Rohit's Indian Restaurant

BU 2677-006 EQ2

Detailed Engineering Evaluation

Quantitative Assessment Report

Christchurch City Council

Rohit's Indian Restaurant Detailed Engineering Evaluation Quantitative Assessment Report

56 – 58 Lichfield Street, Christchurch
Christchurch City Council

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Rohit's Indian Restaurant Building
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Detailed Engineering Evaluation
Quantitative Report - SUMMARY
Final

56 – 58 Lichfield Street, Christchurch

Background

This is a summary of the quantitative report for the building structure at 56-58 Lichfield Street, Christchurch known as Rohit's Indian Restaurant 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 west elevation at roof level
- Superficial damage to the western wall from adjacent building collapse
- 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 foundation have limited capacity and little or no ductility. If these connections fail, a brittle failure mechanism is expected and partial collapse of the building is likely.
b) The precast panel connections to the steel roof portal frames have limited capacity and little or no ductility.
c) The building does not include any collectors to transfer lateral forces into the shear walls along the east and west side of the building. Transfer of these loads relies on the first floor topping slab transfer forces into precast walls.
d) As a result of the presence of precast walls on three sides and internal walls near the southern end of the building, the building's response to lateral loads is torsional in the east-west direction.

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 10% 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, with a width of 12m, 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 Rohit's Indian Restaurant, located at 56-58 Lichfield St, 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. Three relevant sections are:

Section 29 – Information

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

Section 38 – Works

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

Section 51 – Requiring Structural Survey

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

We understand that CERA require a detailed engineering evaluation to be carried out for all buildings (other than those exempt from the Earthquake Prone Building definition in the Building Act). CERA have adopted the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011.

This document sets out a methodology for both initial qualitative and detailed quantitative assessments.

It is anticipated that a number of factors, including the following, will determine the extent of evaluation and strengthening level required:

1. The importance level and occupancy of the building.
2. The placard status and amount of damage.
3. The age and structural type of the building.
4. Consideration of any critical structural weaknesses.

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

2.2 Building Act

Several sections of the Building Act are relevant when considering structural requirements:

Section 112 - Alterations

This section requires that an existing building complies with the relevant sections of the Building Code to at least the extent that it did prior to the alteration.

This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

Section 115 – Change of Use

This section requires that the territorial authority (in this case Christchurch City Council (CCC)) is satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'.

This is typically interpreted by CCC as being 67% of the strength of an equivalent new building. This is also the minimum level recommended by the New Zealand Society for Earthquake Engineering (NZSEE).

Section 121 – Dangerous Buildings

This section was extended by the Canterbury Earthquake (Building Act) Order 2010, and defines a building as dangerous if:

1. In the ordinary course of events (excluding the occurrence of an earthquake), the building is likely to cause injury or death or damage to other property; or
2. In the event of fire, injury or death to any persons in the building or on other property is likely because of fire hazard or the occupancy of the building; or

3. There is a risk that the building could collapse or otherwise cause injury or death as a result of earthquake shaking that is less than a 'moderate earthquake' (refer to Section 122 below); or
4. There is a risk that other property could collapse or otherwise cause injury or death; or
5. A territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

Section 122 – Earthquake Prone Buildings

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

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

Section 124 – Powers of Territorial Authorities

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

Section 131 – Earthquake Prone Building Policy

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

2.3 Christchurch City Council Policy

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

The 2010 amendment includes the following:

1. A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;

2. A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;

3. A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,

4. Repair works for buildings damaged by earthquakes will be required to comply with the above.

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

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

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

2.4 Building Code

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

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

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

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

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

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

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

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

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

3 Earthquake Resistance Standards

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

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

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[i] in Council 16 September 2010, modified the meaning of "dangerous building" to include buildings that were identified as being

EPB's. As a result of this, we would expect such a building would be issued with a Section 124 notice, by the Territorial Authority, or CERA acting on their behalf, once they are made aware of our assessment. Based on information received from CERA to date, this notice is likely to prohibit occupancy of the building (or parts thereof), until its seismic capacity is improved to the point that it is no longer considered an EPB.

3.1.2 Cordoning

– Where there is an overhead falling hazard, or potential collapse hazard of the building, the areas of concern should be cordoned off in accordance with current CERA/Christchurch City Council guidelines.

3.1.3 Strengthening

– Industry guidelines (NZSEE 2006 [2]) strongly recommend that every effort be made to achieve improvement to at least 67%NBS. A strengthening solution to anything less than 67%NBS would not provide an adequate reduction to the level of risk.

– It should be noted that full compliance with the current building code requires building strength of 100%NBS.

3.1.4 Our Ethical Obligation

– In accordance with the IPENZ code of ethics, we have a duty of care to the public. This obligation requires us to identify and inform CERA of potentially dangerous buildings; this would include earthquake prone buildings.

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

4 Background Information

4.1 Building Description

58 Lichfield St is a single building constructed in 1989. The building is a two storey structure constructed of pre-cast concrete walls and a lightweight steel roof supported off steel frames. The first floor is accessed via precast concrete stairs supported on an in situ wall. The building is founded on a pre-existing perimeter basement wall and concrete pads under ends of interior precast walls. A 100mm thick reinforced concrete slab overlays the in-filled pre-existing basement.

The ground floor of the building is divided by the stairway and an internal precast wall into two tenancies. Alibaba's Restaurant occupies the eastern side of the ground floor while the western side appears to have been vacant at the time of the earthquake. Rohit's Indian Restaurant occupies the first floor.

The building is rectangular in shape with a 12m wide street frontage onto Lichfield Street. It is 25m long in the north-south direction. The shallow pitch roof slopes from a central ridge toward steeper sections on the east and west of the building.

The original drawings are dated March 1989 and are stamped by CCC on 5 July 1989. The building appears consistent with this drawing set and no significant alterations appear to have been made.

4.2 Gravity Load Resisting Systems

At roof level, a lightweight Colorsteel roof is supported by steel purlins (spanning in north-south direction) which in turn is supported by transverse steel frames made up of 200UB steel sections that span the full width of the building. The steel sections are fixed to the top of the western longitudinal precast wall panels and face of the eastern longitudinal precast wall panels using weld plates. A Stahlton floor system at first floor level spans east west, slotting into pre-cast pockets in the exterior wall panels and seated on the one story central precast panels. The floor system is fixed using tie bars at 600mm centres into 100mm of topping concrete.

On the northern end of the building, a steel moment frame system is in place to collect gravity loads from the front façade and balcony areas.

4.3 Lateral Load Resisting Systems

Lateral resistance is provided by the precast concrete walls. In the longitudinal direction (north-south), lateral loads at the roof are distributed to the exterior walls at the eastern and western elevations through horizontal steel plate bracing. Lateral loads at the first floor are distributed by the topping to the longitudinal exterior walls and the central longitudinal wall.

East-west (transverse) direction lateral loads at the roof are primarily resisted by out-of-plane bending of the exterior panels at the east and west elevations. Lateral loads are transmitted to the walls through the transverse steel frames. The panels at the south elevation resist a small portion of the tributary roof loads.

East-west lateral loads at the first floor are resisted by the panels at the south elevation (panels 5, 6, 7 and 8), Panel 14 and the transverse insitu wall under the stairs. A steel moment frame is present along the northern end of the building below the first floor. However, it is extremely flexible relative to the pre-cast walls in the remainder of the building. As a result, it does not attract sufficient load to be considered as part of the lateral load resisting structure.

The drawings call for either 663 mesh or H10/H12 bars to be used in the pre-cast panels. Observation on site of the cast in situ wall indicates 663 mesh has been used. The effective steel area for the mesh is less than the H10/H12 bars. Little additional detailing exists on the drawings around panel openings.

