Christchurch City Council

Cecil Courts
BE 1047 EQ2

Detailed Engineering Evaluation

Quantitative Assessment Report

Christchurch City Council

Cecil Courts

Quantitative Assessment Report

16 Cecil Place and 33 Vienna Street, Christchurch, New Zealand

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Summary

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Detailed Engineering Evaluation
Quantitative Report - Summary
Final

Background
This is a summary of the quantitative report for the Cecil Courts residential structures, and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections, and available drawings.

Key Damage Observed
Key damage observed includes:

a) Liquefaction has occurred throughout the site as evident in the ground settlement adjacent to Block J and heaving of the asphalt south of Block E.

b) Garage Block G, H, I and "turret" at Block C appear to be tilting, possibly due to differential settlement.

c) The ground beam foundation has been exposed along the northern elevation of Unit 4 at 16 Cecil Place (within Block A) due to subsidence of the ground.

d) A 4m long section of ground adjacent to the perimeter footing of Unit 9 at 16 Cecil Place (within Block C) collapsed. A void is located in the southeast corner of the foundation as evident by damaged services.

e) Cracking and leaning of masonry landscaping walls throughout the site.

f) Cracking in masonry wall at corners of window and door openings.

g) Horizontal cracking in the order of 1 to 2mm at roof eave level in the transverse masonry walls.

h) Separation at wall junctions between the exterior masonry walls and the adjacent landscaping walls.

i) Humping and differential settlement in the slab on grade in some of the units. The measured differential settlement within a residential unit is up to 76mm over approximately 8m which is considered excessive.

j) Minor cracking at masonry walls at the garages and the laundry building.

k) Cracks in Gib wall and ceiling linings and separation of linings at various junctions.

l) Misalignment of door framing, likely a result of racking and/or localized floor humping.

m) Cracked window/skylight at laundry building.

Aside from the ground conditions which caused the tilt in the North wing, the superstructure performed very well and the observed damage is consistent with the expected building performance, following our review of the structural drawings and site investigations.

Critical Structural Weaknesses

a) Discontinuous walls: Along the rear of the residential units, there are several walls above the first floor level that are not continuous to the foundation. Seismic overturning forces in these walls impose additional loads onto the spandrels.

b) Weak storey and soft storey: In the longitudinal direction, there are long wall piers above the 1st floor. In comparison, the wall is punched by window and door openings below the 1st floor resulting in a weak storey as well as a soft storey condition.

Indicative Building Strength

Based on the information available, and from undertaking a quantitative assessment, the buildings have been assessed to have overall capacities in excess of 34% NBS. The capacities of the residential blocks are generally governed by flexural strength of the narrow wall piers along the front of the buildings and the capacity of the transverse walls to bend out-of-plane. The capacities of the garage blocks and laundry building are governed by the ability of the masonry walls to span out-of-plane to perpendicular walls. The %NBS for each building are summarised below:

Building	% NBS
Residential Blocks (Blocks A – E)	43%
Garages (Blocks G – J)	37%
Laundry Building (Block F)	38%

As the buildings have a calculated seismic capacity of 37-43% NBS they are not defined as being earthquake prone and their seismic performance is legally accepted under the 2004 Building Act. Based on the calculated capacities, the buildings are defined as moderate risk in accordance with the NZSEE [2] guidelines, and have a relative risk of failure of 5-10 times that of a building constructed to the New Building Standard. Based on the form of construction and the seismic load resisting systems present we do not believe that the buildings have a high risk of collapse. It is therefore considered that there is not a high risk imposed to building occupants.

Recommendations

a) Repair damaged structural elements, finishes, and site work.

b) Develop a strengthening works scheme to increase the seismic capacity of the building to at least 67%NBS; this will need to consider compliance with accessibility and fire requirements.

c) Engage a quantity surveyor to determine the costs for strengthening the building.

d) The site needs a full geotechnical assessment to determine the potential for further liquefaction and if ground improvements are needed (this is currently underway).

Contents

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council to undertake a detailed seismic assessment of Cecil Courts, located at 16 Cecil Place and 33 Vienna Street, Christchurch following the Canterbury Earthquake Sequence since September 2010. This report has been prepared by Simpson Gumpertz & Heger (SGH) in conjunction with Opus International Consultants.

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) [3] [4].

2 Compliance

This section contains a brief summary of the requirements of the various statutes and authorities that control activities in relation to buildings in Christchurch at present.

2.1 Canterbury Earthquake Recovery Authority (CERA)

CERA was established on 28 March 2011 to take control of the recovery of Christchurch using powers established by the Canterbury Earthquake Recovery Act enacted on 18 April 2011. This act gives the Chief Executive Officer of CERA wide powers in relation to building safety, demolition and repair. Two relevant sections are:

Section 38 – Works

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

Section 51 – Requiring Structural Survey

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

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

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

1. The importance level and occupancy of the building.

2. The placard status and amount of damage.

3. The age and structural type of the building.

4. Consideration of any critical structural weaknesses.

Christchurch City Council requires any building with a capacity of less than 34% of New Building Standard (including consideration of critical structural weaknesses) to be strengthened to a target of 67% as required under 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).

The Earthquake Prone Building policy for the territorial authority shall apply as outlined in Section 2.3 of this report.

Section 115 – Change of Use

This section requires that the territorial authority is satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'.

This is typically interpreted by territorial authorities as being 67% of the strength of an equivalent new building or as near as practicable. 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.

Where an application for a change of use of a building is made to Council, the building will be required to be strengthened to 67% of New Building Standard or as near as is reasonably practicable.

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:

- increase in the basic seismic design load for the Canterbury earthquake region (Z factor increased to 0.3 equating to an increase of 36 – 47% depending on location within the region);
- 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).

Table 1: %NBS compared to relative risk of failure

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

3.1 Minimum and Recommended Standards

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

3.1.1 Occupancy

The Canterbury Earthquake Order[1] in Council 16 September 2010, modified the meaning of "dangerous building" to include buildings that were identified as being 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 and from the DBH guidance document dated 12 June 2012 [6], 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/territorial authority guidelines.

3.1.3 Strengthening

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

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

3.1.4 Our Ethical Obligation

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

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

4 Background Information

4.1 Building Description

Cecil Court is located at 16 Cecil Place and 33 Vienna Street, Christchurch. The address for the western portion of the site is 16 Cecil place and the eastern portion is 33 Vienna Street. The site consists of twenty 2-storey residential units built in the 1970's. The units are split into five separate blocks that are approximately 21m by 5.2m each. Additionally, there are four garages and a single storey laundry building in the complex. Refer to site plan in Figure 2:

Figure 2: Site Plan

4.1.1 *Attached two storey residential blocks*

Each block consists of four attached two level household units. The structure is estimated to have been designed in the late 1960's and built in the early 1970's.

The roof consists of metal roofing supported by timber purlins on masonry exterior walls. The 1st floor consists of "plyco & Bondec" floor spanning between interior timber beams and timber ledgers bolted onto masonry walls. In addition to exterior of the building, masonry walls also exist between units. The masonry walls are 200mm thick, fully grouted and are reinforced with vertical and horizontal bars. A small lightweight timber framed section exists around the stairwell. The interior walls within each unit are also lightweight timber framed.

The ground floor slab consists of 100mm thick concrete slab on ground. The foundation consists of 600mm deep ground beams below masonry walls and a thickened slab below interior timber walls.

The lateral load resisting system consists of the following:

- Masonry shear walls in the transverse direction at exterior ends of each block and between each household unit. The walls are 200mm thick, fully grouted masonry walls with typical vertical reinforcement at 600mm centres and horizontal bars at 800mm centres. Masonry walls extend from ground floor to roof level where top of block walls match the gable roof pitch.

- Masonry wall piers coupled with spandrel beams in the exterior along longitudinal direction. The walls piers are typically 200mm thick with vertical reinforcement at 600mm centres and horizontal reinforcement typically at 800mm centres. Horizontal bars are added at short piers adjacent to window openings.

- A combination of bond beams spanning between walls and diaphragm action provided by timber floors at 1st floor and GIB ceilings below the roof level distribute lateral loads to the shear walls.

4.1.2 *Detached single level garages*

Each garage is a single level detached structure. The structure is estimated to have been designed in the late 1960's and built in the early 1970's.

Each garage structure consists predominantly of solid filled reinforced masonry exterior walls with some lightweight timber infill. The timber infill is generally the gable end walls. The roof consists of metal roofing supported by spaced timber boards which in turn are supported on timber trusses. The ground floor is concrete slab on ground. The foundation is unknown but is expected to consist of shallow concrete foundations similar to the residential units.

The lateral load resisting system in both directions consists of:

- 200mm thick, fully grouted masonry walls with vertical reinforcement at 600mm centres and horizontal bars at the top of wall.

- A combination of bond beams in masonry wall and diaphragm action provided by timber cross bracing in the plane of the roof distribute the lateral load to the shear walls.

4.1.3 *Detached single level laundry building*

The laundry building is a single level detached structure. The structure is estimated to have been designed in the late 1960's and built in the early 1970's.

The laundry structure consists predominantly of solid filled masonry exterior walls. The roof consists of metal roofing supported by spaced timber boards which are in turn supported by gang-nailed timber trusses. The ground floor is concrete slab on ground. The foundation is unknown but is expected to consist of shallow concrete foundations similar to the residential units.

The lateral load resisting system in both directions consists of:

- 200mm thick, fully grouted masonry walls with vertical reinforcement at 600mm centres and horizontal bars at mid height as was as at the top of wall.
- A combination of bond beams and diaphragm action provided by ceiling linings distribute lateral forces to shear walls.

4.2 Survey

4.2.1 Post 22 February 2011 Rapid Assessment

A structural (Level 2) assessment of the above buildings/property was undertaken on 9th and 10th March 2011 by Opus International Consultants following the February 2011 Christchurch earthquake. Yellow placards (restricted entry) were placed on units 1-4/16 Cecil Place (Block A) due to severe settlements in the slab and risk of subsidence to the south of the structure. Flat 9 at 33 Vienna Street (within Block C) was also given a yellow placard due the presence of a large settlement hole along the south side of the building. All other dwellings in the Cecil Court complex were given green placards as a part of this inspection.

4.2.2 Further Inspections

- Opus International Consultants performed a detailed site walkover on 10 May 2011. This was coupled with a desk study of the historic drill logs to develop an initial appraisal of the suitability of the land and future bearing capacity.
- On 13 October 2011, an Interim Earthquake Structural Damage report was produced by Opus International Consultants. The report cites findings from the previous internal and external walkovers and discusses the implications of observed damage on structural performance. In general, it indicated that the limited structural damage observed could be attributed to ground settlements rather than shaking. Preliminary findings indicated the buildings have performed well and that the buildings should be considered salvageable with repairs to remove the tilt and settlement observed.
- Additional site visits were performed by Opus International Consultants on 26 September 2012. Field measurements were made to produce drawings for the garages and laundry building. Cover metre survey was performed to determine reinforcement spacing within the block walls. Face shells of the block walls at a few locations were chipped out to view the reinforcement in the walls.
- On 26 September 2012, Opus International Consultants completed a survey of the floor levels. The result of the floor level survey is included in Appendix 5

4.3 Original Documentation

Copies of the following construction drawings were provided by CCC:

- Structural Drawings titled "Reclamation Housing – Cecil Street (Cecil St to Thackeray St.)" dated August 1975; 5 Sheets

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

Copies of the design calculations were not provided.

5 Structural Damage

The following damage has been noted:

5.1 Surrounding Buildings

At Tommy Taylor Courts at 7 Cecil Place, which is approximately 70m west of Cecil Courts, we observed significant tilt of the North wing. Evidence of liquefaction was observed throughout the site at Tommy Taylor Courts.

5.2 Residual Displacements

Residual displacements were seen in a number of structural components in the group of apartments.

- Garage Block G (located at 16 Cecil Place) experienced some differential settlement and building rotation due to liquefaction along the southern boundary. The rotation is most severe along the western elevation where the garage is leaning on a timber fence. Rotation is also visible along the eastern elevation (see photos 8-10).
- Garage Block H appears to be tilting to the north.
- Garage Block I appears to be leaning north.
- The turret located at Unit 9 of 16 Cecil Place (within Block C) appears to be tilting to the north.

A copy of the floor level survey is included in Appendix 5.

5.3 Foundations

The foundations of a number of buildings on this site have been affected by liquefaction and the resulting subsidence.

- The ground beam foundation has been exposed along the northern elevation of Unit 4 at 16 Cecil Place (within Block A. See Appendix 1, photo 14).

- A 4m long section of ground adjacent to the perimeter footing of Unit 9 at 16 Cecil Place (within Block C) collapsed. A void is located in the southeast corner of the foundation as evident by damaged services (see Appendix 1, photo 16).

5.4 Primary Structure

5.4.1 Attached Two Storey Residential Blocks

Observed damages consist of:

- Cracking in masonry wall at corners of window and door openings (see photo 30).
- Horizontal cracking in the order of 1 to 2mm at roof eave level in the transverse masonry walls.
- Separation at wall junctions between the exterior masonry walls and the adjacent landscaping walls.
- Humping and differential settlement in the slab on grade in some of the units. Based on floor level survey, the differential settlement within a single unit ranges from 12mm over approximately 3.5m (Unit 3 at 33 Vienna Street) to 76mm over approximately 8m length (Unit 4 at 16 Cecil Place). In some units, the differential settlement exceeds the maximum allowable differential settlement of 25mm over 6m as specified in Clause B1 of the New Zealand Building Code at Serviceability Limit State.

5.4.2 Detached Single Level Garages

Observed damages consist of:

- Minor cracking less than 1mm in the masonry walls of the garages (see Appendix 1, photo 34).

5.4.3 Detached Single Level Laundry Building

Observed damages consist of:

- Minor cracking less than 1mm in the masonry walls of the laundry building.
- The differential settlement within the laundry building is 50mm over approximately 5.5m. This differential settlement exceeds the maximum allowable differential settlement of 25mm over 6m as specified in Clause B1 of the New Zealand Building Code at Serviceability Limit State.

5.5 Non Structural Elements

5.5.1 Attached Two Storey Accommodation Blocks

Observed damages consist of:

- Cracks in GIB wall and ceiling linings and separation of linings at various junctions (photos 36 and 37)
- Misalignment of door framing, likely a result of racking and/or localized floor humping.
- Damage to the joint in the wood siding in front of one unit (photo 22 and 23).
- Cracked window/skylight (see photo 35).

6 General Observations

Aside from differential settlements and humping of the slab on grades, we did not observe any major damage to any of the building structures. There is minor cracking at the masonry walls, which can be caused by a combination of differential ground movement and shaking.

Significant liquefaction has occurred throughout the site as evident in the ground settlements at various locations.

Refer to Geotechnical Desk Study dated 17 May 2011 undertaken by Opus for further information and description of the ground 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" [5] issued on 21 December 2011.

7.1 Qualitative Assessment Summary

A 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.

7.2 Review of 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 each of the buildings and have been considered in the quantitative analysis.

