Tommy Taylor Courts

BU 1048-001 EQ2

Detailed Engineering Evaluation
Quantitative Assessment Report

7 Cecil Place, Waltham, Christchurch

Christchurch City Council

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Detailed Engineering Evaluation
Quantitative Report - SUMMARY
Final V2

Background

This is a summary of the quantitative report for the building structure, 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) Significant liquefaction has occurred throughout the site as evident in the ground displacement in the carpark as well as the north wing.

b) The floor level survey indicates the slope in the ground floor slab in the west wing is up to 40mm over 4.5m which is considered excessive.

c) The north wing has tilted approximately 100mm towards the north leaving a separation at the intersections between the two wings. The tilt created a large gap in the exterior sheathing along the full height of the building.

d) Some minor cracking of exterior finishes typically around window or door openings.

e) Vertical crack of the block wall was observed in the storage room at the southern end of the west wing.

f) Vertical cracks were observed at the junction between the balcony side wall and the building exterior walls.

g) Minor cracks to the interior linings were around corners of window or door openings.

h) Ceiling has partially dropped at unit 13 at the junction between the north wing and the west wing. Daylight can be seen from interior of unit 13 where the north and west wing have been separated.

i) A minor crack was observed below one of the precast concrete stair.

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) Separation between the two wings: The north wing and west wing are not structurally tied together at the floor levels. However, some roof framing members appear to bridge between the two wings. Some localised damage of the roof framing near the intersection is likely.

b) Discontinuous walls: There are several concrete masonry walls at the west wing that are not continuous to the foundation. Seismic overturning forces in these walls impose additional loads and can overwhelm supporting elements. The following summarises where discontinuous walls occur:

- The concrete block bearing wall at the 1st floor between unit 21 and the stairwell is not continuous to the foundation. Instead, it is supported by a concrete beam at 1st floor and perpendicular concrete block walls. The limiting component is the concrete block header above the entry door and the 1st floor concrete beam. The drawings show a notch in the bottom of this beam which appears to be for a sprinkler pipe.

- Slender wall pier at the 1st floor between the dining area and the kitchen of unit 21 is offset from the wall pier at the ground floor below. The wall above is supported on the 1st floor concrete beam. The drawings also indicate a notch in the bottom of this beam which appears to be for a sprinkler pipe.

- Wall pier above the entry door into unit 19 is discontinuous at the ground floor. The wall pier and the 1st floor beam are supported on the block header above the window/door at entry into the unit.

- The exterior wall at the 1st floor at the store room between units 15 and 16 (above the laundry room) is discontinuous.

Indicative Building Strength

a) Based on the information available, and from undertaking a quantitative assessment, the buildings have been assessed to have overall capacities in excess of 33% NBS. The capacities of the 3-storey structures are generally governed by shear strength of the fully grouted block walls. The capacity of the 2-storey portion of the west wing is governed by the overturning of the 1st floor timber walls. This capacity level implies the buildings are considered a moderate risk but their seismic performance is legally accepted under the 2004 Building Act. The %NBS for each building is summarised below:

Building Area	%NBS
West wing (3-storey portion)	34 - 50%
West wing (2-storey portion)	50 - 60%
North wing	35 - 50%

Although the overall building capacity exceeds 33%NBS, there are some localised areas of concern within the 3-storey portion of the west wing where concrete block walls are not continuous to the foundation (see Critical Structural Weakness" section above). Some of the elements supporting these discontinuous shear walls have marginal capacity to resist gravity loading. Earthquake loading impose additional loads and can overwhelm these elements. The computed strength of one such element is approximately 40% NBS.

Recommendations

a) Perform remedial strengthening to support discontinuous shear walls in the west wing. We recommend that these local areas of concern be addressed promptly. This work will all occur in unit 19 at the ground floor.

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.

e) These buildings are not considered earthquake prone and may be occupied per the 2004 Building Act.

Contents

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council to undertake a detailed seismic assessment of the Tommy Taylor Courts, located at 7 Cecil Place, located at 149 Main South Road, Christchurch following the M6.3 Christchurch earthquake on 22 February 2011.

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

The seismic assessment and reporting have been undertaken based on the qualitative and quantitative procedures detailed in the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011.

2 Compliance

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

2.1 Canterbury Earthquake Recovery Authority (CERA)

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

Section 38 – Works

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

Section 51 – Requiring Structural Survey

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

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

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

1. The importance level and occupancy of the building.

2. The placard status and amount of damage.

3. The age and structural type of the building.

4. Consideration of any critical structural weaknesses.

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

2.2 Building Act

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

Section 112 - Alterations

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

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

Section 115 – Change of Use

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

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

Section 121 – Dangerous Buildings

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

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

Section 122 – Earthquake Prone Buildings

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

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

Section 124 – Powers of Territorial Authorities

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

Section 131 – Earthquake Prone Building Policy

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

2.3 Christchurch City Council Policy

Christchurch City Council adopted their Earthquake Prone, Dangerous and Insanitary Building Policy in October 2011 following the Darfield Earthquake on 4 September 2010.

The policy 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:

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

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

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

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

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

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

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

3 Earthquake Resistance Standards

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

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

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

Improvement of Structural Performance	
Legal Requirement	NZSEE Recommendation
The Building Act sets no required level of structural improvement (unless change in use) This is for each TA to decide. Improvement is not limited to 34%NBS.	100%NBS desirable. Improvement should achieve at least 67%NBS
	Not recommended. Acceptable only in exceptional circumstances
Unacceptable	Unacceptable

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

Table 1 below compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% risk of exceedance in 50 years (i.e. 0.2% in the next year).

Table 1: %NBS compared to relative risk of failure

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

3.1 Minimum and Recommended Standards

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

3.1.1 Occupancy

– The Canterbury Earthquake Order[i] in Council 16 September 2010, modified the meaning of “dangerous building” to include buildings that were identified as being EPB’s. As a result of this, we would expect such a building would be issued with a Section 124 notice, by the Territorial Authority, or CERA acting on their behalf, once they are made aware of our assessment. Based on information received from CERA to date, this notice is likely to prohibit occupancy of the building (or parts thereof), until its seismic capacity is improved to the point that it is no longer considered an EPB.

3.1.2 Cordoning

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

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

4 Background Information

4.1 Building Description

Tommy Taylor Courts is located at 7 Cecil Place in Waltham, Christchurch and consists of 2-storey and 3-storey retirement residential units built circa 1999. The building consists of two wings forming an L shape. The overall plan dimension of the north wing is approximately 27m by 10m and the west wing is approximately 10m by 48m.

The north wing and part of the west wing is three storeys high with a pitched roof where the ridge is approximately 10m above grade. The remainder of the west wing is two storeys with ridge at approximately 7.5m above grade. A single-storey storage room exists at the southern end of the west wing. Refer to the site plan in Figure 2 below.

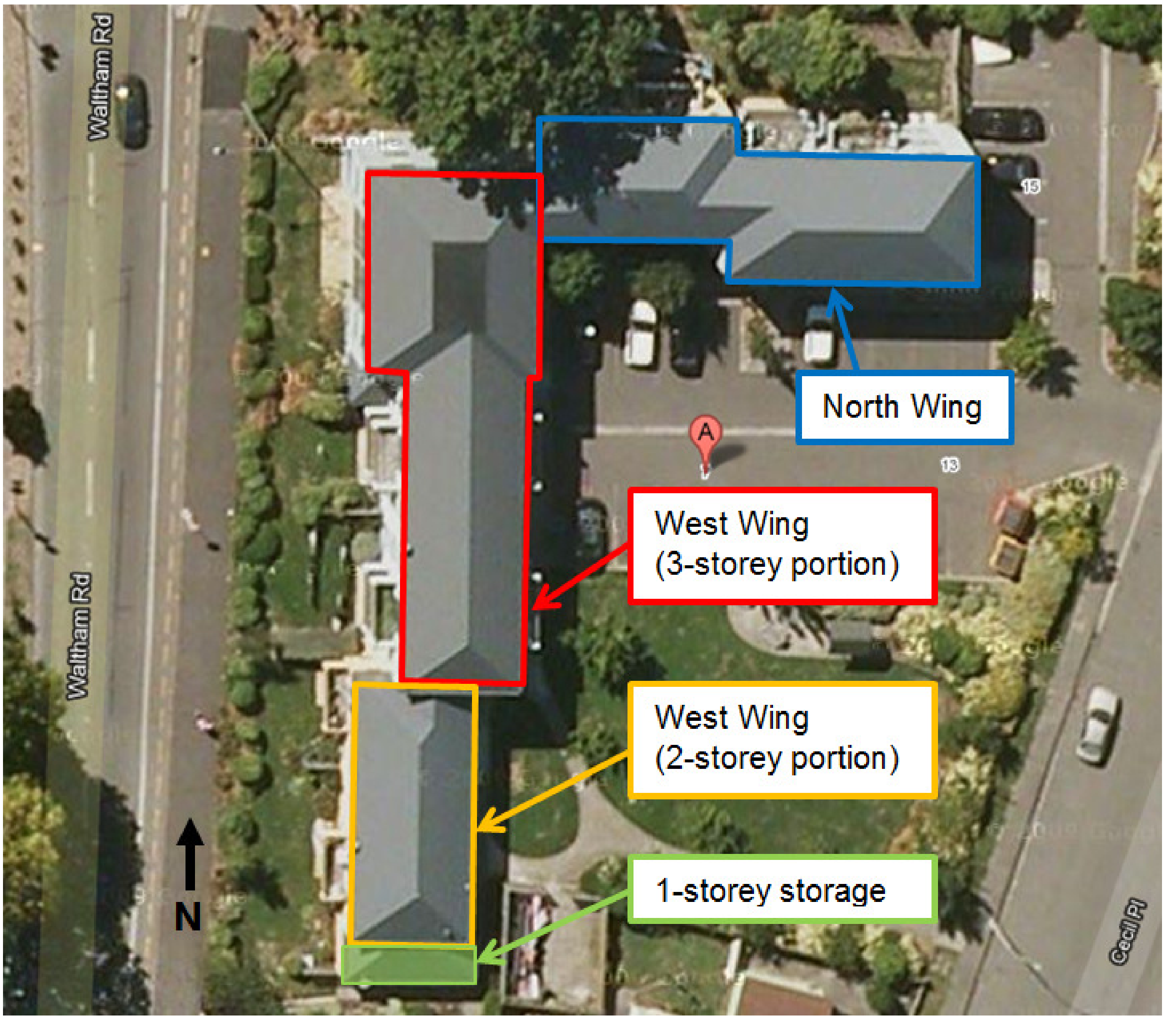


Figure 2 – Site Plan (Source: Google Maps)

The top storey for both wings consists of lightweight timber framed construction supporting a timber trussed roof.

Below the top storey, the structure consists of concrete-topped Unispan precast concrete slab supported by reinforced masonry load bearing walls. The ground floor consists of a concrete slab on grade. The foundation system consists of shallow strip footings under the masonry walls.

The lateral load resisting system consists of:

- Timber framed walls with GIB plasterboard linings and cross bracing in the top storey;
- Below the top storey, the lateral load resisting system consists of 150mm thick solid filled masonry walls reinforced with H12 vertical bars at 600mm centres and D10 horizontal bars at 800mm centres. Slender piers at the ground level have additional H16 vertical reinforcement at the ends of the piers.
- Diaphragm action is provided by the 75mm concrete topping and 663 mesh at levels one and two and by GIB ceilings in the top storey.

5 Survey

5.1 Post 22 February 2011 Rapid Assessment

Opus completed a Level 1 (external) Building Safety Evaluation on 4 March 2011.

5.2 Further Inspections

- On 4 and 10 March 2011, Opus performed site visits following the 6.3 magnitude earthquake on 22 February 2011.
- On 10 January 2012, Opus undertook visual inspections on behalf of the Christchurch City Council following the 6.0 magnitude earthquake on 23 December 2011.
- Additional inspections were performed by Opus on 18 May 2011, 21 June 2011, and 18 June 2012. Visual inspections involved exterior and interior walkover. No finishes were removed.
- Opus also measured the verticality of the building using a smart level on 5 March 2011, 10 March 2011, 11 March 2011, 11 May 2011, and 17 June 2011.
- On 18 June 2012, Opus performed a level survey of the ground floor slab as well as a verticality survey of the building. The survey results are included in the Appendices.

5.3 Original Documentation

Copies of the following drawings were provided by the CCC:

- Architectural drawings titled "Proposed Residential Development by Bryndwr Buildings Ltd.", dated 30 November 1999; Sheets: 1 to 7

- Structural drawings by Harman Halliday – consulting civil and structural engineers, dated December 1999; Sheets 1 to 26

The drawings have been used to confirm the structural systems, investigate potential Critical Structural Weaknesses (CSW's) and identify details which required particular attention.

No calculations were available for review.

6 General Observations

6.1 Ground Damage and Foundations

- Significant liquefaction has occurred throughout the site as evident in the ground displacement in the carpark as well as around the north wing (photos 3 and 4)

Refer to the Geotechnical Desk Study in Appendix 2 for further information and a description on ground damage.