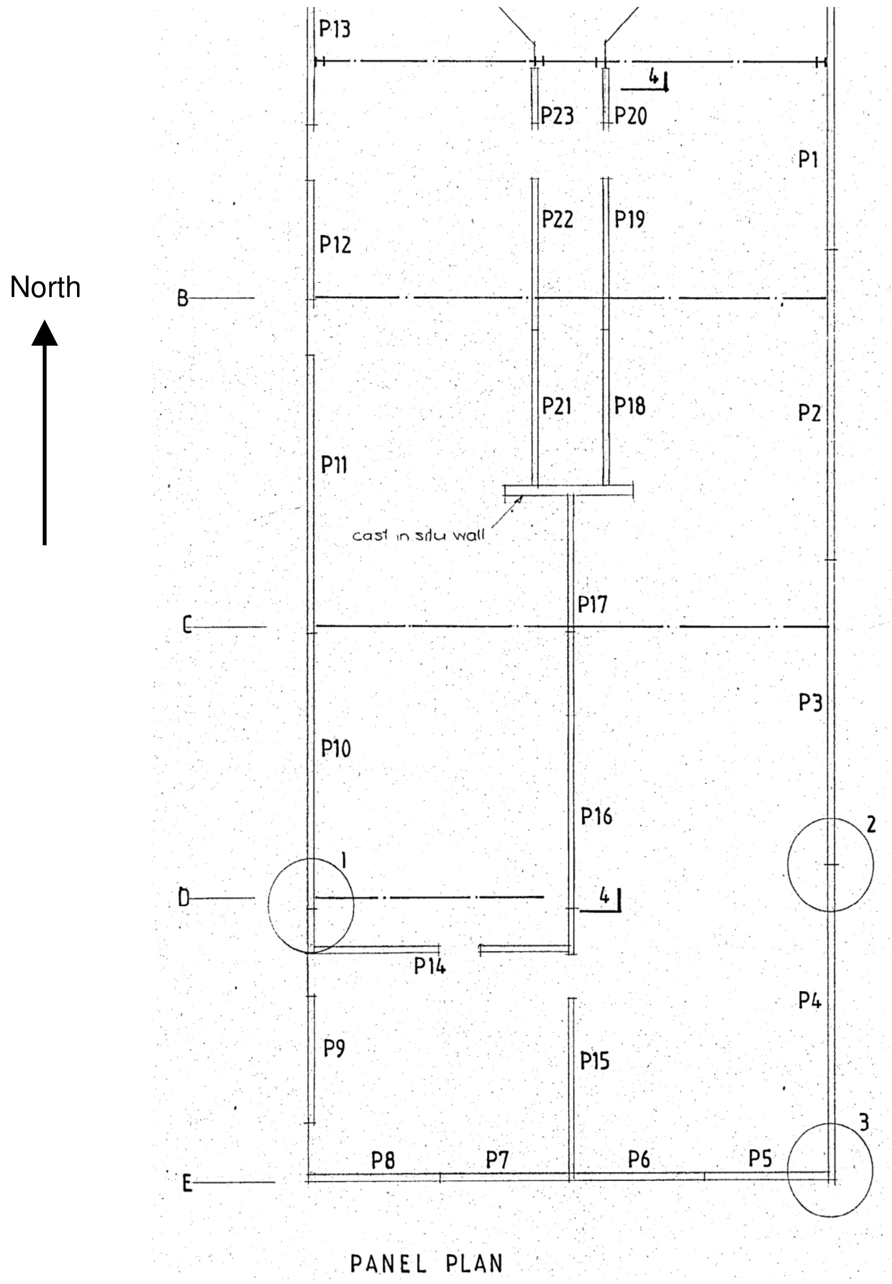


Figure 2: Panel Plan and Lateral Load System

4.4 Original Documentation

Copies of the following construction drawings were provided by CCC on 17 January 2012:

- Development at 56 Lichfield Street Structural drawings, stamped with Christchurch City Council Approval on 5 July 1989. The drawings were prepared by Lewis and Barrow Structural Engineers in April 1989.

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

4.5 CBD Red Zone Cordon

Following the Lyttelton Earthquake of 22 February 2011, the central business district (CBD) suffered major damage to a large proportion of its building stock resulting in a central area of the city was cordoned off and closed to the public, forming what is known as the Red Zone. The Red Zone extent, as of 6 September 2012, is displayed below in Figure 3.

This building is not within the Red Zone and is publicly accessible immediately adjacent to the temporary Bus Exchange.

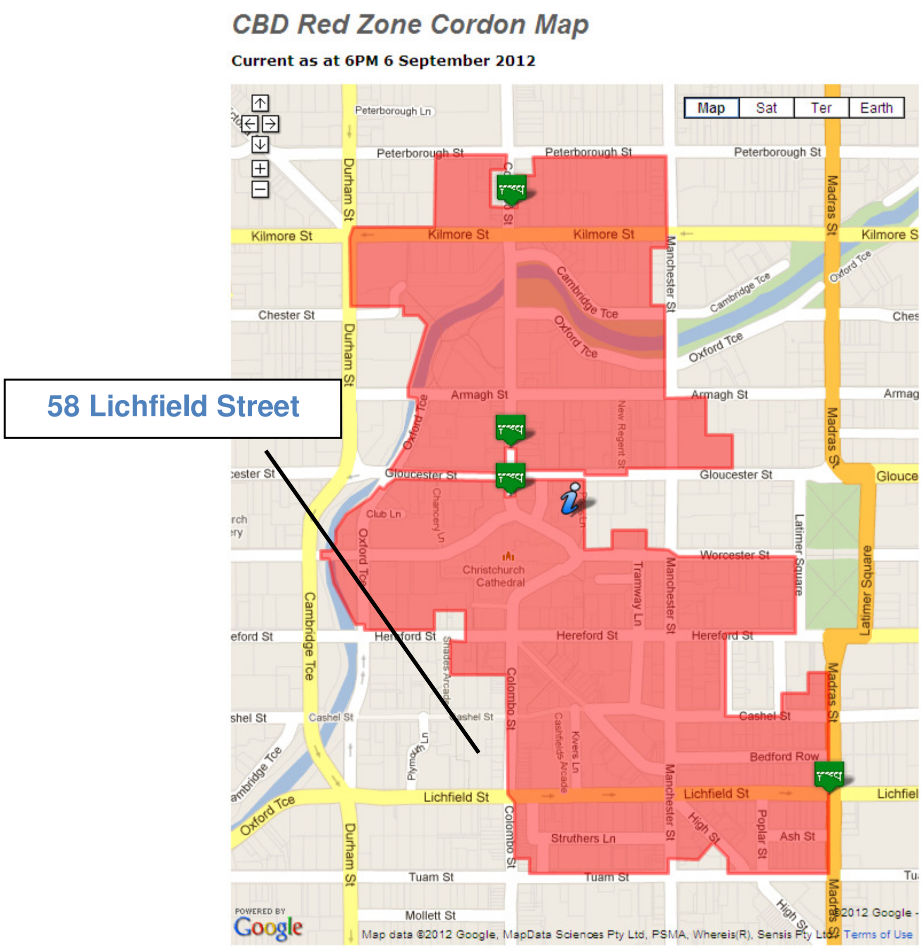


Figure 3: Building Location relative to current Red Zone cordon

5 Survey

5.1 Post 22 February 2011 Rapid Assessment

Level 1 Assessments were undertaken on this building on 28 February 2011and 14 March 2011, both inspections identified hazards from neighbouring buildings.

A structural (Level 2) assessment of the above buildings/property was undertaken on 5 May 2011 and an adjacent building hazard both from an adjacent URM building in Lichfield Street and also at the rear of the Rohit's building was identified. On 28 June 2011, a site inspection confirmed the adjacent building hazard remained. An inspection on 1 August 2011 by Opus International Consultants noted the neighbouring buildings had been demolished and hazards mitigated.