Critical Structural Weaknesses have been identified as follows:

- Discontinuous walls: Along the rear of the residential units, there are several walls above the first floor level that are not continuous to the foundation. Seismic overturning forces in these walls impose additional loads onto the spandrels. See Figure 3 below:

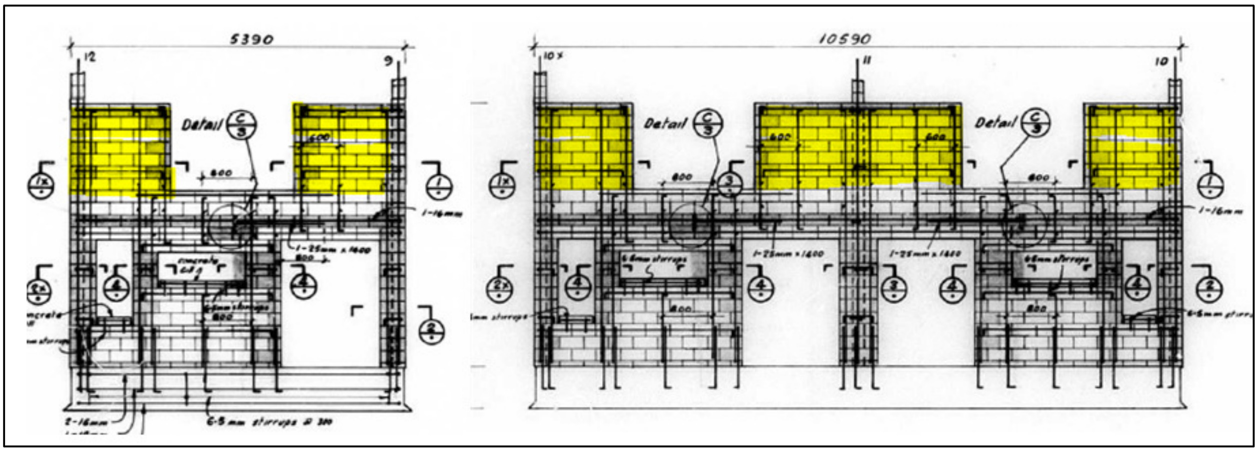

Figure 3: Masonry wall elevation at rear of the Residential block

- Weak storey and soft storey: In the longitudinal direction, there are long wall piers above the 1st floor. In comparison, the wall is punched by window and door openings in the storey below resulting in a weak storey as well as a soft storey condition.

7.3 Quantitative Assessment Methodology

The assessment assumptions and methodology have been included in Appendix 3 of the report due to the technical nature of the content. A brief summary follows:

a. The base shear was calculated from the seismic weight of the building using the spectral values established from NZS1170.5, with an updated Z factor of 0.3 (B1/VM1). The base shear was distributed to different storeys following NZS1170.5.
b. All the buildings (residential units, garages, and laundry building) consist of flexible diaphragms at the 1st floor and/or the roof thus the horizontal forces are distributed to each individual wall lines by tributary area. 2-D models of each wall lines were created in ETABS analysis software.
c. The buildings were assessed as Importance Level 2.
d. The ground condition was taken as site subsoil class D

7.4 Limitations and Assumptions in Results

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

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

a. Simplifications made in the analysis, including boundary conditions such as foundation fixity.

b. Assessments of material strengths based on limited drawings, specifications and site inspections
c. The normal variation in material properties which change from batch to batch.
d. Approximations made in the assessment of the capacity of each element, especially when considering the post-yield behaviour.

7.5 Assessment

A summary of the structural performance of the buildings is shown in Tables 2-4. Note that the values given represent the critical elements in the building, as these effectively define the buildings capacity. Other elements within the building may have significantly greater capacity when compared with the governing elements. This will be considered further when developing the strengthening options.

The building was analysed using a ductility factor (μ) equal to 1.25 due to the fact that the masonry wall pier have minimal horizontal reinforcement.

Table 2: Summary of Seismic Performance – Residential Units, μ = 1.25

Structural Element/System	Failure mode or description of limiting criteria based on displacement capacity of critical element.	% NBS based on assumed capacity
Residential Units: Primary Components (those that are required parts of the lateral resisting system)		
In-plane action of masonry walls piers - Longitudinal direction	Masonry wall piers are 200mm thick fully filled. The walls along the front of the units consist of slender wall piers that are controlled by flexure.	43%
In-plane action of masonry walls piers - Transverse direction	Masonry wall piers are 200mm thick fully filled. The walls along the transverse direction primarily controlled by shear.	100%
Masonry spandrels – Longitudinal direction	The longitudinal walls have various window and door openings creating spandrels that couple wall piers together. As discussed in the CSW section above, some walls above the 1st floor are not continuous to the foundation. The overturning forces in these walls impose additional loads onto the spandrels but the stresses in the spandrels are generally small. The controlling mode of failure is generally in shear.	100%
Floor diaphragm - Longitudinal direction	The floor diaphragm consists of a timber "plyco & Bondec" floor. The shear strength of the timber flooring is limited. The demand on the diaphragm is substantial if block walls rely on the diaphragm for out-of-plane support. However, the connection between the masonry walls and the floor to resist out-of-plane wall loading is limited. As an alternate load path, the masonry walls would span horizontally between perpendicular walls (See discussion below under "Out-of-plane loading for masonry wall".	34% (46% with alternate load path)
Floor diaphragm - Transverse direction	The floor diaphragm consists of a timber "plyco & Bondec" floor. The shear strength of the timber flooring is limited.	50%

Structural Element/System	Failure mode or description of limiting criteria based on displacement capacity of critical element.	% NBS based on assumed capacity
Out-of-plane bending of transverse masonry wall	As per the discussion above in "Floor Diaphragm", the wall-to-floor connections have limited capacity to resist out-of-plane loading from wall. An alternate load path exists where the transverse walls span horizontally between walls at the front and back of the units.	46%

Table 3: Summary of Seismic Performance – Garage Units, μ = 1.25

Structural Element/System	Failure mode or description of limiting criteria based on displacement capacity of critical element.	% NBS based on assumed capacity
Garages: Primary Components (those that are required parts of the lateral resisting system)		
In-plane action for masonry walls - Longitudinal direction	Masonry walls exist along the entire rear of the garages. The walls are 200mm thick fully filled with vertical reinforcement at 600mm centres and horizontal reinforcement at top of wall. The calculated shear stress in the wall is generally small.	100%
In-plane action for masonry walls piers - Transverse direction	Masonry wall existing at the two ends as well as between some parking spaces. They are typically 200mm thick fully filled with vertical reinforcement at 600mm centres and horizontal bars at the top of wall. The calculated shear stresses in the walls are generally small.	100%
Roof diaphragm – Longitudinal direction	The diaphragm for the garages consists of timber braces cut into the skipped straight board. The demand on the diaphragm is substantial if the block walls rely on the diaphragm for out-of-plane support. However, the connections between the masonry walls and the roof to resist out-of-plane wall loading is limited. As an alternate load path, the masonry walls would span horizontally as well as cantilever from ground beam (See discussion below under "Out-of-plane loading for masonry wall"). Given this alternate load path, the lack of a proper diaphragm would unlikely to cause the building to collapse.	16% (37% with alternate load path)
Roof diaphragm – Transverse direction	Diaphragm of the garages consists of timber braces cut into the skipped straight board.	34% (37% with alternate load path)
Out-of-plane bending of masonry wall	As per the discussion above in "Roof Diaphragm", the wall-to-roof connections have limited capacity to resist out-of-plane loading from wall. An alternate load path exists where the wall bends out-of-plane in two-way action where the back wall and ground provide support.	37%

Table 4: Summary of Seismic Performance – Laundry Building, μ = 1.25

Structural Element/System	Failure mode or description of limiting criteria based on displacement capacity of critical element.	% NBS based on assumed capacity
Laundry Building: Primary Components (those that are required parts of the lateral resisting system)		
In-plane action of masonry walls piers - Longitudinal direction	Concrete block wall piers are 200mm thick fully filled. Capacity is limited by flexure in the wall piers at the front of the building.	100%
In-plane action of masonry block walls piers - Transverse direction	Concrete block wall piers are 200mm thick fully filled.	100%
Roof diaphragm – Longitudinal direction	Roof diaphragm of the laundry building consists of GIB sheathing at the ceiling. There are many window openings along the east and west end of the roof/ceiling thus reducing the strength of the GIB to act as a diaphragm. However, given the relatively small size of the building, the masonry walls are expected to be able to span horizontally as well as cantilever from ground beam (See discussion below under "Out-of-plane loading for masonry wall"). Given this alternate load path, the lack of a proper diaphragm would unlikely to cause the building to collapse.	27% (39% with alternate load path)
Roof diaphragm – Transverse direction	Roof diaphragm consists of GIB sheathing at the ceiling.	38%
Out-of-plane bending of masonry wall	As per the discussion above in "Roof Diaphragm", the roof diaphragm has many openings along the east and west end thus substantially reducing its capacity to resist loading in the NS direction. An alternate load path exists where the wall bends out-of-plane in two-way action where the perpendicular walls and ground provide support.	39%

*Alternative load path exists thus the element does not govern building strength

7.6 Discussion

Based on our quantitative assessment, the structures at the Cecil Courts have computed capacities in the overall lateral load resisting system that exceed 34% NBS. The following sections summarise our findings for each of the structural types:

7.6.1 *Attached two storey residential blocks*

In the residential blocks, the flexural strengths of the slender masonry block walls along the front of the buildings limit the overall lateral strength of the building. The calculated strength for these piers are 43%NBS.

The wall-to-floor connections as well as the diaphragm also have limited strength to resist out-of-plane forces in the transverse masonry walls. However, given the relatively small plan dimensions, the masonry walls will span horizontally between wall piers at the front and back of the units thus reducing the demand on the wall anchorages and diaphragms.

The calculated strength for the walls bending out-of-plane (without support from the diaphragm) is 46% NBS thus the buildings are not considered to be earthquake prone.

Although we have identified some Critical Structural Weaknesses in these buildings, the calculated stresses in the walls and spandrels are generally small and the overall lateral load resisting system have capacities that exceed 34% NBS.

7.6.2 *Detached single level garages*

There are a fair amount of masonry walls in the garages such that the in-plane stresses are generally small. Typical for garage structures, the front is open thus the longitudinal direction loading is resisted entirely by the back wall. The roof diaphragms of the garages, which consist of skipped timber boards with some diagonal timber members, have limited capacity to deliver the load to the back wall; especially if the transverse walls rely on the roof diaphragm for out-of-plane support. An alternate load path exists where the transverse walls acts in two-way bending between the slab on ground and the back wall. The calculated strength for the walls bending out-of-plane is approximately 37% NBS.

7.6.3 *Detached single level laundry building*

There is a fair amount of masonry walls such that the in-plane stresses in the walls are generally small. The roof diaphragm consists of GIB sheathing at the ceiling. There are many window/skylight openings along the east and west side of the ceiling thus its capacity to act as a diaphragm for north-south direction loading is greatly reduced. However, given the relatively small footprint of the building, the masonry walls will span horizontally between perpendicular walls. The calculated strength for the walls bending out-of-plane is approximately 39% NBS.

As the buildings have a calculated seismic capacity of 37-43% NBS they are not defined as being earthquake prone and their seismic performance is legally accepted under the 2004 Building Act. Based on the calculated capacities, the buildings are defined as moderate risk in accordance with the NZSEE [2] guidelines, and have a relative risk of failure of 5-10 times that of a building constructed to the New Building Standard. Based on the form of construction and the seismic load resisting systems present we do not believe that the buildings have a high risk of collapse. It is therefore considered that there is not a high risk imposed to building occupants.

8 Summary of Geotechnical Appraisal

8.1 General

Christchurch City Council commissioned Opus International Consultants to undertake a desktop study of the ground conditions beneath Cecil Courts. The result of this study was detailed in a memo dated 17 May 2011, which is included in Appendix 2 of this report. The key points of the study are summarised herein.

8.2 Liquefaction Potential

The historic borehole logs dated between 1890 and 1913 indicate that the site is underlain by variable thicknesses of sand and gravel layers, likely to be susceptible to liquefaction. A gravel layer was encountered at a depth of approximately 24m to 28m. Blue shingle and gravel layers were encountered at shallower depths (6m – 15m) in some of the logs.

The 2004 ECAN Liquefaction study[2] indicates the site as having a moderate to high liquefaction potential under high groundwater conditions. Based on a low groundwater table, ground damage is expected to be moderate, subsidence likely to be between 100mm and 300mm.

No liquefaction was reported following the Darfield Earthquake of 4 September 2010.

The area has been identified to have undergone low to moderate liquefaction[3] as a result of the 22 February 2011 earthquake.

8.3 Summary

The foundations of a number of buildings on this site have been compromised by liquefaction and the resulting subsidence. Units 1 – 4 of 16 Cecil Courts have a real risk of subsidence to the south and Garage Block G has been badly affected.

There are no streams or open watercourses within close proximity of the site, which minimises the potential for lateral spreading.

The SESOC interim advice[4] indicates approximately a 6% per annum probability of another Magnitude 6 – 6.5 earthquake 'close to the Christchurch CBD' over the next 50 years. Liquefaction of a similar order of magnitude, and subsequent damage to buried services, would be expected in such an event. Also liquefaction could occur in large earthquakes on the foot hills, Alpine or other faults.

8.4 Further Work

Due to the ground damage which has occurred at this site, it was proposed to carry out additional geotechnical site investigations. This additional work is currently underway. The objective of the additional geotechnical investigations are to:

1. Determine the ground and groundwater conditions
2. Understand the nature of liquefaction at the site, including depth.
3. Assess the potential for future liquefaction and consequential ground damage.
4. Quantify the magnitude of building rotation that has occurred and monitor on going movement.

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[2] ECan, The Solid Facts on Christchurch Liquefaction

[3] University of Canterbury Liquefaction Map version 1.0 published on NZSEE Clearing House, drive through reconnaissance (23 Feb – 1 March 2011)

[4] Structural Engineering Society NZ – Interim Advice on Christchurch Seismic Design Load Levels

5. Assess the type of damage to building foundations and floors.
6. Assist in the decision whether to repair, redevelop or relocate.
7. Provide geotechnical information for future foundation design.

9 Conclusions

The buildings have been assessed to have overall capacities in excess of 34% NBS. The capacities of the residential blocks are generally governed by flexural strength of the narrow wall piers along the front of the buildings and the capacity of the transverse walls to bend out-of-plane. The capacities of the garage blocks and laundry building are governed by the ability of the masonry walls to span out-of-plane to perpendicular walls. The %NBS for each building are summarised below:

Building	% NBS
Residential Blocks (Blocks A – E)	43%
Garages (Blocks G – J)	37%
Laundry Building (Block F)	38%

The capacity levels noted above imply that the buildings are considered a moderate risk but their seismic performances are legally accepted under the 2004 Building Act.

Ground damage has been moderate to significant at the site. Differential settlement and rotation of the some buildings is visible. Based on the floor level survey, the differential settlement on the ground floor of the some units exceeds 25mm over 6m as specified in Clause B1 of the New Zealand Building Code at Serviceability Limit State.

Structural damage has been limited to cracks in masonry walls, typically near window and door openings and around wall intersections. Some horizontal cracking was observed at the transverse walls in the residential blocks which is indicative of out-of-plane loading.

10 Recommendations

1. Repair damaged structural elements, finishes, and site work.
2. Develop a strengthening works scheme to increase the seismic capacity of the building to at least 67%NBS; this will need to consider compliance with accessibility and fire requirements.
3. Engage a quantity surveyor to determine the costs for strengthening the building.
4. The site needs a full geotechnical assessment to determine the potential for further liquefaction and if ground improvements are needed (currently underway).

11 Limitations

1. This report is based on an inspection of the structure of the buildings and focuses on the structural damage resulting from the 4 September 2010 Darfield Earthquake and the 22 February 2011 Canterbury Earthquake and aftershocks. Some non-structural damage is described but this is not intended to be a complete list of damage to non-structural items.

2. 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.

