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Burwood / Pegasus Community Boardroom

BU 2637-001 EQ2

Detailed Engineering Evaluation

Quantitative Report

Version FINAL

5 Union St (133 Brighton Mall)
New Brighton

INFRASTRUCTURE | MINING & INDUSTRY | DEFENCE | PROPERTY & BUILDINGS | ENVIRONMENT

Burwood / Pegasus Community Boardroom
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Detailed Engineering Evaluation
Quantitative Report
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5 Union St (133 Brighton Mall)
New Brighton

Christchurch City Council

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Date
30 October 2012

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version - FINAL

5 Union St (133 Brighton Mall), New Brighton

Background

This is a summary of the Quantitative report for the building structure. This report assesses the seismic capacity of the building based on the gravity and seismic loads from the current loading standard of NZS1170:2002-2004, the material standards of NZS3101:2006 Concrete and NZS3404:1997 Steel, NZ Society for Earthquake Engineering (NZSEE) guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes as well as further building measurements and non-intrusive testing of the structure on visual inspections of 12th May 2012.

Brief Description

The Burwood / Pegasus Community Boardroom is located at 5 Union Street (also known as 133 Brighton Mall), New Brighton, Christchurch. The single storey building on the property consists the Boardroom and several other commercial tenancies.

From the plans, the date of construction of the building is 1976. The roof structure consists of light-weight metal roofing on timber purlins bolted to the steel roof beams. There is a lightweight metal canopy veranda bordering the west (Union St) and north (Brighton Mall) of the building.

The general construction details of this single level commercial (retail or office) building are steel roof beams attached to precast reinforced concrete columns. The concrete columns and infill concrete wall panels form a boundary fire wall to the eastern side of the building. To the other three sides of the building, steel posts provide support to the other ends of the roof beams. Internal walls are timber framed partitions with plasterboard lining.

Over the whole building area there are unreinforced concrete ground floor slabs. The building foundations are concrete strip footings to the perimeter of the building. Concrete foundation ground beams link the western and eastern perimeter footings.

Key Damage Observed

Key damage observed includes:-

- Minor cracking to internal plasterboard linings.

- Cracking to concrete columns
- Cracking to concrete wall panels
- Cracking and deformation to timber canopy posts
- Yielding of roof bracing

Building Capacity Assessment

Based on the site inspection, available drawings and the results of this quantitative assessment, the overall building capacity is 62% NBS. The building is therefore classified as an Earthquake Risk building.

Critical structural elements are the roof bracing which has yielded and is sagging and needs to be replaced, the steel and concrete columns. To meet a level of at least 67% NBS will require strengthening to these structural elements.

Details of % NBS for each critical building element are itemised below:

- Cantilever Concrete Columns are assessed as 67% NBS.
- Steel portal frame along the western side is assessed to be 62% NBS.

To increase the building seismic capacity to at least 67% NBS, the following strengthening options are can be considered:

- Installing additional steel roof bracing (as well as replacing the yielded existing bracing)
- Strengthening of the existing concrete cantilever columns or the installation of additional new concrete columns with new foundations to support the concrete wall panels
- Modifying the steel portal frames along the western wall by installing new steel tension wall bracing between the existing steel posts.

Pounding Effect

There is a seismic gap of 125 mm between the building and the adjacent building along the eastern boundary. This clearance is adequate to negate the potential effects of building pounding

Recommendations

GHD Limited recommends that further work be undertaken in order to develop the scope of the strengthening and repair options.

When developing a strengthening work scheme to increase the seismic capacity of the lateral structural resisting system as near as practicable to 100% NBS, and to at least 67% NBS, compliance with accessibility and fire requirements will need to be considered.

There are no critical structural weaknesses (CSW) or significant structural hazards for this Earthquake Risk building.

As the building has been assessed to have a %NBS greater than 34% NBS but less than 67% NBS, it is deemed to be Earthquake Risk. It is recommended that the strengthening options be explored to bring the %NBS of the building up to the required 67% in order to comply with Christchurch City Council policy regarding the strengthening of potentially Earthquake Risk buildings.

The building has been assessed as being potentially Earthquake Risk. As a result, it is recommended that the building can remain occupied, as per Christchurch City Council's policy regarding occupancy of potentially Earthquake Prone buildings.

1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of Burwood / Pegasus Community Boardroom.

This report is a Quantitative Assessment of the building structure, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011.

A quantitative assessment involves inspections of the building and a desktop review of existing structural and geotechnical information, including existing drawings and calculations, if available.

The purpose of the assessment is to determine the seismic capacity of the building against the current New Building Standard (NBS). Analyses of the seismic strength of critical building structural elements are assessed against the current NBS loads. Likely building performance and damage patterns are considered in the analyses to identify potential structural weaknesses or collapse hazards. From the identified critical structural weaknesses, the building strength in terms of percentage of new building standard (%NBS) is established.

At the time of this report, other than opening up of the ceiling to inspect the roof space, no intrusive site investigation of the building structure had been carried out. Construction drawings were made available. The building description below is based on our visual inspections and review of the construction plans available.

A Detailed Engineering Evaluation qualitative report produced for this building in March 2012 assessed the building to be 30% NBS which would be classified as an Earthquake Prone Building. From detailed structural analyses of the structure under seismic loading, this quantitative report establishes the strength of the current building and provides for strengthening and repair options for the building to meet a minimum strength standard of 67% NBS. This detailed engineering evaluation report assesses the building to be 62% NBS.

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 carry out a full structural survey before the building is re-occupied.

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

The qualitative assessment is a desk-top and site inspection assessment. It is based on a thorough visual inspection of the building coupled with a review of available documentation such as drawings and specifications. The quantitative assessment involves analytical calculation of the buildings strength and may require non-destructive or destructive material testing, geotechnical testing and intrusive investigation.

It is anticipated that factors determining the extent of evaluation and strengthening level required will include:

- The importance level and occupancy of the building
- The placard status and amount of damage
- The age and structural type of the building
- Consideration of any critical structural weaknesses
- The extent of any earthquake damage

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 any 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)) be satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'. Regarding seismic capacity 'as near as reasonably practicable' has previously been interpreted by CCC as achieving a minimum of 67% NBS however where practical achieving 100% NBS is desirable. The New Zealand Society for Earthquake Engineering (NZSEE) recommend a minimum of 67% NBS.

2.2.1 Section 121 – Dangerous Buildings

The definition of dangerous building in the Act was extended by the Canterbury Earthquake (Building Act) Order 2010, and it now defines a building as dangerous if:

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

Section 122 – Earthquake Prone Buildings

This section defines a building as earthquake prone if its ultimate capacity would be exceeded in a 'moderate earthquake' and it would be likely to collapse causing injury or death, or damage to other property. A moderate earthquake is defined by the building regulations as one that would generate ground shaking 33% of the shaking used to design an equivalent new building.

Section 124 – Powers of Territorial Authorities

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

Section 131 – Earthquake Prone Building Policy

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

2.3 Christchurch City Council Policy

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

The 2010 amendment includes the following:

- A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
- A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;
- A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
- 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.

We anticipate that any building with a capacity of less than 33% NBS (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67% NBS of new building standard as recommended by the Policy.

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

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

2.4 Building Code

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

After the February Earthquake, on 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

- Hazard Factor increased from 0.22 to 0.3 (36% increase in the basic seismic design load)
- Serviceability Return Period Factor increased from 0.25 to 0.33 (80% increase in the serviceability design loads when combined with the Hazard Factor increase)

The increase in the above factors has resulted in a reduction in the level of compliance of an existing building relative to a new building despite the capacity of the existing building not changing.

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 new building standard load requirements have been determined in accordance with the current earthquake loading standard (NZS 1170.5:2004 Structural design actions - Earthquake actions - New Zealand).

The likely capacity of this building has been derived in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) guidelines 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes' (AISPBE), 2006. These guidelines provide an Initial Evaluation Procedure that assesses a buildings capacity based on a comparison of loading codes from when the building was designed and currently. It is a quick high-level procedure that can be used when undertaking a Qualitative analysis of a building. The guidelines also provide guidance on calculating a modified Ultimate Limit State capacity of the building which is much more accurate and can be used when undertaking a Quantitative analysis.

The New Zealand Society for Earthquake Engineering has proposed a way for classifying earthquake risk for existing buildings in terms of %NBS and this is shown in Figure 3.1 below.

Description	Grade	Risk	%NBS	Existing Building Structural Performance	Improvement of Structural Performance	
					Legal Requirement	NZSEE Recommendation
Low Risk Building	A or B	Low	Above 67	Acceptable (improvement may be desirable)	The Building Act sets no required level of structural improvement (unless change in use) This is for each TA to decide. Improvement is not limited to 34%NBS.	100%NBS desirable. Improvement should achieve at least 67%NBS
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally. Improvement recommended		Not recommended. Acceptable only in exceptional circumstances
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement	Unacceptable	Unacceptable

Figure 1 NZSEE Risk Classifications Extracted from Table 2.2 of the NZSEE 2006 AISPBE

Table 3.1 compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% risk of exceedance in 50 years (i.e. 0.2% in the next year). It is noted that the current seismic risk in Christchurch results in a 6% risk of exceedance in the next year.

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

Table 1 %NBS compared to relative risk of failure

4. Building Description

4.1 General

The Burwood / Pegasus Community Boardroom is located at 5 Union Street (also known as 133 Brighton Mall), New Brighton, Christchurch. The single storey building on the property consists the Boardroom and several other commercial tenancies.

From the plans, the date of construction of the building is 1976. The roof structure consists of light-weight metal roofing on timber purlins bolted to the steel roof beams. There is a lightweight metal canopy veranda bordering the west (Union St) and north (Brighton Mall) of the building.

