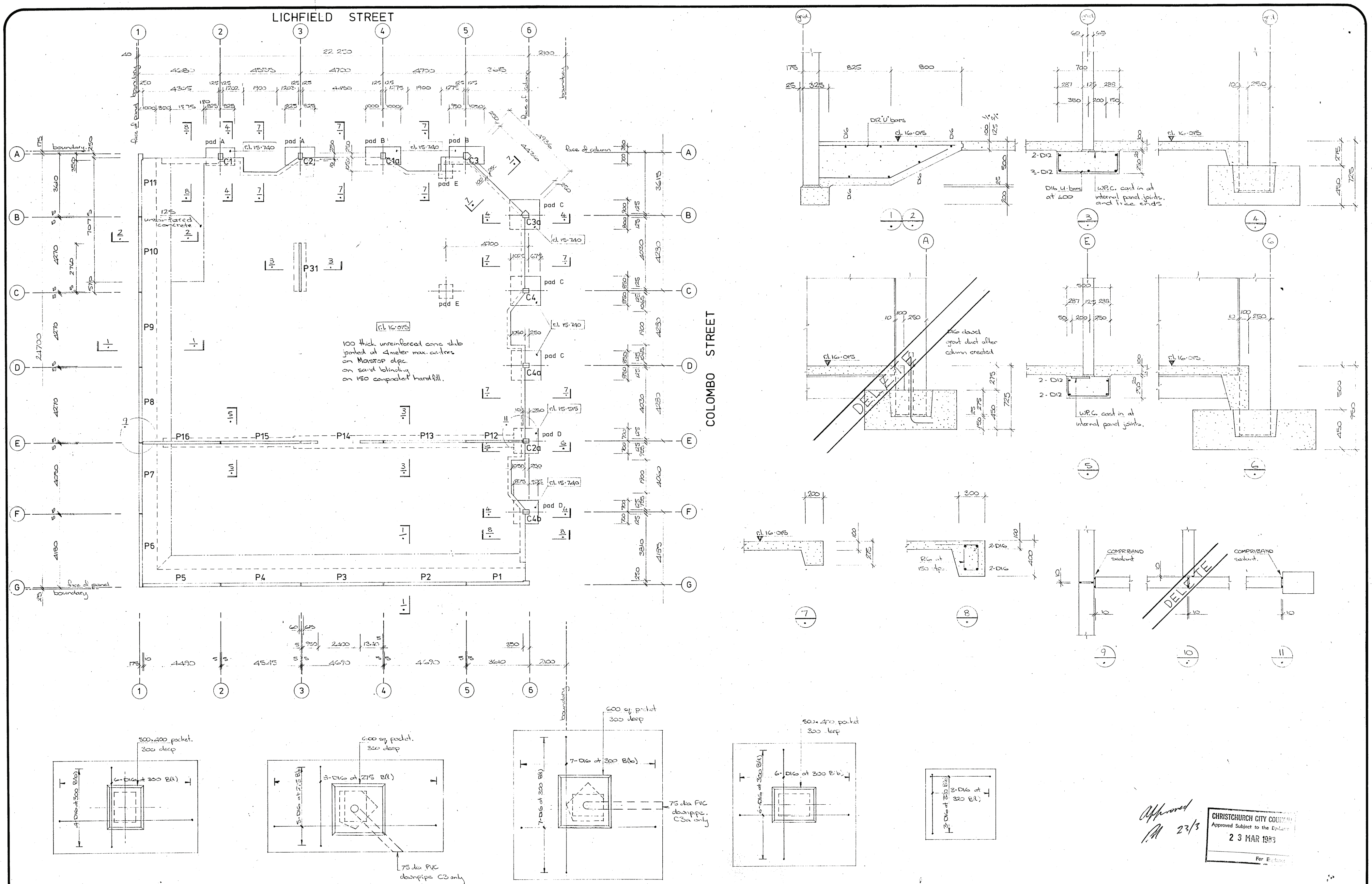
Photograph 9 – General view looking east toward Colombo Street from the south corner of the Rhotis buliding (24 January 2012).

Photograph 10 – General view looking north along the west side of the Rohits building toward Lichfield Street (24 January 2012).

Photograph 11 – General view looking east along Lichfield Street from the west corner of the Rhotis building (24 January 2012).