6.2 Building Exterior

- The north wing has tilted approximately 100mm towards the north leaving a separation at the intersections between the two wings. The tilt created a large gap in the exterior sheathing along the full height of the building (photo 5).

- Minor cracking of exterior finishes typically around window or door openings (photos 10, 16 to 19).

- A vertical crack of the block wall was observed in the storage room at the southern end of the west wing (photo 9).

- Vertical cracks were observed at the junction between the balcony side wall and the building exterior walls (photo 20)

6.3 Building Interior

- The floor level survey indicates the ground floor slab in unit 13 in the west wing slopes up to 40mm over a 4.5m length. This 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.

- Minor cracks to the interior linings were observed around corners of window or door openings (photos 23 to 26).

- The ceiling has partially dropped in unit 13 at the junction between the north wing and the west wing (photo 14). Daylight can be seen from interior of unit 13 where the north and west wing have been separated.

- A minor crack was observed below one of the precast concrete stairs (photo 22).

7 Detailed Seismic Assessment

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

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

7.1 Critical Structural Weaknesses

The term Critical Structural Weakness (CSW) refers to a component of a building that could contribute to increased levels of damage or cause premature collapse of a building.

The Critical Structural Weaknesses are as follows:

a) Separation between the two wings: The north wing and west wing are not structurally tied together at the floor levels. However, some roof framing members appear to bridge between the two wings. Some localised damage of the roof framing near the intersection is likely.

b) Discontinuous walls: There are several reinforced masonry walls at the west wing that are not continuous to the foundation. Seismic overturning forces in these walls impose additional loads and can overwhelm supporting elements. The following points summarise where these discontinuous walls occur (refer to figure 3):

   i. The concrete block bearing wall at the 1st floor between unit 21 and the stairwell is not continuous to the foundation. Instead, it is supported by a concrete beam at 1st floor and perpendicular concrete block walls. The limiting component is the concrete block header above the entry door at the 1st storey and the 1st floor concrete beam (see figure 4).

   ii. The slender wall pier at the 1st level between the dining area and the kitchen of unit 21 is offset from the wall pier at the ground floor below in unit 19. The wall above is supported on the 1st floor concrete beam. The drawings also indicate a notch in the bottom of this beam which exacerbates this condition (see figure 5).

iii. The wall pier at the 1st level above the entry door into unit 19 is also discontinuous at the ground floor. The wall pier and the 1st floor beam are supported on the block header above the window/door into the unit (see figure 6).

iv. The exterior wall at the 1st floor between at the store room between units 15 and 16 (above the laundry room) is discontinuous.

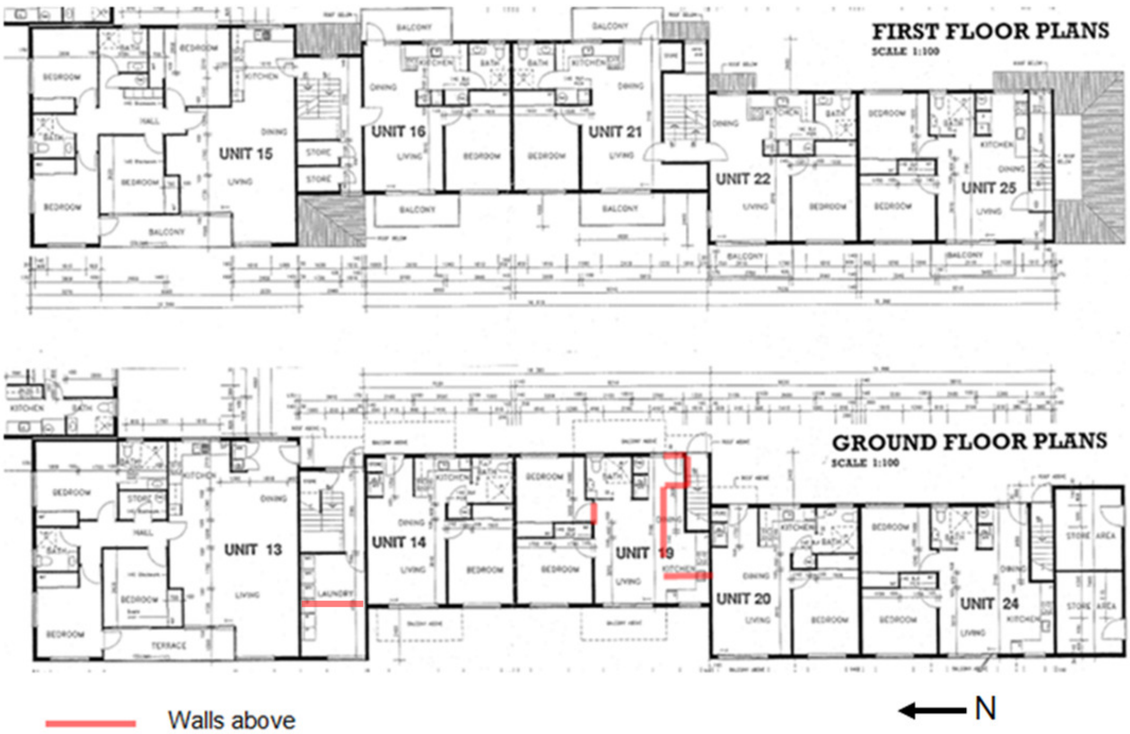


Figure 3 – Locations of discontinuous walls in west wing

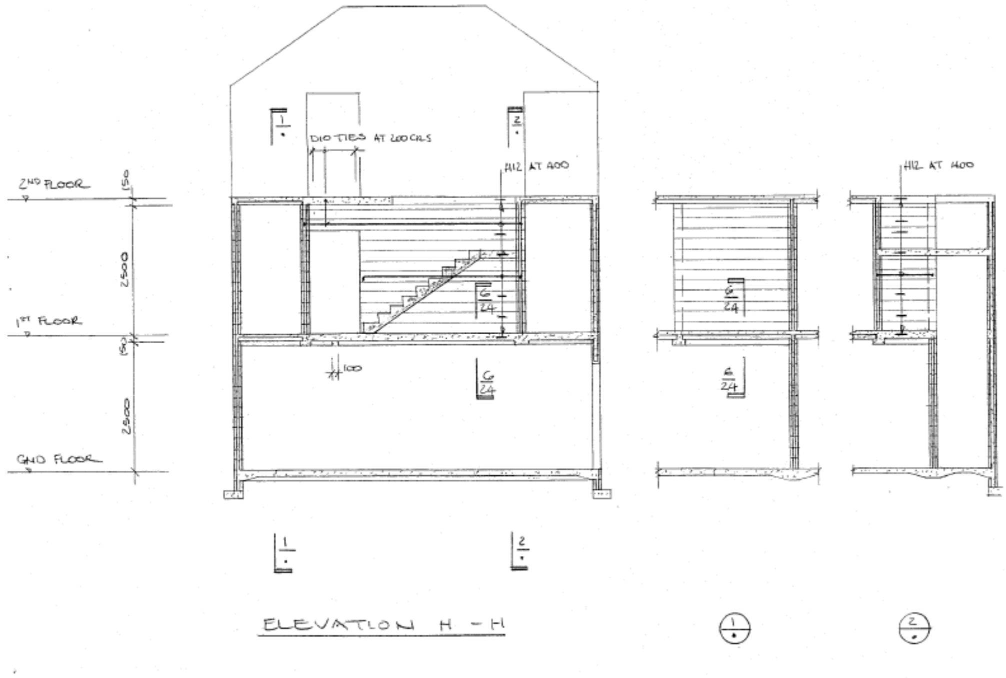


Figure 4 – Elevation of discontinuous wall between unit 21 and the stair

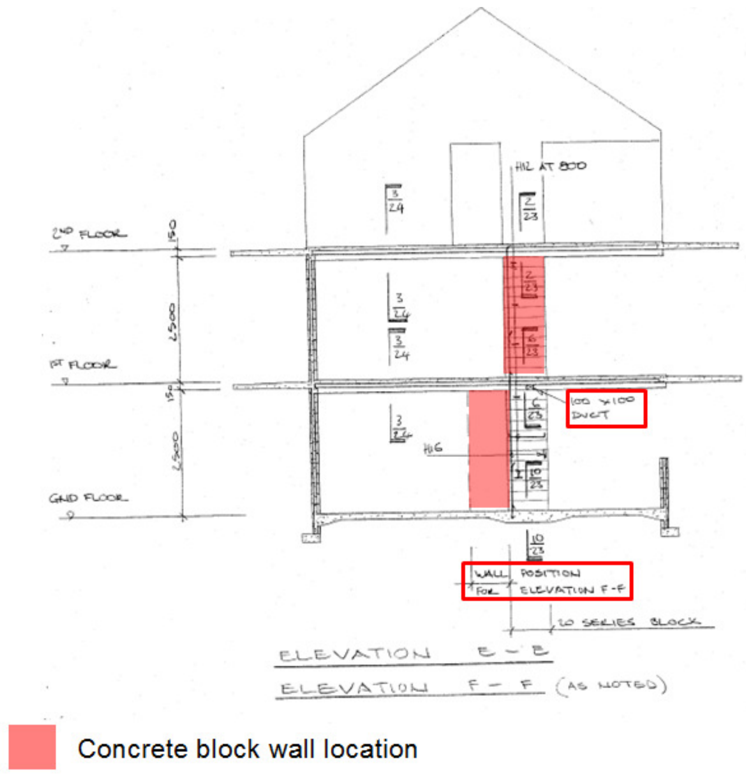


Figure 5 – Elevation of slender wall in unit 21 and 19

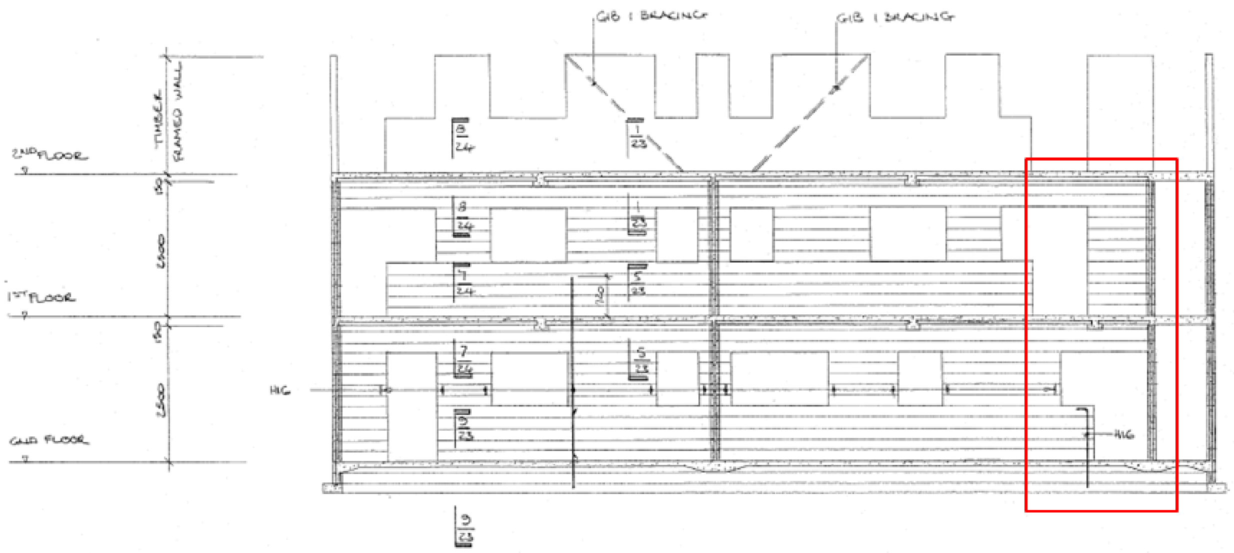


Figure 6 – East elevation showing discontinuous shear wall above entry to unit 19

These conditions have been considered in the analysis of the buildings.

7.2 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:

1. The 3-storey portion of the west wing, which has some CSWs as described in Section 7.1 above, was analysed using a 3-D model created using ETABS analysis software.

2. The north wing and the 2-storey portion of the west wing, which have well distributed walls, were analysed using hand calculations. Lateral forces were distributed to ground floor concrete masonry wall piers based on their relative rigidities.

   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.

   The buildings were assessed as Importance Level 2.

   The timber framed top-storey for all the buildings were analysed as a single storey structure bearing on the rigid concrete masonry structure below. Average wall shear stresses in the GIB sheathing was calculated and compared to shear capacities referenced in NZSEE 2006 [2] guidelines for the "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes.

7.3 Limitations and Assumptions in Results

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

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

- Simplifications made in the analysis, including boundary conditions such as foundation fixity.
- Assessments of material strengths based on drawings and site inspections
- The normal variation in material properties which change from batch to batch.
- Approximations made in the assessment of the capacity of each element.