5.2 Further Inspections

Further inspections were undertaken by Opus International Consultants on 21 December 2011 and 26 March 2012.

6 Structural Damage

The following damage has been noted:

6.1 Surrounding Buildings

Prior to the February 2011 earthquake, R&R Sport held the tenancies in the buildings to the east and west of this building. R&R Sport at 54 Lichfield St has been demolished but it was an unreinforced masonry building which suffered significant damage in the February 2011 event. Falling masonry from this building has damaged the panels along the length of the western wall of Rohit's. This includes damage to the sheathing from the roof onto the upper sections of these walls and removal of this may have compromised weather tightness. A closed in doorway at ground level has been damaged significantly and forced inward. This can be seen in the photographs included in Appendix One. This damage is not structural.

R&R Sport on the corner of Lichfield and Colombo Streets is adjacent to the eastern wall of Rohit's. While this wall is not shared, there is negligible separation between the two buildings and this is may have affected building performance during the earthquakes. R&R Sport is considered earthquake prone with an expected strength of 12%NBS. It contains a number of critical structural weaknesses including a number of brittle failure mechanisms which could lead to partial collapse of the building in a large aftershock.

Because the remaining R&R Sport building abuts the eastern wall of Rohit's, it has not been possible to inspect this side of the building for pounding damage. It is possible that damage has occurred on the exterior of panels P1 – P4 (refer to Figure 2 for panel locations).

643 Colombo Street, formerly known as Peaches & Cream, was a two storey unreinforced masonry building that extended along the south wall of Rohit's Restaurant. While this building suffered significant damage in the February event and falling masonry damaged

the adjacent Penny Lane Records, Rohit's does not appear to have suffered any damage from the collapse.

6.2 Western Elevation Precast Concrete Panels

Cracking has been observed on the top of the western wall at the connections between the steel frames and panels P11 and P12. This is consistent with the lateral roof loads being transferred into the wall panels at the weld plate connections. Inspection of the eastern wall panels of the building has been limited to the interior walls due to proximity of the neighbouring building.

No other damage has been noted to the lateral load resisting system.

6.3 Foundations

Minimal ground settlement was observed on this site (<10mm) and no damage has been observed that could be attributed to ground settlement. No other foundation damage has been observed to the building and as a result, no intrusive investigation has been undertaken at this stage. General Observations and Damage

7 Detailed Seismic Assessment

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

An initial qualitative assessment as outlined in the DEEP guidelines was not undertaken on this building prior to completing a detailed quantitative analysis. Identification of load paths, critical structural weaknesses and collapse hazards has been completed as part of the detailed quantitative analysis.

7.1 Critical Structural Weaknesses

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

a) The steel portal frame that supports the roof structure is fixed to the pre-cast walls that cantilever from the first floor. The fixings are embedded steel plates into the top of the panels. There is minimal distance between the embed and the edge of the concrete and their capacity limits the capacity of the portal frame.

b) Pre-cast concrete wall panels cantilever from the first floor diaphragm and resist lateral loads out of plane for loading in the transverse direction. Flexural failure of this wall out of plane is the likely mode of failure.

c) Along the south wall, limited support is provided at the top of the panel by the roof and this wall provides some lateral support of the roof and majority of its self-weight when loaded in the north-south direction. Flexural failure of this wall out of plane is the likely mode of failure.

d) The building is torsional when loaded in the east west direction due to the small number of walls taking loads in plane and the flexible frame on the northern elevation.

e) Precast concrete walls are connected to the ground slab via single weld plates at each corner of the panel base. These plates are welded to embedded steel "fishtail" plates or angles fixed into the slab. In plane load capacity of the panels is limited by the shear capacity of the connections. The failure mode of these is brittle.

f) The building does not include any collectors to transfer lateral forces in the east-west direction into the shear walls along the south wall of the building. Instead the topping slab on the Stahlton units is relied on to transfer load into either the southern walls (P5-P8) and/or the wall south of gridline D (P14). Very little load is able to be transferred into the cast in situ wall under the stairs in the centre of the building.

g) Connections of horizontal steel plates at roof level are not concentric thus put steel UB roof beams in weak axis bending and torsion. Additionally, loads from the steel plates are ultimately transferred into the concrete panels via embeds at top of precast wall (discussed in bullet item 1 above).

7.2 Quantitative Assessment Methodology

The assessment assumptions and methodology have been included in Appendix 2.

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.

7.3 Limitations and Assumptions in Results

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

- Simplifications made in the analysis, including boundary conditions such as foundation fixity.
- Assessments of material strengths based on limited drawings 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.

7.4 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. Other elements within the building may have significantly greater capacity when compared with the governing elements and may be considered further when developing the strengthening options.

Ductility values have been assigned to elements on a case by case basis with μ = 1.0 being used for elements expected to behave in a brittle way. For singly reinforced panels and steel moment frames which are likely to have more ductility, μ = 1.25 has been chosen.

Table 2: Summary of Seismic Performance

Structural Element/System	**Failure mode or description of limiting criteria based on elastic capacity of critical element**	**Critical Structural Weakness and Collapse Hazard**	**% NBS based on calculated capacity**
Primary Components (those that are required parts of the lateral resisting system)			
Steel portal frames at roof level	Steel portal frames provide lateral resistance in the east-west direction loading to transfer the roof inertia to the precast concrete wall panels that cantilever above the 1st floor. The portal is connected to steel embeds cast into top of the precast concrete panels. The capacity of the portal frame is limited by the steel embeds. Failure mechanism of this connection is likely to be brittle.	Yes	< 10% NBS (μ = 1.0)
Precast concrete walls along east and west elevations acting in out-of-plane bending.	For east-west loading, the precast concrete wall panels cantilever above 1st floor to resist lateral load at roof level. The failure mode is in out of plane flexure.	Yes	15 – 30 % (μ = 1.25)
Precast concrete walls acting in-plane.	Panel is typically connected to ground floor via weld plates at each corner of panel that are welded to "fishtail" plates or steel embed in concrete at ground floor. Capacity of wall panel to resist in plane load is limited by the connections and the failure mode is brittle.	Yes	< 15 % (μ = 1.0)
Precast concrete wall panel 'P14' south of line D	Based on structural drawings, panel 'P14' is not directly connected to the foundation. Instead, the panels are attached to perpendicular panels along line 1 and 1.5, thus imposing additional loads on the perpendicular panels and their attachments. Failure mode is likely to be base connections of the perpendicular walls.	Yes	< 34% (μ = 1.0)
In-situ concrete wall below 1st floor level along line B.5	Concrete shear wall resist lateral load in east-west direction loading. The failure mode is in flexure.	No	40 - 50% NBS (μ = 1.25)
Steel moment frame below 1st floor level along north elevation	Steel moment frame occurs along the north elevation. The moment frame is flexible compare to the concrete walls thus resist relatively small percentage of lateral load.	No	100% NBS (μ = 1.25)
Steel plate bracing at roof level (roof diaphragm in north-	Diagonal steel plates at roof level act as diaphragm to	Yes	< 10% NBS