Appendix B:

Structural Drawings



pad A : 2 off
1650 × 1050 × 450 deep

pad B : 2 off
2000 × 1050 × 450 deep

pad C : 3 off
1700 × 1700 × 450 deep

pad D : 2 off
1400 × 1400 × 450 deep

pad E : 2 off
800 × 800 × 200 deep

ARCH GOODWIN DESIGN LTD. CHRISTCHURCH

Holmes Wood Poole & Johnstone Ltd
Consulting Civil & Structural Engineers
Christchurch Wellington & Auckland

RETAIL DEVELOPMENT
CNR COLOMBO & LICHFIELD STREETS, CHRISTCHURCH

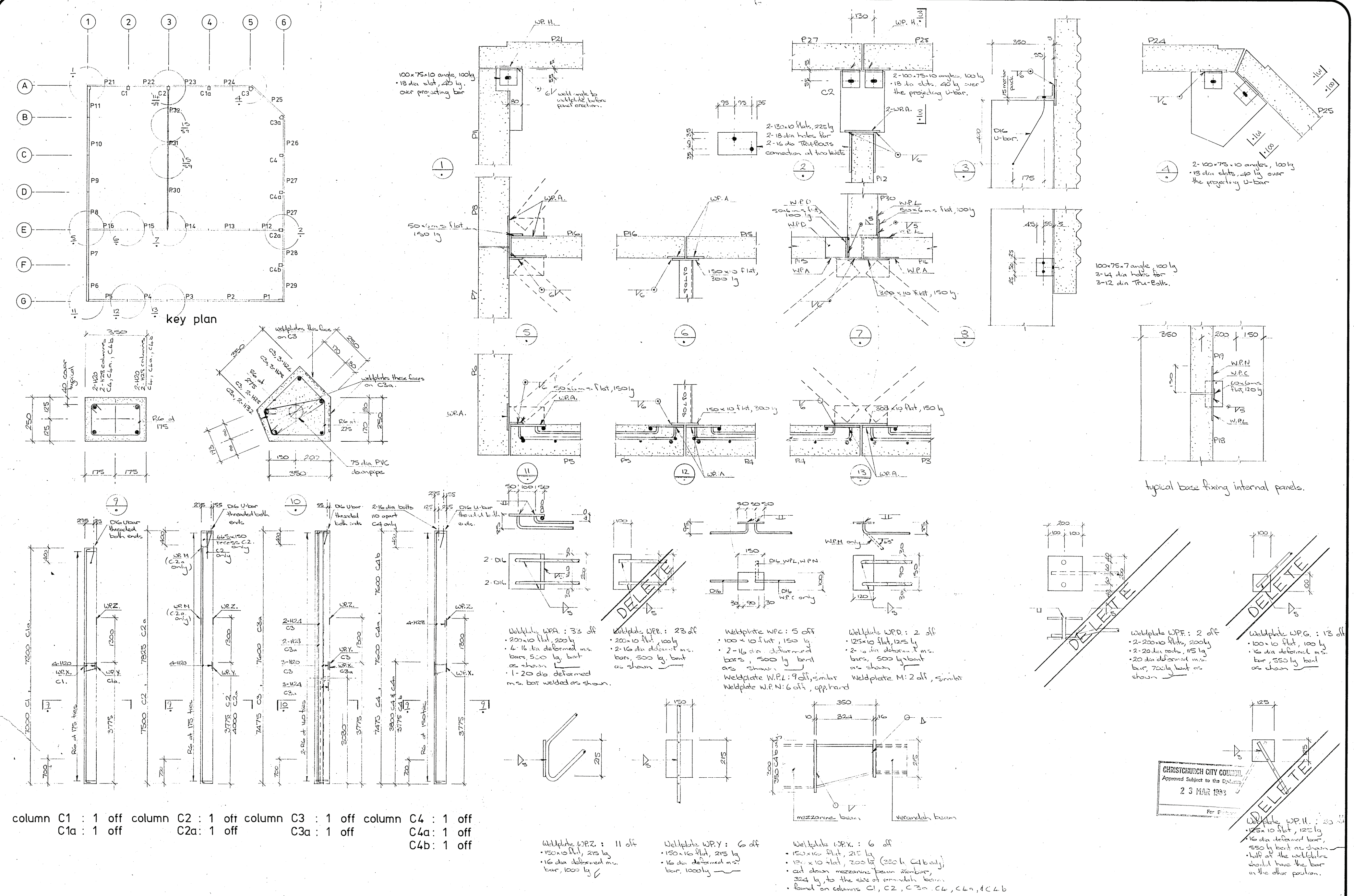
FLOOR PLAN
FOUNDATION DETAILS

Scale	1 : 100 1 : 50
Drawn	T. J. H.
Traced	
Approved	R.a.P.

D	
C	
B	15. 3. 83 Internal walls & column bases altered.
A	26. 1. 83 CONTRACT
	23. 12. 82 PERMIT

2048/S1 B

The Contractor shall verify all dimensions prior to commencing work



ARCH GOODWIN DESIGN LTD. CHRISTCHURCH	**Holmes Wood Poole & Johnstone Ltd** Consulting Civil & Structural Engineers Christchurch Wellington & Auckland	RETAIL DEVELOPMENT CNR COLOMBO & LICHFIELD STREETS, CHRISTCHURCH	PRECAST CONCRETE COLUMN & PANEL LAYOUT	2048/S2B

Scale	1 : 200 / 1 : 50
Drawn	T. J. H.
Traced	
Approved	R.a.P.