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

12 References

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

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

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

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

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

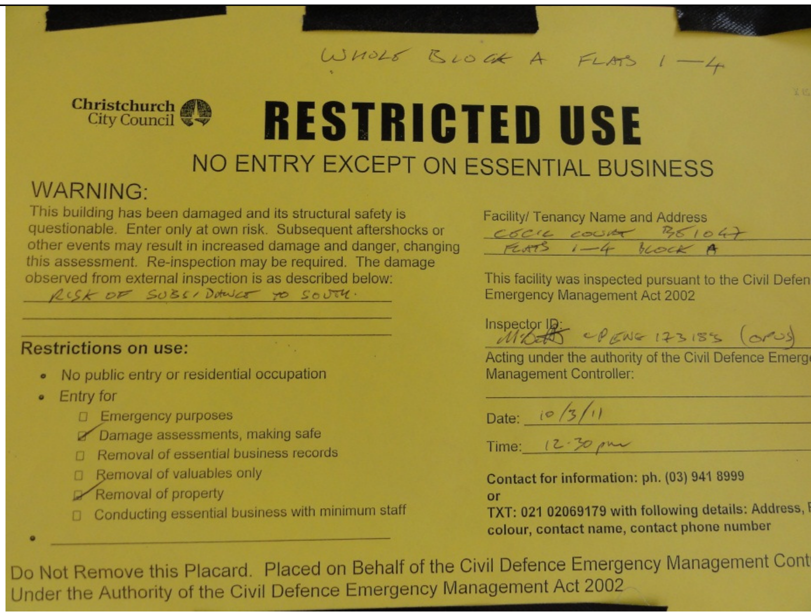
[6] DBH (2012), Guidance for engineers assessing the seismic performance of non-residential and multi-unit residential buildings in greater Christchurch, Department of Building and Housing, June 2012.

Appendix 1 - Photographs

Cecil Courts, Christchurch		
No.	**Item description**	**Photo**
1.	View from entrance to Cecil Courts looking east at Unit 1 at 16 Cecil Court	
2.	View of Units 1 to 4 at 16 Cecil Court	

3.	View from 16 Cecil Courts looking east between Garages	
4.	Garage Block G northeast view	

5.	Rear of Garage Type A	
6.	Laundry Building	

7.	Yellow placard for Units 1-4 at 16 Cecil Place	
8.	Western view of Garage Block G. Garage appears to be tilting to the south	

9.	Settled Garage Block G appears to be leaning on wooden fence	
10.	View of garage leaning on fence from neighbours property	

11.	View of leaning masonry wall near Unit 4 at 16 Cecil Court; Masonry block wall in background exhibits structural damage	
12.	Structural Damage to masonry block wall	

13.	Structural damage to masonry block landscape wall	
14.	Exposed ground beam near Unit 4 of 16 Cecil Place due to ground settlement	

15.	Outside Unit 9 at 16 Cecil Place	
16.	Damage to buried services outside Unit 9 at 16 Cecil Place	

17.	Settlement next to Unit 9 at 33 Vienna Place	
18.	Front facade of Block D, 33 Vienna Street	

19.	Ground damage along the front of Block D, 33 Vienna Street. Damage to siding observed	
20.	Heaving of asphalt at unit 1 of 33 Vienna Street	

21.	Collapsed asphalt in front of Garage Block J	

22.	Wood framed portion typical along the front residential units. Damage to siding was observed at one location	

23.	Close up of damage to wood siding	
24.	Residential unit showing typical finishes typical of ground floor	

25.	Typical floor beam to masonry wall connection	
26.	Cracking at floor beam connection	

27.	Connection between 1st floor sheathing to wall consists of timber ledger bolted into block wall at 600mm centres.	
28.	Typical ceiling at 1st floor consist of GIB sheathing	

29.	Cracking at masonry below 1st floor window	
30.	Cracking of masonry below 1st floor window opening	

31.	During field investigation, masonry block was chipped out to expose reinforcement	
32.	Garage interior	

33.	Garage roof consists of skipped boards spanning between timber trusses	
34.	Minor cracking in garage masonry walls	

35.	Cracked window/skylight at Laundry building	
36.	Cracks in GIB interior linings	

37.	Cracks in GIB linings	

Appendix 2 - Geotechnical Appraisal

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TO	Lindsay Fleming
COPY	Greg Saul, Sheryl Keenan
FROM	Graham Brown
DATE	17 May 2011
FILE	6-QUCCC.01/005SC
SUBJECT	Cecil and Vienna Court - Geotechnical Desk Study

1. Introduction

This memo summarises the findings of a Geotechnical Desk Study and a detailed Site Walkover completed on 10 May 2011. The purpose of this desk study is to provide an initial appraisal of the on the suitability of the land and the future bearing capacity, in accordance with CCC email request of 18 April 2011.

The memo follows an initial Geotechnical Inspection Memo prepared by William Gray dated 11 March 2011.

2. Description of Facility

The site comprises twenty, 2 storey self contained units in five blocks, refer to Site Plan Appendix B. There are four single storey garage blocks and a single storey laundry building. All buildings are formed of masonry block. The buildings are estimated to have been built during the 1970's.

The ground profile is relatively flat, low lying and is typically level with Brougham Street to the north. Cecil Place is located at the western end and Vienna Street at the eastern end of the site.

There is a large green space area bounded by Brougham Street in the central area of the site. The remainder of the site is covered with asphalt and paved areas.

3. Desk Study Results

3.1 *Ground Conditions*

A desk study of well logs in the area from Environment Canterbury records identified four historic drill logs within 300m of the site; refer to Location Plan Appendix A.

Examination of EQC[1] investigations post Darfield Earthquake identified that there are no CPT tests in the vicinity of this site.

A search of Opus database identified two shallow CPT probes undertaken for Orion in nearby Vienna Street and south of the site at 2 Austin Street.

The Logs of the ECan borehole records and Orion CPT tests are all included in Appendix A.

The historic borehole logs dated between 1890 and 1913 indicate that the site is underlain by variable thicknesses of sand and gravel layers, likely to be susceptible to liquefaction. A gravel layer was encountered at a depth of approximately 24m to 28m. Blue shingle and gravel layers were encountered at shallower depths (6m – 15m) in some of the logs.

3.2 Ground and Building Damage

No as built drawings or ground information has been provided by CCC.

A walkover inspection of the exterior of the buildings was completed on 10 May 2011. No interior inspections were conducted.

There is evidence of liquefaction throughout the site resulting in a loss of surface water drainage. Some buildings appear to have suffered from a significant amount of differential settlement resulting in rotation and tilting. A summary of the observed damage is listed below.

- Units 1 – 4 16 Cecil Courts, all yellow stickered due to settlement in ground floor slabs and a risk of subsidence to the south. Ring beam foundation exposed on northern elevation of Unit 4.
- Garage Block G, liquefaction on the southern boundary has resulted in ±150mm of settlement and building rotation. Rotation is severest on the western elevation where the garage is now leaning on timber fence, but also noted on the eastern elevation. Up to 150mm of liquefaction remains behind Garage Block G.
- Garage Block H, appears to be tilting to the north.
- Unit 9 16 Cecil Courts, ‘turret’ appears to be leaning to the north.
- Unit 9 33 Vienna Courts, void on south east corner due to damaged services. Also a 4m long section of ground adjacent to perimeter footing has collapsed.
- Garage Block I, from eastern elevation appears to be leaning north. appears to be tilting to the north by ±50mm.
- Garage Block J, Garage Bay 5, 100mm of ground settlement in front of the entrance.
- Unit 1 33 Vienna Courts, south elevation minor asphalt heave.

[1] Darfield Earthquake 4 September 2010 Geotechnical Land Damage Assessment & Reinstatement Report Tonkin & Taylor for EQC, Stage 1 & 2 2010

As outlined above, no interior inspections of the buildings have been undertaken.

3.3 Liquefaction Hazard

The 2003 ECAN Liquefaction study[2] indicates Brougham Village as having a moderate to high liquefaction potential under high groundwater conditions. Based on a low groundwater table, ground damage is expected to be moderate, subsidence likely to be between 100mm and 300mm.

No liquefaction was reported following the Darfield Earthquake of 4 September 2010.

The area has been identified to have undergone low to moderate liquefaction[3] as a result of the 22 February 2011 earthquake.

4. Appraisal

The foundations of a number of buildings on this site have been comprised by liquefaction and the resulting subsidence. Units 1 – 4 16 Cecil Courts have a real risk of subsidence to the south and Garage Block G has been badly affected.

There are no streams or open watercourses within close proximity of the site, this minimises the potential for lateral spreading.

The SESOC interim advice[4] indicates approximately a 6% per annum probability of another Magnitude 6 – 6.5 earthquake 'close to the Christchurch CBD' over the next 50 years. Liquefaction of a similar order of magnitude, and subsequent damage to buried services, would be expected in such an event. Also liquefaction could occur in large earthquakes on the foot hills, Alpine or other faults.

5. Proposed Geotechnical Investigations

Due to the ground damage which has occurred at this site, it is proposed to carry out the following geotechnical site investigations.

The objective of the proposed geotechnical investigations are to:

a) Determine the ground and groundwater conditions
b) Understand the nature of liquefaction at the site, including depth.
c) Assess the potential for future liquefaction and consequential ground damage.
d) Quantify the magnitude of building rotation that has occurred and monitor ongoing movement.
e) Assess the type of damage to building foundations and floors.
f) Assist in the decision whether to repair, redevelop or relocate.
g) Provide geotechnical information for future foundation design.

The scope of the proposed geotechnical investigations are:

1) Continue smart level readings of buildings to monitor tilt and rotation.

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[2] ECan, The Solid Facts on Christchurch Liquefaction

[3] University of Canterbury Liquefaction Map version 1.0 published on NZSEE Clearing House, drive through reconnaissance (23 Feb – 1 March 2011)

[4] Structural Engineering Society NZ – Interim Advice on Christchurch Seismic Design Load Levels issued 14 April 2011.

2) Borehole to a depth of about 25 m, with Standard Penetration Tests at 1.5 m depth intervals, and install piezometer to monitor groundwater level.
3) Static Cone Penetration Tests (CPT) 4 No.
4) Laboratory soil classification tests on soil samples
5) Excavate and inspect shallow foundations where building damage and/or ground disruption is evident.
6) Hand Auger to 2.0m depth and test to confirm bearing capacity.
7) Assessment and reporting

The location of the proposed borehole, CPTs, Test Pits and Hand Augers are shown on the Annotated Site Location Plan, Appendix C.

Recommendations

1. Carry out geotechnical investigations and assessment as recommended in this memo.
2. Consider the geotechnical conditions, liquefaction hazard and consequential risks in the development of options and decisions for the repair and redevelopment of the site.
3. Further site investigations may be required, depending on the findings of the proposed site investigations.

Attachments:

Appendix A – Location Plan, BH and CPT Records

Appendix B – Annotated Site Plan

Appendix C – Proposed Ground Investigations

Photos showing liquefaction and site damage, 16 Cecil Courts and 33 Vienna Courts

Units1 – 4 16 Cecil Courts

Garage Block G tilting south, up to 150mm settlement.

Liquefaction behind garage

Garage leaning on fence

Exposed ring beam foundation Unit 4 16 Cecil Courts

Garage Block H suspected subsidence on north side

Ground Damage eastern corner of Unit 9 33 Vienna Courts

Ground Damage on eastern elevation of Unit 9 33 Vienna

Possible ground heave of asphalt

Subsidence in front of Garage Block J

APPENDIX A: LOCATION PLAN - BH DATA

Borelog for well M36/1097

Gridref: M36:813-398 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 6.6 +MSD
Driller : not known
Drill Method : Unknown
Drill Depth : -99m Drill Date : 12/02/1913

Environment Canterbury
Your regional council

Scale(m)	Water Level	Depth(m)	Full Drillers Description	Formation Code
	Artesian		Clay & sand	
-10				
-20				
		- 24.3m		sp-ch
-30			Brown shingle	
		- 38.4m		ri
-40			Blue clay & sand	
		- 42.6m		br
			Blue sand	
		- 48.7m		br
-50			Brown sand	
		- 51.2m		br
			Blue shingle	
		- 52.4m		li-1
			Blue sand	
		- 57.3m		li-1
-60			Brown shingle	
		- 63.3m		li-2
			Blue clay & sand	
		- 68.2m		li-2
			Blue shingle	
-70		- 70.1m		li-2
			Blue shingle	
		- 76.2m		li-3
			Brown shingle, water rises 1.8m	
-80		- 79.2m		li-3
-90			Brown sand & shingle	
		- 93.2m		he
			Yellow & Blue clay	
		- 97.8m		he
			Brown shingle, water flows 196.5m3/d & rises 6.7m	
		- 99.0m		bu

Borelog for well M36/0964 page 1 of 2

Environment Canterbury
Your regional council

Gridref: M36:814-399 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 6.2 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -95.3m Drill Date : 6/05/1899

Scale(m): -5, -10, -15, -20, -25, -30, -35, -40, -45

Water Level	Depth(m)	Full Drillers Description	Formation Code
Artesian	-2.09m	Soil	sp
	-6.09m	Clay	sp
	- 21.6m	Gravel (Bl)	sp
	- 24.4m	Blue sand & clay	ch
	- 25.3m	Blue clay & peat	ch
	- 36.9m	Gravel (Br) wl +0.3m	ri
	- 38.3m	Peat	br
	- 39.3m	Clay (Bl)	br
	- 42.0m	Gravel (Br) wl +0.6	br
	- 51.8m	Sand br	br

br

Borelog for well M36/0964 page 2 of 2

Gridref: M36:814-399 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 6.2 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -95.3m Drill Date : 6/05/1899

Environment Canterbury
Your regional council

Water Level: Artesian

Depth(m)	Full Drillers Description	Formation Code
- 51.8m	Sand br	br
- 53.9m	Clay y	br
- 69.5m	Gravel Brown wl +1.2m	li
- 70.1m	Peat	li-2
- 75.9m	Clay (Bl)	li-2
- 79.2m	Gravel (Br) wl +2.1m	li-3
- 81.7m	Yellow sandy gravel	li-3
- 82.9m	Clay sandy y	he
- 84.7m	Sand y	he
- 85.6m	Gravel br	he
- 90.2m	Yellow sand	he
- 93.9m	Sand & clay y	he
- 95.3m	Gravel Brown wl +7.9m	bu

Scale(m): -50, -55, -60, -65, -70, -75, -80, -85, -90, -95

Borelog for well M36/5121

Gridref: M36:818-399 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 6.2 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -104.5m Drill Date : 21/10/1890

Environment Canterbury
Your regional council

Water Level	Depth(m)	Full Drillers Description	Formation Code
Artesian	-1.82m	Surface soil and sand	sp?
	- 14.0m	Blue gravel	sp?
	- 21.3m	Blue sand	ch
	- 27.4m	Blue clay	ch
	- 38.7m	Blue gravel and sand- 1st strata water rise within 1.5m of surface	li
	- 39.3m	Peat and clay	br
	- 40.2m	Brown gravel- water rise within 0.6m of surface	br
	- 43.0m	Blue sand	br
	- 50.9m	Brown sand	br
	- 57.3m	Yellow clay	br
	- 67.7m	Brown sand- 3rd strata water within 0.3m of surface	li
	- 69.5m	Yellow clay	li
	- 80.8m	Brown gravel and sand	li
	- 98.1m	Yellow quick sand	he
	- 103.6m	Yellow clay	he
	- 104.5m	Brown gravel- got water to rise 6.7m at a flow of 24 gals	bu

Borelog for well M36/1048 page 1 of 2

Gridref: M36:815-398 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 6.3 +MSD
Driller : not known
Drill Method : Unknown
Drill Depth : -99.3m Drill Date :

Environment Canterbury
Your regional council

Water Level: Artesian

Depth(m)	Full Drillers Description	Formation Code
-1.20m	Surface soil & sand	sp
-6.00m	Blue shingle	sp
-7.59m	Blue clay	sp
- 15.2m	Blue sand	ch
- 21.3m	Blue shingle	sp
- 27.4m	Blue clay	ch
- 39.6m	Brown shingle	ri
- 40.8m	Blue clay & peat	br
- 42.0m	Brown shingle	br
- 49.9m	Brown sand	br