The general construction details of this single level commercial (retail or office) building are steel roof beams attached to precast reinforced concrete columns. The concrete columns and infill concrete wall panels form a boundary fire wall to the eastern side of the building. To the other three sides of the building, steel posts provide support to the other ends of the roof beams. Internal walls are timber framed partitions with plasterboard lining.

Over the whole building area there are unreinforced concrete ground floor slabs. The building foundations are concrete strip footings to the perimeter of the building. Concrete foundation ground beams link the western and eastern perimeter footings. The construction plans are included in Appendix B. The building appears to be well constructed with no apparent critical structural weaknesses. Key structural details of the building are shown in Figure 2 below.

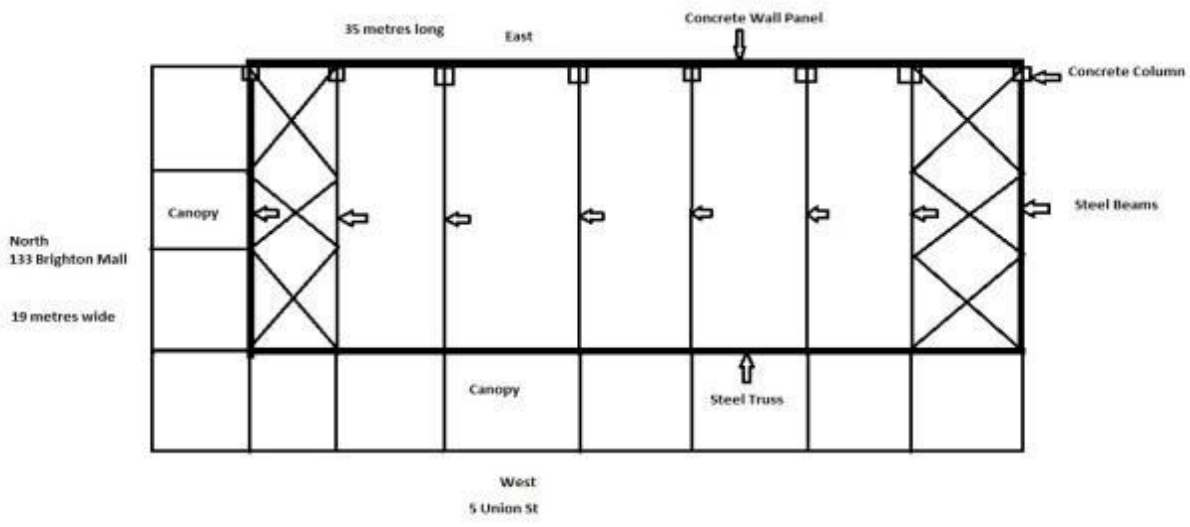


Figure 2 Plan sketch showing key structural elements

4.2 Gravity Load Resisting System

Gravity loads from the metal roofing are transferred into the timber roof purlins, then onto the steel portal beams or steel trusses. From the beams or trusses, the loads are then transferred to the steel or concrete columns located around the building perimeter. Column loads are carried into the foundation pads or strip footing which transfer the loads directly into the ground.

Canopy loads are supported by timber posts and the building lintel beams. The timber posts transfer the loads to pad foundations which then transfer the loads into the ground.

4.3 Lateral Load Resisting System

The lateral load resisting system consists of steel portal frames and concrete shear walls. Lateral loads from the roof level are either transferred through the roof bracing to the steel portal frames or directly into the concrete shear walls.

For lateral loading across (west – east direction) the building, loads at roof level are transferred to the steel roof beams and into the eastern cantilever concrete columns which are connected to the concrete foundation beams. The concrete columns are primary elements providing seismic restraint to the building and these columns are the most critical structural elements for seismic actions across the building. This is evident from the quake damage cracking found near the base of these columns.

For lateral loading along (north – south direction) the building, loads at roof level are transferred through the steel roof bracing into the western steel portal truss frame and also into the concrete columns and infill shear wall panels along the eastern side of the building. From the portal frame columns and infill walls, lateral loads are transferred into the continuous foundation beams below the ground floor slab. With the stiff concrete infill wall panels along the eastern side and a flexible steel moment frame along the western side, seismic lateral load eccentricities to the building are created. The roof bracing system redistributes theses load eccentricities into the concrete columns.

The photographs in Appendix A and plans in Appendix B show that all primary steel members are joined by fully welded connections.

5. Assessment

5.1 Site Inspection

An inspection of the building was undertaken on the 12th of May 2012. Both the interior and exterior of the building was inspected. The building was observed to have a green placard in place. No inspection of the foundations of the structure was able to be undertaken.

The inspection consisted of observing the building to determine the structural systems and likely behaviour of the building during an earthquake. The site was assessed for damage, including observing the ground conditions, checking for damage in areas where damage would be expected for the structure type observed and noting general damage observed throughout the building in both structural and non-structural elements.

5.2 Investigation & Opening Up Work

Assess to the roof space was gained to inspect and photograph the roof framing and supporting structure and to inspect for structural damage.

5.3 Modelling

2D frame analyses using Microstran was undertaken to model the building structure for 100% NBS loads. The loads from the analyses were then checked against the structural member capacities derived from the various material codes (NZS3101 concrete and NZS3404 steel). Critical members were then identified and various options for replacement, modification or strengthening of these were considered for these members. These proposed options were then re-analysed and compared with the NBS loads. These options were further modified until the weakest members have sufficient strength capacity to resist at least 67% NBS loading.

5.4 Calculations

The calculations were undertaken to check the strength capacity of various structural elements such as roof bracing, portal frames, columns and foundations.

Tension capacity of the roof bracing was checked against the applied loads. Concrete column bending and shear capacities were checked against the applied loads. From the frame model analyses member loads were checked against member strength capacities. Foundation loads were checked against the foundation strength capacities.

Structural elements that were identified to be less than 67% NBS capacity from the modelling and analyses were considered for strengthening or replacement to meet a minimum seismic loading of 67% NBS to the building.

5.4.1 Building Demand

Steel Framed Portion of the Structure

Self-weight of the structure was calculated from the Table A1 of Appendix A of NZS 1170.1: 2002.

For ductile and limited ductility structures with seismic-resisting systems located along two perpendicular directions, the specified actions may be assumed to act separately along each of these two horizontal directions as set out in Cl 5.3.1.1 of NZS 1170.5: 2004.

Individual member bending moment, shear force and axial force demands were extracted from a finite element analysis of the frame.

5.4.2 Seismic Weight Coefficient

The elastic site hazard spectrum for horizontal loading, C(T), for the building was derived from Equation 3.1(1) of NZS 1170.5;

$$\boldsymbol{C(T) = C_h\, Z\, R\, N(T.D)}$$

Where

$C_h(T)$ = the spectral shape factor determined from CL 3.1.2

Z = the hazard factor from CL 3.1.4 and the subsequent amendments which increased the hazard factor to 0.3 for Christchurch

R = the return period factor from Table 3.5 for an annual probability of exceedance of 1/500 for an Importance Level 2 building

N(T,D) = the near-fault scaling facto from CL 3.1.6

The structural performance factor, S_P, was calculated in accordance with CL 4.4.2

$$\mathbf{S_P = 1.33 - 0.3\mu}$$

Where μ, the structural ductility factor, was taken as 2.00.

The seismic weight coefficient was then calculated in accordance with Cl 5.2.1.1 of NZS 1170.5: 2011. For the purposes of calculating the seismic weight coefficient a period, T_1, of 0.4 was assumed for the building. The coefficient was then calculated using Equation 5.2(1);

$$\mathbf{C_d(T_1) = \frac{C(T_1)S_P}{k_\mu}}$$

Where

$$\mathbf{k_\mu = \frac{(\mu - 1)T_1}{0.7} + 1}$$

5.4.3 Member Bending Moment Capacity (Section 5.1 of NZS 3404:1997)

A member bent about the section major principle axis shall satisfy:

$$\boldsymbol{M_x^* \le \Phi M_{sx}} \text{ and}$$

$$\boldsymbol{M_x^* \le \Phi M_{bx}}$$

Where

$$\boldsymbol{M_x^* = the\ design\ bending\ moment\ from\ analysis}$$

$$\boldsymbol{\Phi = the\ strength\ reduction\ factor\ from\ Table\ 3.3\ of\ NZS\ 3604: Part\ 1\ 1997}$$

$$M_{sx} = \textit{the nominal section capacity in bending, as specified in Clause } 5.2 \textit{ of NZS } 3404\text{: }\textit{Part } 1\ 1997$$

$$M_{bx} = \textit{the nominal member capacity in bending, as specified in Clause } 5.3 \textit{ or } 5.6 \textit{ of NZS } 3404\text{: }\textit{Part } 1\ 1997$$

For hollow sections, the nominal member capacity in bending, M_{bx}, always equals the nominal section capacity in bending, M_{sx}, according to clause 5.6.1.4 of NZS 3404: Part 2 1997.