D	
C	
B	15. 3. 83 Weldplate details altered
A	26. 1. 83 CONTRACT
	23. 12. 82. PERMIT

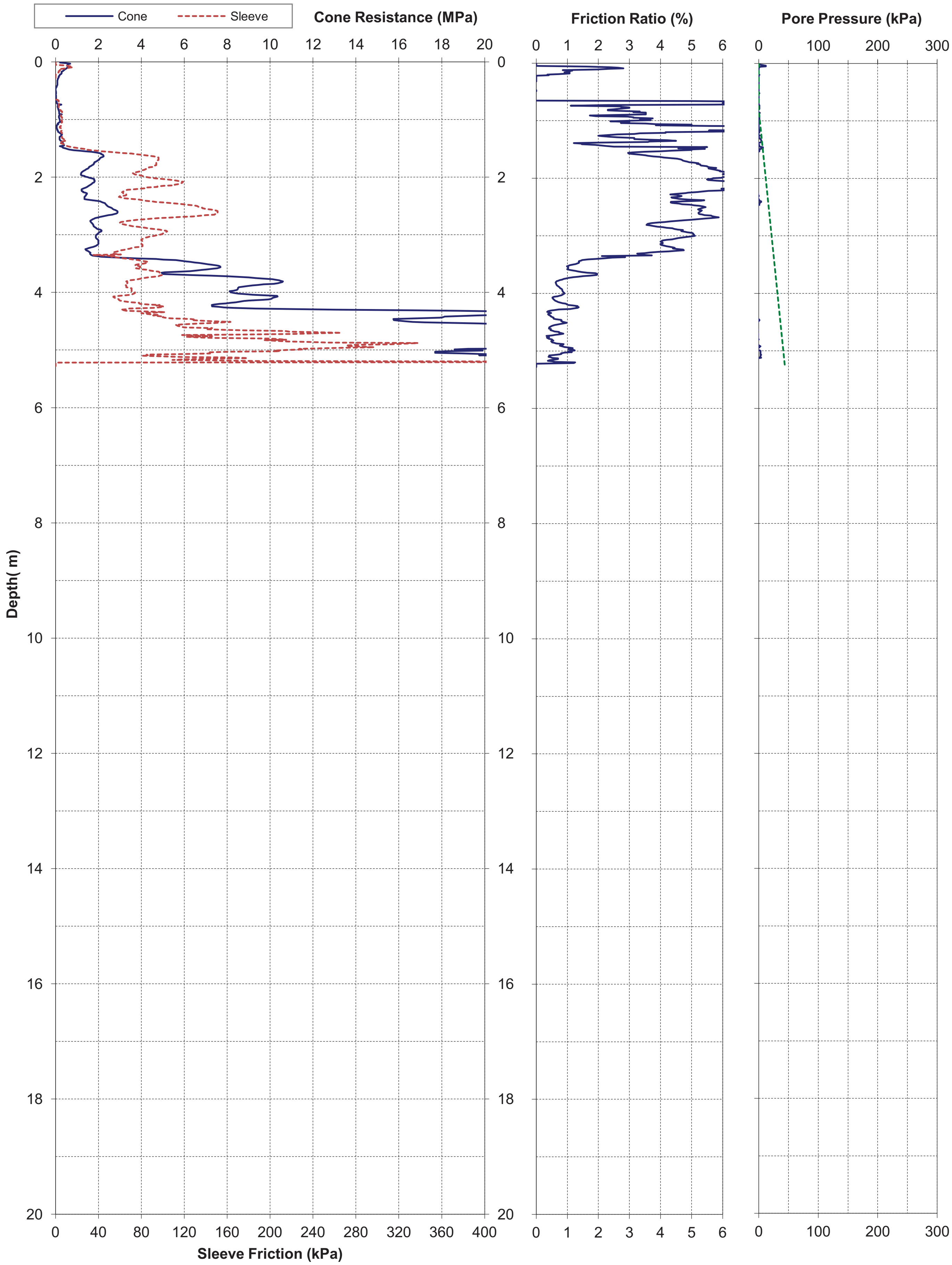
The Contractor shall verify all dimensions prior to commencing work

Appendix C:

Environment Canterbury Well and CCC CPT Logs

Project:	Christchurch 2011 Earthquake - CERA Ground Investigations			Page:	1 of 1	CPT-CBD-68
Test Date:	14-Sep-2011	Location:	Central City	Operator:	Perry	
Pre-Drill:	1.5m	Assumed GWL:	0.8mBGL	Located By:	Survey GPS	
Position:	2480662.8mE	5741411.3mN	6.66mRL	Coord. System:	NZMG & MSL	
Other Tests:				Comments:		

CERA Canterbury Earthquake Recovery Authority



Bore or Well No: M35/1486

Well Name:

Owner: LICHFIELD CAR PARK

Environment Canterbury
Your regional council

Street of Well:	LICHFIELD ST	**File No:**	
Locality:	CITY	**Allocation Zone:**	Christchurch/West Melton
NZGM Grid Reference:	M35:8052-4145 QAR 4		
NZGM X-Y:	2480520 - 5741450		
Location Description:	Bore no 3	**Uses:**	Foundation/Investigation Bore
ECan Monitoring:			
Well Status:	Casing Retrieved / Abandoned		

