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Porritt Park Garage
BU 0706-003 EQ2

Detailed Engineering Evaluation
Qualitative Report
Version FINAL

845 Avonside Drive, Wainoni

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

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Qualitative Report
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845 Avonside Drive, Wainoni

Christchurch City Council

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Date
12/12/13

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Porritt Park Garage

BU 0706-003 EQ2

Detailed Engineering Evaluation

Qualitative Report - SUMMARY

Version FINAL

845 Avonside Drive, Wainoni

Background

This is a summary of the Qualitative report for 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 and visual inspections on 1st May 2012 only.

Building Description

The Porritt Park Garage is located at next to the hockey fields at Porritt Park, 845 Avonside Road, Avondale, Christchurch. The single story, concrete masonry building is currently used as a garage for storing equipment. There is a small covered yard directly to the north of the building. The building is assumed to have been constructed at the same time as the adjacent Porritt Park Grandstand Complex in 1973. No drawings of the building were available.

Key Damage Observed

The lightweight timber framed roof over the yard adjacent to the building has detached from the north-western face of the building during the earthquakes. No other structural damage was observed during our inspection. It should be noted that the interior of the building was unable to be inspected.

Ground damage in the form of liquefaction is widespread over the site. Significant amounts of sand and silt have been ejected in the areas around the building, particularly to the west of the building between the building and the hockey pitch. Severe ground cracks and lateral spreading were observed near the river approximately 200m south of the Garage. Settlement of the ground is evident in this area of the site. The ground behind the retaining wall approximately 4m to the north of the building has settled approximately 200mm.

Critical Structural Weaknesses

The following potential critical structural weakness has been identified.

- Plan Irregularity (30% Reduction)
- Significant Site Characteristics (30% Reduction)

20% NBS

Ground damage in the form of liquefaction and resulting settlement has a high probability of reoccurring on the site. Due to the nature of the concrete masonry structure (single storey, slab foundation), liquefaction and settlement are unlikely to cause premature collapse of this part of the building. However, it should be noted that lateral spreading around the concrete masonry building is likely to cause damage that may lead to premature collapse. The roof over the yard has already collapsed as a result of these factors. The site characteristics have been assessed as a 'significant' factor in terms of the Detailed Engineering Evaluation. This is in accordance with NZSEE guidelines.

Indicative Building Strength (from IEP and CSW assessment)

Based on the information available, and using the NZSEE Initial Evaluation Procedure, the original capacity of the building has been assessed to be in the order of 22% NBS and post-earthquake capacity in the order of 20% NBS (10% reduction based on the damage observed). The building's post-earthquake capacity excluding critical structural weaknesses is in the order of 40% NBS.

The building has been assessed to have a seismic capacity in the order of 20% NBS and is therefore potentially Earthquake Prone.

Recommendations

As the structure has been assessed as potentially Earthquake Prone, we recommend that the building remain closed as per Christchurch City Council's Earthquake Prone Buildings policy until further detailed assessment of the structure and ground conditions is undertaken and if necessary, strengthening options explored.

Given the enclosed information we would recommend a series of additional location specific geotechnical assessments, including testing and investigation, be completed.

1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of Porritt Park Garage.

This report is a Qualitative 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 qualitative 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 likely building performance and damage patterns, to identify any potential critical structural weaknesses or collapse hazards, and to make an initial assessment of the likely building strength in terms of percentage of new building standard (%NBS).

At the time of this report, no intrusive site investigation, detailed analysis, or modelling of the building structure had been carried out. Construction drawings were not made available. The building description below is based on our visual inspections only.

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 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 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 Porritt Park Garage is located next to the hockey fields at Porritt Park, 845 Avonside Road, Wainoni, Christchurch. The single story building is currently used as a garage for storing equipment. There is a small covered yard directly to the north of the building. The building is assumed to have been constructed at the same time as the adjacent Porritt Park Grandstand Complex in 1973.

Key structural details of the building are shown in Figure 2 below.