Structural Element/System	**Failure mode or description of limiting criteria based on elastic capacity of critical element**	**Critical Structural Weakness and Collapse Hazard**	**% NBS based on calculated capacity**
south direction loading)	resist lateral loads in north-south direction. Steel plates are welded to UB roof beams. The connections are not concentric thus induce torsion and weak axis bending to the steel UB framing at the roof. Additionally, the capacities of the connections to the eastern and western precast walls are not adequate. Failure is likely in a brittle mode. Especially at embed on top of western wall where minimal distances exist between embed and edge of concrete.		(μ = 1.0)
Concrete topping at 1st floor level	In east-west direction loading, the concrete topping acts as a diaphragm to redistribute load from roof as well as inertial load from 1st floor to walls and moment frame below.	No	60% NBS (μ = 1.25)
Collectors	Collectors are not provided to transfer lateral loads into shear walls along the transverse direction. Load path depends on axial forces in topping slab to deliver seismic forces into in-situ wall between B.5 or precast wall panel "P14".	Yes	< 34% NBS (μ = 1.25)
Secondary Components (those that are not required parts of the lateral load resisting system but which must be able to maintain their gravity load capacity while the building under goes deformation due to earthquake loading)			
Out-of-plane loading of precast concrete wall panel along south elevation	The out-of-plane loading is resisted by combination of cantilever action (from 1st floor slab) with some support at the roof level. In the north-south direction loading, the roof diaphragm consists of steel plates which are flexible. Thus the roof provides little support in the out-of-plane loading of the panels. Additionally, opening in 1st floor occurs adjacent to panel 'P7' approximately the entire length, Thus the panel relies on roof and one attachment to perpendicular wall for out-of-plane support.	Yes	30 - 40% (μ_p = 2)
Precast stair	Details of precast stair show steel RHS embed. RHS is grouted into recesses at supporting slab with approximately 300mm bearing length. The bearing length is generous and story drift is expected to be small thus it is not considered a CSW.	No	NA

7.5 Discussion

The seismic capacity of the building is governed by the capacity of the connection from the precast concrete walls to the foundations, with this connection having a capacity of 10-20% 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 connections between the precast walls and foundation 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.

The connections of the steel roof frames to the top of the panels on the west wall pose a risk of brittle failure. These connections have a capacity of 40%NBS and damage to the panels immediately below the connections indicate that full capacity of the panel is unlikely to be able to develop before the connection fails. The potential for partial collapse of the building resulting from this type of failure is high. The relative risk to the public due to the current carpark access along this side of the building should be noted as being high.

8 Summary of Geotechnical Appraisal

8.1 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.

8.2 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.

8.3 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.

8.4 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.

9 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:

North-south direction loading (longitudinal direction)

- Strengthen the roof diaphragm.
- Strengthen purlin connection to south wall for out-of-plane loading. Strengthen light gauge steel purlins as required to increase compression capacity.
- Improve transfer of shear forces in exterior precast walls
- Strengthen connection between steel UB and top of precast panels.

East-west direction loading (transverse direction)

- Strengthen connection between existing UB beams and the top of the concrete wall.
- Provide supplemental support under UB beams.
- Strengthen foundation.
- Strengthening to address lateral forces in the east-west direction
- Repair of all current earthquake induced damage to the building.

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

10 Conclusions

a) The seismic performance of the original building is governed by the shear capacity of the pre-cast wall embed plates along the eastern and western walls due to failure of the steel plate connections. These sections have an expected strength of 10 %NBS. The building is therefore considered to be earthquake prone in accordance with the Building Act 2004.

b) Connection of the steel roof frame to the top of the precast panels is also an area of concern. The capacity of the roof framing is limited by the embedded steel weld plates and is approximately 25%NBS. This prevents the pre-cast walls from developing their full capacity.

c) Torsional action of the building has a major effect on the loads that are transferred to the walls of the building. This exacerbates the poor performance of a number of elements.

d) The building contains a number of critical structural weaknesses which include connections between the roof steel and precast walls, lack of load path for transfer of lateral loads to the south walls and a lack of load path to transfer lateral loads from the first floor diaphragm into the pre-cast walls.

e) Liquefaction hazard for the site is considered low.

f) The building contains a number of failure mechanisms which could lead to partial collapse of the building. We recommend that the existing cordon around the adjacent building be extended to cordon off access to the full perimeter of the building as soon as possible. The width of the cordon should be 12m, based on 1.5 times the maximum building height.

g) While it is likely to be possible to strengthen the building to at least 67%NBS, we do not believe that this will be economically feasible.

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

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

12 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 Canterbury Earthquake and aftershocks only. Some non-structural damage is described but this is not intended to be a complete list of damage to non-structural items.

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

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

13 References

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

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

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

Appendix 1 - Photographs

North Elevation (Lichfield Street)

Western Elevation

South Elevation

Panel Connection to floor

First Floor precast flooring system

Roof connection at top of panels

Damage noted at top of panel 12 (Western wall)

Damage noted at top of panels 11 (Eastern wall)

Appendix 2 – Quantitative Assessment Methodology and Assumptions

A2.1. 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.
- ASCE/SEI 41-06, *Seismic Rehabilitation of Existing Buildings,* Structural Engineering Institute of the American Society of Civil Engineers, 2007.

A2.2. Analysis Parameters

The following parameters are 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
  $\mu = 1.25$ (nominally ductile)
- Structural performance factor — Cl. 2.6.2.2, NZS3101:2006
  $S_p = 0.925$

- Material properties

Table A1: Analysis Material Properties

Concrete nominal compressive strength, f'_c (MPa) (1)	25
Mild reinforcing nominal yield strength, f_y (MPa) (2)	275
High strength reinforcing nominal yield strength, f_y (MPa) (2)	414

Notes:
1. Based on guidance from *NZSEE 2006,* probable concrete compressive strength is based on a value of 1.5 times the nominal compressive strength (Cl. 7.1.1)
2. Based on guidance from *NZSEE 2006,* probable reinforcement yield strength is based on a value of 1.08 times the nominal yield strength (Cl. 7.1.1)

- Effective section properties

Table A2: 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
  $G + E_u + \Psi_E Q$

- Floor live loading — Table 3.1 Part G, AS/NZS1170.1
  Q = 2.0 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 = 2694 kN

A2.3. Assessment Methodology

Static & Modal Response Spectrum Analysis

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

A 3D model was set up using the structural analysis program ETABS, and effective section properties for structural members were taken from Table A2 above. The floor diaphragms were modelled as flexible diaphragms.

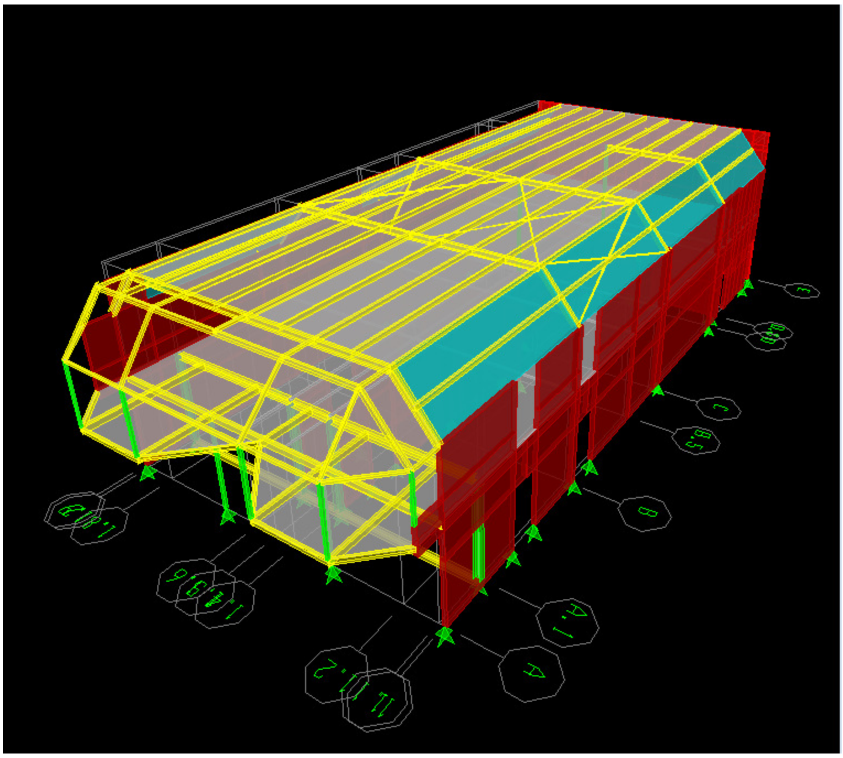


Figure A1: ETABS model of the Structure (Northwest corner)

The fundamental building periods output from ETABS are:

- T_1 = 0.08 sec (E/W Dir)
- T_2 = 0.03 sec (N/S Dir)

An equivalent static analysis was also carried out as a consistency check of the MRS analysis output. The Central Library structure is classified as an irregular structure, per NZS1170.5, Clause 4.5. For structures that are classified as irregular, the base shear from the MRS analysis shall be scaled up to 100% of the equivalent static method base shear, as required by NZS1170.5, Clause 5.2.2.2. The base shears resulting from the equivalent static method are:

- V_{ES} = 1,960 kN (E/W direction)
- V_{ES} = 1,960 kN (N/S direction)

The base shears resulting from the MRS are:

- V_{MRS} = 818 kN (E/W direction)
- V_{MRS} = 754 kN (N/S direction)

The forces from the MRS analysis were scaled up by 2.4 and 2.6 in the E/W and N/S directions, respectively.