Drill Date:		**Water Level Count:**	0
Well Depth:	13.70m -GL	**Strata Layers:**	7
Initial Water Depth:		**Aquifer Tests:**	0
Diameter:		**Isotope Data:**	0
		Yield/Drawdown Tests:	0
Measuring Point Alt:	7.96m MSD QAR 4	**Highest GW Level:**	
GL Around Well:	0.00m -MP	**Lowest GW Level:**	
MP Description:		**First Reading:**	
		Last Reading:	
Driller:	not known	**Calc. Min. GWL:**	-0.30m -MP
Drilling Method:	Unknown	**Last Updated:**	18 Oct 2006
Casing Material:		**Last Field Check:**	
Pump Type:	None Installed		
Yield:		**Screens:**	
Drawdown:		**Screen Type:**	No Screen
Specific Capacity:		**Top GL:**	
		Bottom GL:	
Aquifer Type:	Unknown		
Aquifer Name:	Springston Formation		

Borelog for well M35/1486

Gridref: M35:8052-4145 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 7.96 +MSD
Driller : not known
Drill Method : Unknown
Drill Depth : -13.7m Drill Date :

Water level: -0.3CalcMin

Depth(m)	Full Drillers Description	Formation Code
-1.80m	Top filling coal & ashes etc	fi
-3.00m	Brown sand clay	sp?
-4.00m	Sand	sp?
-9.50m	Shingle	sp?
- 10.0m	Shingle with some sand	sp?
- 11.9m	Sand	sp?
- 13.7m	Blue shingle	sp?

Scale(m): -5, -10

Bore or Well No:	M35/1917		
Well Name:			
Owner:	BEATHS		

Environment Canterbury
Your regional council

Street of Well:	CNR CASHEL & COLOMBO STS	**File No:**	
Locality:	CHRISTCHURCH	**Allocation Zone:**	Christchurch/West Melton
NZGM Grid Reference:	M35:807-415 QAR 4		
NZGM X-Y:	2480700 - 5741500		
Location Description:	MIDDLE OF RIGHT OF WAY FROM LICHFIELD ST	**Uses:**	
ECan Monitoring:			
Well Status:	Not Used		

Drill Date:		**Water Level Count:**	0
Well Depth:	126.70m -GL	**Strata Layers:**	22
Initial Water Depth:	9.14m -MP	**Aquifer Tests:**	0
Diameter:		**Isotope Data:**	0
		Yield/Drawdown Tests:	0
Measuring Point Alt:	6.60m MSD QAR 3	**Highest GW Level:**	
GL Around Well:	0.00m -MP	**Lowest GW Level:**	
MP Description:		**First Reading:**	
		Last Reading:	
Driller:	not known	**Calc. Min. GWL:**	2.80m -MP
Drilling Method:	Unknown	**Last Updated:**	18 Oct 2006
Casing Material:		**Last Field Check:**	
Pump Type:	Unknown		
Yield:		**Screens:**	
Drawdown:		**Screen Type:**	
Specific Capacity:		**Top GL:**	
		Bottom GL:	
Aquifer Type:	Flowing Artesian		
Aquifer Name:	Wainoni Gravel		

Borelog for well M35/1917 page 1 of 2

Gridref: M35:807-415 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 6.6 +MSD
Driller : not known
Drill Method : Unknown
Drill Depth : -126.7m Drill Date :

Environment Canterbury Regional Council

Water Level: Artesian

Depth(m)	Full Drillers Description	Formation Code
-4.50m	Sandy clay	sp?
-8.19m	Shingle	sp?
- 18.2m	Sand	ch
- 21.9m	Sandy clay	ch
- 22.8m	Peat	ch
- 36.5m	Shingle	ri
- 39.6m	Sandy clay	br
- 43.2m	Shingle	br
- 51.8m	Sand	br
- 53.0m	Sand and peat	br
- 56.3m	Sandy clay	br
- 57.7m	Shingle	li-1
- 60.9m	Sandy clay	li-2
- 63.3m	Shingle	li-2

Borelog for well M35/1917 page 2 of 2

Gridref: M35:807-415 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 6.6 +MSD
Driller : not known
Drill Method : Unknown
Drill Depth : -126.7m Drill Date :

Water Level	Depth(m)	Full Drillers Description	Formation Code
Artesian		Shingle	
	- 73.1m		li-2
		Sandy clay	
	- 76.2m		li-2
		Shingle	
	- 94.4m		li-he
		Sandy clay	
	- 97.5m		he
		Shingle	
	- 105.1m		bu
		Shingle and wood	
	- 106.6m		bu
		Sandy clay	
	- 114.3m		sh
		Clay	
	- 115.0m		sh
		Shingle	
	- 126.7m		sh-wa

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

Bore or Well No:	M35/2200
Well Name:	
Owner:	BALLANTYNE, J.& CO. LTD.