Scale(m): -5, -10, -15, -20, -25, -30, -35, -40, -45

Borelog for well M36/1048 page 2 of 2

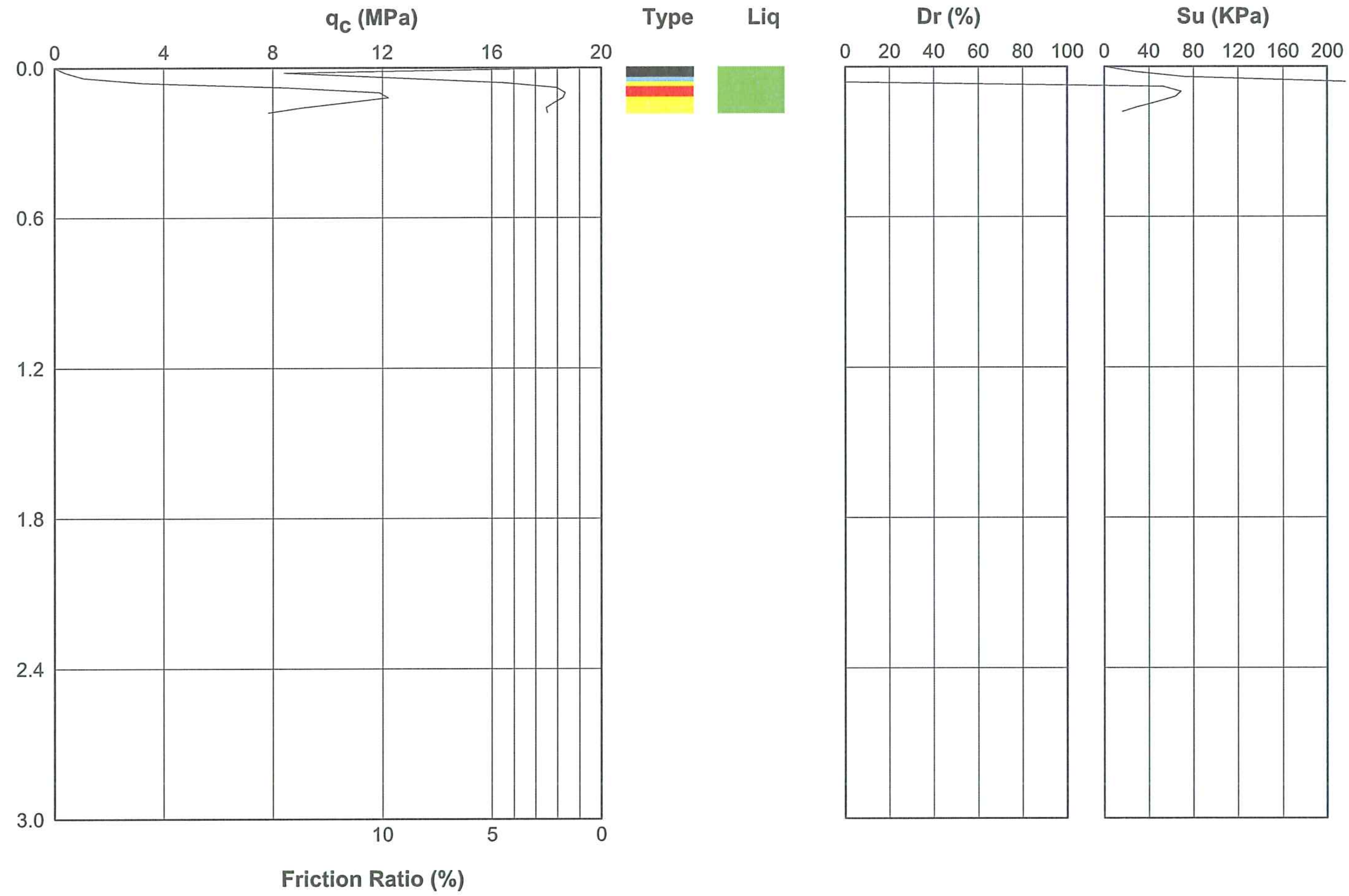
Gridref: M36:815-398 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 6.3 +MSD
Driller : not known
Drill Method : Unknown
Drill Depth : -99.3m Drill Date :

Environment Canterbury
Your regional council

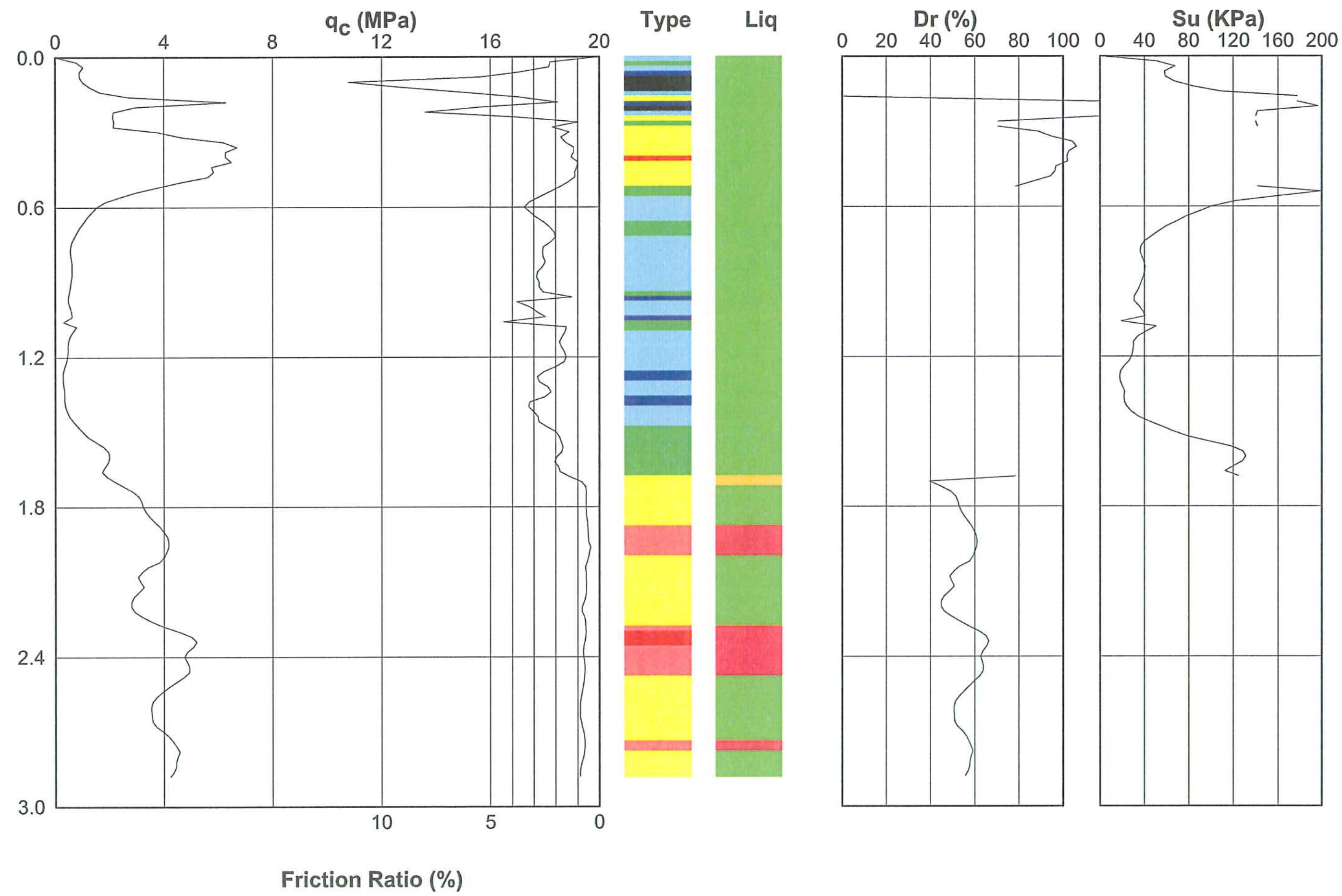
Water Level	Depth(m)	Full Drillers Description	Formation Code
Artesian	49.9m	Brown sand	br
	- 51.8m	Blue sand	br
	- 53.6m	Blue sand & clay	br
	- 56.6m	Blue clay	br
	- 70.1m	Brown shingle	li
	- 76.2m	Blue clay	li-2
	- 84.7m	Brown shingle	li-3
	- 86.2m	Brown sand	he
	- 89.0m	Brown shingle	he
	- 89.9m	Brown sand	he
	- 92.3m	Brown shingle water rises 1.8m	he
	- 95.0m	Yellow clay	he
	- 99.3m	Brown shingle water rises 6.0m	bu

Scale(m): -50, -55, -60, -65, -70, -75, -80, -85, -90, -95

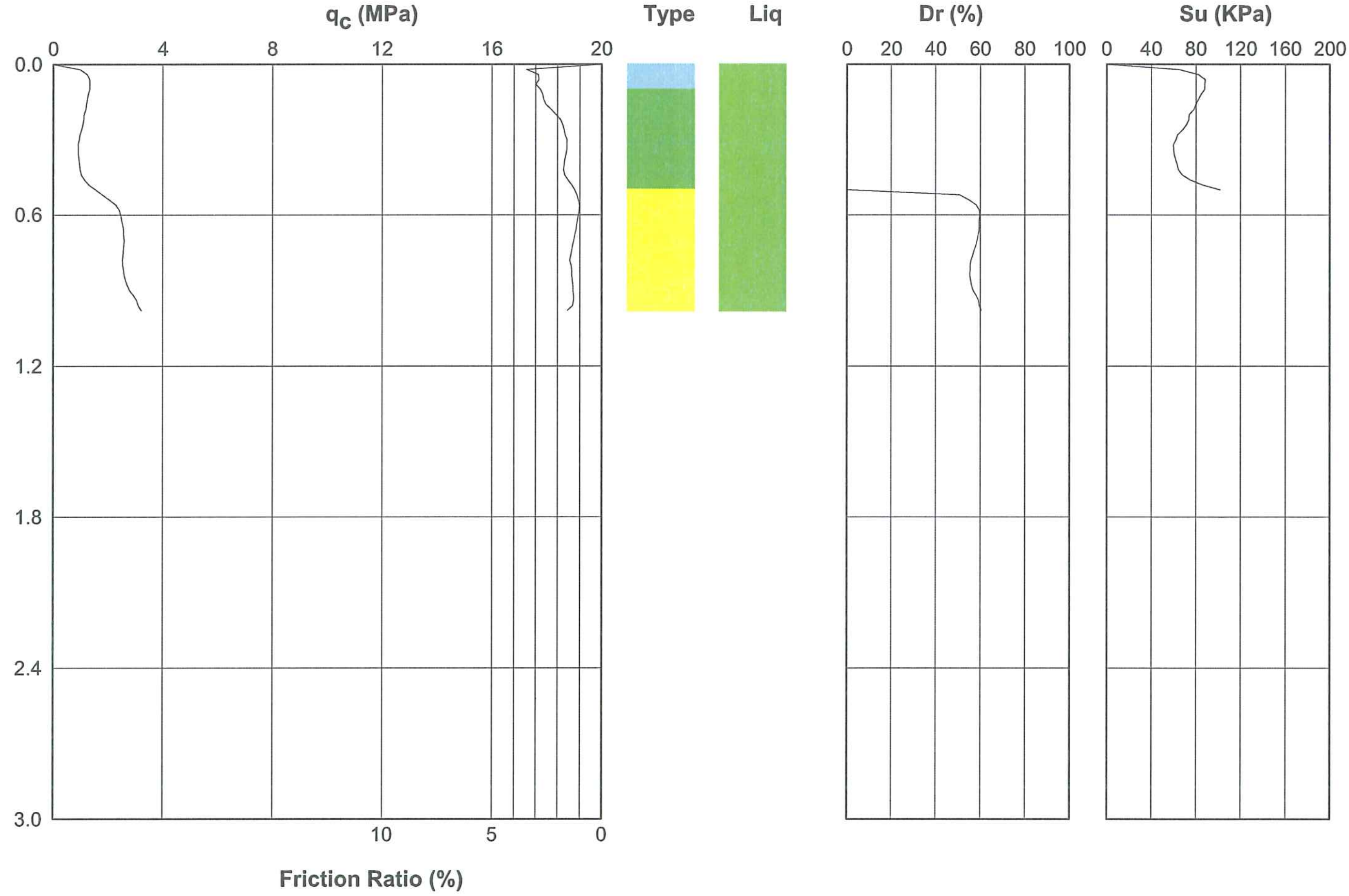
Site 185 Orion Pole Network Investigation Vienna Street



Site 186 Orion Pole Network Investigation

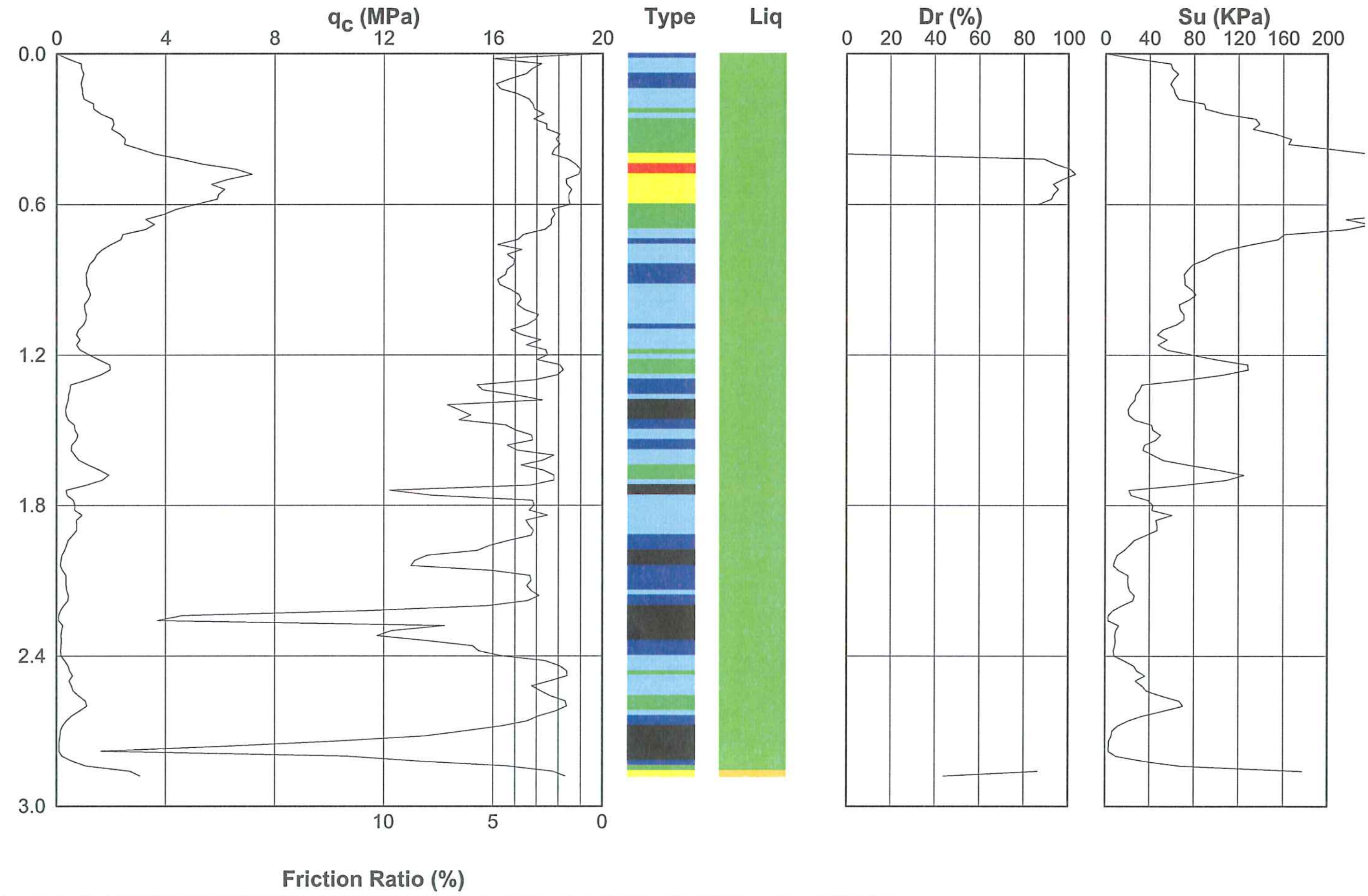


Site 191 Orion Pole Network Investigation



Site 192 Orion Pole Network Investigation

Anglis street 500m south



(A) Block G Tilting 150mm South, liquefaction

(B) Ring beam of foundation exposed unit 4/15 Cecil Court.