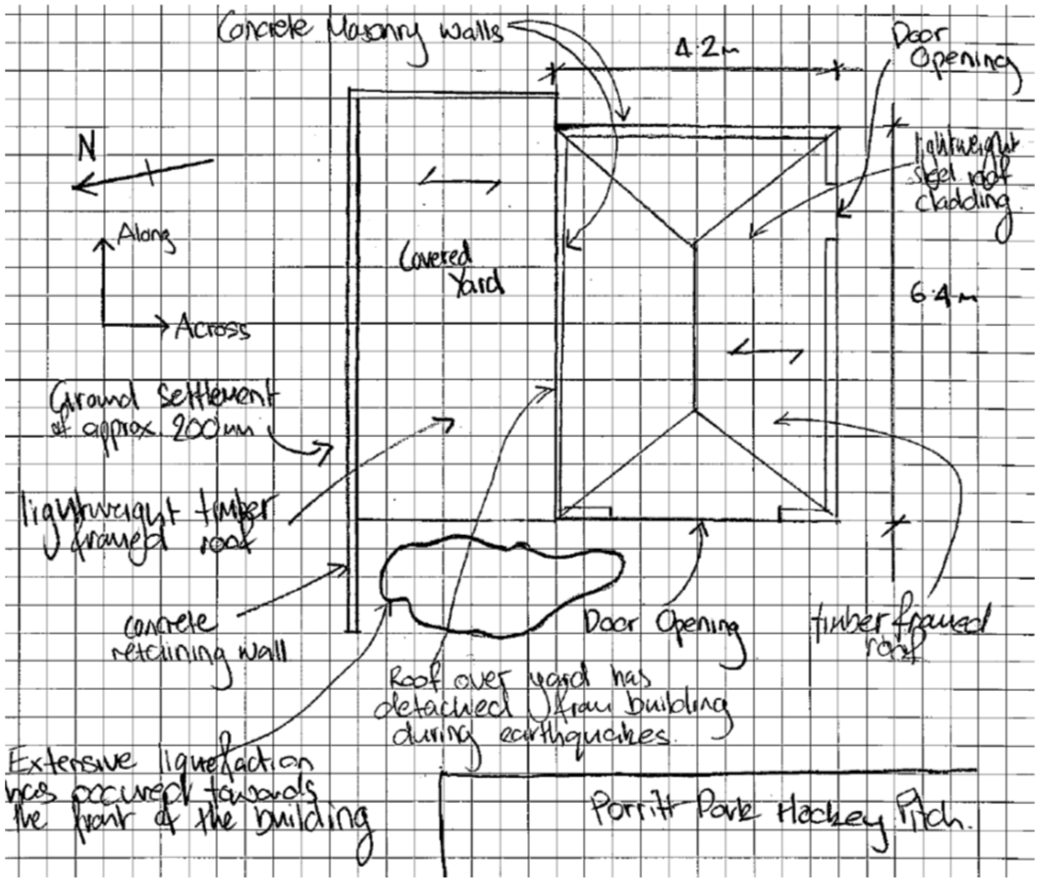


Figure 2 Plan sketch showing key structural elements

The dimensions of the rectangular building are approximately 6.4 m wide by 4.2 m long and 3.5 m tall. The overall footprint of the building is approximately 28 m^2.

We were unable to gain access inside the building during our inspection. As a result, descriptions of the building's structural systems have been inferred from an inspection of the exterior of the building only. No drawings of the building have been made available.

4.2 Gravity Load Resisting System

The gravity loads acting on the structure are resisted by timber roof framing and concrete masonry load-bearing walls.

Gravity loads from the lightweight steel roof cladding are supported by timber rafters or trusses. The timber trusses/rafters supported by concrete masonry walls on all four sides of the building. Cast-in-situ concrete lintels transfer gravity loads around the door openings in the masonry walls. Gravity loads are transferred through the load-bearing concrete masonry walls and into the foundations. The foundations are likely to consist of a reinforced concrete slab on grade with a reinforced concrete thickening around the perimeter of the building. It should be noted that intrusive investigations of the foundations to confirm their configuration were not carried out.

The lightweight timber roof covering the adjacent yard was supported on the northern edge by a timber framed fence extending above the reinforced concrete retaining wall and was nailed to the timber framed roof of the concrete masonry building along the southern edge. The nails along the southern edge have pulled out during the earthquake shaking and the roof has detached as a result.

4.3 Lateral Load Resisting System

Lateral loads acting on the structure of the building are resisted by concrete masonry walls. The lateral forces are distributed from the roof structure to the concrete masonry walls. The dimensions of the building are relatively small and as a result, diaphragm action of the roof is unlikely to be required to transfer the forces. The general stiffness of the timber framing is likely to be adequate to transfer the forces to the concrete masonry walls.