Environment Canterbury
Your regional council

Street of Well:	CNR CASHEL & COLOMBO STS	**File No:**	
Locality:	CHRISTCHURCH	**Allocation Zone:**	Christchurch/West Melton
NZGM Grid Reference:	M35:806-415 QAR 4		
NZGM X-Y:	2480600 - 5741500		
Location Description:		**Uses:**	Foundation/Investigation Bore
ECan Monitoring:			
Well Status:	Casing Retrieved / Abandoned		

Drill Date:		**Water Level Count:**	0
Well Depth:	12.10m -GL	**Strata Layers:**	5
Initial Water Depth:		**Aquifer Tests:**	0
Diameter:		**Isotope Data:**	0
		Yield/Drawdown Tests:	0
Measuring Point Ait:	6.70m MSD QAR 3	**Highest GW Level:**	
GL Around Well:	0.00m -MP	**Lowest GW Level:**	
MP Description:		**First Reading:**	
		Last Reading:	
Driller:	not known	**Calc. Min. GWL:**	-0.20m -MP
Drilling Method:	Unknown	**Last Updated:**	18 Oct 2006
Casing Material:		**Last Field Check:**	
Pump Type:	None Installed		
Yield:		**Screens:**	
Drawdown:		**Screen Type:**	No Screen
Specific Capacity:		**Top GL:**	
		Bottom GL:	
Aquifer Type:	Unknown		
Aquifer Name:			

Borelog for well M35/2200

Gridref: M35:806-415 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 6.7 +MSD
Driller : not known
Drill Method : Unknown
Drill Depth : -15m Drill Date :

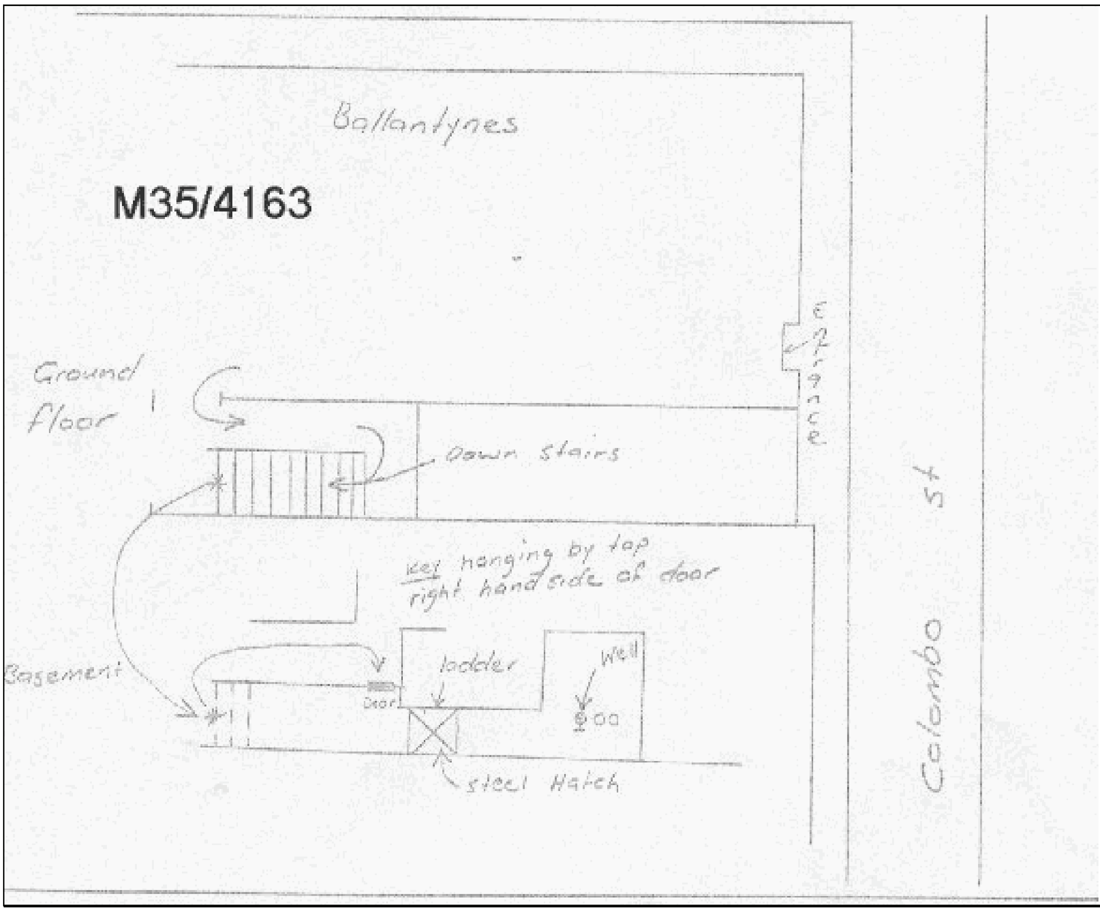
Scale(m)	Water Level	Depth(m)	Full Drillers Description	Formation Code
	-0.2CalcMin	-0.30m	As for bore no.1	sp?
			Sand	
		-3.59m	Gravel	sp?
-5		-4.19m	Gravel	sp?
-10		- 12.1m	Fine sand	sp?
		- 15.0m		ch