(C) Block H possible tilt north.

(D) unit 9/15 cecil court, leaning north.

(E) Ground collapse along eastern elevation

(F) Block I tilting north

(G) Block J subsidence in front of garages.

(H) Heave in asphalt.

CECIL + VIENNA COURTS
GEOTECH SITE VISIT 10/05/11
Graham Brown.

SITE PLAN
APPROX SCALE 1:400
0 5m 10m

Scope of geotech investigations
- Street level readings on all buildings
- BH to 25m.
- 4 x CPT tests
- Shallow Hand Augers to 2.0m depth.
- Test pits

CECIL + VIENNA COURTS
APPROX SCALE 1:500
0 5 10m

Appendix 3 - Methodology and Assumptions

A3.1. 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.

A3.2. 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. 2.2.14B, 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 for all buildings

Concrete masonry block nominal compressive strength, f'_m (MPa)	10
Concrete nominal compressive strength, f'_c (MPa) (1)	25
Mild reinforcing nominal yield strength, f_y (MPa) (2)	275

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.

- Section properties of Concrete Masonry Walls

Table A3: Average weight and equivalent solid thickness of Concrete Masonry Walls

Wall Weights and Areas

(Excerpted from Design of Reinforced Masonry Structures, published by CMACN)

Average Weight of Completed Wall[1] (psf) and Equivalent Solid Thickness (in)

		Hollow Concrete Block												Equivalent Solid Thickness[2] Inches			
		Lightweight 103 pcf				Mediumweight 115 pcf				Normalweight 135 pcf							
Wall Thickness		6"	8"	10"	12"	6"	8"	10"	12"	6"	8"	10"	12"	6"	8"	10"	12"
Solid grouted wall		52	75	93	118	58	78	98	124	63	84	104	133	5.6	7.5	9.6	11.6
vertical cores grouted at	16" o.c.	41	60	69	88	47	63	80	94	52	66	86	103	4.5	5.8	7.2	8.5
	24" o.c.	37	55	61	79	43	58	72	85	48	61	78	94	4.1	5.2	6.3	7.5
	32" o.c.	36	52	57	74	42	55	68	80	47	58	74	89	4.0	4.9	5.9	7.0
	40" o.c.	35	50	55	71	41	53	66	77	46	56	72	86	3.8	4.7	5.7	6.7
	48" o.c.	34	49	53	69	40	45	64	75	45	55	70	83	3.7	4.6	5.5	6.5

[1] The above table gives the average weights of completed walls of various thickness in pounds per square foot of wall face area. An average amount has been added into these values to include the weight of bond beams and reinforcing steel. Weight of grout is assumed at 140 pcf.

[2] Equivalent solid thickness means the calculated thickness of the wall if there were not hollow cores, and is obtained by dividing the volume of solid material in the wall by the face area of the wall. This Equivalent Solid Thickness (EST) is for the determination of area for structural design only, e.g. fs = P/(EST)b. It is NOT to be used to obtain fire ratings. Fire rating thickness is based either on equivalent solid thickness of ungrouted units only or solid grouted walls.

(http://www.angelusblock.com/products/technical_articles_wall_weights.cfm)

- 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 = 1.5 kPa – General Areas
  Q = 0.5 kPa – Non-habitable roof spaces

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

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

  Building seismic weights of different buildings are as follows:
  Apartments = 966 KN
  Launderette = 109 KN
  Garage A = 298 KN
  Garage B = 230 KN

A3.3. Assessment Methodology

Static Analysis

The seismic assessment was undertaken by completing static analysis for the building in accordance with NZS 1170.5:2004.

A 2D model was set up using the structural analysis program ETABS, and effective section properties for structural members were taken from Table A2 above. Diaphragms of the buildings consist of timber sheathing or GIB ceiling and are considered to be flexible diaphragms. Thus lateral load are distributed based on tributary area and are inputted to each individual wall lines.

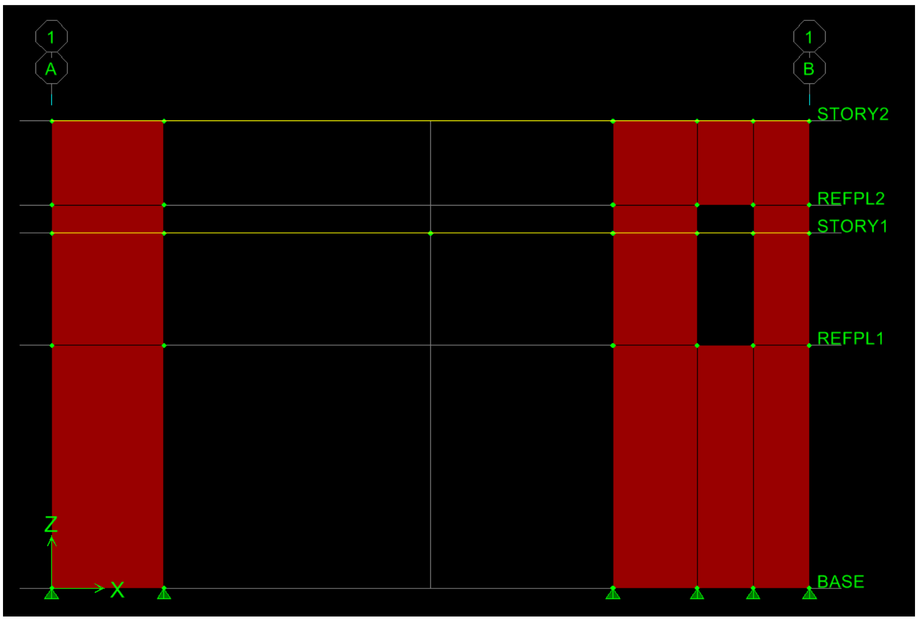


Figure A1: 2D ETABS model of 2-Storey Apartment single front wall section

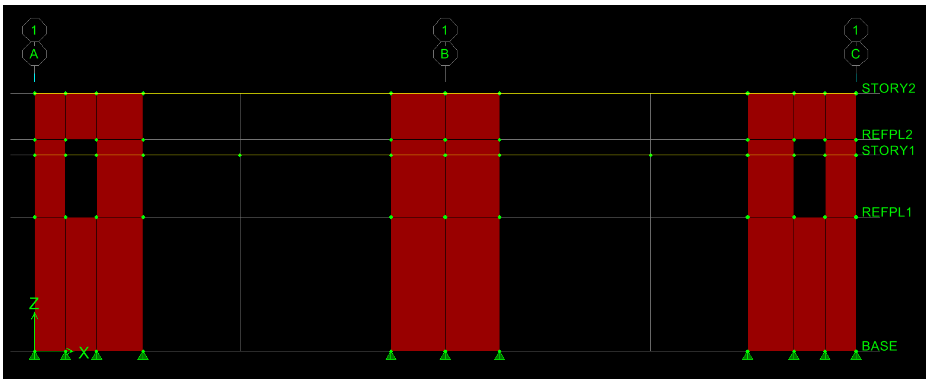


Figure A2: 2D ETABS model of 2-Storey Apartment double front wall section

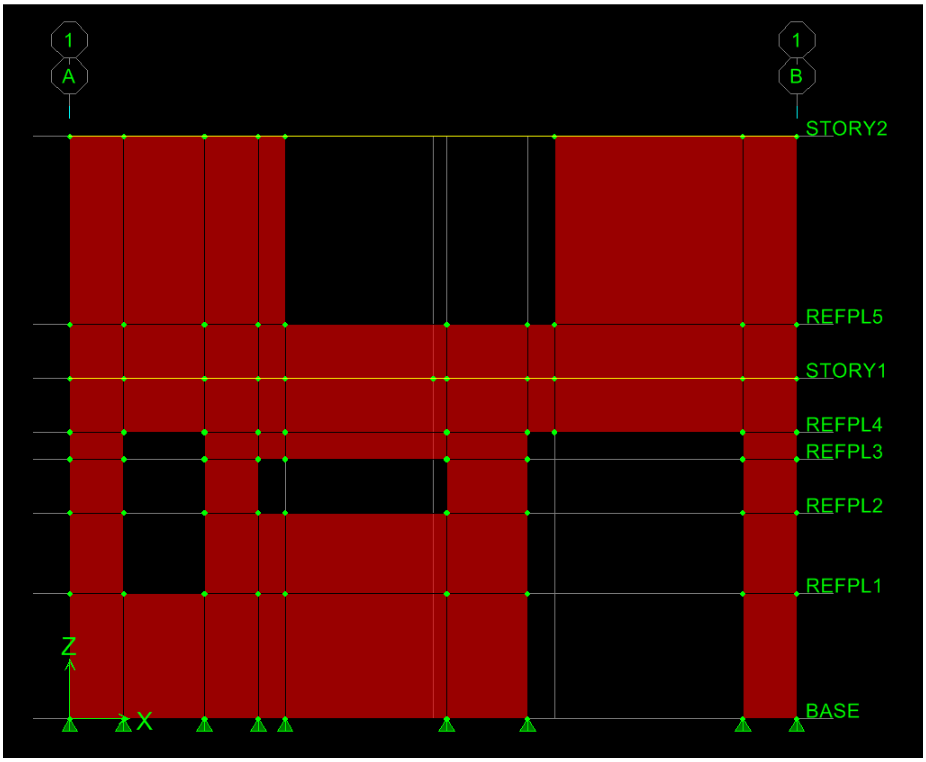


Figure A3: 2D ETABS model of 2-Storey Apartment single rear wall section

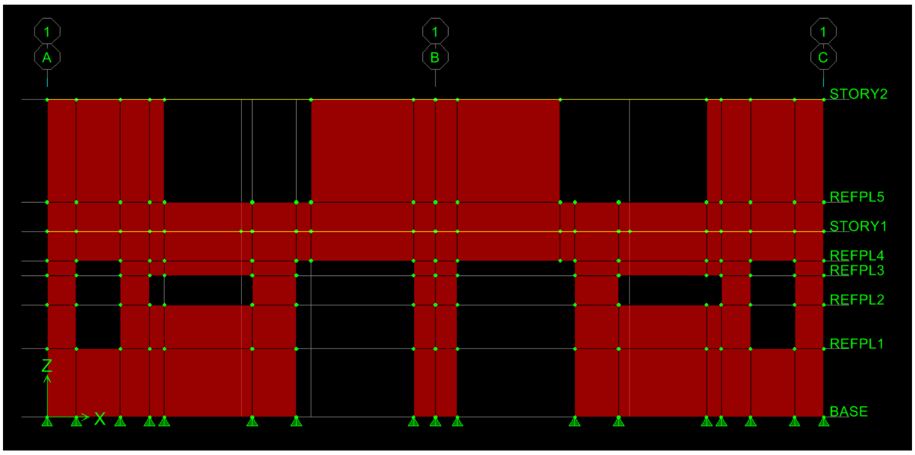


Figure A4: 2D ETABS model of 2-Storey Apartment double rear wall section

The fundamental building periods were assumed to be less than the lower bound limit of 0.4s which is a conservative assumption.

An equivalent static analysis was carried out to perform the seismic assessment of the building. The base shears resulting from the equivalent static method are:

Table A5: Base shear from equivalent static method

Building	Base shear -E/W direction (KN)	Base shear –N/S direction (KN)
Apartment	703	703
Launderette	79	79
Garage A	217	217
Garage B	167	167

The building was analysed as having limited ductility (μ = 1.25) and the design actions were applied separately in each perpendicular direction.

Element Demand to Capacity

Element force demands were extracted from the equivalent static 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.

Out of plane loading of masonry walls

To assess the capacity of the masonry walls to resist out-of-plane loading without the aid of roof and/or floor diaphragm, 2D models of various walls were created using SAP2000 analysis software. Uniform face loading was applied to the walls. Resulting forces in the walls are obtained from the software and compared to the calculated strength of the walls.

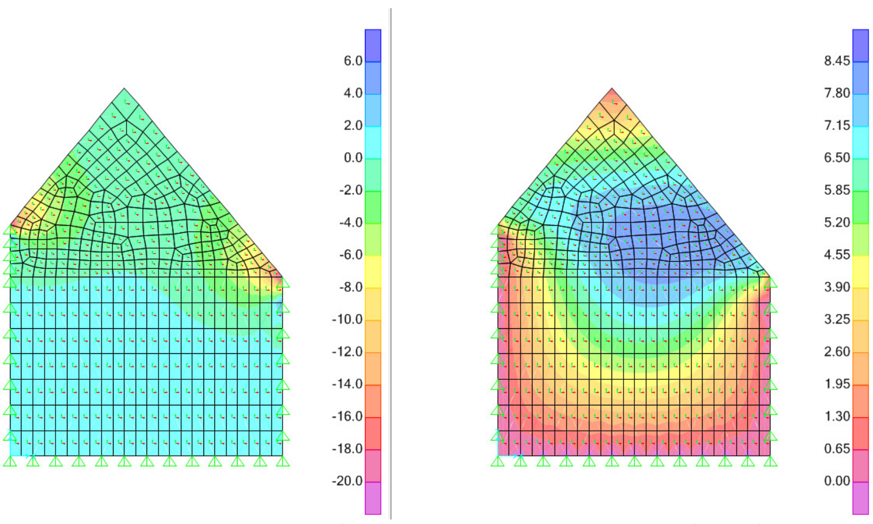


Figure A5: Out-of-plane analysis for transverse walls at Residential Blocks

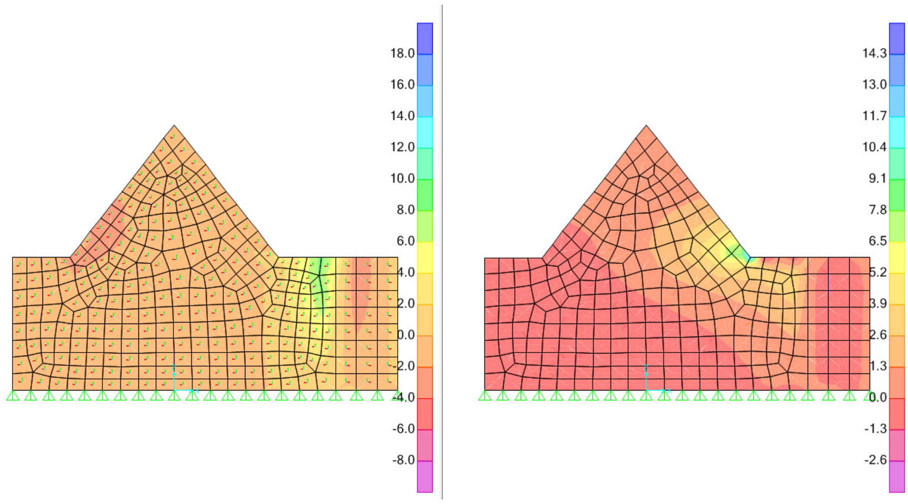


Figure A5: Out-of-plane analysis for transverse walls at Garage Blocks

Appendix 4 – CERA DEE Spreadsheet

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Cecil Courts - Residential	Reviewer:	Alistair Boyce
Building Address: Unit / No: / Street	16 Cecil Place	CPEng No:	209860
Legal Description:		Company:	Opus International
		Company project number:	QUCC1.92
		Company phone number:	33635400
GPS south: (Degrees / Min / Sec)		Date of submission:	23/11/2012
GPS east:		Inspection Date:	11/07/2012
		Revision:	Final
Building Unique Identifier (CCC):	BE 1047 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	silty sand	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	