The building was analysed as having ductility (μ = 1.25) and the design actions were applied separately in each perpendicular direction, with 100% for the first axis plus 30% on the second axis, and then 30% on the first axis and 100% on the second axis, as required by NZS1170.5, Clause 5.3.1.2.

<u>Element Demand to Capacity</u>

Element force demands were extracted from the MRS analysis and compared to calculated capacities based on the material properties assumed in Table A1. The results of these demand to capacity checks are summarized in further detail in the report and reported as %NBS.

Appendix 3 – Geotechnical Appraisal

7 February 2012

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

6-QUCC.55

Geotechnical Desk Study, 56 - 58 Lichfield St

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 56 to 58 Lichfield 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) Alibaba – A restaurant located on Lichfield Street.
2) Rohits – An Indian style restaurant located on Lichfield Street.

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 of the foundations of the building are available and extracts are included in Appendix B of this report. The building is two storeys with an in filled basement.

The foundations consist of a 100mm thick reinforced concrete slab supported on hardfill and demolition bricks. Internal columns are supported on shallow concrete pads to basement level.

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 60m east of the site and terminated in shallow gravels at a depth of approximately 5m. Logs of relevant borehole 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 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 56 - 58 Lichfield Street due to the Canterbury Earthquake and aftershock sequence following the 4 September 2010 earthquake.

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).

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

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 differential 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, dependent on the location of the epicentre. It is expected that the probability of occurrence is likely to decrease with time, following periods of reduced seismic activity.

[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

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 Rohits restaurant (24 January 2012).

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

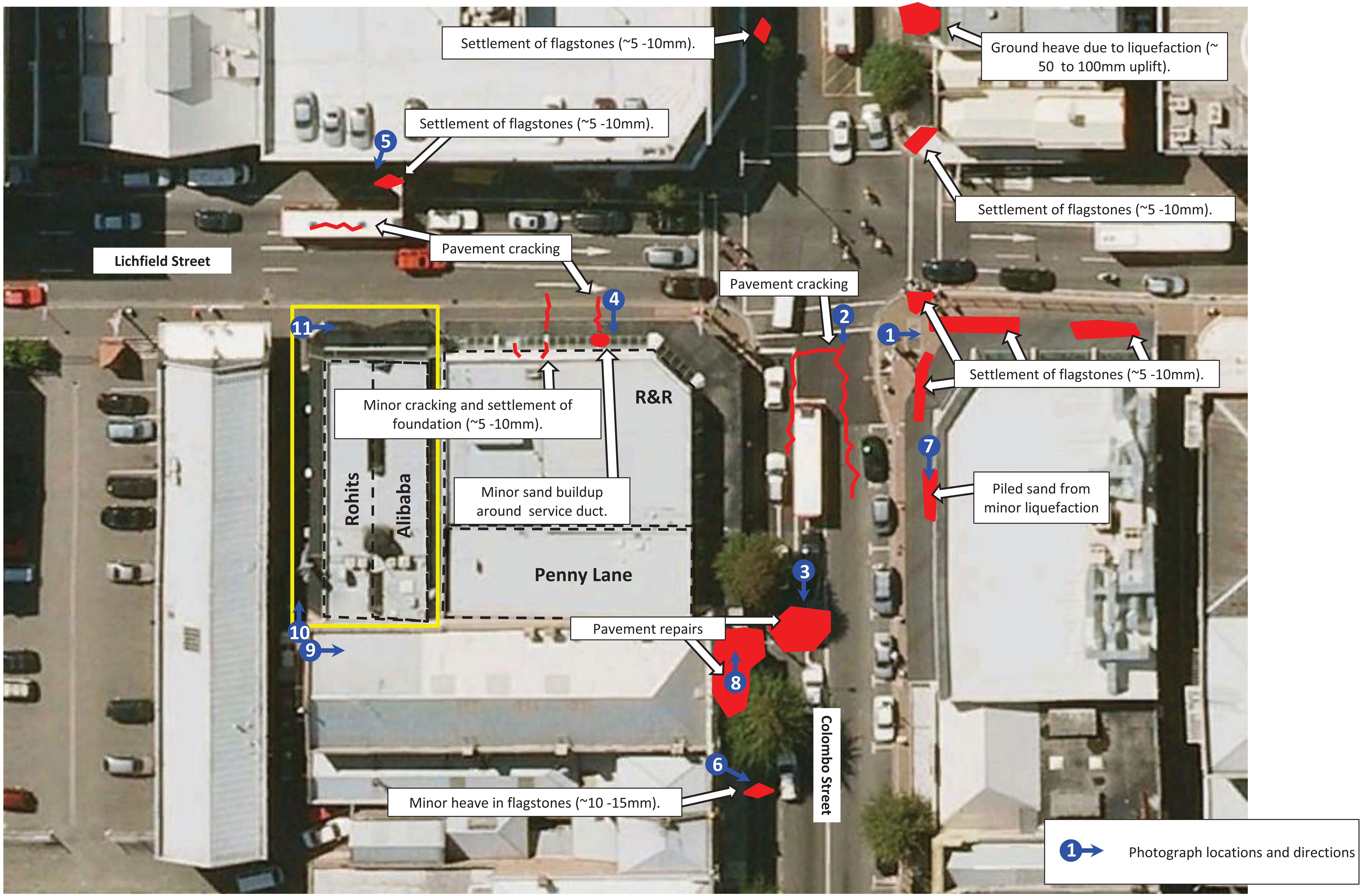
Photograph 7 – Piling of sand ejected by minor liquefaction on the east side of Colombo Street across from R&R (24 January 2012).

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 Rohits 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 Rohits building (24 January 2012).



OPUS

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20 Moorhouse Ave
PO Box 1482
Christchurch, New Zealand
Tel: +64 3 363 5400 Fax: +64 3 365 7857

Project: 56 - 58 Lichfield Street
Geotechnical Desktop Study

Project No.: 6-QUCCC.55

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: 56 - 58 Lichfield
Geotechnical Desktop Study