The western wall has a large opening to allow equipment to be stored in the garage. The southern wall also has a smaller door opening. The openings can be seen in Photographs 1 and 4. The eastern wall of the building has no openings and as a result it is expected to be stiffer than the western concrete masonry wall.

There is also a large opening along the western elevation of the yard with in-situ concrete retaining walls along the northern and eastern elevations of the yard.

These factors are likely to offset the centre of stiffness of the building and result in additional torsional forces being induced during earthquake shaking. The plan irregularity has been considered significant for the building in accordance with NZSEE guidelines.

5. Assessment

An inspection of the building was undertaken on the 1st of May 2012. Only the exterior of the building was inspected. No inspection of the interior of the building or 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.

The %NBS score determined for the building has been based the IEP procedure described by the NZSEE based on the information obtained from visual observation of the building only.

6. Damage Assessment

6.1 Surrounding Buildings

The Porritt Park Grandstand Complex to the south of the Porritt Park Garage was observed to have a moderate amount of structural damage resulting from the recent seismic activity. The Grandstand Complex is approximately 10m from the Garage and also suffered significant and extensive ground damage in the form of lateral spreading, liquefaction and ground settlement. Cracking in reinforced concrete and masonry elements is widespread throughout the Grandstand Complex structure.

6.2 Residual Displacements and General Observations

No significant residual displacement of the building was observed during our inspection. Due to the extensive liquefaction that has occurred at the site and around the building, it is expected that some minor overall settlement of the structure has occurred.

A check of the building level and verticality was undertaken with a spirit level during our inspection. The checks indicated that the building is still relatively level and that the building had not settled differentially.

The lightweight timber framed roof over the yard adjacent to the building has detached from the north-western face of the building during the earthquakes. The roof was supported on the northern edge by a timber framed fence extending above the reinforced concrete retaining wall and was nailed to the timber framed roof of the concrete masonry building along the southern edge. The nails along the southern edge have pulled out and the roof has detached as a result. It is likely that the roof has detached as a result of the supporting retaining wall moving due to lateral spreading and settlement of the retained slope.

No other structural damage to the building was observed during our inspection. It should be noted that the interior of the building was unable to be inspected.

6.3 Ground Damage

Significant ground damage was observed throughout the Porritt Park site during our inspections. Severe ground cracks and lateral spreading were observed near the river approximately 200m south of the Garage. Extensive liquefaction and ground settlement was observed throughout the site. Significant amounts of sand and silt were still present on the ground throughout the site.

7. Critical Structural Weakness

7.1 Short Columns

No short columns were observed in the building.

7.2 Plan Irregularity

The western external concrete masonry wall of the building has a large door opening. The stiffness of this wall is expected to be lower than that of the eastern wall at the opposite end of the building which is continuous and has no openings. There is also a large opening along the western elevation of the yard with in-situ concrete retaining walls along the northern and eastern elevations of the yard.

As a result, the distribution of concrete masonry walls and in-situ concrete retaining walls throughout the building is irregular and may lead to additional torsional forces acting on the building during an earthquake.

For the purposes of the IEP assessment of the building and determination of the %NBS score, the effects of the large opening in the western face of the building has been assessed as ‘significant’ in accordance with the NZSEE guidelines.

7.3 Roof

Although the roof structure was unable to be inspected, due to the relatively small size of the building it is not expected that significant diaphragm action will be required. The general stiffness of the timber framing is likely to be adequate to transfer the lateral forces to the perimeter concrete masonry walls.

7.4 Staircases

The building does not contain a staircase.

7.5 Site Characteristics

Ground damage in the form of liquefaction and resulting settlement has a high probability of reoccurring on the site. Due to the nature of the concrete masonry structure (single storey, slab foundation), liquefaction and settlement are unlikely to cause premature collapse of this part of the building. However, it should be noted that lateral spreading around the concrete masonry building is likely to cause damage that may lead to premature collapse. The roof over the yard has already collapsed as a result of these factors. The site characteristics have been assessed as a ‘significant’ factor in terms of the Detailed Engineering Evaluation. This is in accordance with NZSEE guidelines.

8. Geotechnical Consideration

8.1 Site Description

Porritt Park Garage is located in Wainoni, and is accessed from Avonside Drive. The Avon River presently flows north immediately to the west of the site. However, the river previously meandered around the southern, eastern and northern boundaries of the reserve until realigned and widened to allow for increased recreational use in the early twentieth century. The previous course of the river still contains water, and is connected to the main branch at both ends.