Building

No. of storeys above ground:	2	single storey = 1	Ground floor elevation (Absolute) (m):	0.00
Ground floor split?	no		Ground floor elevation above ground (m):	
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	
Building height (m):	5.00		height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	110			
Age of Building (years):	40		Date of design:	1965-1976
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	multi-unit residential		Brief strengthening description:	
Use (upper floors):	multi-unit residential			
Use notes (if required):				
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	Timber purlins
Floors:	timber	joist depth and spacing (mm)	
Beams:	timber	type	
Columns:	other (note)	typical dimensions (mm x mm)	
Walls:	fully filled concrete masonry	#N/A	

Lateral load resisting structure

Lateral system along:	fully filled CMU	**Note: Define along and across in detailed report!**	note total length of wall at ground (m):	26
Ductility assumed, μ:	1.25		wall thickness (m):	0.6
Period along:	0.40	##### enter height above at H31	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	fully filled CMU		note total length of wall at ground (m):	8
Ductility assumed, μ:	1.25		wall thickness (m):	0.6
Period across:	0.40	##### enter height above at H31	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

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

Non-structural elements

Stairs:	timber	describe supports	
Wall cladding:	other light	describe	
Roof Cladding:	Metal	describe	
Glazing:	timber frames		
Ceilings:	strapped or direct fixed		gib ceiling
Services(list):			

Available documentation

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

Damage

Site: (refer DEE Table 4-2)

Site performance:	Poor	Describe damage:	
Settlement:	25-100m	notes (if applicable):	
Differential settlement:	1:150 or more	notes (if applicable):	
Liquefaction:	2-5 m²/100m³	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	moderate to substantial (1 in 5)	notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	yes	Describe:	Discontinuous shear wall
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

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

Recommendations

	Level of repair/strengthening required:	significant structural and strengthening	Describe:	as described in report
	Building Consent required:	yes	Describe:	
	Interim occupancy recommendations:	partial occupancy	Describe:	
Along	Assessed %NBS before:	43% ##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Quantitative
	Assessed %NBS after:	43%		
Across	Assessed %NBS before:	50% ##### %NBS from IEP below		
	Assessed %NBS after:	50%		

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Cecil Courts - Garage	Reviewer:	Alistair Boyce
	Unit No: / Street	CPEng No:	209860
Building Address:	16 / Cecil Place	Company:	Opus International
Legal Description:		Company project number:	6-QUCC1.92
		Company phone number:	33635400
	Degrees / Min / Sec		
GPS south:		Date of submission:	23/11/2012
GPS east:		Inspection Date:	11/07/2012
		Revision:	Final
Building Unique Identifier (CCC):	BE 1047 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	silty sand	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	0.00
Ground floor split?	no		Ground floor elevation above ground (m):	
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	
Building height (m):	4.50		height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	93			
Age of Building (years):	40		Date of design:	1965-1976
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	parking		Brief strengthening description:	
Use (upper floors):	other (specify)			
Use notes (if required):				
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	steel truss	truss depth, purlin type and cladding	gang nail truss, 2.2m deep
Floors:	other (note)	describe sytem	Concrete slab on ground
Beams:	cast-insitu concrete	overall depth x width (mm x mm)	
Columns:	timber	typical dimensions (mm x mm)	
Walls:	fully filled concrete masonry	#N/A	

Lateral load resisting structure

Lateral system along:	fully filled CMU	**Note: Define along and across in**	note total length of wall at ground (m):	18
Ductility assumed, μ:	1.25	**detailed report!**	wall thickness (m):	0.6
Period along:	0.40	##### enter height above at H31	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	fully filled CMU		note total length of wall at ground (m):	20
Ductility assumed, μ:	1.25		wall thickness (m):	0.6
Period across:	0.40	##### enter height above at H31	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

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

Non-structural elements

Stairs:	other (specify)	describe	no stair
Wall cladding:	exposed structure	describe	
Roof Cladding:	Metal	describe	
Glazing:	other (specify)		
Ceilings:	none		
Services(list):			

Available documentation

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

Damage

Site:
(refer DEE Table 4-2)

Site performance:	Poor	Describe damage:	
Settlement:	25-100m	notes (if applicable):	
Differential settlement:	1:150 or more	notes (if applicable):	
Liquefaction:	2-5 m²/100m³	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	moderate to substantial (1 in 5)	notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

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

Recommendations

	Level of repair/strengthening required:	significant structural and strengthening	Describe:	as described in report
	Building Consent required:	yes	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	
Along	Assessed %NBS before:	37% ##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Quantitative
	Assessed %NBS after:	37%		
Across	Assessed %NBS before:	37% ##### %NBS from IEP below		
	Assessed %NBS after:	37%		

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Cecil Courts - Laundry	Reviewer:	Alistair Boyce
	Unit No: Street	CPEng No:	209860
Building Address:	16 Cecil Place	Company:	Opus International
Legal Description:		Company project number:	6-QUCC1.92
		Company phone number:	3635400
	Degrees Min Sec		
GPS south:		Date of submission:	23/11/2012
GPS east:		Inspection Date:	11/07/2012
		Revision:	Final
Building Unique Identifier (CCC):	BE 1047 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	silty sand	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	0.00
Ground floor split?	no		Ground floor elevation above ground (m):	
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	
Building height (m):	4.00		height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	40			
Age of Building (years):	40		Date of design:	1965-1976
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	other (specify)		Brief strengthening description:	
Use (upper floors):				
Use notes (if required):	Residential laundry			
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber truss	truss depth, purlin type and cladding	
Floors:	other (note)	describe sytem	Concrete slab on ground
Beams:	cast-insitu concrete	overall depth x width (mm x mm)	
Columns:	timber	typical dimensions (mm x mm)	
Walls:	fully filled concrete masonry	#N/A	

Lateral load resisting structure

Lateral system along:	fully filled CMU	**Note: Define along and across in detailed report!**	note total length of wall at ground (m):	12
Ductility assumed, μ:	1.25		wall thickness (m):	0.6
Period along:	0.40	##### enter height above at H31	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	fully filled CMU		note total length of wall at ground (m):	7
Ductility assumed, μ:	1.25		wall thickness (m):	0.6
Period across:	0.40	##### enter height above at H31	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

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

Non-structural elements

Stairs:	other (specify)	describe	no stair
Wall cladding:	exposed structure	describe	
Roof Cladding:	Metal	describe	
Glazing:	other (specify)		
Ceilings:	none		
Services(list):			

Available documentation

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

Damage

Site:
(refer DEE Table 4-2)

Site performance:	Poor	Describe damage:	
Settlement:	25-100m	notes (if applicable):	
Differential settlement:	1:150 or more	notes (if applicable):	
Liquefaction:	2-5 m²/100m³	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	moderate to substantial (1 in 5)	notes (if applicable):	

Building:

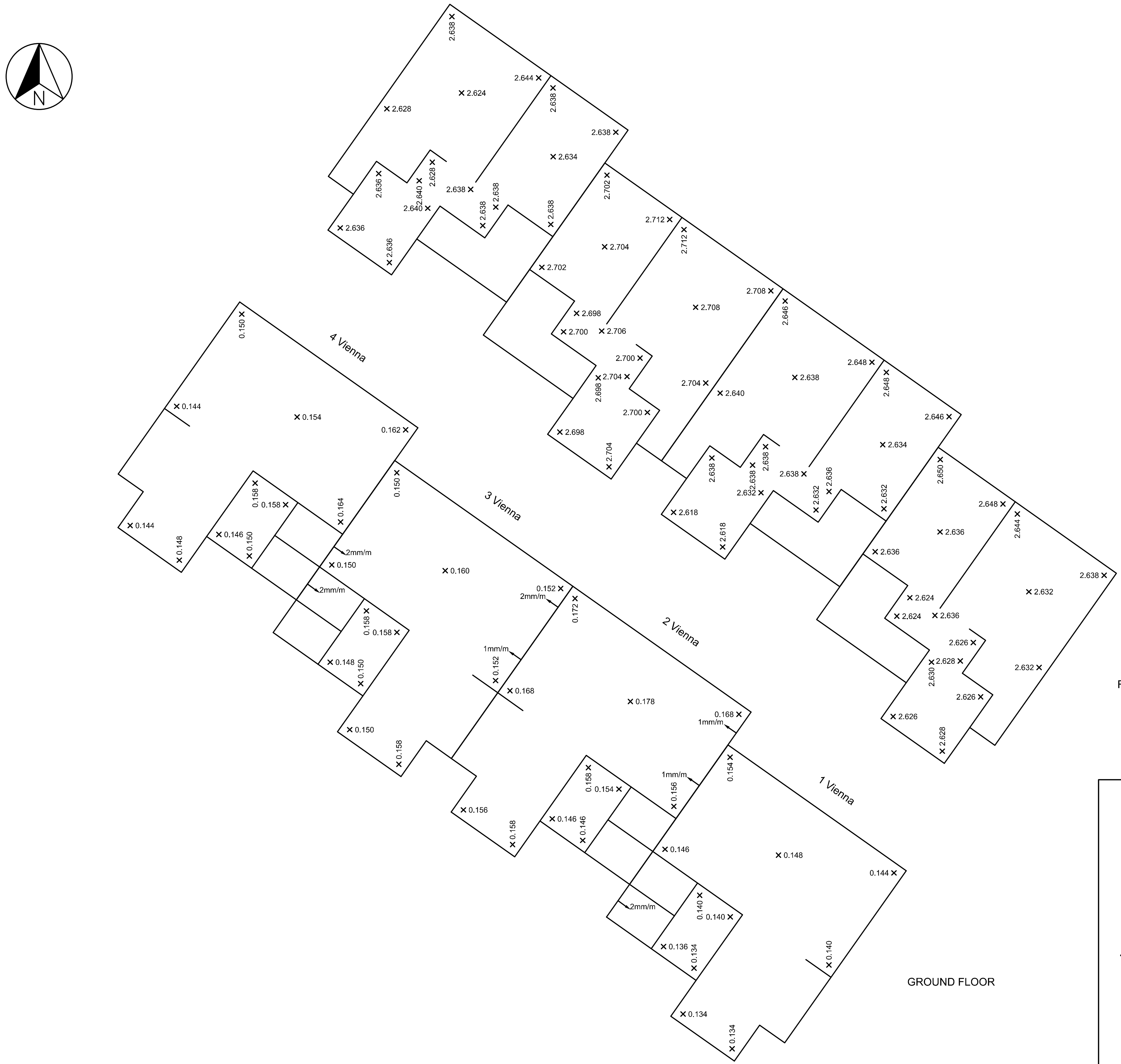
	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

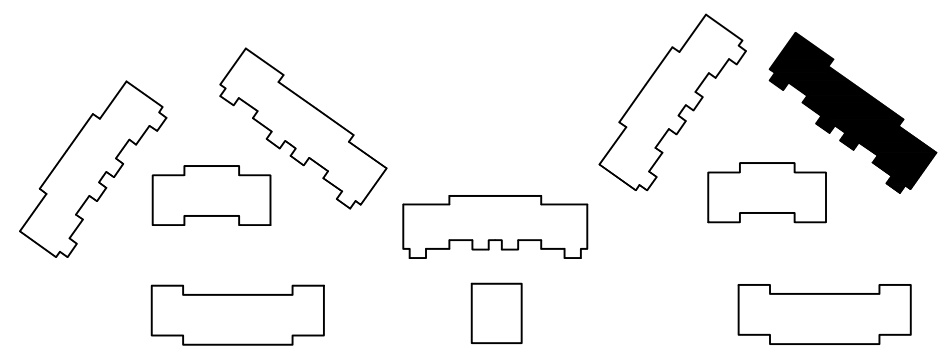
$$Damage\_Ratio = \frac{(\%NBS\,(before) - \%NBS\,(after))}{\%NBS\,(before)}$$

Recommendations

	Level of repair/strengthening required:	significant structural and strengthening	Describe:	as described in report
	Building Consent required:	yes	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	
Along	Assessed %NBS before:	39% ##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Quantitative
	Assessed %NBS after:	39%		
Across	Assessed %NBS before:	38% ##### %NBS from IEP below		
	Assessed %NBS after:	38%		

Appendix 5 – Level Survey





Revision	Amendment	Approved	Revision Date

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Drawn	Designed	Approved	Revision Date
B JARRATT		S BECKER	28/09/2012

Project No.	Scale
6QUCC1.92	1:50 @ A1 & 1:100 @ A3

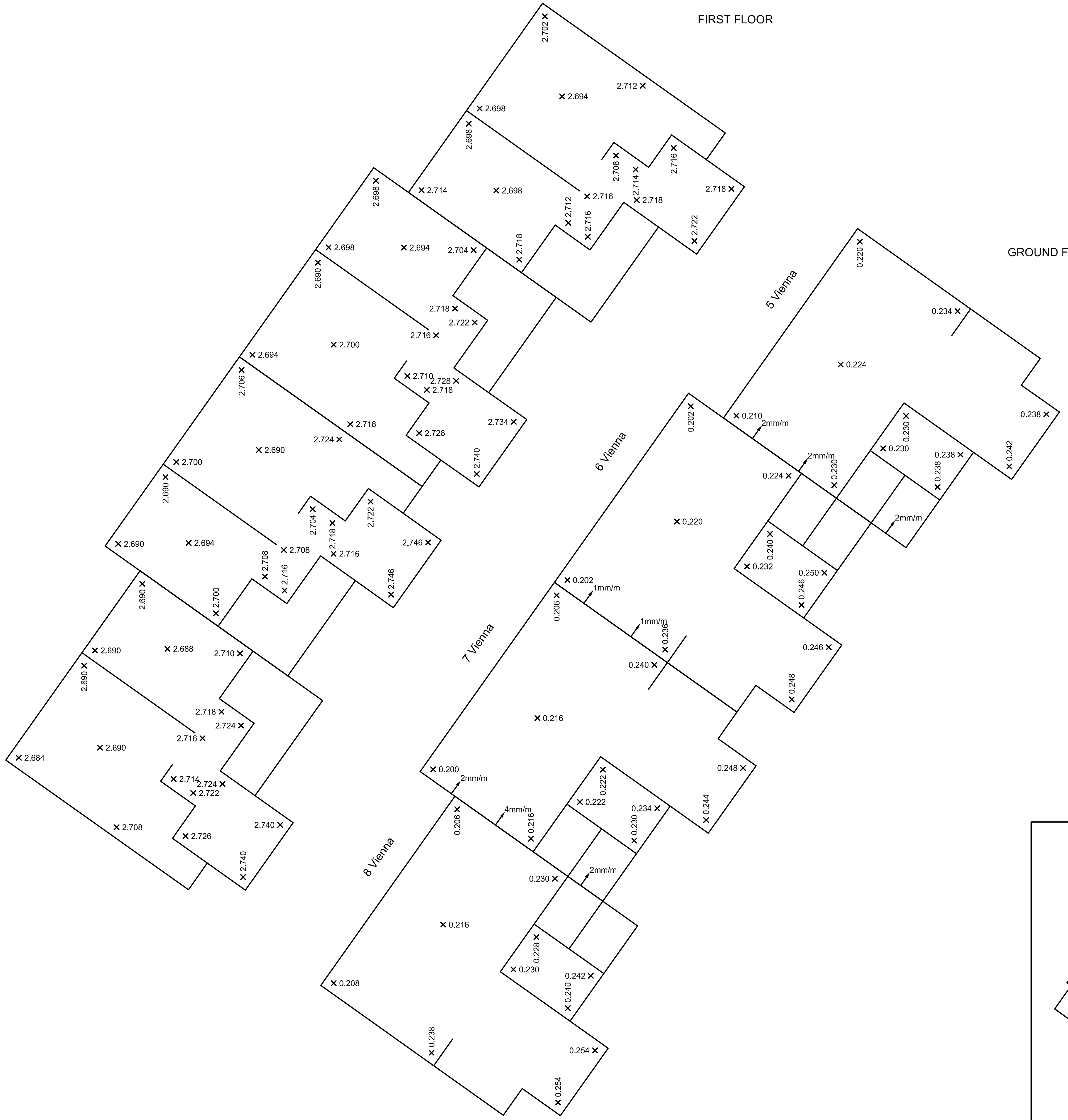
Project
CHRISTCHURCH CITY COUNCIL
CECIL COURTS