Project No.: 6-QUCCC.55

Client: Christchurch City Council

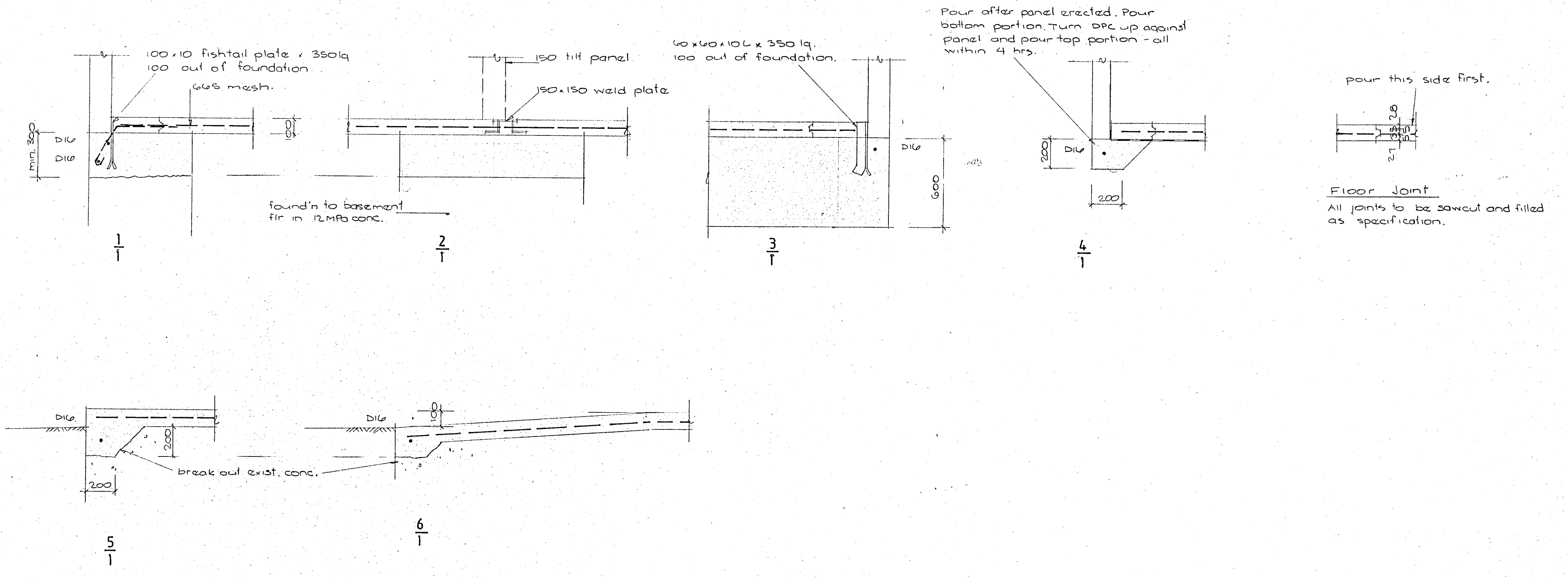
Figure 2 - Site Location Plan

Completed by: Shane Greene - 24 January 2012
Engineering Geologist

Date Drawn: 24/01/2012

Appendix B:

Structural Drawings



89-1280
Sheet 6

CHRISTCHURCH CITY COUNCIL
Approved Subject to the By-Laws
5 JUL 1989
For Engineer

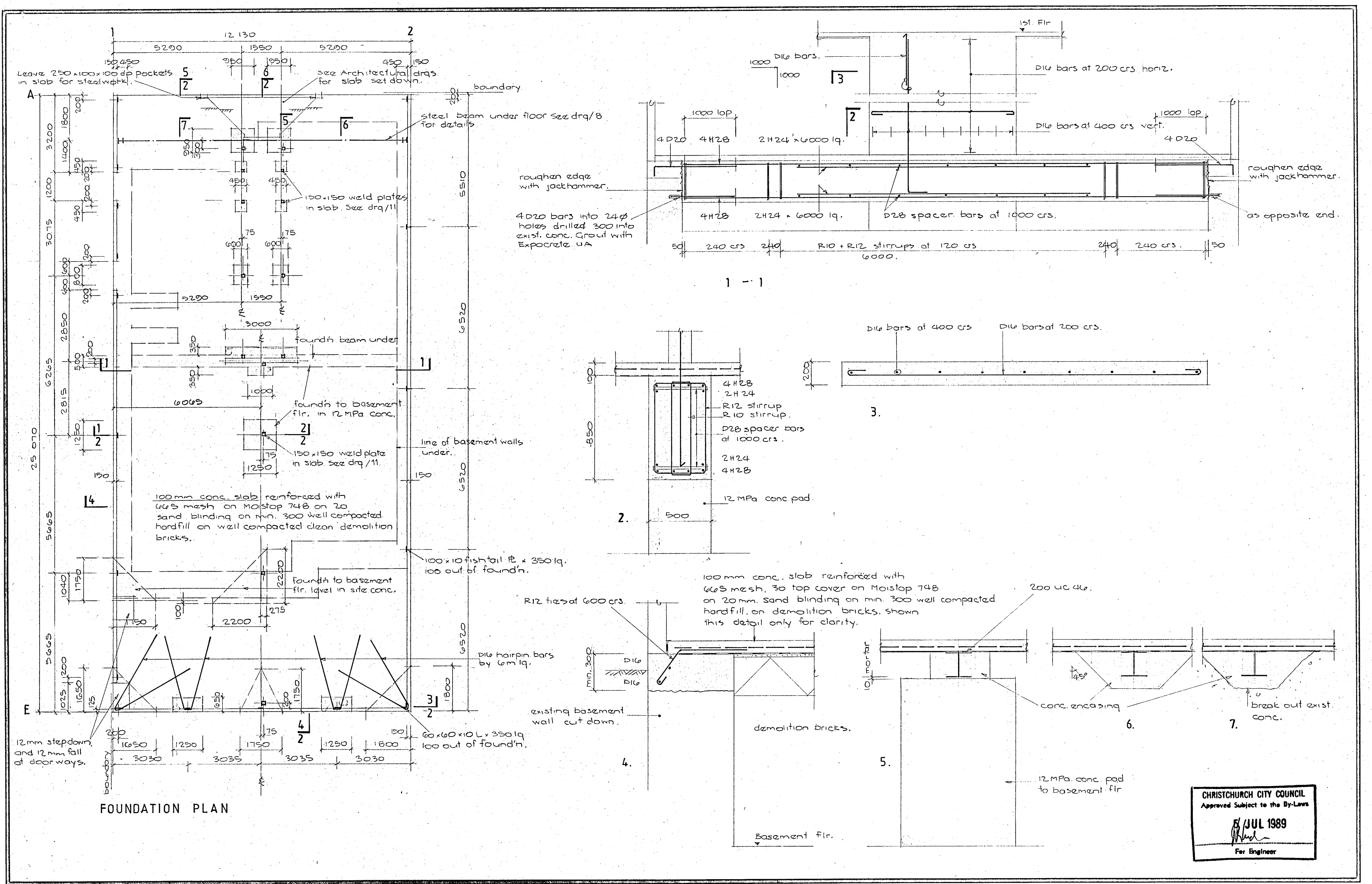
LEWIS & BARROW
CONSULTING CIVIL AND STRUCTURAL ENGINEERS
183 Hereford St Christchurch New Zealand
P.O Box 13-282 Telephone: 64-320

DEVELOPMENT AT 56 LICHFIELD STREET

FOUNDATIONS

DSN. LC DRN. HS TCD. HS CKD. GM DATE. 4.89	SCALES	FILE	DRAWING
	1:20	3577	S2 OF 12

Contractor shall verify all dimensions before starting work



LEWIS & BARROW
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183 Hereford St Christchurch New Zealand
P.O Box 13-282 Telephone: 64-320