5.4.4 Member Shear Capacity (Section 5.9 of NZS 3404:1997)

A member web subjected to shear force, V^*, shall satisfy:

$$V^* \leq \Phi V_v$$

Where

$$V^* = \textit{the design shear force from analysis}$$

$$\Phi = \textit{the strength reduction factor from Table } 3.3 \textit{ of NZS } 3404\text{: }\textit{Part } 1\ 1997$$

$$V_v = \textit{the nominal section capacity of the web, as determined in Clause } 5.11.2 \textit{ of NZS } 3404\text{: }\textit{Part } 1\ 1997$$

5.4.5 Member Shear and Bending Moment Interaction (Section 5.12 of NZS 3404:1997)

A member subjected to bending moment, M_x^*, and shear force, V^*, shall have its nominal web shear capacity, V_{vm}, calculated using the equations set out in Clause 5.12.2 of NZS 3404: Part 1 1997. The web design shear capacity in the presence of bending moment shall satisfy

$$V^* \leq \Phi V_{vm}$$

Where

$$V^* = \textit{the design shear force from analysis}$$

$$\Phi = \textit{the strength reduction factor from Table } 3.3 \textit{ of NZS } 3404\text{: }\textit{Part } 1\ 1997$$

$$V_{vm} = \textit{the nominal section capacity of the web, modefied for the presence of bending as determined in Clause } 5.12.2 \textit{ of NZS } 3404\text{: }\textit{Part } 1\ 1997$$

5.4.6 Member Axial Capacity (Section 6 of NZS 3404:1997)

A concentrically loaded member subject to a design axial compressive force, N^*, shall comply with both:

$$N^* \leq \Phi N_s$$

$$N^* \leq \Phi N_c$$

Where

$$N^* = \textit{the design axial force from analysis}$$

$$\Phi = \textit{the strength reduction factor from Table } 3.3 \textit{ of NZS } 3404\text{: }\textit{Part } 1\ 1997$$

$$N_s = \textit{the nominal section capacity, as determined in Clause } 6.2.1.1 \textit{ of NZS } 3404\text{: }\textit{Part } 1\ 1997$$

$$N_c = \text{the nominal member capacity , as determined in Clause 6.3 of NZS 3404: Part 1 1997}$$

5.4.7 Member Combined Axial and Bending Moment Capacity (Section 8 of NZS 3404:1997)

A member subject to uniaxial bending and axial actions need not be checked for combined actions when the axial force is not significant as defined by Cl 8.1.4 of NZS 3404:1997. The design axial force shall be considered significant unless it complies with:

$$N^* \leq 0.05\Phi N_s \text{ if the member is subject to uniaxial bending and is an I or channel section}$$

$$N^* \leq 0.05\Phi N_c \text{ if the member is subject to uniaxial bending and is any other cross section}$$

Where axial force is considered significant, the following general design provision should be satisfied:

$$M_x^* \leq \Phi M_{rx}$$

Where

$$M_x^* = \text{the design bending moment from analysis}$$

$$\Phi = \text{the strength reduction factor from Table 3.3 of NZS 3404: Part 1 1997}$$

$$M_{rx} = \text{the nominal section moment capacity, reduced by axial force as specified in Clause}$$

$$\text{8.3.2.1 of NZS 3404: Part 1 1997}$$

5.4.8 Connection Bolt Shear Capacity (Section 9 of NZS 3404:1997)

A bolt subject to a design shear force shall satisfy (Cl 9.3.2.1 of NZS 3404:1997);

$$V_f^* \leq \Phi V_f$$

Where

$$V_f^* = \text{the design shear action from analysis}$$

$$\Phi = \text{the strength reduction factor from Table 3.3 of NZS 3404: Part 1 1997}$$

$$V_f = \text{the nominal bolt shear capacity, as specified in Clause}$$

$$\text{9.3.2.1 of NZS 3404: Part 1 1997}$$

5.4.9 Connection Bolt Tension Capacity (Section 9 of NZS 3404:1997)

A bolt subject to a design tensile force shall satisfy (Cl 9.3.2.2 of NZS 3404:1997);

$$N_{tf}^* \leq \Phi N_{tf}$$

Where

$$N_{tf}^* = \text{the design tensile action from analysis}$$

$$\Phi = \text{the strength reduction factor from Table 3.3 of NZS 3604: Part 1 1997}$$

$$N_{tf} = \text{the nominal bolt tensile capacity, as specified in Clause}$$

$$\text{9.3.2.2 of NZS 3404: Part 1 1997}$$

5.4.10 Connection Bolt Combines Shear and Tension Capacity (Section 9 of NZS 3404:1997)

A bolt required to resist both design shear and design tension forces at the same time shall satisfy (Cl 9.3.2.3 of NZS 3404:1997);

$$\left(\frac{V_f^*}{\Phi V_f}\right)^2 + \left(\frac{N_{tf}^*}{\Phi N_{tf}}\right)^2 \leq 1.0$$

Where

$N_{tf}^* = \textit{the design tensile action from analysis}$

$\Phi = \textit{the strength reduction factor from Table } 3.3 \textit{ of NZS } 3604{:}\ \textit{Part } 1\ 1997$

$N_{tf} = \textit{the nominal bolt tensile capacity, as specified in Clause } 9.3.2.2 \textit{ of NZS } 3404{:}\ \textit{Part } 1\ 1997$

$V_f^* = \textit{the design shear action from analysis}$

$V_f = \textit{the nominal bolt shear capacity, as specified in Clause } 9.3.2.1 \textit{ of NZS } 3404{:}\ \textit{Part } 1\ 1997$

6. Damage Assessment

6.1 Surrounding Buildings

This building is located between roads and a commercial property. There is a 124 mm seismic gap between the boardroom building and the adjoining eastern building. During the inspections there was no apparent damage to the adjacent building.

6.2 Residual Displacements and General Observations

Some minor residual displacements of the structure were noticed during our inspection of the building. There is a canopy post that appears to be bowed from lateral displacement of its footing. Minor cracking was noted to the precast concrete wall panel and the precast concrete columns.

There is minor cracking to the interior plasterboard linings to the interior walls and ceilings of the law office but this is not considered to be significant. A number of ceiling tiles are displaced. Several cracked and shattered windows were replaced after the recent large seismic events. Some minor movement and cracking was identified to some of the timber posts supporting the canopies.

There is evidence of some yielding of the roof braces but no sign of any failure of the steel members or their connections. From the crack damaged columns, broken windows and sagging roof bracing, this indicates that there is possibly insufficient bracing along the western side of the building.

Other than some minor crack damage to concrete columns and walls, timber posts and to interior plasterboard linings and a bowed post, there was no other structural damage found to the building.

6.3 Ground Damage

No significant ground damage was observed during the site inspections. There is no evidence of foundation settlement other than one bowed timber post supporting the canopy as a likely consequence of localised ground liquefaction.

7. Analysis

7.1 Seismic Parameters

Earthquake loads shall be calculated using New Zealand Code.

◗ Site Classification	D
◗ Seismic Zone factor (Z)	
(Table 3.3, NZS 1170.5:2004, NZBC Clause B1 Structure)	0.30 (Christchurch)
◗ Annual Probability of Exceedance	
(Table 3.3, NZS 1170.0:2002)	1/500 (ULS) Importance Level 2
◗ Annual Probability of Exceedance	
(Table 3.3, NZS 1170.0:2002)	1/25 (SLS)
◗ Return Period Factor (Ru)	
(Table 3.5, NZS 1170.5:2004)	1.0 (ULS)
◗ Return Period Factor (Rs)	
(Table 3.5, NZS 1170.5:2004 & NZBC Clause B1)	0.33 (SLS)
◗ Ductility Factor (μ)	2.00
◗ Performance Factor (Sp)	0.70
◗ Gravitational Constant (g)	9.81 m/sec2

An increased Z factor of 0.30 for Christchurch has been used in line with recommendations from the Department of Building and Housing recommendations resulting in a reduced % NBS score.

7.2 Structural Ductility Factor

A structural ductility factor of 2.0 has been assumed in both the long and short direction of the building based on the steel and concrete portal frame system as indicated on the available drawings. The portal frames have been assessed as the limiting structural elements in terms of the ductility of the structure and the ability to dissipate energy during an earthquake. As a result, the structural ductility factor of 2.0 associated with the moment resisting steel portal frames has been used for this purpose of the Detailed Engineering Evaluation Quantitative Assessment.

8. Geotechnical Investigation

Burwood/Pegasus Community Boardroom is located in New Brighton, Christchurch and is accessed from Union Street. The site is predominantly flat and approximately 1m above mean sea level. The site is also 350m west of the sea at Pegasus Bay, and approximately 500 east of the Avon River.

8.1 Published Information on Ground Conditions

8.1.1 Published Geology

The geological map of the area[1] indicates that the site is underlain by Holocene soils of the Christchurch Formation, comprising dominantly sand derived from fixed or semi-fixed dune and beach deposits.

8.1.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that twelve boreholes are located within a 200m radius of the site). Of these boreholes, three of them are summarised below.

Bore Name	Dist. From site	Log Summary
M35/2306	50m NE	0 to 42.0m Sand and Clay 42.0 to 48.7m Gravel 48.7 to 72.8m Blue & brown Sand 72.8 to 93.8m Clay and Gravel 93.8 to 96.4m Brown Gravel (WT at 4.6m bgl)
M35/2374	100m NW	0 to 41.7m Sand and Clay 41.7 to 49.9m Blue Gravel 49.9 to 70.4m Blue Clay & Sand 70.4 to 88.6m layers of clay and gravel (WT at 4.6m bgl)
M35/2412	120m E	0 to 42.6m Blue Sand 42.6 to 44.1m Clay & Peat 44.1 to 47.2m Blue Gravel 47.2 to 50.2m Clay & Peat 50.2 to 68.9m Yellow Sand & Clay 68.9 to 76.8m Yellow Sand & Gravel 76.8 to 103.6m Layers of Blue Clay, Brown Gravel, and Clay & Peat (WT at 3.66m bgl)

Table 8.1 ECan Bore Log Summary Table

[1] Brown, L. J. and Weeber J.H. 1992: *Geology of the Christchurch Urban Area*. Institute of Geological and Nuclear Sciences 1:25,000 Geological Map 1. IGNS Limited: Lower Hutt.

It should be noted that the boreholes were sunk for groundwater extraction and not for geotechnical purposes. Therefore, the amount of material recovered and available for interpretation and recording will have been variable at best and may not be representative. The logs have been written by the well driller and not a geotechnical professional or to a standard. In addition strength data is not recorded.

8.1.3 EQC Geotechnical Investigations

The Earthquake Commission has undertaken geotechnical testing in the area of the site. Information pertaining to this investigation is included in a Tonkin and Taylor Report[2]. Within 200 m of the property two investigation probes points were undertaken, the results of which are detailed below in Table 8.2.