7.4 Quantitative Assessment

A summary of the structural performance of the building is shown in the following tables. Note that the values given represent the critical elements in the building, as these effectively define the building's capacity. As noted in Appendix A2.2 Analysis Parameters, the building was analysed using a ductility factor (μ) equal to 1.25 due to the fact that majority of the reinforced block walls are controlled by shear.

Modes of failure that do not govern the building's performance are not included in the table except as noted for cases where higher ductility factors have led to the component being classified as non-critical.

Table 2: Summary of Seismic Performance for West Wing 3-Storey Portion – μ = 1.25

Structural Element/System	Failure mode or description of limiting criteria based on displacement capacity of critical element.	Critical Structural Weakness and Collapse Hazard	% NBS based on assumed capacity
Primary Components (those that are required parts of the lateral resisting system			
Typical concrete block walls - Longitudinal direction (north-south)	Concrete block wall piers are 150mm thick fully filled. The walls along the longitudinal direction primarily fail in shear. Some slender piers fail in flexure.	No	34 – 50%
Typical concrete block walls - Transverse direction (east-west)	Concrete block wall piers are 150mm thick fully filled. The walls along the transverse direction primarily fail in shear.	No	50 – 70%
Timber framed walls sheathed with GIB at top storey level – Longitudinal direction	Timber framed walls sheathed in GIB provide lateral resistance to the top storey. These walls are generally controlled by their overturning capacity since no	No	50 – 60%

Structural Element/System	Failure mode or description of limiting criteria based on displacement capacity of critical element.	Critical Structural Weakness and Collapse Hazard	% NBS based on assumed capacity
(north-south)	holdowns were detailed on the drawings.		
Timber framed walls sheathed with GIB at top storey level – Transverse direction (east-west)	Timber framed walls sheathed in GIB provide lateral resistance to the top storey. These walls are generally controlled by their overturning capacity since no hold-downs were detailed on the drawings.	No	50 – 70%
Diaphragm strength of typical slab	The diaphragm must resist the storey shear imposed on it and transfer it to lower level lateral resisting system. The storey shear is distributed linearly along the span of the diaphragm. The shear strength of the concrete and wire-mesh of the topping slab can resist the diaphragm shear stresses.	No	100%
Discontinuous shear wall between unit 21 and stairway.	The concrete block wall between unit 21 and the stairway is not continuous to the foundation; instead, it relies on a concrete beam at the 1st floor and perpendicular concrete block walls for vertical support. The limiting element in this load path is the concrete block header above the entry door at the first storey. The failure mode is in flexure. Since the wall provides gravity support for 1st and 2nd floor slabs, failure of these elements could lead to local loss of gravity support. Note that the 1st floor beam is notched at the doorway. See Figure 4.	Yes	40%
Discontinuous shear wall above entry to unit 19.	Wall pier above the entry door into unit 19 is discontinuous at the ground floor. The wall pier and the 1st floor beam are supported on the block header above the window/door into the unit. The failure mode is flexure in the header.	Yes	50%
Discontinuous slender wall pier in unit 21	The slender wall pier between the dining area and the kitchen in unit 21 is not continuous to the foundation. The wall pier below along the same line in unit 19 is offset to the west. The pier above is supported on a 250mm by 250mm concrete beam on the 1st floor. The drawings also show a notch in the bottom of this beam. The complex load path is loaded to its full capacity by gravity only loads. In theory, the first story wall will not resist lateral seismic loads because it is free to rotate about its support at the bottom west corner. See Figure 5.	Yes	See discussion
Discontinuous shear wall at store room between units 15 and 16	The exterior wall pier at the 1st floor at the store room between units 15 and 16 (above the laundry room) is discontinuous and is supported by transverse walls along each side of the laundry room at the ground floor. A window opening occurs at this wall and the failure mode is shear in the spandrels above and below this window.	Yes	34%

Secondary Components (those that are not required parts of the lateral load resisting system but which must be able to maintain their gravity load capacity while the building under goes deformation due to earthquake loading)			
Stairs	The stairs are precast concrete construction that is dowelled into in-situ concrete slab at landing. The dowels consist of (4) H10 bars extended 300mm into the in-situ concrete landing. Drift of the building is expected to be low thus the stair are not considered a life safety hazard.	No	NA
Out-of-plane loads on typical block wall piers	The block wall is generally supported at first and second floor. The reinforcement in the wall can resist out-of-plane bending from inertial forces of the wall self-weight.	No	100%

Table 3: Summary of Seismic Performance for West Wing, 2-Storey Portion – μ = 1.25

Structural Element/System	Failure mode or description of limiting criteria based on displacement capacity of critical element.	Critical Structural Weakness and Collapse Hazard	% NBS based on assumed capacity
Primary Components (those that are required parts of the lateral resisting system			
Typical concrete block walls - Longitudinal direction (north-south)	Concrete block wall piers are 150mm thick fully filled. The walls along the longitudinal direction primarily fail in shear. Some slender piers fail in flexure.	No	75%
Typical concrete block walls - Transverse direction (east-west)	Concrete block wall piers are 150mm thick fully filled. The walls along the transverse direction primarily fail in shear.	No	100%
Timber framed walls sheathed with GIB at top storey level – Longitudinal direction (north-south)	Timber framed walls sheathed in GIB provide lateral resistance to the top storey. These walls are generally controlled by their overturning capacity since no holdowns were detailed on the drawings.	No	50 – 60%
Timber framed walls sheathed with GIB at top storey level – Transverse direction (east-west)	Timber framed walls sheathed in GIB provide lateral resistance to the top storey. These walls are generally controlled by their overturning capacity since no holdowns were detailed on the drawings.	No	50 – 70%
Diaphragm strength of typical slab	The diaphragm must resist the storey shear imposed on it and transfer it to lower level lateral resisting system. The storey shear is distributed linearly along the span of the diaphragm. The shear strength of the concrete and wire-mesh of the topping slab can resist the diaphragm shear stresses.	No	100%

Secondary Components (those that are not required parts of the lateral load resisting system but which must be able to maintain their gravity load capacity while the building under goes deformation due to earthquake loading)			
Stairs	The stairs are precast concrete construction that is dowelled into in-situ concrete slab at landing. The dowels consist of (4) H10 bars extended 300mm into the in-situ concrete landing. Drift of the building is expected to be low thus the stair are not considered a life safety hazard.	No	NA
Out-of-plane loads on typical block wall piers	The block wall is generally supported at first and second floor. The reinforcement in the wall can resist out-of-plane bending from inertial forces of the wall self-weight.	No	100%

Table 3: Summary of Seismic Performance for North Wing – μ = 1.25 (unless noted otherwise)

Structural Element/System	Failure mode or description of limiting criteria based on displacement capacity of critical element.	Critical Structural Weakness and Collapse Hazard	% NBS based on calculated capacity
Primary Components (those that are required parts of the lateral resisting system)			
150mm grouted concrete block - Longitudinal direction (east-west)	Concrete block wall piers are 150mm thick fully filled. The walls along the transverse direction primarily fail in shear. Some slender wall piers fail in flexure.	No	35 – 50%
150mm grouted concrete block - Transverse direction (north-south)	Concrete block wall piers are 150mm thick fully filled. The walls along the transverse direction primarily fail in shear.	No	70%
Timber framed walls sheathed with GIB at top storey level – Longitudinal direction (east-west)	Timber framed walls sheathed in GIB provide lateral resistance to the top storey. These walls are generally controlled by their overturning capacity since no holdowns were detailed on the drawings.	No	40 – 50%
Timber framed walls sheathed with GIB at top storey level – Transverse direction (north-south)	Timber framed walls sheathed in GIB provide lateral resistance to the top storey. These walls are generally controlled by their overturning capacity since no holdowns were detailed on the drawings.	No	50 – 70%
Diaphragm strength of typical slab	The diaphragm must resist the storey shear imposed on it and transfer it to lower level lateral resisting system. The storey shear is distributed linearly along the span of the diaphragm. The shear strength of the concrete and wire-mesh of the topping slab can resist the diaphragm shear stresses.	No	100%

Secondary Components (those that are not required parts of the lateral load resisting system but which must be able to maintain their gravity load capacity while the building under goes deformation due to earthquake loading)			
Stairs	The stairs are precast concrete construction that is dowelled into in-situ concrete slab at landing. The dowels consist of (4) H10 bars extended 300mm into the in-situ concrete landing. Drift of the building is expected to be low thus the stair are not considered a life safety hazard.	No	NA
Out-of-plane loads on typical block wall piers	Block wall is generally supported at first and second floor. The reinforcement in the wall can resist out-of-plane bending from inertial forces of the wall self-weight.	No	100%

7.5 Discussion

Based on our quantitative assessment, both the west and north wings have computed strengths in the overall lateral load resisting system that exceed 33% NBS. In general, the strength of the building is limited by shear strength of the masonry block walls. The stresses in the walls along the longitudinal direction are generally higher than those along the transverse because the party walls between units are generally longer and have fewer openings.

Given that these buildings exceed 33% but are not greater than 67%NBS, they are considered a moderate risk but their seismic performance is legally accepted under the 2004 Building Act.

Although the buildings overall capacity exceeds 33%NBS, there are some localised areas of concern. One area of concern is the discontinuous shear wall at the 1st floor in the west wing between unit 21 and the stairwell. As shown in Figure 4 above, this wall is a bearing wall that relies on a complex load path for gravity support. The limiting element is the concrete block header above the entry door to unit 21. Overturning forces from east-west direction earthquake loading impose additional vertical loads and the computed strengths of the header is approximately 40% NBS. This wall should be supported by adding a column below in unit 19 to allow for continued use of the building due to the complex load path and dependence on uncertain detailing.

The discontinuous wall pier above the entry to unit 19 bears on the door header at unit 19. See Figure 6. This condition should be addressed for long term use of the building.

The discontinuous exterior wall above the laundry room should also be addressed for long term use of the building.

At unit 21, between the dining area and the kitchen, a discontinuous wall condition occurs where the slender piers at the 1st and ground levels are offset. The pier above is supported on a concrete beam at the 1st floor. See Figure 5. Per our calculations, the capacity to support gravity loading is marginal. The wall above should be supported by adding a column in unit 19 below for continued use of the building due to the complex load path and dependence on uncertain detailing.

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 Tommy Taylor Courts. The result of this study was detailed in a memo dated 18 May 2011, which is included in the appendix 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 competent 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 2003 ECAN Liquefaction study 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.

The area has been identified to have undergone low to moderate liquefaction as a result of the 22 February 2011 earthquake. However, aerial photographs taken on 24 February 2011 show extensive liquefaction, particularly on the north eastern side of the site. Notable features include up to 150mm of ground heave in the eastern carpark and collapse of the footpath on Brougham Street.

Differential settlement and rotation of the North wing is clearly visible. There is a large gap, estimated at up to 150mm, at two locations between the two wings.

A detailed floor survey was completed for each building and is included in the appendix of this report.

8.3 Monitoring Results

During the site visit of 5 March 2011 Opus established a monitoring programme to record building movement using a smart level. The smart level was used to measure the offset from vertical at 27 key locations on the perimeter of the building. The results of readings taken on 5/03, 10/03, 11/03 and 11/05 are presented in the memo dated 18 May 2011.

The readings confirmed that the north wing has tilted up to 22mm/m towards the north (Brougham Street). This equates to 170mm of tilt over the 8m height of the structure. Rotation occurred between the 5/03 and 10/03 readings.

8.4 Further Work

A full geotechnical assessment is required to determine the potential for further liquefaction and if ground improvements are needed.

9 Remedial Options

The damaged elements discussed in Section 6 must be repaired. These include:

- Removing the tilt in the north wing.
  - After the tilt of north wing is corrected, repair exterior finish at the intersection between the two wings.
  - Repair damages to exterior finishes including crack repair around window, door openings, and at balcony side walls.
  - Repair damages in interior finishes including crack repair of GIB, and reinstatement of partially fallen ceilings.

In addition to the repair listed above, the building will require strengthening, with a target of increasing the seismic performance to as near as practicable to 100%NBS, and at least 67%NBS. Any conceptual strengthening scheme would have to address:

- Load paths below discontinuous walls at unit 21.
- Existing block walls that require strengthening.
- The excessive floor slope at unit 13.

The site also needs a full geotechnical assessment to determine the potential for further liquefaction and if ground improvements are needed. This should be done prior to implementing repair and strengthening schemes.

10 Conclusions

The buildings have been assessed to have overall capacities in excess of 33% NBS. The capacities of the 3-storey structures are generally governed by shear strength of the fully grouted block walls. The capacity of the 2-storey portion of the west wing is governed by the overturning of the 1st floor timber walls. This capacity level implies the buildings are considered a moderate risk but their seismic performance is legally accepted under the 2004 Building Act. The %NBS for each building is summarised below:

Building Area	%NBS
West wing (3-storey portion)	34 - 50%
West wing (2-storey portion)	50 - 60%
North wing	35 - 50%

However, there are some localised areas of concern within the 3-storey portion of the west wing where concrete block walls are not continuous to the foundation (see item b below). Earthquake loading impose additional loads into elements that support these discontinuous walls. The computed strengths of these elements are approximately 40 to 50% NBS but depends on a complex and unreliable load path.