Bore or Well No:	M35/4163
Well Name:	BALLANTYNES
Owner:	BALLANTYNES COMPANY LTD

Street of Well:	COLOMBO ST	**File No:**	
Locality:	CHRISTCHURCH	**Allocation Zone:**	Christchurch/West Melton
NZGM Grid Reference:	M35:8060-4149 QAR 3		
NZGM X-Y:	2480600 - 5741490		
Location Description:	IN BASEMENT	**Uses:**	Water Level Observation
ECan Monitoring:	Monthly Manual		
Well Status:	Active (exist, present)		

Drill Date:	09 Mar 1960	**Water Level Count:**	481
Well Depth:	65.00m -GL	**Strata Layers:**	18
Initial Water Depth:	5.97m -MP	**Aquifer Tests:**	0
Diameter:	100mm	**Isotope Data:**	0
		Yield/Drawdown Tests:	0
Measuring Point Ait:	2.22m MSD QAR 1	**Highest GW Level:**	7.51m from MP
GL Around Well:	4.17m -MP	**Lowest GW Level:**	5.67m from MP
MP Description:	Pressure gauge nut	**First Reading:**	07 May 1984
		Last Reading:	14 Feb 2011
Driller:	Job Osborne (& Co/Ltd)	**Calc. Min. GWL:**	5.94m -MP
Drilling Method:	Cable Tool	**Last Updated:**	21 Sep 2006
Casing Material:		**Last Field Check:**	14 Feb 2011
Pump Type:	Unknown		
Yield:	0 l/s	**Screens:**	
Drawdown:	0 m	**Screen Type:**	
Specific Capacity:		**Top GL:**	
		Bottom GL:	
Aquifer Type:	Flowing Artesian		
Aquifer Name:	Linwood Gravel		

Date	Comments
	WELL ORIGINALLY USED IN A HEAT EXCHANGE SYSTEM FOR AN AIR CONDITIONING PLANT.ALSO M35/2280,4164,4165
12 Jun 2001	MP lowered with 17cm, Old water level data referenced to new MP



Ballantynes
M35/4163
Entrance
Ground floor
down stairs
key hanging by top right hand side of door
Basement
ladder
Well
Door
steel Hatch
Colombo St

Borelog for well M35/4163

Gridref: M35:8060-4149 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 6.39 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Cable Tool
Drill Depth : -68.3m Drill Date : 9/03/1960

Water Level	Depth(m)	Full Drillers Description	Formation Code
Artesian	-0.89m	Filling	fi
	-5.80m	Blue clay	sp?
	- 11.9m	Blue gravel	sp?
	- 21.3m	Blue sand	ch
	- 23.8m	Blue clay	ch
	- 36.0m	Brown gravel	ri
	- 39.9m	Blue clay	br
	- 40.9m	Hard Blue sand	br
	- 44.2m	Brown gravel	br
	- 53.4m	Brown sand	br
	- 54.0m	Blue clay	br
	- 56.4m	Hard sand with layers of clay	br
	- 58.8m	Brown gravel	li-1
	- 59.1m	Brown sand	li-2
	- 59.5m	Brown gravel	li-2
	- 60.0m	Brown sand	
	- 61.3m	Blue clay	
	- 68.3m	Brown gravel	li-2

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

Bore or Well No:	M35/7383
Well Name:	
Owner:	A1 HOTEL

Environment Canterbury
Your regional council

Street of Well:	CNR CASHEL/COLOMBO STS	**File No:**	
Locality:	CHRISTCHURCH	**Allocation Zone:**	Christchurch/West Melton
NZGM Grid Reference:	M35:8066-4151 QAR 4		
NZGM X-Y:	2480660 - 5741510		
Location Description:		**Uses:**	
ECan Monitoring:			
Well Status:	Not Used		

Drill Date:	28 May 1900	**Water Level Count:**	0
Well Depth:	127.40m -GL	**Strata Layers:**	25
Initial Water Depth:	9.10m -MP	**Aquifer Tests:**	0
Diameter:	76mm	**Isotope Data:**	0
		Yield/Drawdown Tests:	0
Measuring Point Ait:	6.50m MSD QAR 3	**Highest GW Level:**	
GL Around Well:	0.00m -MP	**Lowest GW Level:**	
MP Description:		**First Reading:**	
		Last Reading:	
Driller:	Job Osborne (& Co/Ltd)	**Calc. Min. GWL:**	2.80m -MP
Drilling Method:	Hydraulic/Percussion	**Last Updated:**	21 Sep 2006
Casing Material:	STEEL	**Last Field Check:**	
Pump Type:	Unknown		
Yield:	0 l/s	**Screens:**	
Drawdown:	0 m	**Screen Type:**	
Specific Capacity:		**Top GL:**	
		Bottom GL:	
Aquifer Type:	Flowing Artesian		
Aquifer Name:	Wainoni Gravel		