Bore Name	Dist. From Site	Log Summary
CPT – NBT – 43	180m S	0 – 1.0 m Silt
		1 – 5.2 m Dense to medium dense coarse beach Sand
		5.2 – 7.0 m Silty sand to fine beach Sand
		7.0 – 18.3 m Dense medium beach Sand
		18.3 – 19.1 m Sandy Clay
		19.1 – 22.9 m Dense medium beach Sand
		22.9 – 24.1 m Sandy Clay & Clay interbedded
		24.1 – 27.8 m Dense fine Sand
CPT – NBT - 44	250m W	0 – 1.0 m Soft silt
		1.0 – 9.0 m Dense medium beach Sand
		9.0 – 19.0 m Dense fine beach Sand
		19.0 – 20.4 m Sandy Clay to Clay interbeds
		20.4 – 23.5 m Dense fine Sand
		23.5 – 24.7 m Clay to Sandy Clay interbeds
		24.7 – 32.3 m Dense fine to medium Sand
		32.3 – 36.1 m Sandy Clay
		36.1 – 39.7 m Dense medium Sand
		39.7 – 40.8 m Very stiff Clay

Table 8.2 EQC Geotechnical Investigation ECan Bore Log Summary Table

8.1.4 Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has indicated the site is situated within the Green Zone, indicating that repair and rebuild may take place.

[2] Tonkin & Taylor Ltd (2011): Christchurch Earthquake Recovery, *Geotechnical Factual Report, New Brighton*

Land in the CERA green zone has been divided into three technical categories. These categories describe how the land in expected to perform in future earthquakes.

The site is indicated as being within the TC2 (yellow) zone[3]. This means that moderate to significant land damage from liquefaction is possible in future significant earthquakes.

8.1.5 Post-Earthquake Liquefaction Observations

Aerial photography taken following the 22 February 2011 earthquake examined for the purposes of this investigation showed no signs of liquefaction (see Figure 8.1 below).

Figure 8.1 Post February 2011 Earthquake Aerial Photography[4]

8.2 Seismicity

8.2.1 Nearby Faults

There are many faults in the Canterbury region, however only those considered most likely to have an adverse effect on the site are detailed below.

[3] CERA, Map of Technical Categories, http://cera.govt.nz/maps/technical-categories

[4] Aerial Photography Supplied by Koordinates sourced from http://koordinates.com/layer/3185-christchurch-post-earthquake-aerial-photos-24-feb-2011/

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	140 km	NW	8.3	~300 years
Greendale (2010) Fault	28 km	SW	7.1	~15,000 years
Hope Fault	110 km	N	7.2~7.5	120~200 years
Kelly Fault	120 km	NW	7.2	~150 years
Porters Pass Fault	70 km	NW	7.0	~1100 years

Table 8.3 Summary of Known Active Faults

Recent earthquakes since 22 February 2011 have identified the presence of a new active fault system / zone underneath Christchurch City and the Port Hills. Research and published information on this system is in development and not generally available and average recurrence intervals are yet to be estimated.

8.2.2 Ground Shaking Hazard

This seismic activity has produced earthquakes of Magnitude-6.3 with peak ground accelerations (PGA) up to twice the acceleration due to gravity (2g) in some parts of the city. This has resulted in widespread liquefaction throughout Christchurch.

New Zealand Standard NZS 1170.5:2004 now quantifies the Seismic Hazard factor for Christchurch as 0.30, being in a moderate to high earthquake zone. This value has been provisionally upgraded recently (from 0.22) to reflect the seismicity hazard observed in the earthquakes since 4 September 2010.

8.3 Field Investigations

In order to further understand the ground conditions at the site, intrusive testing comprising one CPTU investigation was conducted at the site on 04 April 2012.

The location of this test is tabulated in Table 8.4.

Investigation	Depth (m bgl)	Easting (NZMG)	Northing (NZMG)
CPT 001	20	2487982	5744329

Table 8.4 Coordinates of Investigation Locations

The CPTU investigation was undertaken by McMillans Drilling Ltd on 04 April 2012 to a target depth of 20m below ground level.

Interpretation of output graphs[5] from the investigation showing Cone Tip Resistance (q_c), Friction Ratio (Fr), Inferred Lithology and Inferred Liquefaction Potential are presented in Table 8.5 and Table 8.6.

Please refer to Appendix D for further detail.

[5] McMillans Drilling CPT data plots, Appendix D.

8.4 Ground Conditions Encountered

Depth (m)	Lithology [1]	ConeTip Resistance q_c (MPa)	Friction Ratio Fr (%)	Relative Density Dr (%)
0 – 20.0	SAND	10 – 20	0.8	80 – 100

Table 8.5 Summary of CPT-Inferred Lithology

The ground conditions encountered in the investigation comprise sand to 20m. This is consistent with the ECan boreholes and the EQC investigations, which show medium dense to dense sands (with some layers of clay) to ~30m bgl. It is anticipated the sands are underlain by gravels below 40m bgl.

8.5 Liquefaction Assessment

8.5.1 Parameters used in Analysis

Assumptions made for the analysis process are as follows:

- D50 particle sizes for the site soil (sands) from CPT soil analysis
- Hazard factor for Christchurch Z = 0.30
- Importance Level 2, post seismic event (50-year design life)- R = 1.0
- Spectral shape factor C = 1.12 (for class D, E)
- PGA a_h= Z· R· C = (0.30) (1.0) (1.12) = 0.34g.

The following equation has been used to approximate soil unit weight from the CPT investigation data: [6]

$$\gamma = \frac{\gamma_w Gs}{2.65}\left(0.27\log Fr\ +\ 0.36\log\left(\frac{qc}{p_{atm}}\right) + 1.236\right)$$

This gave unit weight values ranging between 19.0 and 20.0 kN/m^3 (saturated).

The liquefaction analysis process has been conducted using the methodology from Robertson & Wride[7], and from the NZGS Guidelines[8].

8.5.2 Results of Liquefaction Analysis

The results of the liquefaction analysis, as outlined in Table 8.6, indicate that depths to 20m are considered not liquefiable.

[6] Robertson P.K., & Cabal K.L. (2010): *Estimating soil unit weight from CPT*. Gregg Drilling & Testing Inc.: Signal Hill, California, USA.

[7] Robertson, P.K. & Wride, C.E. (1998): *Evaluating cyclic liquefaction potential using the cone penetration test.* Canadian Geotechnical Journal, Vol 35, pp.442–459.

[8] Cubrinovski M., McManus K.J., Pender M.J., McVerry G., Sinclair T., Matuschka T., Simpson K., Clayton P., Jury R. 2010: *Geotechnical earthquake engineering practice: Module 1 – Guideline for the identification, assessment and mitigation of liquefaction hazards*. NZ Geotechnical Society

Depth (m)	Lithology	Triggering Factor F_L	Liquefaction Susceptibility [9]
0 – 20.0	Beach SAND	> 1.9	*Not Liquefiable*

Table 8.6 Summary of Liquefaction Susceptibility

Please refer to Appendix C for the Liquefaction Analysis spreadsheets.

8.5.3 Interpretation of Analysis

Overall, the site is considered to be not susceptible to liquefaction. This interpretation is supported by the field evidence which showed there was no liquefaction at the site in both the Darfield and Christchurch earthquakes, and by EQC investigations in Appendix C.

8.6 Interpretation of Ground Conditions

8.6.1 Liquefaction Potential

The site is considered prone to minor to moderate amounts of liquefaction during further earthquakes as evidenced by:

- Inspections of the site on 25 January and 4 April 2012 noting no signs of liquefaction;
- CERA's classification of TC2, indicating minor to moderate land damage from liquefaction is likely in future significant earthquakes. However, the results in Appendix C indicate liquefaction is not likely; and,
- The ground conditions underlying the site are understood to be medium dense to dense sands.

8.6.2 Slope Failure and/or Rockfall Potential

The site is located within New Brighton, a flat suburb in eastern Christchurch. Global slope instability and Rockfall potential are considered negligible. However, any localised retaining structures and/or embankments should be further investigated to determine the site-specific slope instability potential.

8.6.3 Foundation Recommendations

The soil class of **D** (in accordance with NZS 1170.5:2004) previously reported is still considered appropriate, and this should be adopted for the site.

Based on the information presented above, we recommend the following for the subject site:

- All foundations be specifically-designed by a suitably qualified and experienced geotechnical engineer; and,
- Ground improvement works are not recommended.

[9] Table 6.1, NZGS Guidelines Module 1 (2010)

9. Results

Critical structural elements are the sagging steel roof bracing which appear to have yielded. These should be replaced with new bracing. The structural analysis of the other structural elements has identified critical items which impact on the %NBS to the building.

The critical structural item for seismic actions along the building (in north – south direction) is:

- The bending strength of the portal frame columns along the western wall which is assessed as 62% NBS

The critical structural item for seismic actions across the building (in east – west direction) is:

- The concrete column bending strength which is assessed as 67% NBS

Structural weaknesses identified are the roof bracing, the portal frame along the western side and the cantilever concrete columns.

10. Conclusions

The overall building has been assessed to have a seismic capacity in the order:

62% of NBS along the building

67% of NBS across the building.

Therefore this building is classified as an Earthquake Risk building with 62%NBS.

The identified structural items for strengthening are the portal frame steel columns along the western wall for seismic actions along the building and the concrete columns for seismic actions acting across the building

Strengthening of these items or reducing the seismic load to these items by installing new structural elements will increase the seismic capacity of the building.

The most effective strengthening option will be to install new steel roof bracing to the building as this will reduce the seismic loads along the building acting on the critical structural elements Also installing steel bracing between columns along the windows to the western wall would be an option to strengthen the portal frame along the western elevation..