The slope in the ground floor slab at unit 13 is excessive.

We have identified the following critical structural weaknesses:

a) Separation between the two wings: The north wing and west wing are not structurally tied together at the floor levels. However, some roof framing members appear to bridge between the two wings. Some localised damage at the roof near the intersection is likely.

b) Discontinuous walls: There are several concrete masonry walls at the west wing that are not continuous to the foundation. Seismic overturning forces in these walls impose additional loads and can overwhelm supporting elements. The following summarises where discontinuous walls occur:

- The concrete block bearing wall at the 1st floor between unit 21 and the stairwell is not continuous to the foundation. Instead, it is supported by a concrete beam at 1st floor and perpendicular concrete block walls. The limiting component is the concrete block header above the entry door to unit 21 and the 1st floor concrete beam.

- Slender wall pier at the 1st floor between the dining area and the kitchen of unit 21 is offset from the wall pier at the ground floor below in unit 19. The wall above is supported on the 1st floor concrete beam. The drawings also indicate a notch in the bottom of this beam which worsens this condition.

- Wall pier above the entry door into unit 19 is discontinuous at the ground storey. The wall pier and the 1st floor beam are supported on the block header above the window/door at entry into the unit.

- The exterior 1st storey wall above the laundry room is discontinuous.

c) Ground damage has been moderate to significant at the site. Differential settlement and rotation of the north wing is clearly visible. Based on the floor level survey, the differential settlement on the ground floor of the north wing is approximately 100mm generally sloping towards the north and the verticality survey indicates a 100mm lean to the north.

d) Superstructure damage has been most severe at the junction between the north and south wings. This is due to the tilt in the north wing.

e) Other superstructure damage has been limited to minor cracks in GIB and exterior wall sheathing around window and door openings, cracks around balcony side walls, and minor cracking in concrete block wall toward the south of the west wing.

11 Recommendations

1. Remove the tilt of the north wing and repair the associated damage.
2. Perform remedial strengthening to support discontinuous shear walls in the west wing. We recommend that the two discontinuous shear walls in unit 21 be supported by adding in two columns in the unit 19 at the ground storey. This work should be addressed promptly.
3. 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.
4. Engage a quantity surveyor to determine the costs for strengthening the building.
5. The site needs a full geotechnical assessment to determine the potential for further liquefaction and if ground improvements are needed.

12 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.

13 References

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

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

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

[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.

Appendix 1 - Photographs

Tommy Taylors Court, Christchurch, NZ

No.	Item description	Photo
1.	Overall view of Tommy Taylor Court apartments looking west (north wing in the front and west wing in the back)	
	North wing	
2.	View of north wing viewing from southeast	

3.	Vertical ground displacement at parking lot area to the east of north wing	
4.	Tilting of north wing	

5.	Separation of north wing and west wing	
6.	North side of north wing	

7.	Roof trusses	
	West wing	
8.	West façade of west wing viewing from the southwest	

9.	Vertical crack in the masonry wall inside the storage room at the southern end of the west wing	
10.	Crack in the exterior finish where it joins the balcony	

11.	Separation of north and west wing as seen from the southern junction	
12.	Gap between north and west wing masonry walls	

13.	Masonry unit face shell separated at base of wall	
14.	Separation of west and north wing as seen within unit 13	

15.	Front view of balcony in west wing	

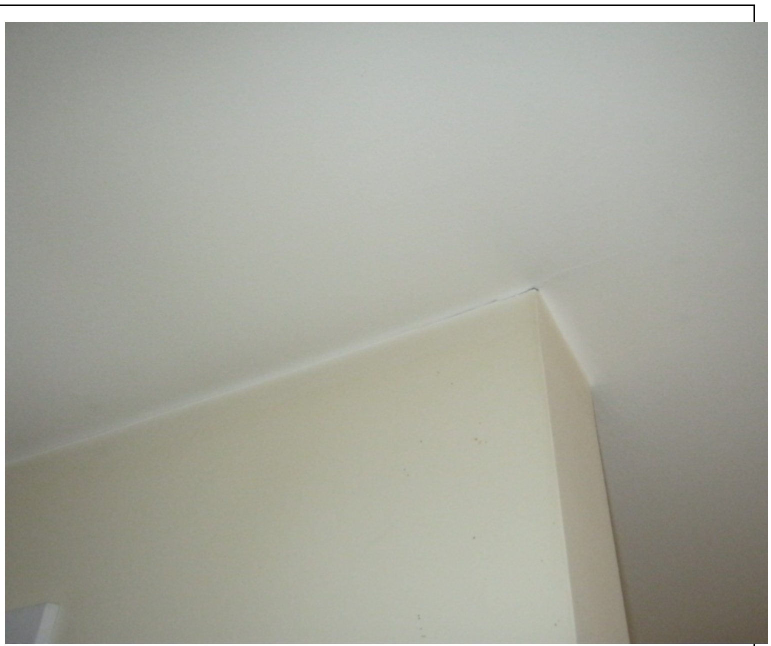
16.	Cracks in finishes near corner of window opening	
17.	Horizontal crack of exterior finishes adjacent to window opening indicating flexural yielding in wall pier	

18.	Vertical crack in the wall finish below a window opening	
19.	Horizontal crack in the wall below balcony	

20.	Gap between masonry wall and balcony in west wing	
21.	Cracks in the entrance slab of west wing due to settlement	

22.	Crack in the precast stairs in west wing	
23.	Typical horizontal crack in the wall near an opening	

24.	Vertical crack in the wall finishes over an opening in west wing	
25.	Typical crack in finishes over an opening	

26.	Separation between timber framed wall and roof ceiling in west wing	

Appendix 2 - Geotechnical Appraisal

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TO	Lindsay Fleming
COPY	Greg Saul, Sheryl Keenan
FROM	Graham Brown
DATE	18 May 2011
FILE	6-QUCCC.01/005SC
SUBJECT	Tommy Taylor Court - Geotechnical Desk Study

1. Introduction

This memo summarises the findings of a Geotechnical Desk Study and a detailed Site Walkover completed on 11 May 2011. The purpose of this desk study is to provide an initial appraisal 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 Assessment prepared by Tim Browne dated 5 March 2011, and a Geotechnical update based on monitoring results interpreted by William Gray on 10 and 11 March 2011.

2. Description of Facility

The site comprises two and three storey retirement units in an 'L' shaped formation, refer to Site Plan Appendix B. The north–south wing is 50m long and the east-west wing is 38m long. The east–west wing has suffered the most severe damage.

The complex was opened in 2001 and is formed of reinforced masonry block.

The ground profile is relatively flat, low lying and is typically level with Brougham Street to the north and Waltham Road to the west. The complex is accessed from Cecil Place on the eastern side of the site.

The grounds are landscaped with grassed areas, paving and shrubs.

3. Desk Study

3.1 *Ground Conditions*

A desk study of well logs in the area from Environment Canterbury records identified four historic drill logs within 250m 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 close 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 competent 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 Construction Drawings

A copy of the Construction Drawings prepared by Harman Halliday have been obtained from CCC records. No site specific ground investigation data has been provided by CCC.

The drawings indicate that the buildings are founded on a shallow perimeter strip footing. The footings are typically only 350mm wide with nominal steel reinforcement supporting a 100mm thick ground floor slab. The footings are founded approximately 400mm below existing ground level. In some locations the footing width is increased to 750mm.

3.3 Ground and Building Damage

A walkover inspection of the exterior of the buildings was completed on 11 May 2011. No interior inspections were conducted.

As outlined in the 5 March 2011 report, there is evidence of significant liquefaction throughout the site. Notable features include up to 150mm of ground heave in the eastern carpark and collapse of the footpath on Brougham Street.

Differential settlement and rotation of the east–west wing is clearly visible. There is a large gap, estimated at up to 150mm, at two locations between the east–west wing and the north-south wing, refer to the annotated Site Plan Appendix B. The building has been monitored by both Opus and CCC.

3.4 Monitoring Results

During the site visit of 5 March 2011 Opus established a monitoring programme to record building movement using a smart level. The smart level was used to measure the offset from vertical at 27 key locations on the perimeter of the building. The results of readings taken on 5/03, 10/03, 11/03 and 11/05 are presented in Appendix C.

The readings confirmed that the east-west wing has tilted up to 22mm/m towards the north (Brougham Street). This equates to 170mm of tilt over the 8m height of the structure. Rotation occurred between the 5/03 and 10/03 readings with no further rotation since.

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

In addition to monitoring undertaken by Opus, CCC Survey Department set up monitoring stations on the south elevation of the east-west wing, refer Appendix C. Targets were positioned 1.0m above ground level and also below gutter level to monitor movement. The first set of readings were taken on 15/03, with 7 subsequent inspections. All movement recorded has been less than 4.0mm indicating the east–west wing has been stable since readings commenced on 15/03.

3.5 Liquefaction Hazard

The 2003 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. Aerial photographs taken on 24 February 2011 show extensive liquefaction, particularly on the north eastern side of the site.

4. Appraisal

4.1 Interpretation of Monitoring Results and Site Observations

As a result of the 22 February earthquake and subsequent liquefaction, the east-west wing has rotated as a block by up to 170mm to the north. The maximum rotation appears to have occurred at the eastern end of this wing. Rotation at the western end is approximately 110mm. The north-south wing does not appear to have been as adversely affected by the earthquake, rotation less than 60mm.

The construction drawings have confirmed the 3 storey building is founded on shallow footings. This information confirms that the east-west wing has subsided on the north side.

Subsurface investigations are recommended to confirm the cause of the rotation. The rotation is suspected to be caused by either liquefaction induced consolidation or bearing capacity loss of the shallow soils. Our investigations will seek to identify the cause of the rotation and identify potential remedial solutions.

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 the

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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.

structure would be expected in such an event. Also liquefaction could occur in large earthquakes from the Canterbury foot hills, Alpine or other faults.

5. Proposed Geotechnical Investigations

Due to the ground damage and subsidence 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) Assess the bearing capacity of the soils beneath the shallow foundations and floors.
e) Assist in the decision whether to repair or replace.
f) Provide geotechnical information for future foundation design.

The scope of the proposed geotechnical investigations are:

1) Borehole to a depth of about 25 m in the north east corner of the site, with Standard Penetration Tests at 1.5 m depth intervals, and install piezometer to monitor groundwater level.
2) Static Cone Penetration Tests (CPT) 4 No.
3) Laboratory soil classification tests on soil samples.
4) Excavate and inspect the shallow foundations at key locations to check for voids, liquefaction induced consolidation.
5) Hand Auger to 2.0m depth and test to confirm the soil strata bearing capacity.
6) Assessment and reporting.

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

6. 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 – Monitoring Results

Photos showing liquefaction and site damage, Tommy Taylor Courts

North South Wing,

North South Wing

View of East – West Wing

View of East- West Wing tilting north.

Liquefaction damage to footpath

Heave of asphalt in Carpark


150mm gap at the junction of the two wings


Smaller gap on northern elevation.