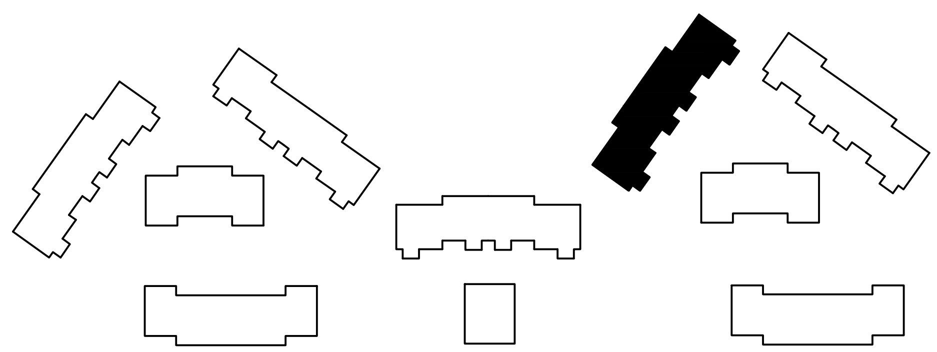
Sheet
FLOOR LEVEL SURVEY
1 TO 4 VIENNA STREET

Drawing No.	Sheet. No.	Revision
6/1366/299/2604	1/7	R0

1:50 @ A1 / 1:100 @ A3 — 0 1 2 3 4 5 m

Original Sheet Size A1 [841x594] Plot Date 10/10/12 @ 14:46 p:\projects\6-quake.01\ccc_residential units\cecil & vienna court\survey\6qucc1.92 cecil courts floor level survey\c3d\6qucc1.92 cecil street.dwg - 1-4 Vienna





1:50 @ A1 0 1 2 3 4 5 m
1:100 @ A3

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Project: CHRISTCHURCH CITY COUNCIL
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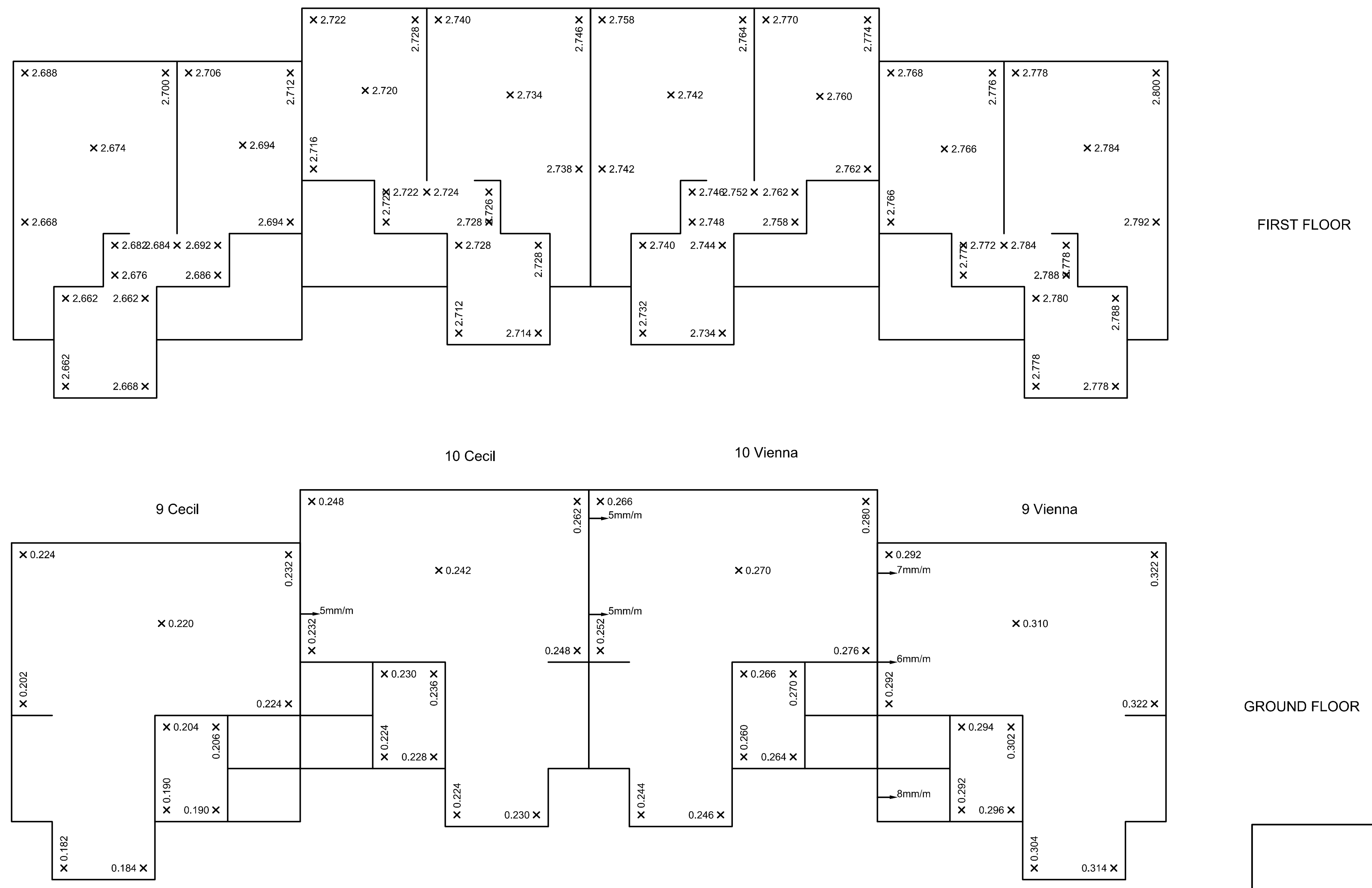
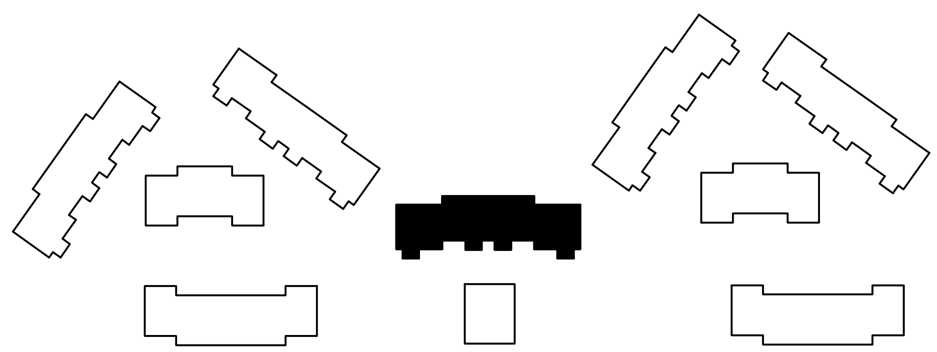
Sheet: FLOOR LEVEL SURVEY
5 TO 8 VIENNA STREET

Project No.	Scale	Drawing No.	Sheet. No.	Revision
6QUCC1.92	1:50 @ A1 & 1:100 @ A3	6/1366/299/2604	2/7	R0

Original Sheet Size A1 [841x594] Plot Date 10/10/12 @ 14:46 p:\projects\6-quake.01\ccc_residential units\cecil & vienna court\survey\6qucc1.92 cecil courts floor level survey\c3d\6qucc1.92 cecil street.dwg - 5-8 Vienna





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CHRISTCHURCH CITY COUNCIL
CECIL COURTS

Sheet
FLOOR LEVEL SURVEY
9 & 10 VIENNA & CECIL STREET

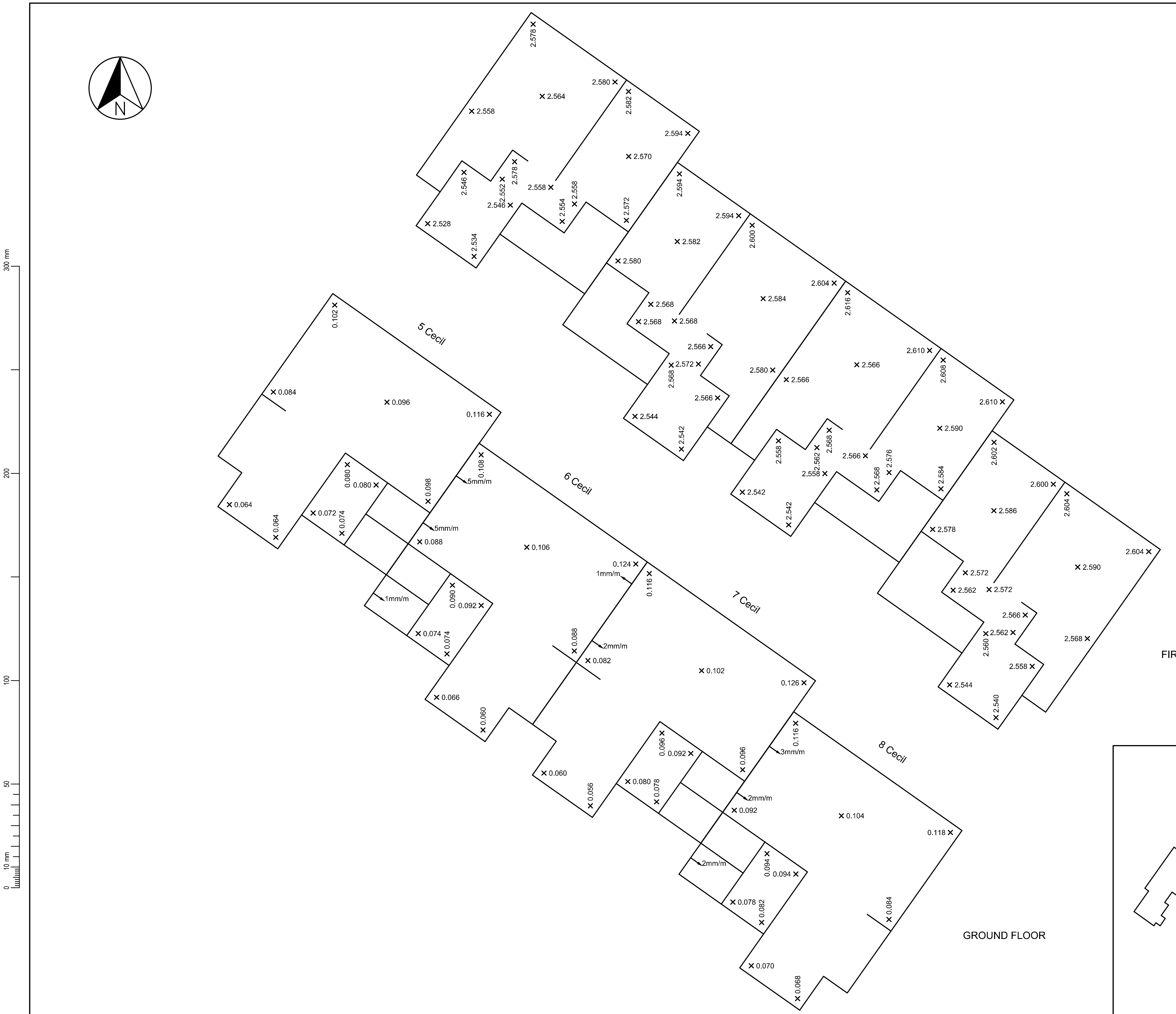
Drawn	Designed	Approved	Revision Date
B JARRATT		S BECKER	28/09/2012

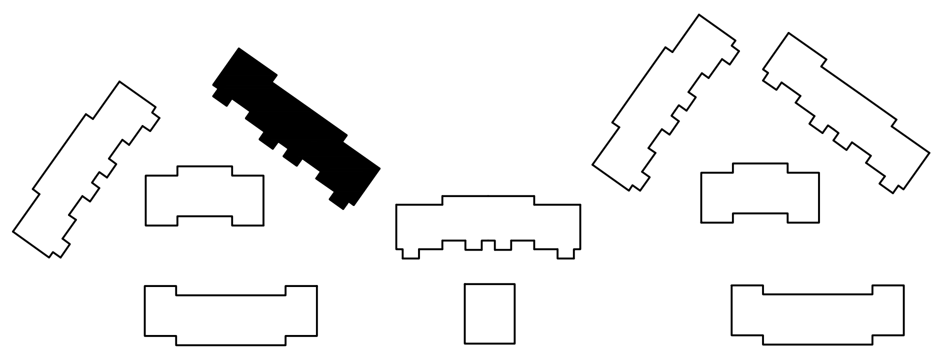
Project No.	Scale	Drawing No.	Sheet. No.	Revision
6QUCC1.92	1:50 @ A1 & 1:100 @ A3	6/1366/299/2604	3/7	R0

1:50 @ A1
1:100 @ A3
0 1 2 3 4 5 m

300 mm 200 100 50 10 mm 0

Original Sheet Size A1 [841x594] Plot Date 10/10/12 @ 14:46 p:\projects\6-quake.01\ccc_residential units\cecil & vienna court\survey\6qucc1.92 cecil courts floor level survey\c3d\6qucc1.92 cecil street.dwg - 9-10 Vienna & Cecil





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CHRISTCHURCH CITY COUNCIL
CECIL COURTS

Sheet
FLOOR LEVEL SURVEY
5 TO 8 CECIL STREET

Drawn	Designed	Approved	Revision Date
B JARRATT		S BECKER	28/09/2012

Project No.	Scale	Drawing No.	Sheet. No.	Revision
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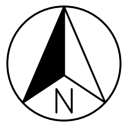