DEVELOPMENT AT 56 LICHFIELD STREET

FOUNDATIONS

	SCALES	FILE	DRAWING
DSN. LC DRN. HS TCD. HS CKD. ghl DATE. 4.89	1:100 1:50 1:20	3577	S1 OF 12

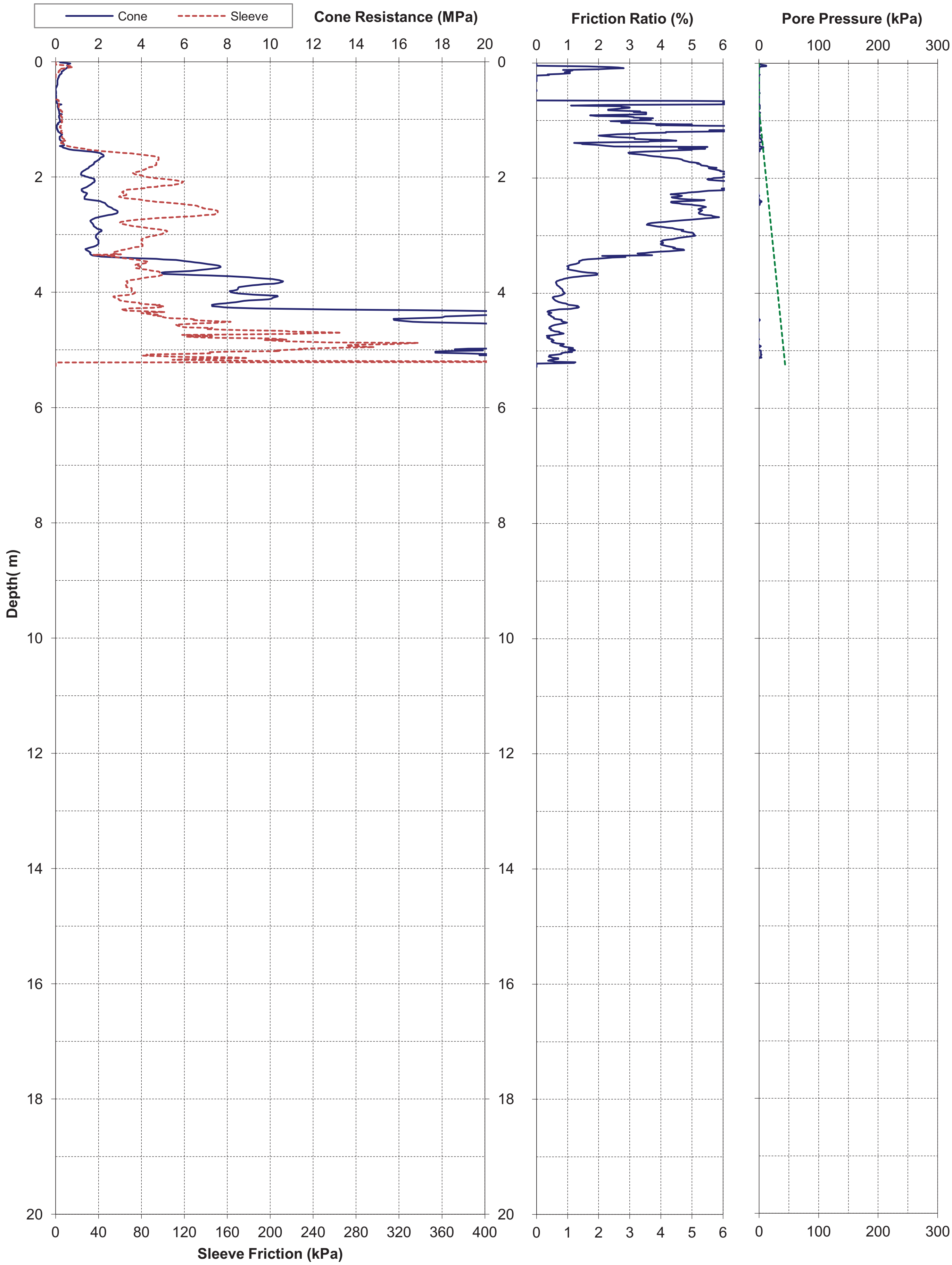
Contractor shall verify all dimensions before starting work

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



T+T Ref: 52000.3400 Printed: 28/09/2011 1:31 p.m. Template: CPT Graph Template v0.41.xls

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

Scale(m)	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?
-5	-9.50m	Shingle	sp?
-10	- 10.0m	Shingle with some sand	sp?
	- 11.9m	Sand	sp?
	- 13.7m	Blue shingle	sp?

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 Ait:	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 :

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 :

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

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

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

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?
		-4.19m	Gravel	sp?
-5				
-10				
		- 12.1m	Fine sand	sp?
		- 15.0m		ch

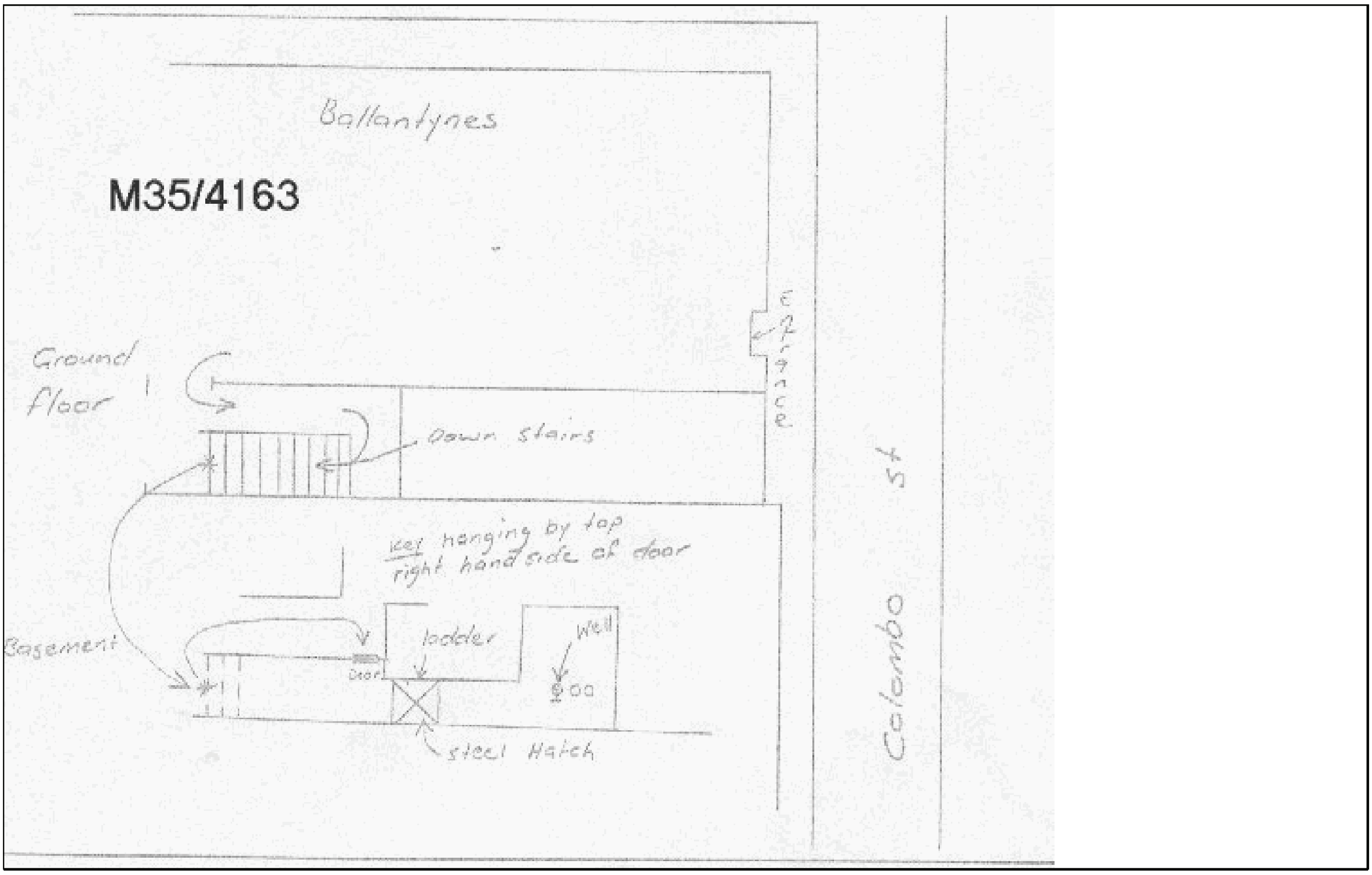
Bore or Well No:	M35/4163
Well Name:	BALLANTYNES
Owner:	BALLANTYNES COMPANY LTD