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

Water Level: Artesian

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

Scale(m): -10, -20, -30, -40, -50, -60, -70, -80, -90

Borelog for well M36/0964 page 1 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
-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

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

Borelog for well M36/0964 page 2 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): -50, -55, -60, -65, -70, -75, -80, -85, -90, -95

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

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

Scale(m): -10, -20, -30, -40, -50, -60, -70, -80, -90, -100

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	Depth(m)	Full Drillers Description	Formation Code
Artesian	-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 :

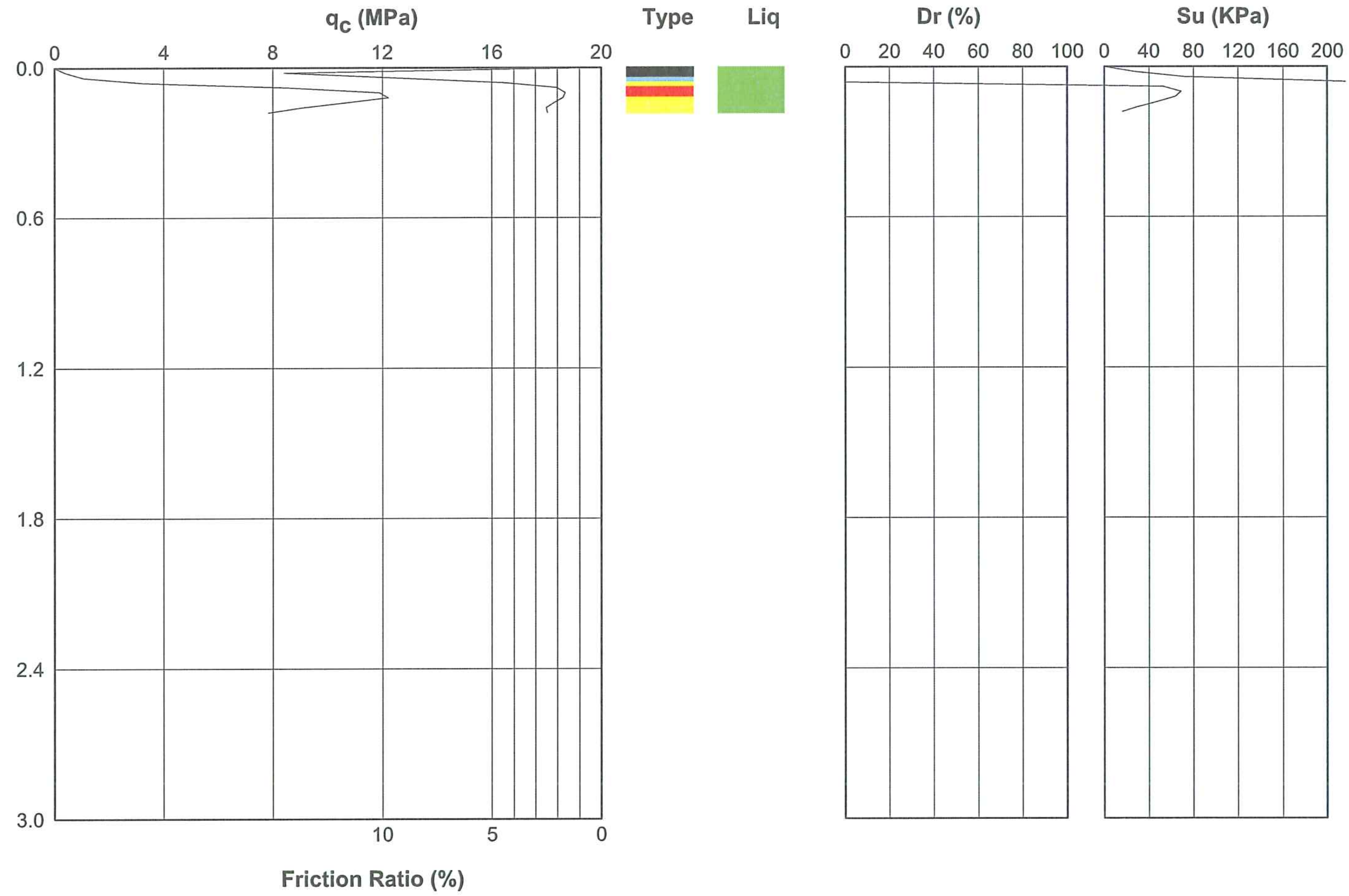
Environment Canterbury
Your regional council

Water Level: Artesian

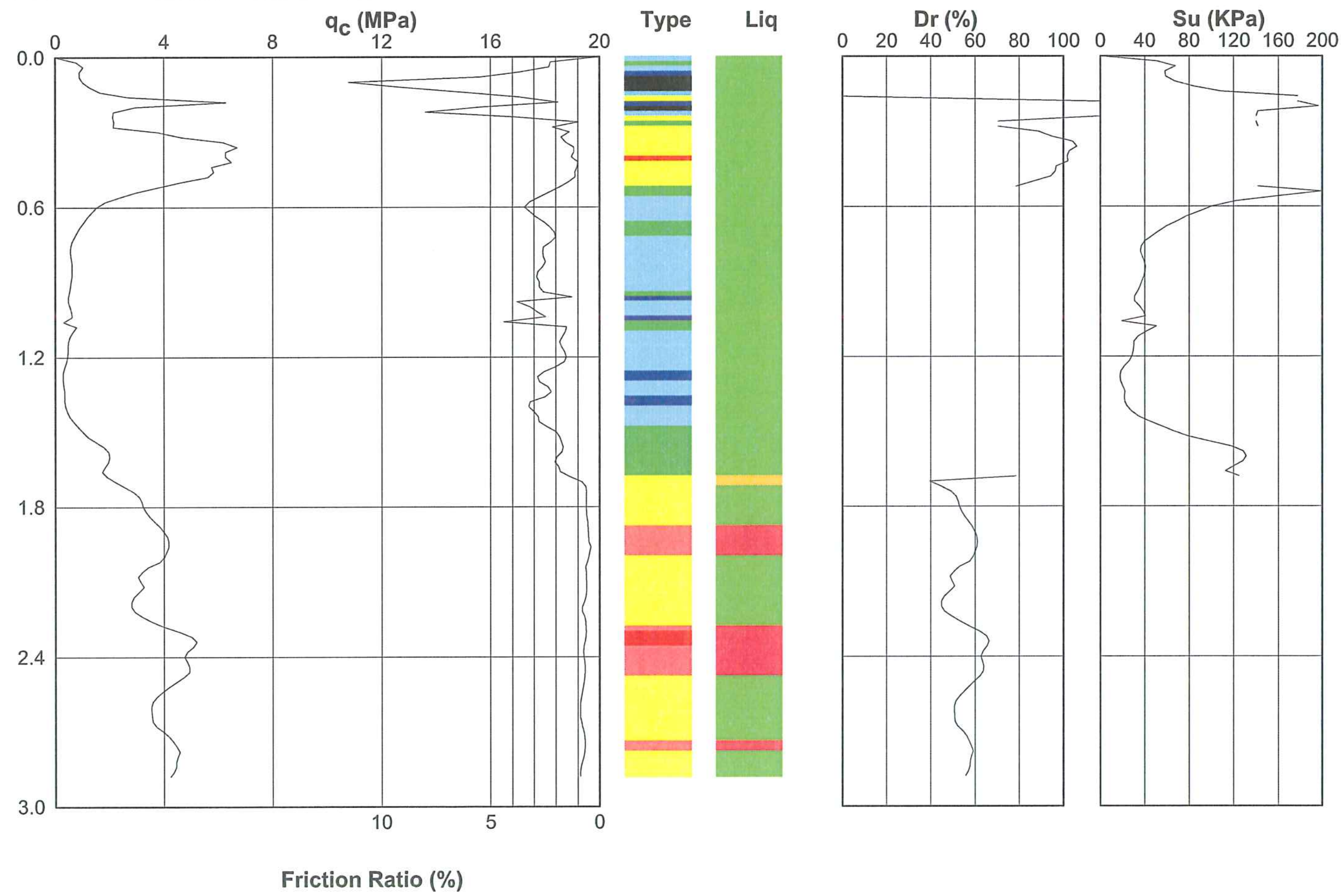
Depth(m)	Full Drillers Description	Formation Code
49.9m	Brown sand	br
- 51.8m	Blue sand	br
- 53.6m	Blue sand & clay	br
- 56.6m	Blue clay	br
- 70.1m	Brown shingle	li
- 76.2m	Blue clay	li-2
- 84.7m	Brown shingle	li-3
- 86.2m	Brown sand	he
- 89.0m	Brown shingle	he
- 89.9m	Brown sand	he
- 92.3m	Brown shingle 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

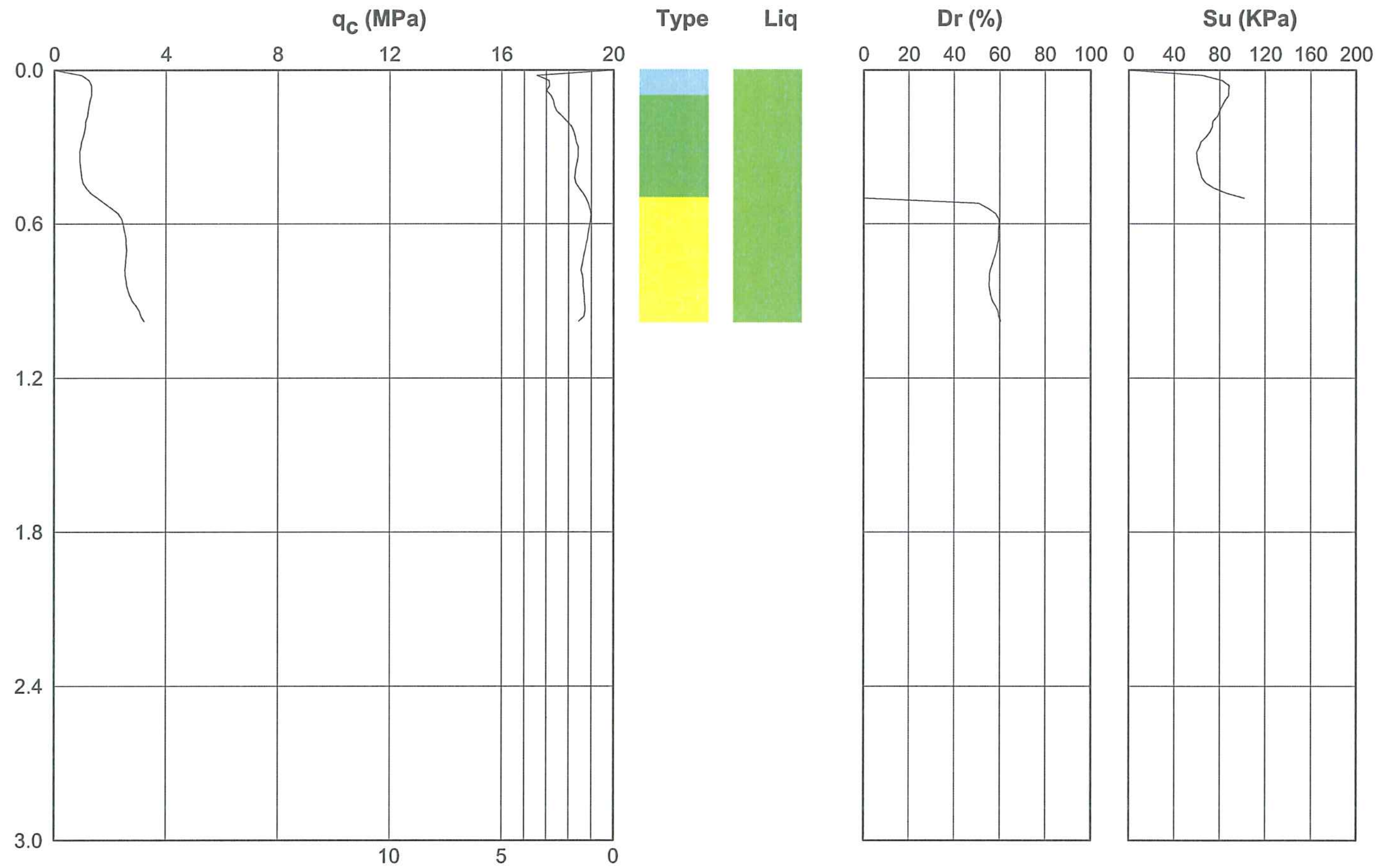
Site 185 Orion Pole Network Investigation Vienna Street



Site 186 Orion Pole Network Investigation



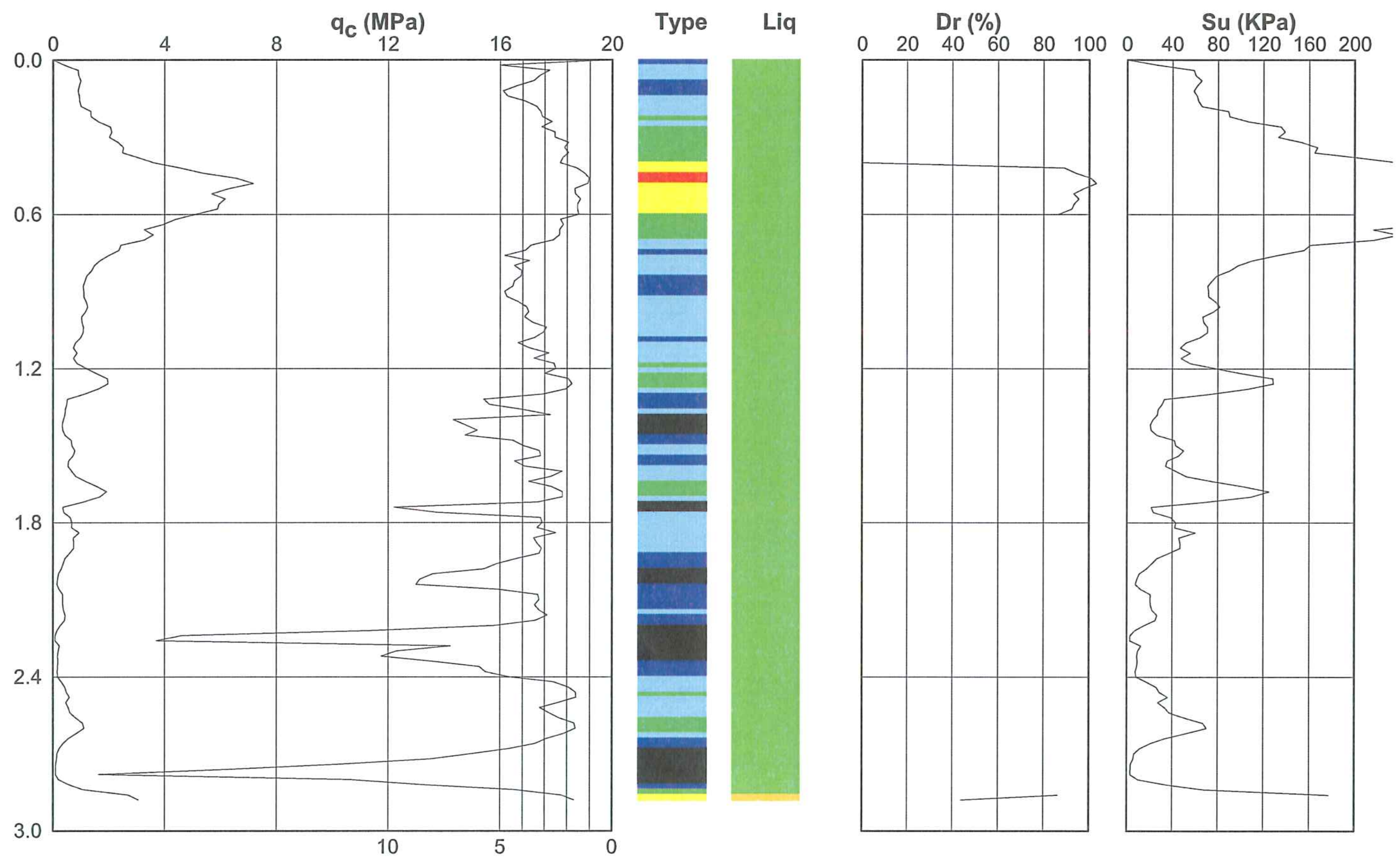
Site 191 Orion Pole Network Investigation