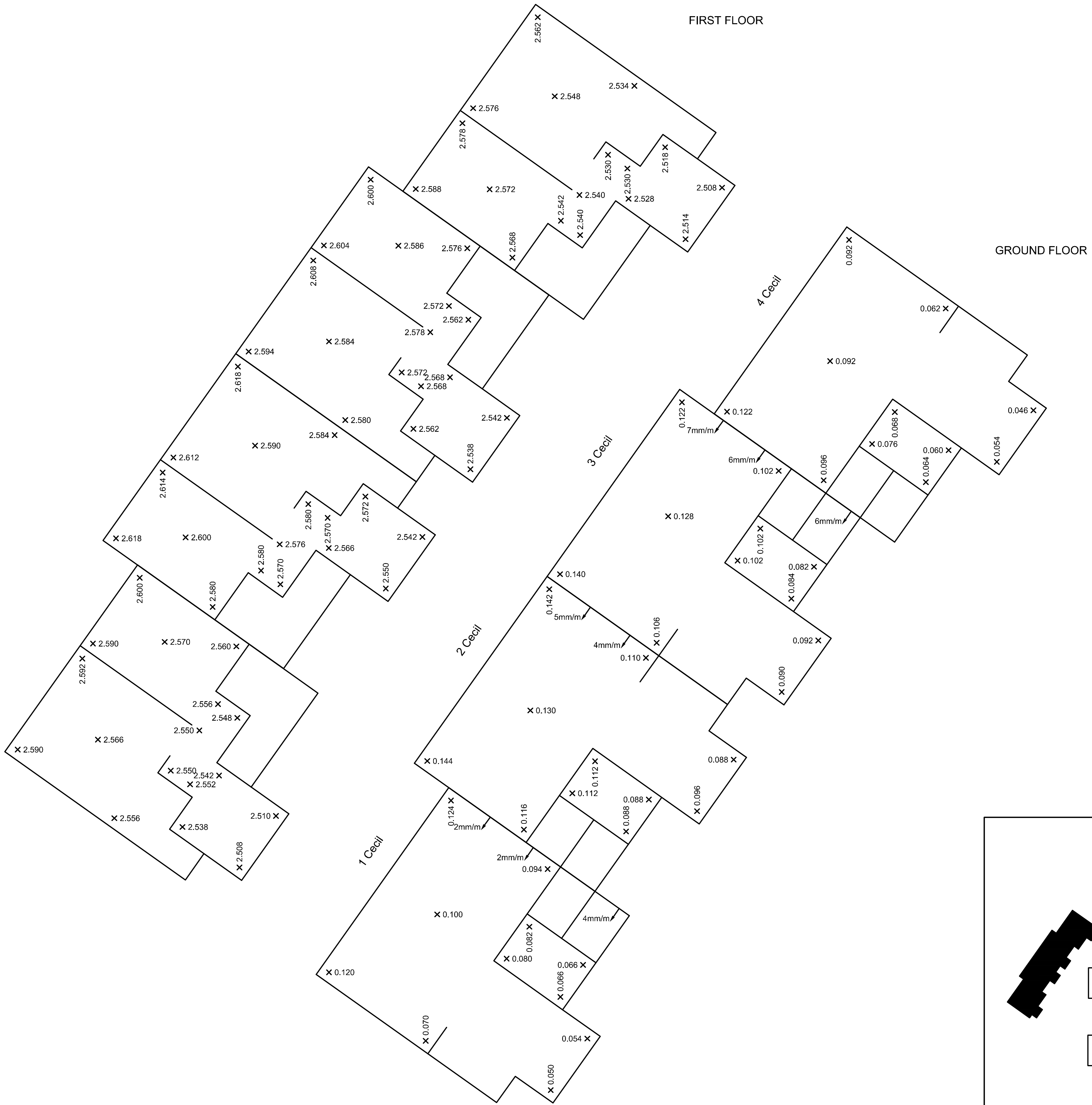

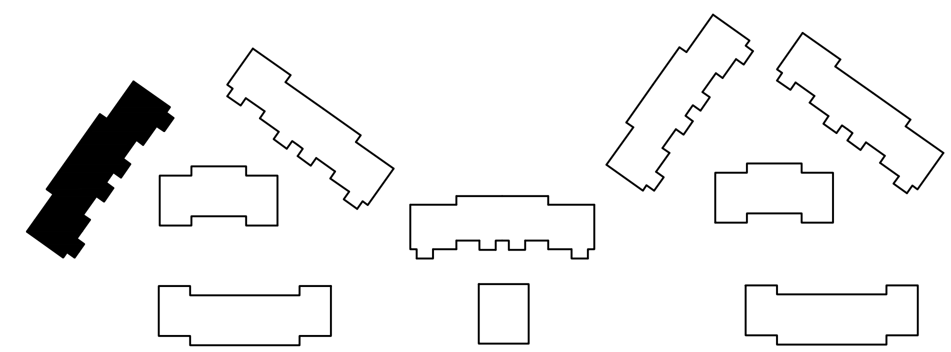

1:50 @ A1 / 1:100 @ A3 — 0 1 2 3 4 5 m

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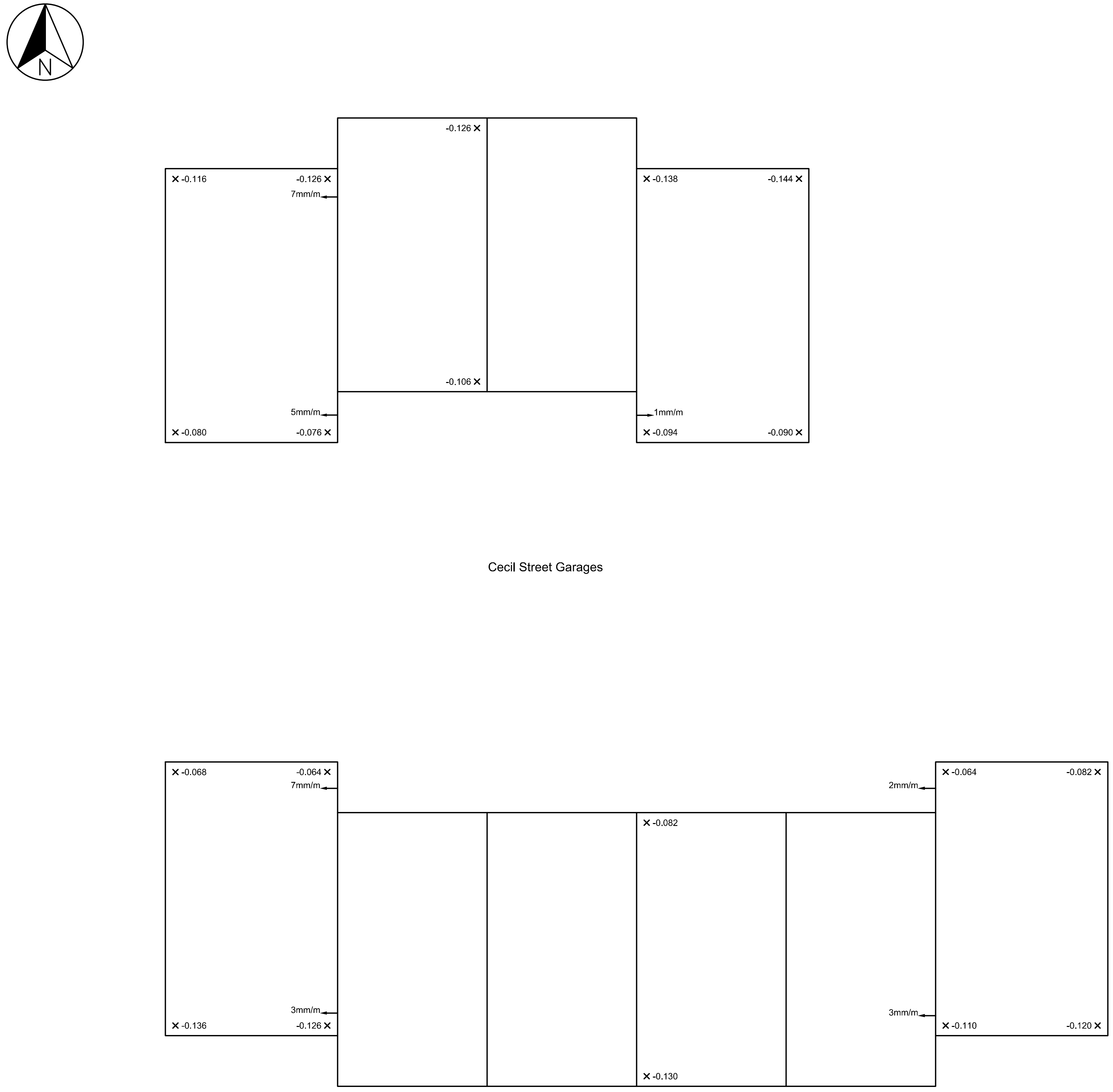
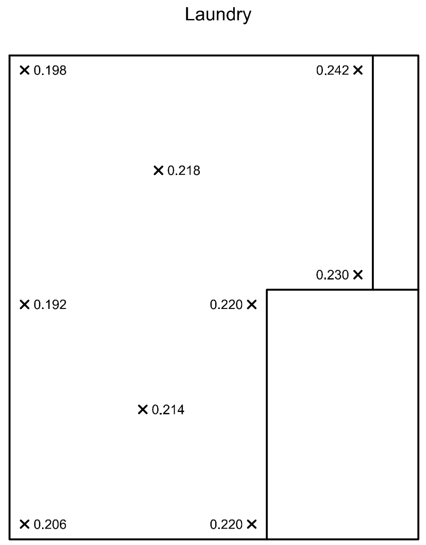
CECIL COURTS

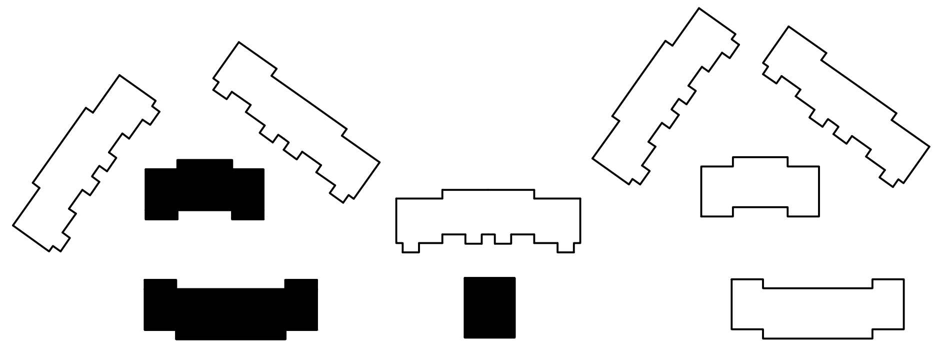
Sheet: FLOOR LEVEL SURVEY
1 TO 4 CECIL STREET

Project No.	Scale	Drawing No.	Sheet. No.	Revision
6QUCC1.92	1:50 @ A1 & 1:100 @ A3	6/1366/299/2604	5/7	R0

Original Sheet Size A1 [841x594] Plot Date 10/10/12 @ 14:46 p:\projects\6-quake.01\ccc_residential units\cecil & vienna court\survey\6qucc1.92 cecil courts floor level survey\c3d\6qucc1.92 cecil street.dwg - 1-4 Cecil







Revision	Amendment	Approved	Revision Date

1:50 @ A1 / 1:100 @ A3 — 0 1 2 3 4 5 m

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CECIL COURTS

Sheet: FLOOR LEVEL SURVEY
CECIL STREET GARAGES AND LAUNDRY

Drawn	Designed	Approved	Revision Date
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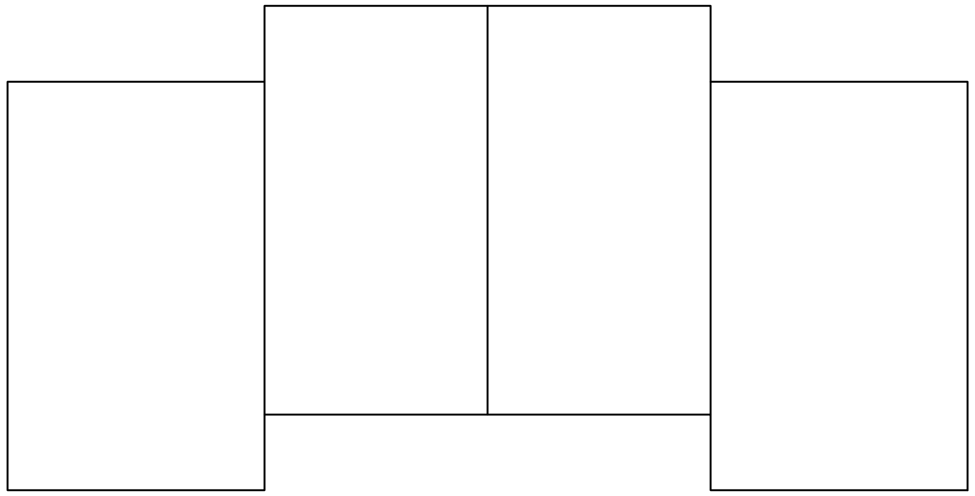
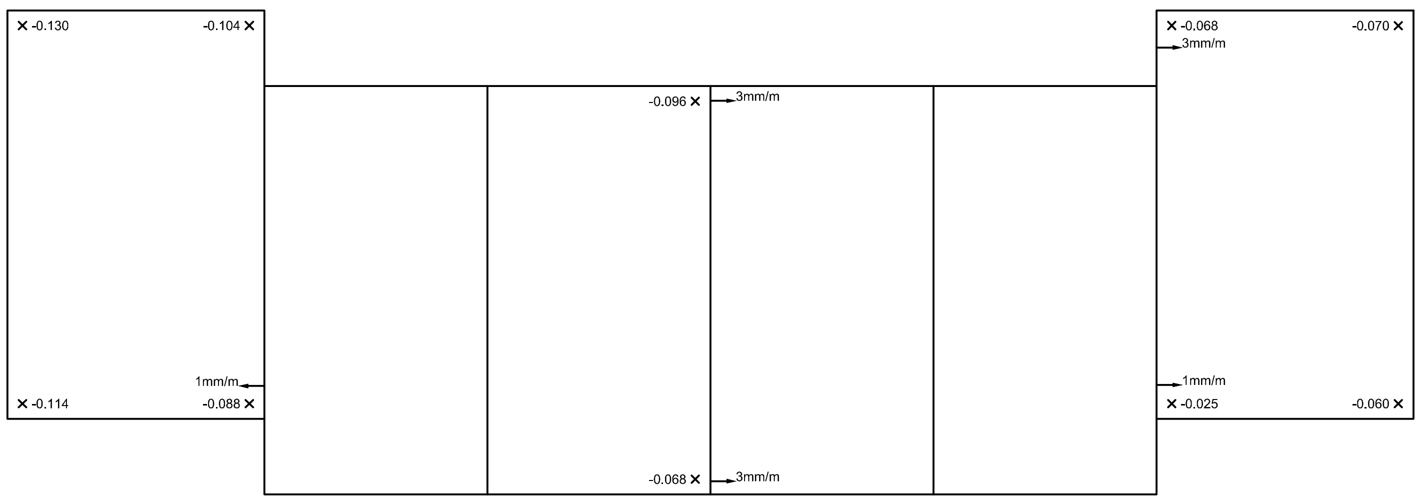
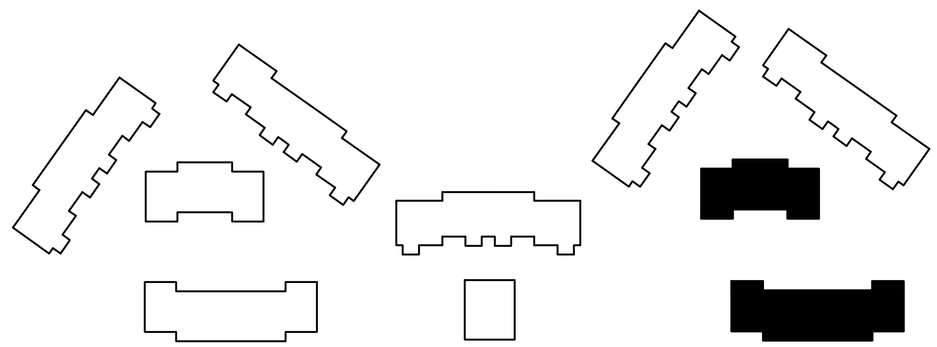
Project No.	Scale	Drawing No.	Sheet. No.	Revision
6QUCC1.92	1:50 @ A1 & 1:100 @ A3	6/1366/299/2604	6/7	R0

Original Sheet Size A1 [841x594] Plot Date 10/10/12 @ 14:46 p:\projects\6-quake.01\ccc_residential units\cecil & vienna court\survey\6qucc1.92 cecil courts floor level survey\c3d\6qucc1.92 cecil street.dwg - Cecil Garages and Laundry



Vienna Street Garages





1:50 @ A1
1:100 @ A3
0 1 2 3 4 5 m

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CECIL COURTS

Sheet:

FLOOR LEVEL SURVEY
VIENNA STREET GARAGES

Project No.	Scale	Drawing No.	Sheet. No.	Revision
6QUCC1.92	1:50 @ A1 & 1:100 @ A3	6/1366/299/2604	7/7	R0

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