Date	Comments
	HOTEL DEMOLISHED & REPLACED BY PRESENT BEATHS BUILDING.ALSO M35/7382

Borelog for well M35/7383

Gridref: M35:8066-4151 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 6.5 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -127.4m Drill Date : 28/05/1900

Water Level	Depth(m)	Full Drillers Description	Formation Code
Artesian	-4.59m	Sand & clay	sp?
	-9.10m	Blue shingle	sp?
	- 18.3m	Blue sand	ch
	- 21.9m	Blue sand & clay	ch
	- 23.5m	Clay & peat	ch
	- 37.8m	Brown shingle,water 0.3m below	ri
	- 39.9m	Blue sand & clay	br
	- 44.2m	Brown shingle	br
	- 51.8m	Yellow sand	br
	- 53.3m	Blue sand	br
	- 56.1m	Blue sand & clay	br
	- 56.7m	Yellow sand & clay	br
	- 60.4m	Brown shingle,water 0.9m above	li
	- 61.3m	Yellow sand & clay	li
	- 73.8m	Brown shingle	li

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

Borelog for well M35/7383 page 2 of 2

Gridref: M35:8066-4151 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 6.5 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -127.4m Drill Date : 28/05/1900

Environment Canterbury
Your regional council

Scale(m)	Water Level	Depth(m)	Full Drillers Description	Formation Code
-70	Artesian	- 73.8m	Brown shingle	li
		- 75.0m	Yellow sand & clay	li
		- 76.2m	Blue sand & clay	li
		- 76.8m	Yellow clay	li
-80, -90		- 95.4m	Brown shingle.water 2.1m above ,good flow @ 80.8m	li-he
		- 98.5m	Yellow sand & clay	he
-100		- 104.5m	Brown shingle,water rises 3.65m,good flow @ 100.6m	bu
		- 107.0m	Brown sand & gravel	bu
-110		- 111.3m	Blue sand & clay	sh
		- 117.0m	Yellow sand & clay	sh
-120		- 127.4m	Brown shingle	wa

Unknown No:	M35/16105
Well Name:	CCC BorelogID 5488
Owner:	CCC borelog

Environment Canterbury
Your regional council

Street of Well:		**File No:**	
Locality:		**Allocation Zone:**	Christchurch/West Melton
NZGM Grid Reference:	M35:80518-41442 QAR 3		
NZGM X-Y:	2480518 - 5741442		
Location Description:		**Uses:**	Foundation/Investigation Bore
ECan Monitoring:			
Well Status:	Filled in		

Drill Date:	01 Jan 1965	**Water Level Count:**	0
Well Depth:	6.40m -GL	**Strata Layers:**	4
Initial Water Depth:		**Aquifer Tests:**	0
Diameter:		**Isotope Data:**	0
		Yield/Drawdown Tests:	0
Measuring Point Ait:	7.96m MSD QAR 4	**Highest GW Level:**	
GL Around Well:	0.00m -MP	**Lowest GW Level:**	
MP Description:		**First Reading:**	
		Last Reading:	
Driller:		**Calc. Min. GWL:**	
Drilling Method:		**Last Updated:**	27 Mar 2008
Casing Material:		**Last Field Check:**	
Pump Type:			
Yield:		**Screens:**	
Drawdown:		**Screen Type:**	
Specific Capacity:		**Top GL:**	
		Bottom GL:	
Aquifer Type:			
Aquifer Name:			

Borelog for well M35/16105

Gridref: M35:80518-41442 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 7.96 +MSD
Well name : CCC BorelogID 5488
Drill Method : Not Recorded
Drill Depth : -6.4m Drill Date : 1/01/1965

Scale(m)	Water Level	Depth(m)	Full Drillers Description	Formation Code
0 to -3		-3.00m	fill	
-3 to -3.7		-3.70m	blue sand	
-3.7 to -4.6		-4.60m	gravel	
-4.6 to -6.4		-6.40m	blue sand and gravel	

Unknown No:	M35/16112
Well Name:	CCC BorelogID 5496
Owner:	CCC borelog

Environment Canterbury
Your regional council

Street of Well:		**File No:**	
Locality:		**Allocation Zone:**	Christchurch/West Melton
NZGM Grid Reference:	M35:80772-41436 QAR 3		
NZGM X-Y:	2480772 - 5741436		
Location Description:		**Uses:**	Foundation/Investigation Bore
ECan Monitoring:			
Well Status:	Filled in		