Friction Ratio (%)

Site 192 Orion Pole Network Investigation

[illegible] street 500m south



Friction Ratio (%)

CPT. HANDAUGER + SCALA TEST CPT + ~~BH~~ TEST. QUCCC.01 ~~005E~~ 005.SC

Cracking both sides possible heave

150mm heave in asphalt

(+) survey control point @ ceil level

TOMMY TAYLOR COURTS

PROPOSED GROUND INVESTIGATIONS

CPT on grassed Area

Tommy Taylor Courts, 7 Cecil Place.

Label	Location	Wall Orientation	Tilt amount (mm/m) and direction of tilt (5-3-11)	7-3-11	10/3/11	11/3/11	11/5/11
1	NE corner, east side	NS	9E	10	2E	0	6E
2	NE Corner north side	EW	11N	12	**22N**	**22N**	**15N**
3	Left of entry foyer south side	EW	16N	23 (moved by window)		Not measured due to wall shape	
4	Second Floor, by door to apartment 12	EW	16N	17			
5	By entry east side	NS	2W	3			1E
6	By entry east side	EW	4W	4		0	2N
7	On SE corner on 3 story block	NS	0	0		**9E**	**9E**
8	On SE corner on 3 story block	EW	4N	0		0	2N
9	On horizontal slab by window in stairwell	EW	3N	Not measured			
10	N/A						
11	Top floor on wall by door	NS	0	Not measured			
12	Top floor on wall by door	EW	2W	Not measured			
13	On 2 storey block, east side	NS	4W	Not measured		**7E**	**4E**
14	On 2 storey block, SE corner	NS	2W	5			2E
15	On 2 storey block, SE corner	EW	12N	7			10N
16	Second Floor, by door to apartment 12	NS	8E	8			
17	At apex of building East side	NS	0	0			
18	At apex of building East side	EW	11N	11			
19	SE corner of EW block	NS	5W	4	7E	2E	
20	SE corner of EW block	EW	12N	11	**19N**	**15N**	
21	On column, NW end	NS	7E	7		4E	2E
22	On column, NW end	EW	2N	2			8N
23	Northern side, NW end	EW	11S	11			10S
24	Northern Side NW end	EW	7N	7	**17N**	**17N**	**12N**
25	Northern Side NW end	NS	9E	9			**2E**
26	Northern Side, NE end	EW	3N	3	**13N**	**13N**	**13N**
27	Northern Side, NE end	EW	8N	5	**17N**	**17N**	**15N**

Tommy Taylor Courts, 7 Cecil Place.

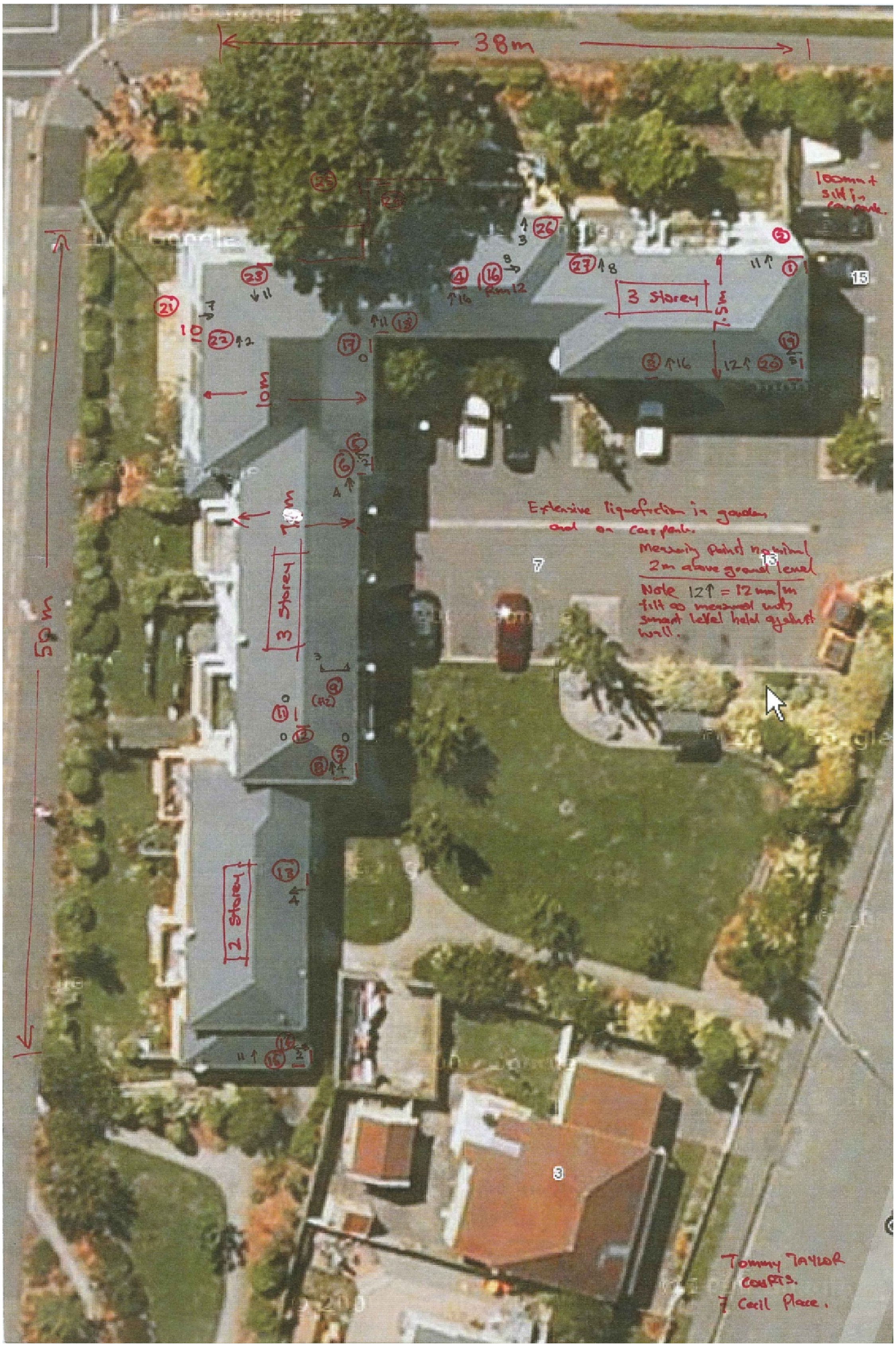
Notes from G Brown 11/5/11:

Based on site measurements taken there has been no significant movement since readings taken by William Gray on 11/3/11, estimated accuracy of smart level readings ±2mm/m.

Survey Control Point observed on south wall of Unit 1 at ground floor window sill level. I recommend ongoing monitoring of the displacements, request copy of CCC survey results.

Recommendations for subsurface investigations to follow.

Tommy Taylor Courts, 7 Cecil Place.



Site Plan showing monitoring locations, and amount/direction of movement at 5/03/11

Appendix 3 - Quantitative Assessment 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. 2.2.14$_B$, B1/VM1
  $Z = 0.30$

- Return period factor — Table 3.5, NZS1170.5
  $R_u = 1.0$ (*Importance* Level 2 structure, 50 year design life)

- Ductility factor — Cl. 2.6.1.2, NZS3101:2006
  $\mu = 1.25$ (nominally ductile)

- Structural performance factor — Cl. 2.6.2.2, NZS3101:2006

  $S_p = 0.925$

- Material properties

Table A1: Analysis Material Properties for all buildings

Shear strength of timber walls sheathed in GIB, v_n (kN/m)	3 per side
Concrete block nominal compressive strength, f'_m (MPa)	10
Concrete nominal compressive strength, f'_c (MPa) (1)	30
Mild reinforcing nominal yield strength, f_y (MPa) (2)	300
High strength reinforcing nominal yield strength, f_y (MPa) (2)	430

Notes:

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

- Effective section properties

Table A2: Effective section properties from NZS3101:2006

Table C6.6 – Effective section properties, I_e

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

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

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

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

West wing (3-storey portion) = 3990KN
West wing (2-storey portion) = 1150 KN
North wing = 3450 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 3D model of the 3-storey portion of the west wing was set up using the structural analysis program ETABS, and effective section properties for structural members were taken from Table A2 above. The floor diaphragms were modelled with shell elements and treated as rigid diaphragms.

For the north wing and the 2-storey portion of the west wing, hand calculations were performed. Shears in concrete masonry wall piers are distributed based on relative rigidity of the wall piers assuming floor diaphragms are rigid.

The timber framed top-storey for all the buildings was analysed as a single storey structure was that sits atop the rigid concrete masonry structure below. Average wall shear stresses in the GIB sheathing was calculated and compare to shear capacity referenced in NZSEE 2006 [2] guidelines for the “Assessment and Improvement of the Structural Performance of Buildings in Earthquakes.

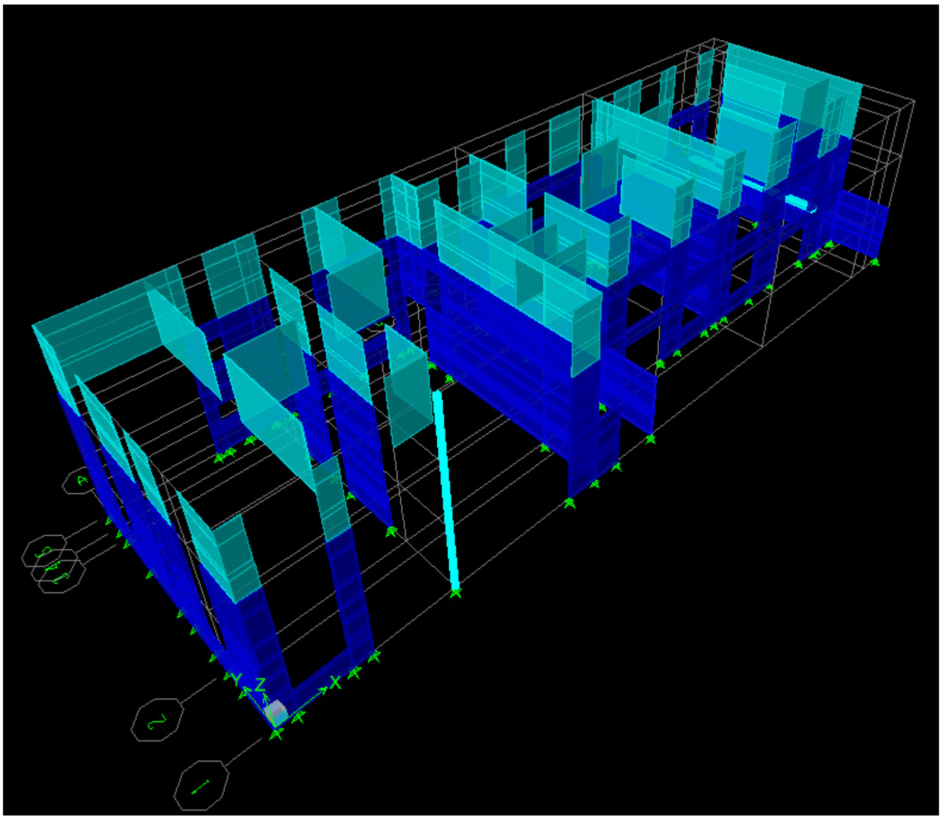


Figure A1: ETABS model of 3-storey portion of the west wing

The fundamental building periods are:

Table A3: Fundamental time periods of buildings

Building	Time period -E/W direction (s)	Time period –N/S direction (s)
West wing (3-storey portion)	0.06	0.13
West wing (2-Storey portion)	0.2	0.2
North wing	0.2	0.2

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 A4: Base shear from equivalent static method

Building	Base shear -E/W direction (KN)	Base shear –N/S direction (KN)
West wing (3-storey portion)	2,900	2,900
West wing (2-Storey portion)	840	840
North wing	2,510	2,510

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

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 summarised in further detail in the report and reported as %NBS.