Environment Canterbury
Your regional council

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)		

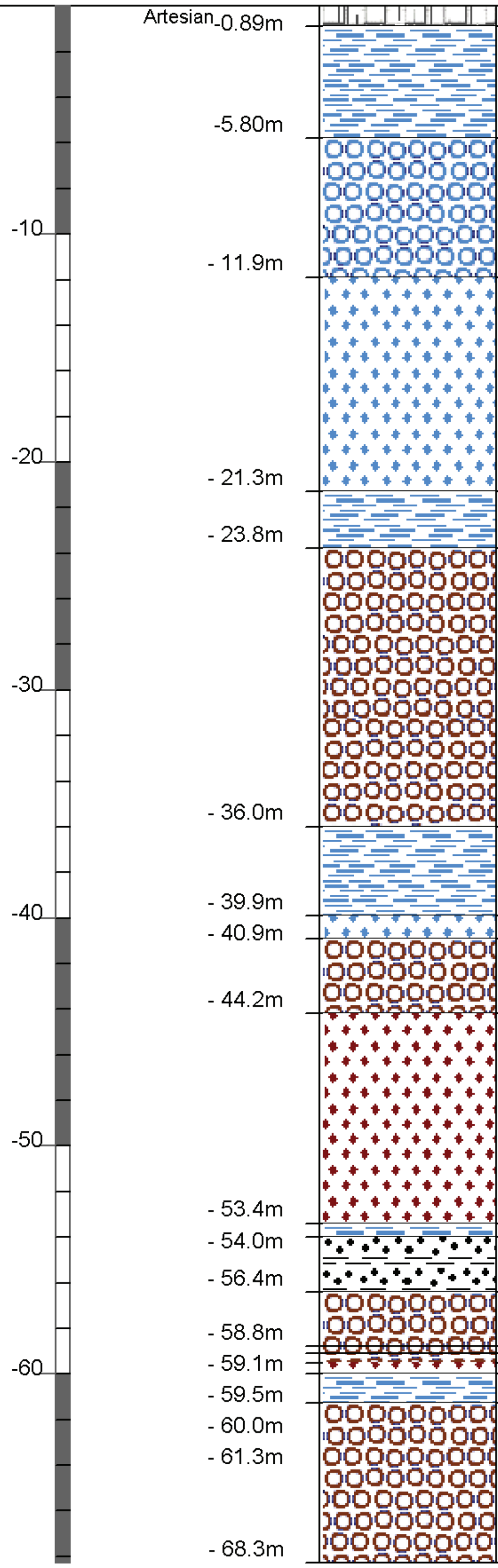
Drill Date:	09 Mar 1960	**Water Level Count:**	481
Well Depth:	65.00m -GL	**Strata Layers:**	18
Initial Water Depth:	5.97m -MP	**Aquifer Tests:**	0
Diameter:	100mm	**Isotope Data:**	0
		Yield/Drawdown Tests:	0
Measuring Point Ait:	2.22m MSD QAR 1	**Highest GW Level:**	7.51m from MP
GL Around Well:	4.17m -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
Ground floor
Entrance
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



Depth(m)	Full Drillers Description	Formation Code
-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-1
- 59.5m	Brown gravel	li-2
- 60.0m	Brown sand	li-2
- 61.3m	Blue clay	li-2
- 68.3m	Brown gravel	li-2

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 page 1 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): -10, -20, -30, -40, -50, -60

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

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

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

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

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

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	
-5		-9.40m	blue gravel and sand	
-10		- 12.2m	blue sand	

Appendix 4 – DEE Spreadsheet

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name: Rohits Indian Restaurant
Building Address: Unit: — No: 56-58 Street: Lichfield Street
Legal Description: CB 19B/966 Pt Section 1011
GPS south: (Degrees / Min / Sec)
GPS east:
Building Unique Identifier (CCC): BU 2677-006 EQ2

Reviewer: Alistair Boyce
CPEng No: 209860
Company: Opus International
Company project number:
Company phone number:
Date of submission: 13/09/2012
Inspection Date: 26/03/2012
Revision: Final
Is there a full report with this summary? yes

Site

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

Max retaining height (m):
Soil Profile (if available):
If Ground improvement on site, describe:
Approx site elevation (m): 20.00

Building

No. of storeys above ground: 2 (single storey = 1)
Ground floor split? no
Storeys below ground: 0
Foundation type: isolated pads, no tie beams
Building height (m): 8.00
Floor footprint area (approx): 300
Age of Building (years): 23

Ground floor elevation (Absolute) (m): 0.00
Ground floor elevation above ground (m):
if Foundation type is other, describe: Founded on pre-existing basement walls
height from ground to level of uppermost seismic mass (for IEP only) (m): 8
Date of design: 1976-1992

Strengthening present? no
If so, when (year)?
And what load level (%g)?
Brief strengthening description:

Use (ground floor): commercial
Use (upper floors): commercial
Use notes (if required): Hospitality - restaurant
Importance level (to NZS1170.5): IL2

Gravity Structure

Gravity System: load bearing walls
Roof: steel framed — rafter type, purlin type and cladding: steel purlins, 200UB frame, lightweight steel roof
Floors: precast concrete with topping — unit type and depth (mm), topping thickness: Stahlton beams T1 160, 75mm topping
Beams: none — overall depth x width (mm x mm):
Columns: other (note) — typical dimensions (mm x mm):
Walls: load bearing concrete — #N/A

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along: concrete shear wall — note total length of wall at ground (m): 25
Ductility assumed, μ: 1.00 — wall thickness (m): 150
Period along: 0.08 — 0.00 from parameters in sheet — estimate or calculation?
Total deflection (ULS) (mm): — estimate or calculation?
maximum interstorey deflection (ULS) (mm): — estimate or calculation?

Lateral system across: concrete shear wall — note total length of wall at ground (m): 12
Ductility assumed, μ: 1.00 — wall thickness (m): 150
Period across: 0.03 — 0.01 from parameters in sheet — 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: precast, half height — describe supports: Supported by cast in situ wall
Wall cladding: plaster system — describe: Gib linings only
Roof Cladding: Metal — describe:
Glazing: aluminium frames
Ceilings: light tiles
Services(list):

Available documentation

		original designer name/date
Architectural	partial	B Wayne Seebeck
Structural	full	Lewis & Barrow
Mechanical	none	
Electrical	none	
Geotech report	full	Unknown

Damage

Site: (refer DEE Table 4-2)

Site performance: Good — Describe damage: Minor

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: 0-20mm/20m — notes (if applicable):
Damage to area: slight — notes (if applicable):

Building:

Current Placard Status: yellow

Along — Damage ratio: — Describe how damage ratio arrived at: Minor damage observed
Describe (summary):

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

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

Diaphragms — Damage?: yes — Describe: 1st floor concrete diap
CSWs: — Damage?: yes — Describe: roof-wall cxns, wall-grd cxns, torsional,
Pounding: — Damage?: no — Describe:
Non-structural: — Damage?: no — Describe:

Recommendations

Level of repair/strengthening required: significant structural and strengthening — Describe: as described in report
Building Consent required: yes — Describe:
Interim occupancy recommendations: do not occupy — Describe:

Along — Assessed %NBS before: — ##### %NBS from IEP below
Assessed %NBS after: 10%
If IEP not used, please detail assessment methodology: Quantitative

Across — Assessed %NBS before: — ##### %NBS from IEP below
Assessed %NBS after: 10%

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: 8m

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):	0.08	0.03
(%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, $(NBS\%)_b = (\%NBS)_{nom} \times A \times B \times C \times D \times E$ — $\%NBS_b$: **#DIV/0!** | **#DIV/0!**

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

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

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

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

3.6. Other factors, Factor F

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

OPUS