Drill Date:	01 Jan 1968	**Water Level Count:**	0
Well Depth:	12.20m -GL	**Strata Layers:**	4
Initial Water Depth:	-3.00m -MP	**Aquifer Tests:**	0
Diameter:		**Isotope Data:**	0
		Yield/Drawdown Tests:	0
Measuring Point Ait:	7.81m MSD QAR 4	**Highest GW Level:**	
GL Around Well:	0.00m -MP	**Lowest GW Level:**	
MP Description:		**First Reading:**	
		Last Reading:	
Driller:		**Calc. Min. GWL:**	
Drilling Method:		**Last Updated:**	27 Mar 2008
Casing Material:		**Last Field Check:**	
Pump Type:			
Yield:		**Screens:**	
Drawdown:		**Screen Type:**	
Specific Capacity:		**Top GL:**	
		Bottom GL:	
Aquifer Type:			
Aquifer Name:			

Borelog for well M35/16112

Gridref: M35:80772-41436 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 7.81 +MSD
Well name : CCC BorelogID 5496
Drill Method : Not Recorded
Drill Depth : -12.2m Drill Date : 1/01/1968

Scale(m)	Water Level	Depth(m)	Full Drillers Description	Formation Code
		-0.50m	fill and bricks	
		-4.00m	clay	
		-9.40m	blue gravel and sand	
		- 12.2m	blue sand	

Scale marks: -5, -10

Appendix 5 – CERA DEE Spreadsheet

Detailed Engineering Evaluation Summary Data

V1.11

Location

Building Name: R & R Sport
Unit | No: | Street
Building Address: 645-647 Colombo Street
Legal Description:
Degrees | Min | Sec
GPS south:
GPS east:
Building Unique Identifier (CCC): BU 2677 005 EQ2

Reviewer: Alistair Boyce
CPEng No: 209860
Company: Opus International Consultants
Company project number: 6-QUCCC.54
Company phone number: 03 363 5400
Date of submission: 13-Sep-12
Inspection Date: 21-Dec-11
Revision: Final
Is there a full report with this summary? yes

Site

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

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

Building

No. of storeys above ground: 2 — single storey = 1
Ground floor split? no
Storeys below ground 0
Foundation type: strip footings
Building height (m): 7.50
Floor footprint area (approx): 575
Age of Building (years): 29

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): 7.5
Date of design: 1976-1992

Strengthening present? no
Use (ground floor): retail
Use (upper floors): retail
Use notes (if required):
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: steel framed — rafter type, purlin type and cladding
Floors: other (note) — describe sytem Timber framed mezzanine floor
Beams:
Columns: precast concrete — typical dimensions (mm x mm)
Walls: load bearing concrete — #N/A

Lateral load resisting structure

Lateral system along: multi-level tilt panel
Ductility assumed, μ: 1.25
Period along: 0.40 ##### enter height above at H31
Total deflection (ULS) (mm):
maximum interstorey deflection (ULS) (mm):

Note: Define along and across in detailed report!

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

Lateral system across: multi-level tilt panel
Ductility assumed, μ: 1.25
Period across: 0.27 ##### enter height above at H31
Total deflection (ULS) (mm):
maximum interstorey deflection (ULS) (mm):

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

Separations:

north (mm): — leave blank if not relevant
east (mm):
south (mm):
west (mm): 50

Non-structural elements

Stairs:
Wall cladding: precast panels — thickness and fixing type
Roof Cladding: Metal — describe
Glazing: aluminium frames
Ceilings:
Services(list):

Available documentation

Architectural — original designer name/date
Structural full — original designer name/date Holmes Wood Poole & Johnstone, Dec. 1982
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: — Describe damage:
Settlement: 0-25mm — 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):

Across — Damage ratio: #DIV/0!
Describe (summary):

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

Diaphragms — Damage?: yes — Describe: Roof diaphragm failure
CSWs: — Damage?: yes — Describe: Lack of ductility and load paths
Pounding: — Damage?: no — Describe:
Non-structural: — Damage?: yes — Describe:

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: ##### %NBS from IEP below
Assessed %NBS after: 12%

If IEP not used, please detail assessment methodology: Quantitative seismic assessment

Across — Assessed %NBS before: ##### %NBS from IEP below
Assessed %NBS after: 12%

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): 1976-1992
h_n from above: 7.5m

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	0.27
(%NBS)nom from Fig 3.3:		

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

	along	across
Final $(\%NBS)_{nom}$:	**0%**	**0%**

2.2 Near Fault Scaling Factor

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

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

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:
Z_{1992}, from NZS4203:1992
Hazard scaling factor, **Factor B:** #DIV/0!

2.4 Return Period Scaling Factor

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

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)	1.00	1.00
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	1.00	1.00

2.6 Structural Performance Scaling Factor:

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

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

	along	across
$\%NBS_b$:	**#DIV/0!**	**#DIV/0!**

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

3.1. Plan Irregularity, factor A: 1
3.2. Vertical irregularity, Factor B: 1
3.3. Short columns, Factor C: 1
3.4. Pounding potential — Pounding effect D1, from Table to right 1.0
Height Difference effect D2, from Table to right 1.0
Therefore, Factor D: 1
3.5. Site Characteristics 1

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

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

3.6. Other factors, Factor F

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

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

OPUS