Appendix 4 - Floor Level and Verticality Survey

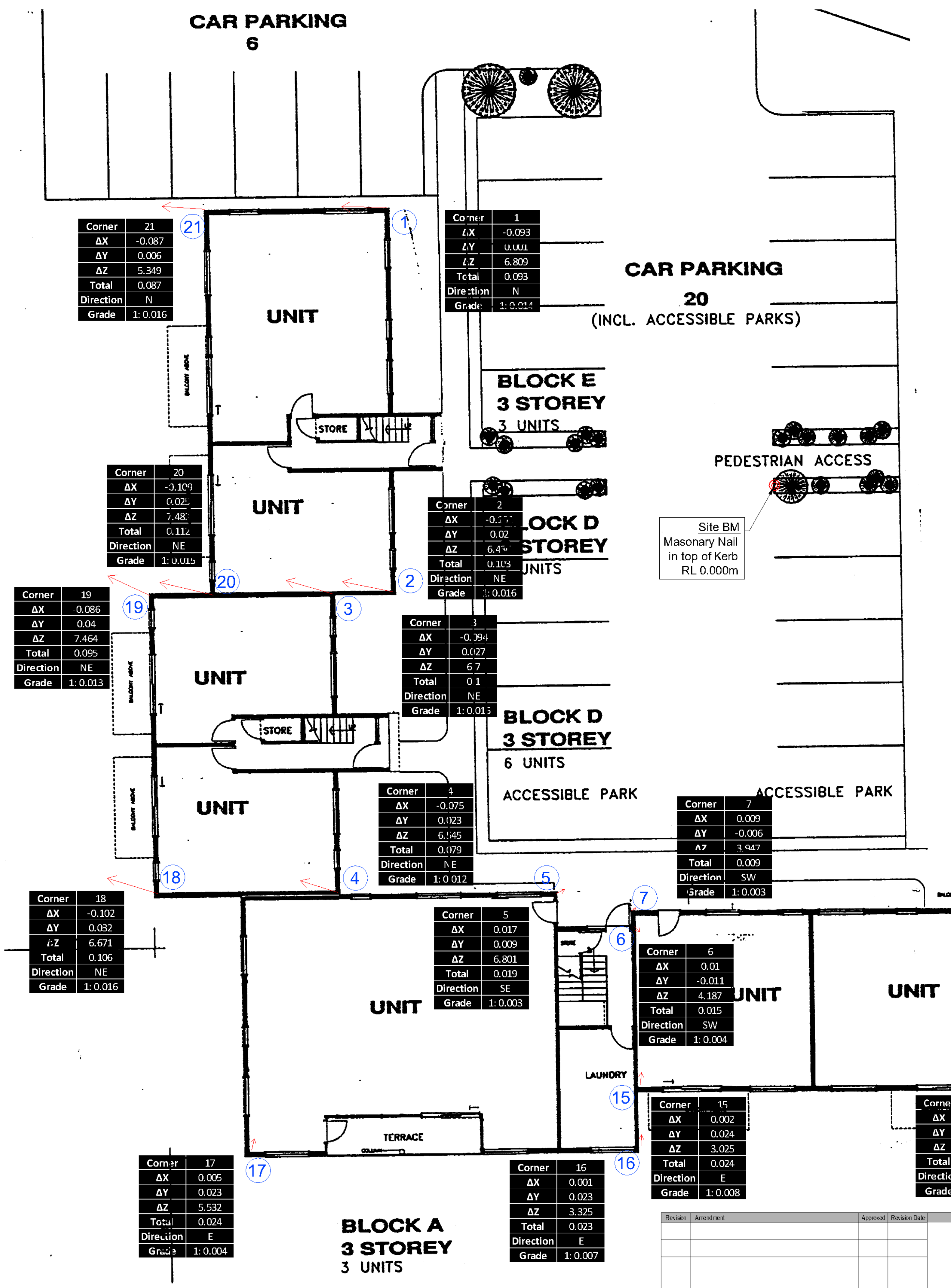
VERTICALITY ASSESSMENT NOTES:

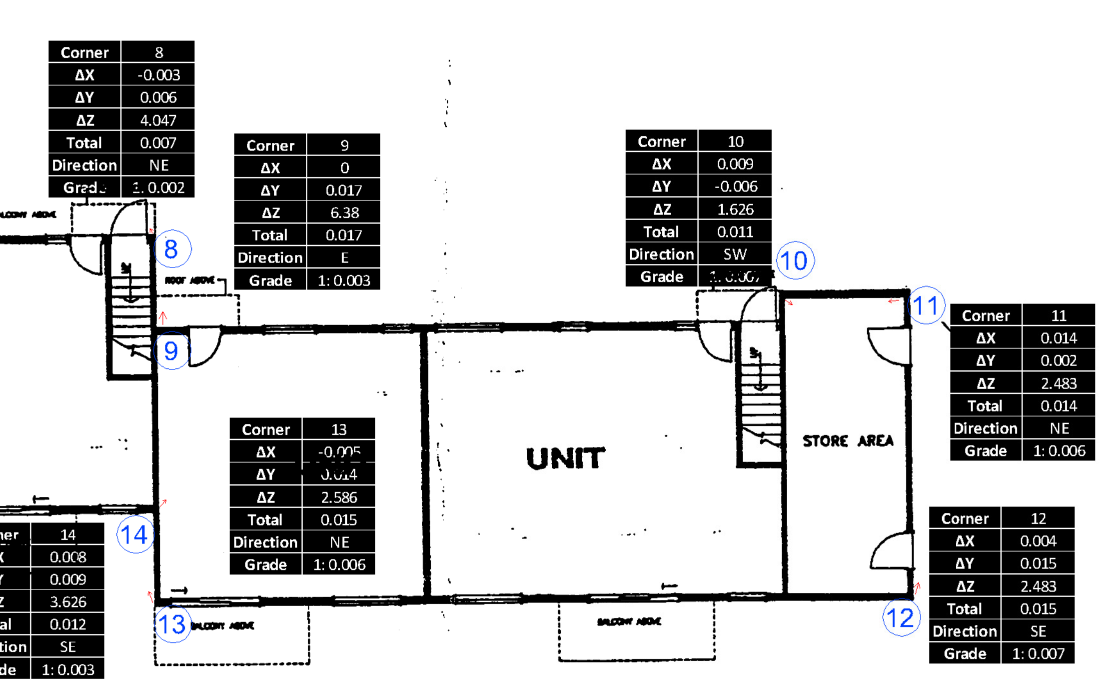
Verticality is assessed by surveying ɘatures which would have been architecturally vertical at time of construction or pre quake.

Due to objects obscuring views to thɛ entire height of walls / features full vertical assessment was unable to be undertaken

The results are shown in dx, dy, dz fırmat for easy interpretation.
dz value shows the height over whicł the verticality assessment was undertaken.
Total value shows the total lean in mn and Grade and the Direction in relation to the outside wall.

Coordinate and orientation datums aɛ assumed and based on control established onsite. A site benchmarklocated on the top of the kerb in the middle of the carpark is shown on the diagram.





Y
X

Revision	Amendment	Approved	Revision Date

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Drawn	Designed	Approved	Revision Date
C Yaxley		S Becker	20/06/2012

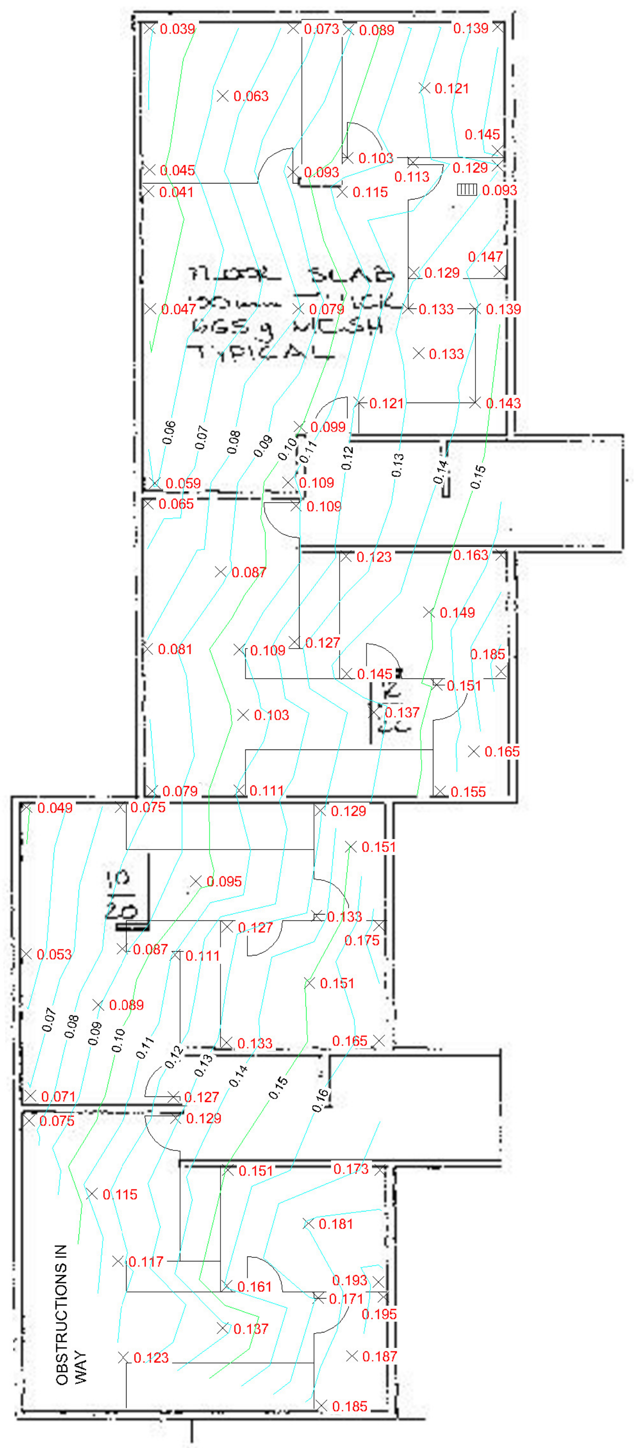
Project No.	Scale
6QUCCC.86	Not to Scale

Project
Tommy Taylor Courts
Levels and Verticality survey

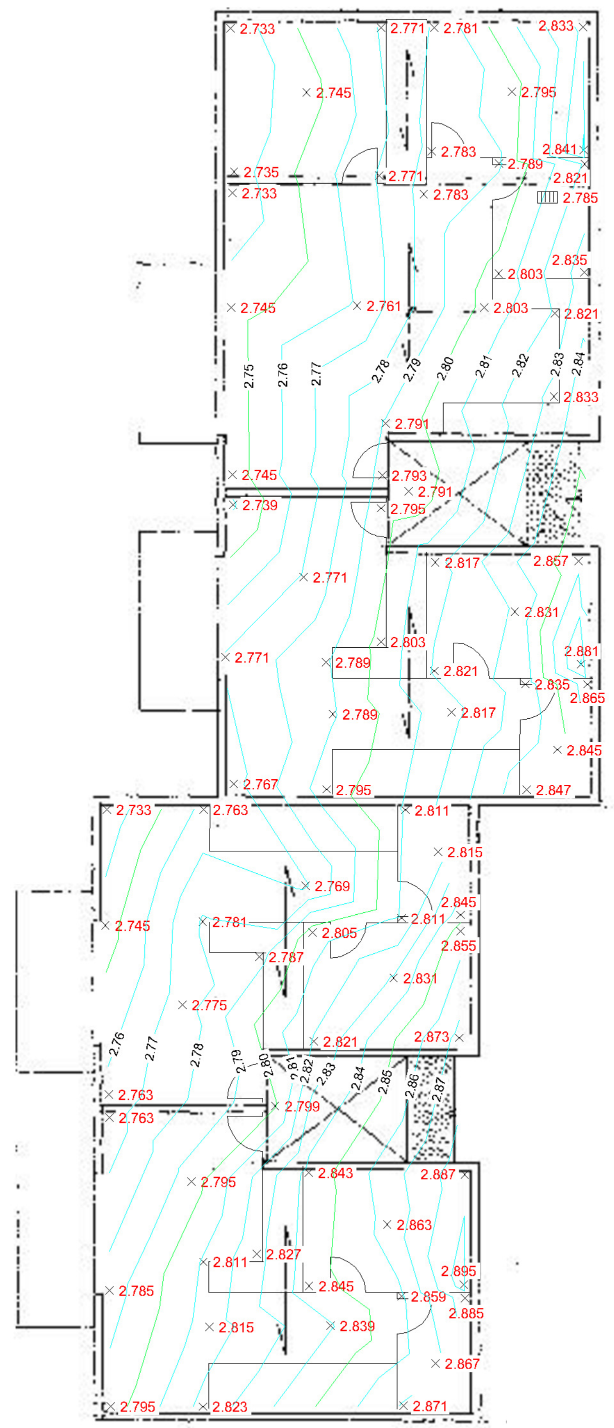
Sheet
Verticality Data

Drawing No.	Sheet No.	Revision
6/1366/272/2604	1	R0

Original Sheet Size A1 [841x594] Plot Date 05/07/12 @ 11:31 p:\projects\6-quake.01\ccc\ residential units\tommy taylor courts\tommy taylor courts\survey\civil 3d\tommy taylor level and verticality 180 1612.dwg - Verticality