The subject area is low lying and topographically is typically flat. It is approximately 2m above sea level, and approximately 4km west of the coast.

8.2 Published Information on Ground Conditions

8.2.1 Published Geology

The geological map of the area[1] indicates that the site is underlain by Holocene soils of the Yaldhurst Member, sub-group of the Springston Formation, comprising alluvial sand and silt overbank deposits.

A north-south oriented band of marine sand of fixed and semi-fixed dunes (Christchurch Formation) is located nearby to the east.

8.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that numerous boreholes are located within a 200m radius of the site (see Table 2), although none are within the property itself. Those with deeper lithographic logs indicate the area to be underlain by layers of alternating layers of sand and clay, with gravel below ~25m.

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35/5362	92.7m	-	150m S
M35/12097	3.0m	-	100m S
M35/12098	3.3m	-	100m S
M35/12099	3.3m	-	100m S
M35/12647	6.1m	-	200m E

Table 2 ECan Borehole Summary

It should be noted that the purpose of the boreholes the well logs are associated with, 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

[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. Lower Hutt. Institute of Geological and Nuclear Sciences Limited.

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.

8.2.3 EQC Geotechnical Investigations

The Earthquake Commission has undertaken geotechnical testing in the area of the site. Information pertaining to this investigation is included in the Tonkin & Taylor Report for Wainoni[2]. Four CPT investigations were conducted along the eastern boundary of the park across the river, approximately 200m from the subject buildings, as summarised below in Table 3.

Bore Name	Grid Reference	Depth (m bgl)	Log Summary
CPT WAI 36	2484689 mE	0 – 1	Surface Soil
	5743611 mN	1 – 3.6	SILT mixtures (silty clay to sandy silt)
		3.6 – 17.7	Medium dense to dense SAND (WT at 1.6m bgl)
CPT WAI 37	2484773 mE	0 – 1	Surface Soil
	5743521 mN	1 – 2.4	SILT mixtures (silty clay to sandy silt)
		2.4 – 19	Dense SAND, with occasional silt/clay lenses (WT at 2.6m bgl)
CPT WAI 38	2484755 mE	0 – 1	Surface Soil
	5743469 mN	1 – 4	SILT mixtures (silty clay to sandy silt)
		4 – 22.9	Dense SAND, with occasional silt/clay lenses
CPT WAI 39	2484671 mE	0 – 2.2	Surface Soil and Clays
	5743354 mN	2.2 – 8.8	Loose to medium dense SAND
		8.8 – 9.8	SILT mixtures (silty clay to sandy silt)
		9.8 – 19.5	Medium dense to dense SAND (WT at 1.5m bgl)

Table 3 EQC Geotechnical Investigation Summary Table

Initial observations of the CPT results indicate the soils typically comprise a surface layer of fines (clay and silt), underlain by sand with occasional silt/clay lenses.

8.2.4 Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has published areas showing the Green Zone Technical Category in relation to the risk of future liquefaction and how these areas are expected to perform in future earthquakes.

The site is situated within the Green Zone. Within this, it is classified Technical Category Not Applicable, as the property is considered non-residential.

[2] Tonkin and Taylor . September 2011: Christchurch Earthquake Recovery, Geotechnical Factual Report, Wainoni

8.2.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows signs of major liquefaction (including lateral spreading) across the site, and throughout surrounding streets, as shown in Figure 3. Significant liquefaction-induced damage is also evident across the hockey turf surfaces.

Figure 3 Post February 2011 Earthquake Aerial Photography [3]

8.2.6 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated (near the surface) to comprise silts, clays, and loose to dense sand, underlain by multiple strata of gravel, clay and sand.

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

8.3 Seismicity

8.3.1 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.

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	130 km	NW	~8.3	~300 years
Greendale (2010) Fault	25 km	W	7.1	~15,000 years
Hope Fault	100 km	N	7.2~7.5	120~200 years
Kelly Fault	100 km	NW	7.2	~150 years
Porters Pass Fault	60 km	NW	7.0	~1100 years

Table 4 Summary of Known Active Faults[4,5]

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

8.3.2 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 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.

The site has a 475-year PGA (peak ground acceleration) of ~0.4 (Stirling et al, 2002[4]). Combining this with the anticipated geology and estimated bedrock depths in excess of 500m, the ground shaking hazard is expected to be relatively high.