Installing interior bracing walls across the building would also be effective in improving the %NBS acting across the building.

There are no critical structural weaknesses (CSW) or significant structural hazards for this building and normal usage and occupancy of the building may be permitted. As the building has been assessed to have 62%NBS, it is deemed to be an Earthquake Risk building.

11. Recommendations

GHD Limited recommends that strengthening works be carried out to the building to achieve at least 67% NBS.

In developing a strengthening scheme to increase the seismic capacity of the building to or as near is practical to 100% NBS, the work will need to consider the accessibility and fire requirements of the building.

As the building has been assessed to have a %NBS greater than 34% NBS but less than 67% NBS, it is deemed to be Earthquake Risk. It is recommended that strengthening options be explored to bring the %NBS of the building up to the required 67% in order to comply with Christchurch City Council policy regarding the strengthening of potentially Earthquake Risk buildings.

The building has been assessed as being potentially Earthquake Risk. As a result, it is recommended that the building can remain occupied, as per Christchurch City Council's policy regarding occupancy of potentially Earthquake Prone buildings.

12. Limitations

12.1 General

This report has been prepared subject to the following limitations:

- The foundations of the building were unable to be inspected.
- Other than creating openings in ceilings to access the roof space, no intrusive structural investigations have been undertaken.
- No intrusive geotechnical investigations have been undertaken.
- No level or verticality surveys have been undertaken.
- No material testing has been undertaken.

It is noted that this report has been prepared at the request of Christchurch City Council and is intended to be used for their purposes only. GHD accepts no responsibility for any other party or person who relies on the information contained in this report.

12.2 Geotechnical Limitations

The data and advice provided herein relate only to the project and structures described herein and must be reviewed by a competent geotechnical engineer before being used for any other purpose. GHD Limited (GHD) accepts no responsibility for other use of the data by third parties.

Where drill hole or test pit logs, cone tests, laboratory tests, geophysical tests and similar work have been performed and recorded by others under a separate commission, the data is included and used in the form provided by others. The responsibility for the accuracy of such data remains with the issuing authority, not with GHD.

The advice tendered in this report is based on information obtained from the desk study investigation location test points and sample points. It is not warranted in respect to the conditions that may be encountered across the site other than at these locations. It is emphasised that the actual characteristics of the subsurface materials may vary significantly between adjacent test points, sample intervals and at locations other than where observations, explorations and investigations have been made. Subsurface conditions, including groundwater levels and contaminant concentrations can change in a limited time. This should be borne in mind when assessing the data.

It should be noted that because of the inherent uncertainties in subsurface evaluations, changed or unanticipated subsurface conditions may occur that could affect total project cost and/or execution. GHD does not accept responsibility for the consequences of significant variances in the conditions and the requirements for execution of the work.

The subsurface and surface earthworks, excavations and foundations should be examined by a suitably qualified and experienced Engineer who shall judge whether the revealed conditions accord with both the assumptions in this report and/or the design of the works. If they do not accord, the Engineer shall modify advice in this report and/or design of the works to accord with the circumstances that are revealed.

An understanding of the geotechnical site conditions depends on the integration of many pieces of information, some regional, some site specific, some structure specific and some experienced based. Hence this report should not be altered, amended or abbreviated, issued in part and issued incomplete in any way without prior checking and approval by GHD. GHD accepts no responsibility for any circumstances which arise from the issue of the report which have been modified in any way as outlined above.