NORTH WING GROUND FLOOR PLAN



NORTH WING FIRST FLOOR PLAN

Note. All levels are in terms of an assumed datum.
Datum Point: Is a masonary nail in the car park which has been given the RL 0.000m

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Drawn	Designed	Approved	Revision Date
J Morrow		S Becker	20/06/2012

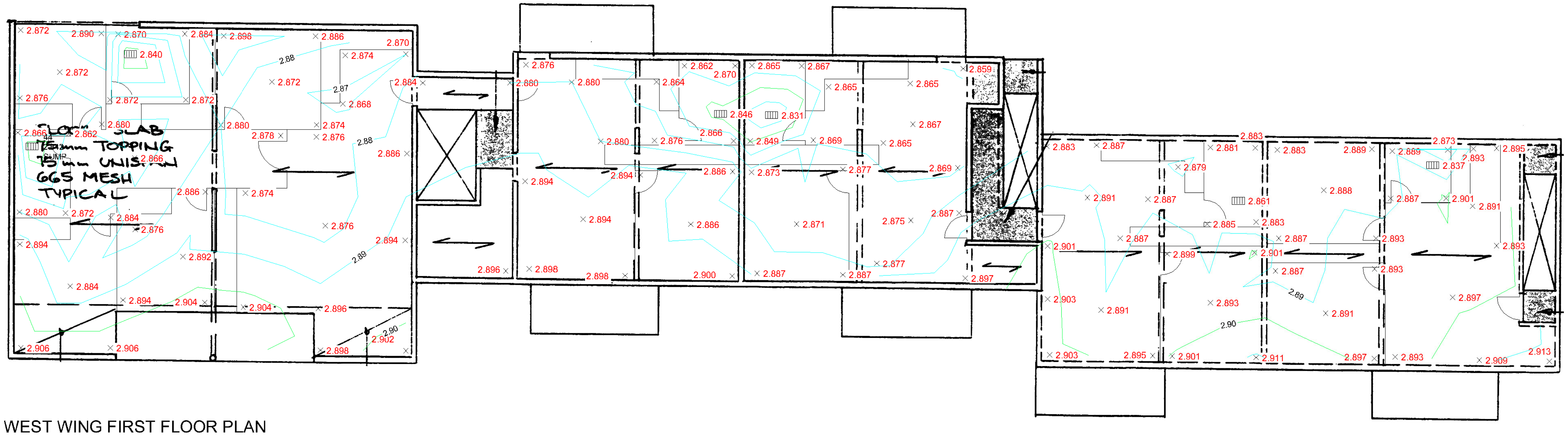
Project: Tommy Taylor Court:s Levels and Verticality survey

Sheet: North Wing Flocr Levels

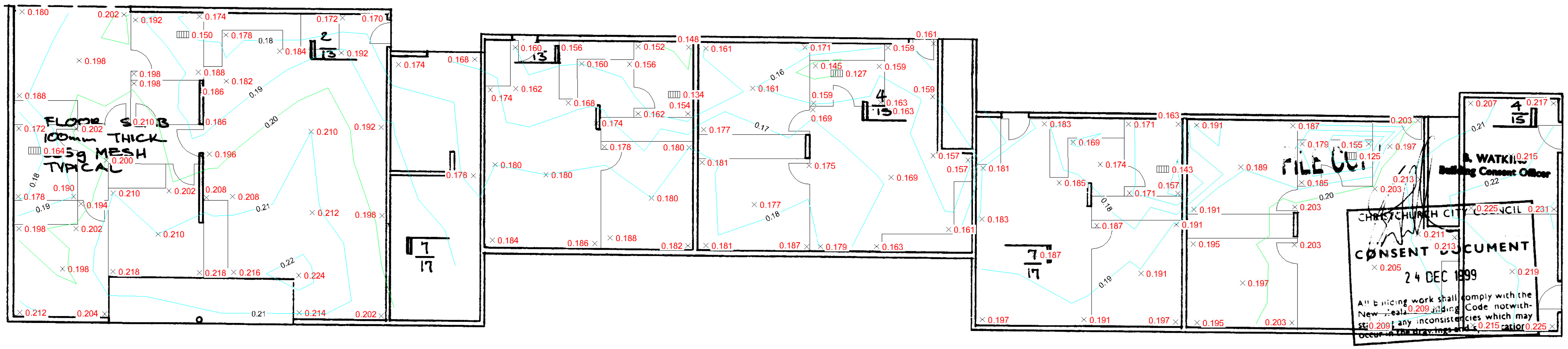
Project No.	Scale	Drawing No.	Sheet. No.	Revision
6QUCCC.86	1:75 @ A1, 1: 150 @ A3	6/1366/272/2604	2	R0

1:150 @ A1 1:300 @ A3 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 m

Original Sheet Size A1 [841x594] Plot Date 05/07/12 @ 11:31 p:\projects\6-quake.01\ccc\ residential units\tommy taylor courts\tommy taylor courts\survey\civil 3d\tommy taylor level and verticality 180 1612.dwg - North Wing Levels



WEST WING FIRST FLOOR PLAN



WEST WING GROUND FLOOR PLAN

Note. All levels are in terms of an assumed datum.
Datum Point: Is a masonary nail in the car park which has been given the
RL 0.000m

1:150 @ A1
1:300 @ A3 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 m

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Drawn	Designed	Approved	Revision Date
J Morrow		S Becker	20/06/2012

Project No.	Scale
6QUCCC.86	1:75 @ A1, 1: 150 @ A3

Project
Tommy Taylor Courts
Levels and Verticality survey

Sheet
West Wing Floor Levels

Drawing No.	Sheet No.	Revision
6/1366/272/2604	3	R0

Original Sheet Size A1 [841x594] Plot Date 05/07/12 @ 11:31 p:\projects\8-quake.01\cccl residential units\tommy taylor courts\tommy taylor courts\survey\civil 3d\tommy taylor level and verticality 180 1612.dwg - West Wing Levels

Appendix 5 – CERA DEE Spreadsheets

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Tommy Taylor Courts - North Wing	Reviewer:	Alistair Boyce
	Unit / No: 7 / Street: Cecil Place	CPEng No:	209860
Building Address:	7 Cecil Place	Company:	Opus International
Legal Description:		Company project number:	
		Company phone number:	
GPS south: (Degrees / Min / Sec)		Date of submission:	17/12/2012
GPS east:		Inspection Date:	17/6/2012
		Revision:	Final V2
Building Unique Identifier (CCC):	BU 1048-001 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:	3	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):	10.00		height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	270			
Age of Building (years):	13		Date of design:	1992-2004
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:	precast concrete with topping	unit type and depth (mm), topping	75mm topping over 75mm Unispan
Beams:	none	overall depth x width (mm x mm)	
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):	20
Ductility assumed, μ:	1.25		wall thickness (m):	0.6
Period along:	0.20	##### 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):	52
Ductility assumed, μ:	1.25		wall thickness (m):	0.6
Period across:	0.20	##### 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:	precast, half height	describe supports	
Wall cladding:	plaster system	describe	
Roof Cladding:	Metal	describe	
Glazing:	aluminium frames		
Ceilings:	strapped or direct fixed		gib ceiling
Services(list):			

Available documentation

Architectural	full	original designer name/date	Harman Halliday
Structural	full	original designer name/date	Harman Halliday
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:	100-200mm	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:	yellow		
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:	pounding
Pounding:	Damage?:	yes	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:	do not occupy	Describe:	
Along	Assessed %NBS before:	35% ##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Quantitative
	Assessed %NBS after:	35%		
Across	Assessed %NBS before:	35% ##### %NBS from IEP below		
	Assessed %NBS after:	35%		

IEP

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

Period of design of building (from above):	1992-2004	h_n from above:	m
Seismic Zone, if designed between 1965 and 1992:		not required for this age of building	
		Design Soil type from NZS4203:1992, cl 4.6.2.2:	

along across

	along	across
Period (from above):	0.2	0.2
(%NBS)nom from Fig 3.3:		
Note:1 for specifically design public buildings, to the code of the day: pre-1965 = 1.25; 1965-1976, Zone A =1.33; 1965-1976, Zone B = 1.2; all else 1.0		1.00
Note 2: for RC buildings designed between 1976-1984, use 1.2		1.0
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)		1.0
Final (%NBS)nom:	**0%**	**0%**

2.2 Near Fault Scaling Factor

	along	across
Near Fault scaling factor, from NZS1170.5, cl 3.1.6:		1.00
Near Fault scaling factor (1/N(T,D), **Factor A**:	1	1

2.3 Hazard Scaling Factor

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

2.4 Return Period Scaling Factor

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

2.5 Ductility Scaling Factor

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

2.6 Structural Performance Scaling Factor:

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

2.7 Baseline %NBS, $(NBS\%)_b$ = $(\%NBS)_{nom}$ x A x B x C x D x E **$\%NBS_b$:** **#DIV/0!** **#DIV/0!**

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

3.1. Plan Irregularity, factor A: 1

3.2. Vertical irregularity, Factor B: 1

3.3. Short columns, Factor C: 1

3.4. Pounding potential

Pounding effect D1, from Table to right 1.0

Height Difference effect D2, from Table to right 1.0

Therefore, Factor D: 1

3.5. Site Characteristics 1

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

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

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max value =1.5, no minimum		
Rationale for choice of F factor, if not 1		

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)

List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

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

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Tommy Taylor Courts - West Wing	Reviewer:	Alistair Boyce
	Unit / No: 7 / Street: Cecil Place	CPEng No:	209860
Building Address:	7 Cecil Place	Company:	Opus International
Legal Description:		Company project number:	
		Company phone number:	
	Degrees / Min / Sec		
GPS south:		Date of submission:	17/12/2012
GPS east:		Inspection Date:	17/6/2012
		Revision:	Final V2
Building Unique Identifier (CCC):	BU 1048-001 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:	3	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):	10.00		height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	480			
Age of Building (years):	13		Date of design:	1992-2004
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:	precast concrete with topping	unit type and depth (mm), topping	75mm topping over 75mm Unispan
Beams:	none	overall depth x width (mm x mm)	
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):	76
Ductility assumed, μ:	1.25		wall thickness (m):	0.6
Period along:	0.13	##### 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):	38
Ductility assumed, μ:	1.25		wall thickness (m):	0.6
Period across:	0.06	##### 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:	precast, half height	describe supports	
Wall cladding:	plaster system	describe	
Roof Cladding:	Metal	describe	
Glazing:	aluminium frames		
Ceilings:	strapped or direct fixed		gib ceiling
Services(list):			

Available documentation

Architectural	full	original designer name/date	Harman Halliday
Structural	full	original designer name/date	Harman Halliday
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
Pounding:	Damage?:	yes	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:	34%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Quantitative
	Assessed %NBS after:	34%			
Across	Assessed %NBS before:	34%	##### %NBS from IEP below		
	Assessed %NBS after:	34%			

IEP

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

Period of design of building (from above):	1992-2004	h_n from above:	m
Seismic Zone, if designed between 1965 and 1992:		not required for this age of building	
		Design Soil type from NZS4203:1992, cl 4.6.2.2:	

along across

	along	across
Period (from above):	0.13	0.06
(%NBS)nom from Fig 3.3:		

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

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

2.2 Near Fault Scaling Factor

	along	across
Near Fault scaling factor, from NZS1170.5, cl 3.1.6:		1.00
Near Fault scaling factor (1/N(T,D), **Factor A**:	1	1

2.3 Hazard Scaling Factor

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

2.4 Return Period Scaling Factor

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

2.5 Ductility Scaling Factor

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

2.6 Structural Performance Scaling Factor:

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

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

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

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

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

Table for selection of D1	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Alignment of floors within 20% of H	**0.7**	**0.8**	**1**
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Table for Selection of D2	Severe	Significant	Insignificant/none
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Height difference > 4 storeys	0.4	0.7	1
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3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max value =1.5, no minimum		
Rationale for choice of F factor, if not 1		

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

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