Appendix A
Photographs

North Elevation at 133 Brighton Mall

West Elevation at 5 Union St

South Elevation at Beresford St

Moderate Crack damage to concrete column

Moderate Crack Damage to concrete column

Roof beam to concrete column

Roof beam to concrete column at courtyard

Welded connection roof beam to concrete column

Welded connection concrete column to concrete wall panel

Welded beam to post joint

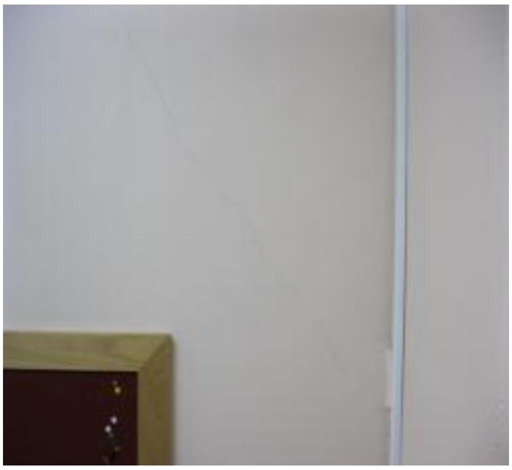
Western wall line beam and post

Southern wall line

Crack to Concrete wall panel

Concrete wall crack

Gap between column and wall



Concrete wall concrete

Plaster lining crack at wall - ceiling

Sagging (stretched) roof bracing

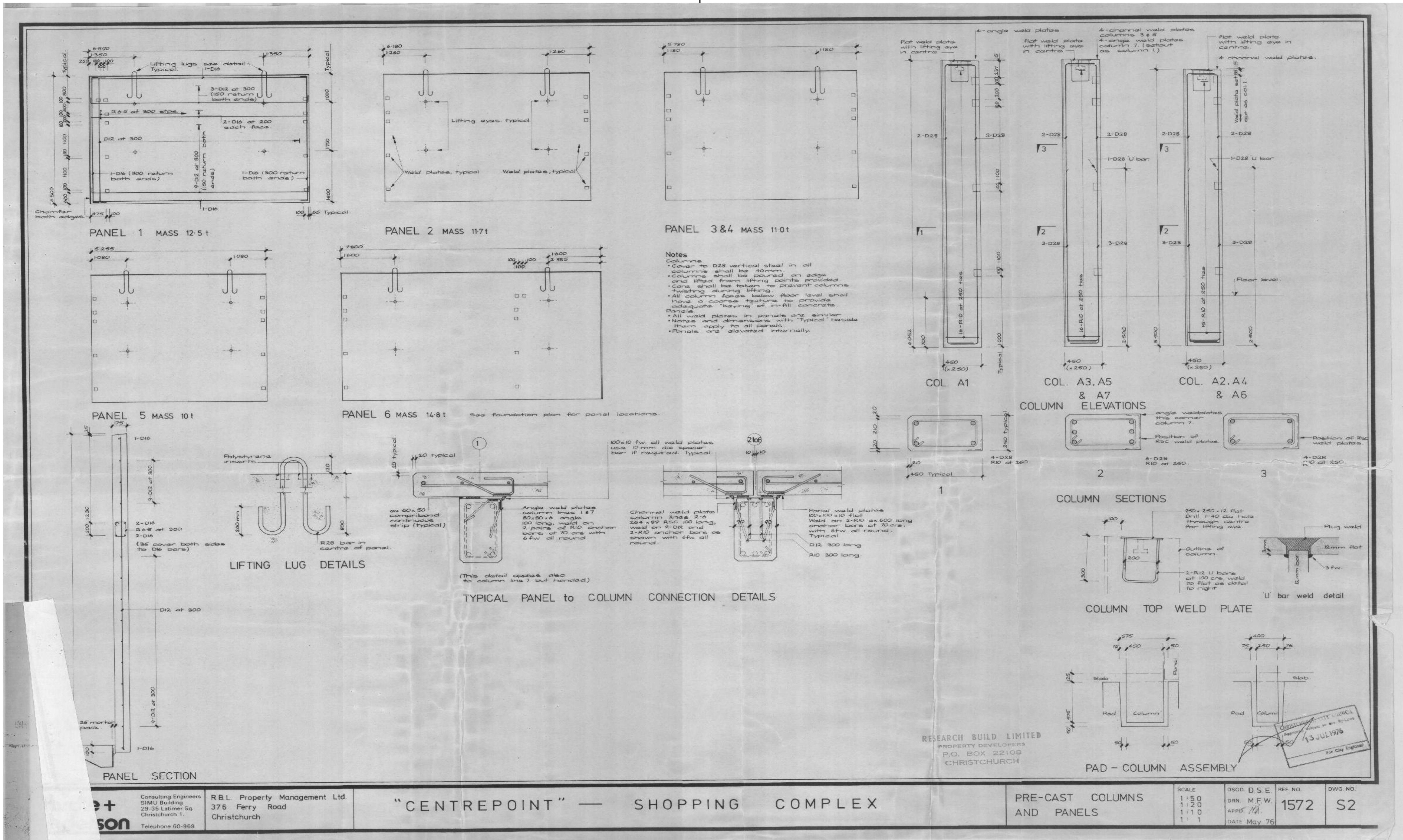
Roof purlins and roof bracing

Crack in timber post of canopy

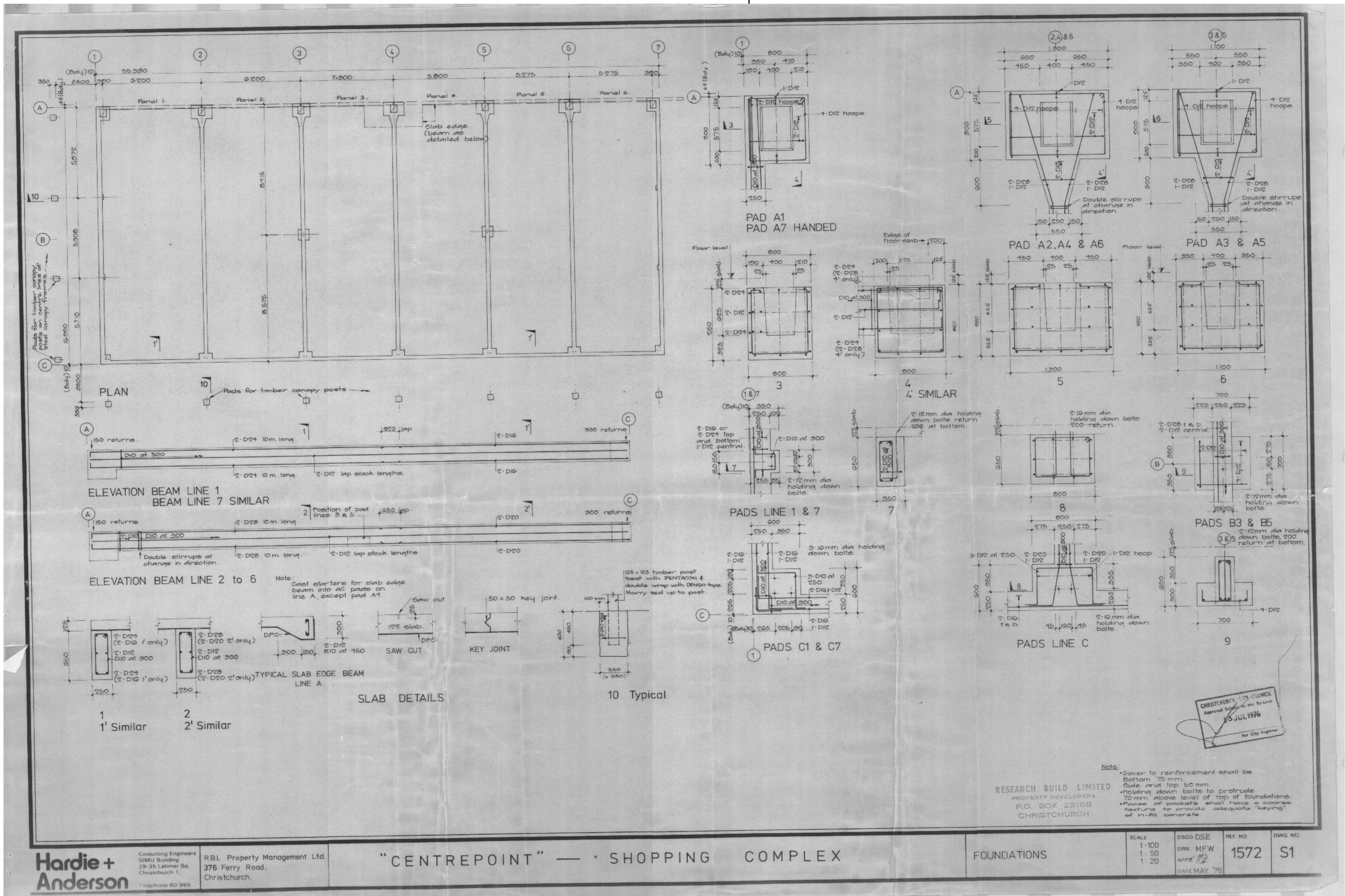
Bowing timber post to canopy

Appendix B

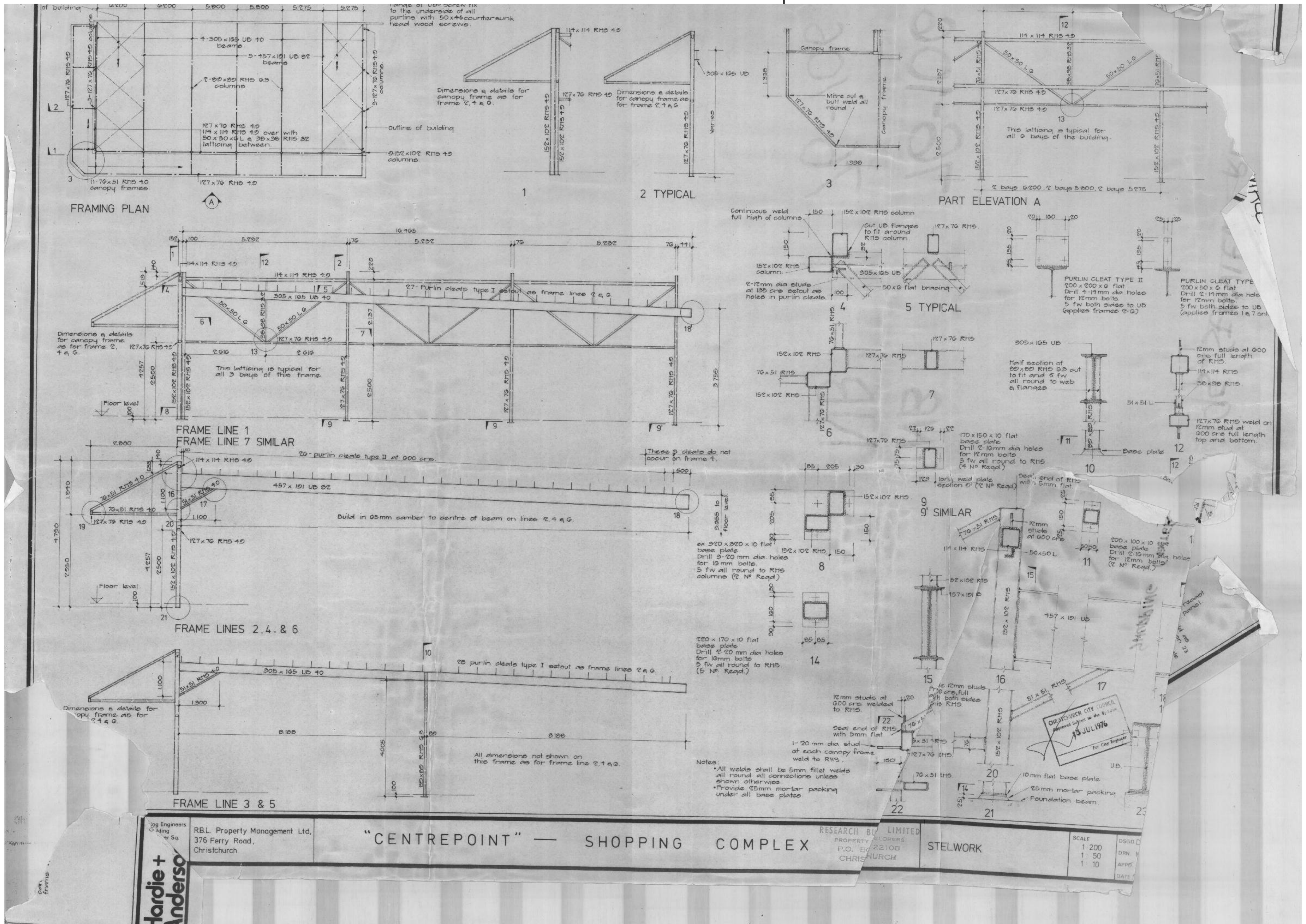
Existing Drawings / Sketches



Concrete Wall Panels and Columns



Foundations and floor slab



Steel beams, posts and bracing

Appendix C

Geotechnical Investigation Results and Liquefaction Analysis

CPT ANALYSIS NOTES

Soil Type

Interpretation using chart of Robertson & Campanella (1983). This is a simple but well proven interpretation using cone tip resistance (q_C) and friction ratio (f_R) only. No normalisation for overburden stress is applied. Cone tip resistance measured with the piezocone is corrected with measured pore pressure (u_C).

- sand (and gravel)
- silt-sand
- silt
- clay-silt
- clay
- peat

Liquefaction Screening

The purpose of the screening is to highlight susceptible soils, that is sand and silt-sand in a relatively loose condition. This is not a full liquefaction risk assessment which requires knowledge of the particular earthquake risk at a site and additional analysis. The screening is based on the chart of Shibata and Teparaksa (1988).

- high susceptibility
- medium susceptibility
- low susceptibility

High susceptibility is here defined as requiring a shear stress ratio of 0.2 to cause liquefaction with D_{50} for sands assumed to be 0.25 mm and for silty sands to be 0.05 mm.

Medium susceptibility is here defined as requiring a shear stress ratio of 0.4 to cause liquefaction with D_{50} for sands assumed to be 0.25 mm and for silty sands to be 0.05 mm.

Low susceptibility is all other cases.

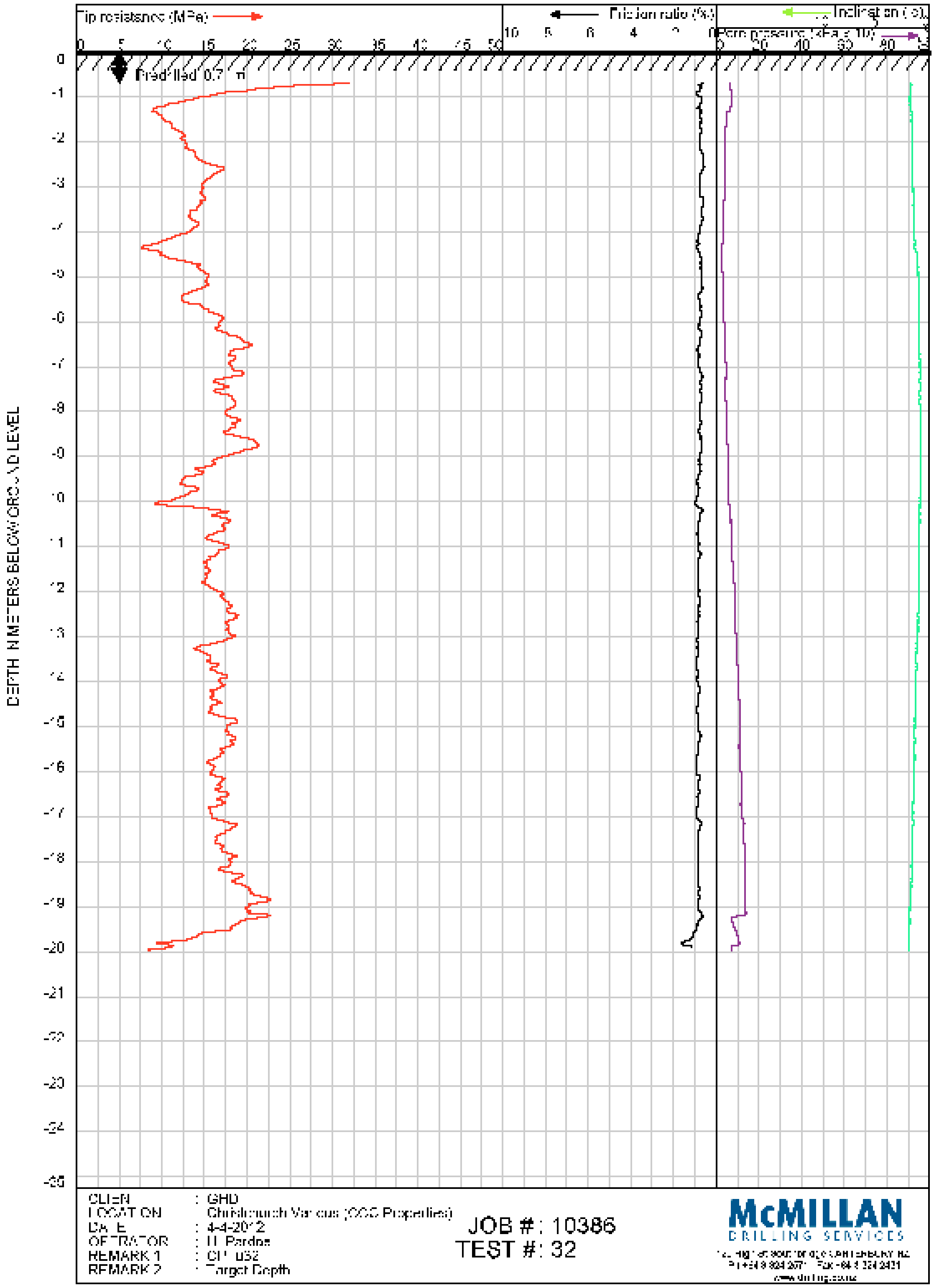
Relative Density (D_R)

Based on the method of Baldi et. al. (1986) from data on normally consolidated sand.

Undrained Shear Strength (S_U)

Derived from the bearing capacity equation using $S_U = (q_C - \sigma_{VO})/15$.

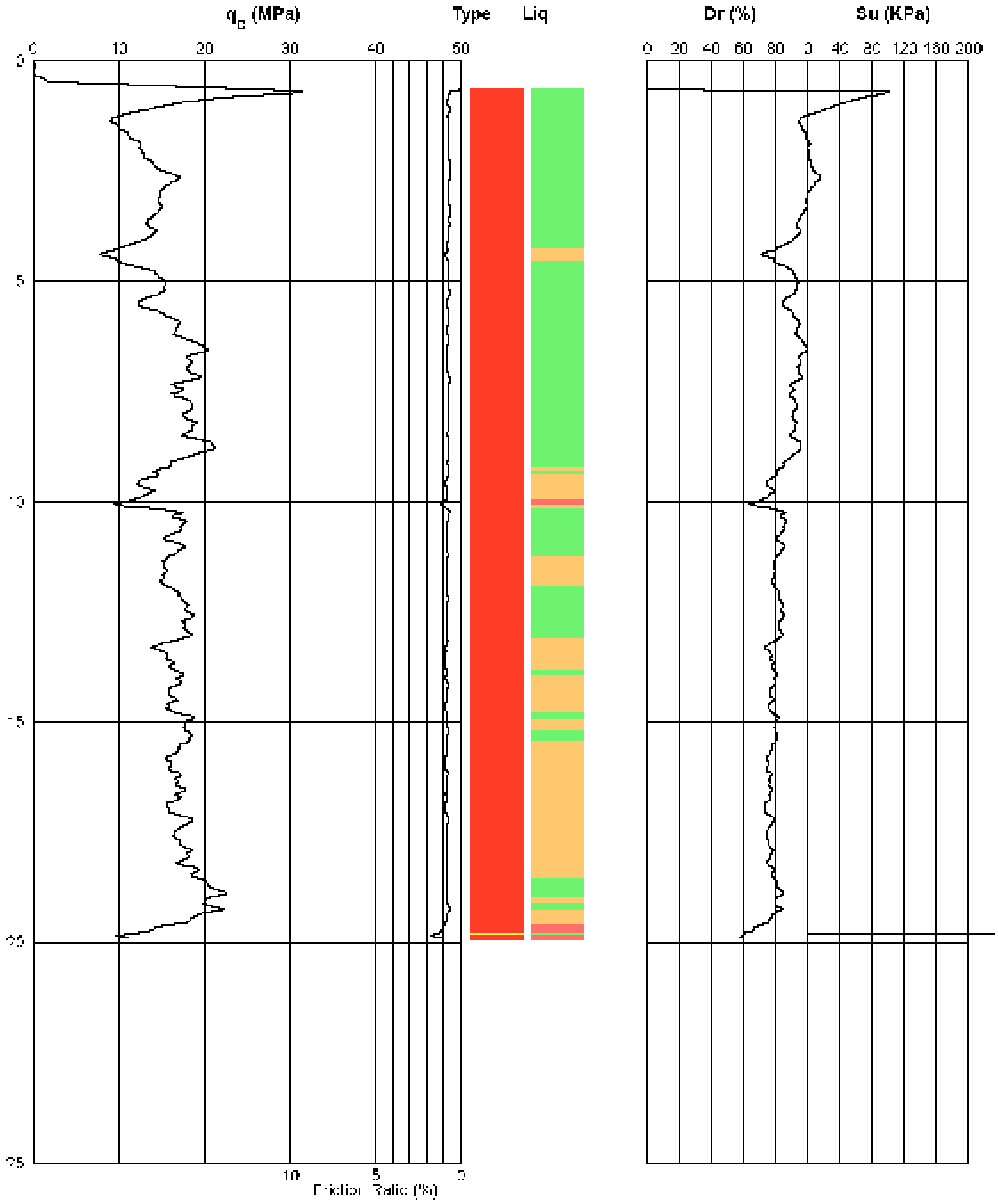
McMILLAN
DRILLING SERVICES



51/30596/32/

Detailed Engineering Evaluations Quantitative Report
Burwood / Pegasus Community Boardroom

PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



Job No:	10356	Date:	4-4-2012
CPT No:	CPTu32	Operator:	H. Pardoe
Project:	GHD	Remark:	Target Depth
Location:	Christchurch Various (CCC Properties)		

McMILLAN DRILLING SERVICES

SOIL LIQUEFACTION SUSCEPTIBILITY ASSESSMENT

LOCATION : CPT 01
PROJECT : Burwood-Pegasus Community Board Room
JOB NO : 51 30596 32
DATE : 26 April 2012

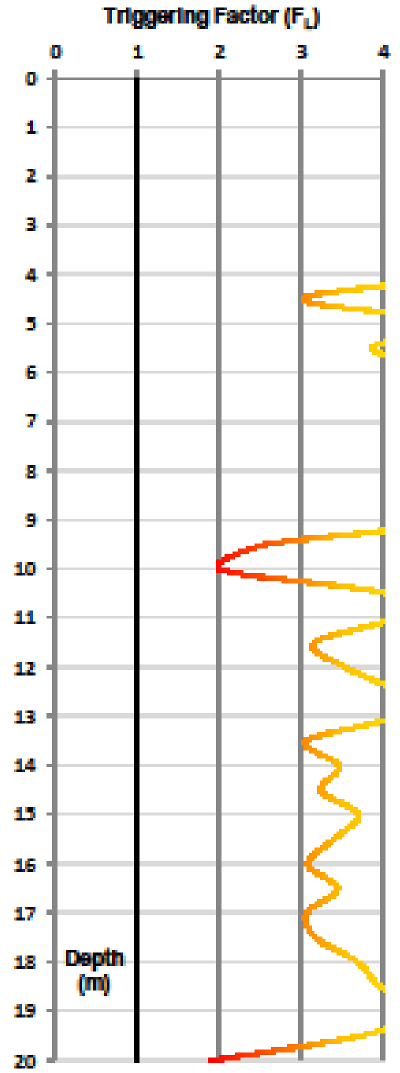
SHEET : 1
CALCULATED BY : MH
CHECKED BY :

CPT No: CPT 01
GWL: 3.7 m
a_{max}: 0.34 g

p_{atm}: 101 kPa
Bore depth: 20.0 m
EQ Magnitude: 5

Soil Level From	Soil Level To	Thick-ness (m)	γ, Soil Unit Weight (kN/m³)	Cone Resistance q_c (MPa)	Sleeve Friction Stress, f_s (MPa)	Soil Total Stress σ_{vo} (kPa)	Soil Effective Stress σ'_{vo} (kPa)	Cyclic Stress Ratio, CSR	Cyclic Resistance Ratio, CRR	Soil Behavior (Robertson, 2010)	Triggering Factor F_L	Liquefaction Potential
0	0.5	0.5	19.7	7.86	0.17	9.8	9.8	0.00		SANDS: clean sand to silty sand		NL
0.5	1	0.5	19.9	15.33	0.16	19.8	19.8	0.00		SANDS: clean sand to silty sand		NL
1	1.5	0.5	18.8	10.05	0.08	29.2	29.2	0.00		SANDS: clean sand to silty sand		NL
1.5	2	0.5	19.2	12.65	0.10	38.8	38.8	0.00		SANDS: clean sand to silty sand		NL
2	2.5	0.5	19.3	15.54	0.10	48.4	48.4	0.00		SANDS: clean sand to silty sand		NL
2.5	3	0.5	19.5	14.74	0.11	58.2	58.2	0.00		SANDS: clean sand to silty sand		NL
3	3.5	0.5	19.2	14.08	0.09	67.8	67.8	0.00		SANDS: clean sand to silty sand		NL
3.5	4	0.5	19.3	13.36	0.10	77.4	74.5	0.22	1.29	SANDS: clean sand to silty sand		NL
4	4.5	0.5	18.8	9.88	0.08	86.8	79.0	0.24	0.71	SANDS: clean sand to silty sand	3.01	Negligible
4.5	5	0.5	19.4	15.50	0.11	96.6	83.8	0.25		SANDS: clean sand to silty sand		NL
5	5.5	0.5	19.3	12.31	0.11	106.2	88.5	0.25	0.98	SANDS: clean sand to silty sand	3.84	Negligible
5.5	6	0.5	19.7	16.81	0.13	116.0	93.5	0.26		SANDS: clean sand to silty sand		NL
6	6.5	0.5	19.9	20.39	0.15	126.0	98.5	0.27		SANDS: clean sand to silty sand		NL
6.5	7	0.5	19.9	18.15	0.15	135.9	103.5	0.28		SANDS: clean sand to silty sand		NL
7	7.5	0.5	19.6	16.77	0.12	145.7	108.4	0.28		SANDS: clean sand to silty sand		NL
7.5	8	0.5	19.7	17.53	0.13	155.6	113.4	0.28		SANDS: clean sand to silty sand		NL
8	8.5	0.5	19.8	17.71	0.14	165.5	118.4	0.29		SANDS: clean sand to silty sand		NL
8.5	9	0.5	19.7	17.29	0.13	175.3	123.3	0.29		SANDS: clean sand to silty sand		NL
9	9.5	0.5	19.4	12.44	0.11	185.0	128.1	0.29	0.75	SANDS: clean sand to silty sand	2.55	Negligible
9.5	10	0.5	19.3	10.09	0.11	194.7	132.9	0.29	0.59	SANDS: clean sand to silty sand	2.01	Negligible
10	10.5	0.5	19.8	17.21	0.14	204.6	137.8	0.29	1.19	SANDS: clean sand to silty sand		Negligible
10.5	11	0.5	19.8	17.79	0.14	214.4	142.8	0.29	1.20	SANDS: clean sand to silty sand		Negligible
11	11.5	0.5	19.6	15.42	0.13	224.2	147.7	0.29	0.92	SANDS: clean sand to silty sand	3.15	Negligible
11.5	12	0.5	19.7	16.86	0.13	234.1	152.7	0.29	1.03	SANDS: clean sand to silty sand	3.53	Negligible
12	12.5	0.5	19.8	18.61	0.15	244.0	157.7	0.29	1.19	SANDS: clean sand to silty sand		Negligible
12.5	13	0.5	19.9	18.61	0.15	253.9	162.7	0.29	1.18	SANDS: clean sand to silty sand		Negligible
13	13.5	0.5	19.7	15.65	0.14	263.8	167.7	0.28	0.86	SANDS: clean sand to silty sand	3.05	Negligible
13.5	14	0.5	19.8	16.73	0.15	273.7	172.7	0.28	0.96	SANDS: clean sand to silty sand	3.45	Negligible
14	14.5	0.5	19.7	16.63	0.14	283.6	177.6	0.27	0.89	SANDS: clean sand to silty sand	3.23	Negligible
14.5	15	0.5	19.9	17.57	0.16	293.5	182.7	0.27	1.00	SANDS: clean sand to silty sand	3.69	Negligible
15	15.5	0.5	19.8	16.90	0.15	303.5	187.7	0.27	0.90	SANDS: clean sand to silty sand	3.38	Negligible
15.5	16	0.5	19.8	15.77	0.15	313.3	192.7	0.26	0.80	SANDS: clean sand to silty sand	3.07	Negligible
16	16.5	0.5	19.7	17.79	0.14	323.2	197.6	0.26	0.88	SANDS: clean sand to silty sand	3.43	Negligible
16.5	17	0.5	19.8	15.87	0.15	333.1	202.6	0.25	0.77	SANDS: clean sand to silty sand	3.05	Negligible
17	17.5	0.5	19.8	16.49	0.14	343.0	207.6	0.25	0.79	SANDS: clean sand to silty sand	3.17	Negligible
17.5	18	0.5	19.9	18.11	0.16	352.9	212.7	0.24	0.90	SANDS: clean sand to silty sand	3.68	Negligible
18	18.5	0.5	20.0	19.02	0.16	362.9	217.7	0.24	0.95	SANDS: clean sand to silty sand	3.95	Negligible
18.5	19	0.5	20.1	20.22	0.18	373.0	222.9	0.24	1.06	SANDS: clean sand to silty sand		Negligible
19	19.5	0.5	20.0	18.14	0.16	383.0	228.0	0.23	0.87	SANDS: clean sand to silty sand	3.71	Negligible
19.5	20	0.5	19.3	8.61	0.12	392.6	232.7	0.23	0.44	SAND MIXTURES: silty sand to sandy silt	1.90	Negligible

Liquefaction Susceptibility



Appendix D

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Burwood/Pegasus Community Board Room			Reviewer:	Stephen Lee
	Unit	No:	Street	CPEng No:	1006840
Building Address:		5	Union Street	Company:	GHD
Legal Description:	Lot 1 DP 6726			Company project number:	513059632
				Company phone number:	(03) 3780900
	Degrees	Min	Sec		
GPS south:	43	30	29.07	Date of submission:	
GPS east:	172	43	39.05	Inspection Date:	25/01/12
				Revision:	
Building Unique Identifier (CCC):	BU 2637-001 EQ2			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	0
Soil type:	silty sand	Soil Profile (if available):	Sand and Clay
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):	1.00

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	1.30
Ground floor split?	no		Ground floor elevation above ground (m):	0.30
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	Concrete slab
Building height (m):	4.50		height from ground to level of uppermost seismic mass (for IEP only) (m):	4.3
Floor footprint area (approx):	576			
Age of Building (years):	37		Date of design:	1976-1992
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	commercial		Brief strengthening description:	
Use (upper floors):				
Use notes (if required):	Commercial offices/community boardroom			
Importance level (to NZS1170.5):	IL2			

<u>Gravity Structure</u>

Gravity System:	frame system		
Roof:	steel framed	rafter type, purlin type and cladding	460UB frame @6.2 centres, 295 x 50mm purlins @ 600 centres.
Floors:	concrete flat slab	slab thickness (mm)	
Beams:	steel non-composite	beam and connector type	Weld plates
Columns:	other (note)	typical dimensions (mm x mm)	Precast concrete 460x250mm & 180x100 RHS
Walls:	load bearing concrete	#N/A	

Lateral load resisting structure

Lateral system along:	concrete shear wall		**Note: Define along and across in detailed report!**	note total length of wall at ground (m):	36
Ductility assumed, μ:	1.25			wall thickness (m):	0.175
Period along:	0.01	0.01	from parameters in sheet	estimate or calculation?	calculated
Total deflection (ULS) (mm):				estimate or calculation?	
maximum interstorey deflection (ULS) (mm):				estimate or calculation?	
Lateral system across:	other (note)			describe system	Steel frame and possible cantilevering of concrete columns
Ductility assumed, μ:	1.25				
Period across:	0.10	0.00		estimate or calculation?	estimated
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):	125	
south (mm):		
west (mm):		

Non-structural elements

Stairs:			
Wall cladding:	other light	describe	Glazing to north, west and south
Roof Cladding:	Membrane	substrate	
Glazing:	aluminium frames		
Ceilings:	light tiles		
Services(list):			

Available documentation

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

Damage

Site:
(refer DEE Table 4-2)

Site performance:		Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

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

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

Recommendations

	Level of repair/strengthening required:	none		Describe:	
	Building Consent required:	no		Describe:	
	Interim occupancy recommendations:	full occupancy		Describe:	
Along	Assessed %NBS before:	30%	30% %NBS from IEP below	If IEP not used, please detail assessment methodology:	
	Assessed %NBS after:	30%			
Across	Assessed %NBS before:	30%	30% %NBS from IEP below		
	Assessed %NBS after:	30%			

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

Period of design of building (from above): 1976-1992

h_n from above: 4.3m

Seismic Zone, if designed between 1965 and 1992: B

not required for this age of building

not required for this age of building

	along	across
Period (from above):	0.01	0.1
(%NBS)nom from Fig 3.3:	16.0%	16.0%

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	1.20
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 $(\%NBS)_{nom}$:	**19%**	**19%**

2.2 Near Fault Scaling Factor

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

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

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:	0.30
Z_{1992}, from NZS4203:1992	0.8
Hazard scaling factor, **Factor B:**	3.333333333

2.4 Return Period Scaling Factor

Building Importance level (from above):	2	
Return Period Scaling factor from Table 3.1, **Factor C**:	1.00	

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)	1.50	1.50
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:	1.00	1.00
Ductiity Scaling Factor, **Factor D**:	1.00	1.00

2.6 Structural Performance Scaling Factor:

	along	across
Sp:	0.850	0.850
Structural Performance Scaling Factor **Factor E**:	1.176470588	1.176470588

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

	along	across
$\%NBS_b$:	**75%**	**75%**

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

3.1. Plan Irregularity, factor A:	severe	0.4
3.2. Vertical irregularity, Factor B:	insignificant	1
3.3. Short columns, Factor C:	insignificant	1
3.4. Pounding potential — Pounding effect D1, from Table to right		1.0
Height Difference effect D2, from Table to right		1.0
Therefore, Factor D:		1
3.5. Site Characteristics	insignificant	1

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

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

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum	1.0	1.0
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)	**0.40**	**0.40**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	**30%**	**30%**
4.4 Percentage New Building Standard (%NBS), (before)		**30%**

GHD

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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
Final	D Lee	Hamish Mackinven		Stephen Lee		30 Oct 12