8.4 Slope Failure and / or Rockfall Potential

The topography surrounding the site is typically flat, and hence rockfalls are not considered to be a hazard at this site. However, given the site's proximity to the Avon River, it is considered possible that lateral spreading may occur in the area.

[4] Stirling, M.W, McVerry, G.H, and Berryman K.R. (2002) A New Seismic Hazard Model for New Zealand, Bulletin of the Seismological Society of America, Vol. 92 No. 5, pp 1878-1903, June 2002.

[5] GNS Active Faults Database

In addition, any localised retaining structures should be investigated to better establish the site-specific slope instability.

8.5 Liquefaction Potential

The site is considered to be at major risk from liquefaction during further earthquakes as evidenced by:

- Significant liquefaction at the site following the events of 4th September 2010 (Mw 7.1), 22nd February (Mw 6.3, 2.0g) and 13th June 2011 (Mw 5.6-6.3, 1.5g); and,
- Anticipated ground conditions comprising sand and silt layers considered to be highly liquefiable.

Lateral spreading also occurred following the September and February earthquakes. Due to the property being an island, this spreading propagated in all directions. The surface of the ground across the park was significantly cracked.

Further investigation is recommended to better determine subsoil conditions. From this, a more comprehensive liquefaction assessment could be undertaken.

8.6 Recommendations

Given the anticipated ground conditions, we recommend that further investigation is undertaken. Specifically, we recommend two to four CPT investigations and one machine-drilled borehole be conducted to target depths of 20m bgl.

A soil class of **E** (in accordance with NZS 1170.5:2004) should be adopted for the site. However, this is subject to confirmation following the assessment of intrusive ground investigation results.

8.7 Conclusions & Summary

This assessment is based on a review of the geology and existing ground investigation information, and observations from the Christchurch earthquakes since 4 September 2010.

The site appears to be situated on stratified alluvial deposits, comprising layers of silt, clay, loose to dense sand, and gravel (gravel at depth). Associated with this the site also has major liquefaction potential. The proximity to the Avon River highlights the potential for lateral spreading, observed on 4th September 2010, and 22nd February 2011.

It is recommended that intrusive investigation comprising two to four piezocone CPT's be conducted. This will allow a more comprehensive liquefaction and/or ground condition assessment to be made.

A soil class of **E** (in accordance with NZS 1170.5:2004) should be adopted for the site

9. Survey

A check of the building level and verticality was undertaken with a spirit level during our inspection. The checks indicated that the building is still relatively level and that the building had not settled differentially.

10. Initial Capacity Assessment

10.1 % NBS Assessment

The building has had its capacity assessed using the Initial Evaluation Procedure based on the information available. The buildings capacity excluding critical structural weaknesses and the capacity of any identified weaknesses are expressed as a percentage of new building standard (%NBS) and are in the order of that shown below in Table 5. These capacities are subject to confirmation by a more detailed quantitative analysis.

Item	%NBS
Building excluding CSW's	45
Plan Irregularity (30% Reduction)	22
Significant Site Characteristics (30% Reduction)	
Damage to Structure (10% Reduction)	20

Table 5 Indicative Building and Critical Structural Weaknesses Capacities based on the NZSEE Initial Evaluation Procedure

Following an IEP assessment, the building has been assessed as achieving 20% New Building Standard (NBS). The building is therefore considered potentially Earthquake Prone as it achieves less than 34% NBS. This score has been reduced by 10% based on the damage to the roof structure that has resulted in the roof covering the yard detaching from the concrete masonry building.

10.2 Seismic Parameters

The seismic design parameters based on current design requirements from NZS1170:2002 and the NZBC clause B1 for this building are:

- Site soil class: E, NZS 1170.5:2004, Clause 3.1.3, Soft Soil
- Site hazard factor, Z = 0.3, NZBC, Clause B1 Structure, Amendment 11 effective from 1 August 2011
- Return period factor R_u = 0.5, NZS 1170.5:2004, Table 3.5, Importance Level 1 structure with a 50 year design life.

An increased Z factor of 0.3 for Christchurch has been used in line with recommendations from the Department of Building and Housing resulting in a reduced % NBS score.

10.3 Expected Structural Ductility Factor

A structural ductility factor of 1.25 has been assumed both along and across the building based on the concrete masonry wall system observed and the assumed construction date of 1973. The concrete masonry walls are likely to be lightly reinforced. As a result, the ductility of the concrete masonry walls is expected to be nominal.

10.4 Discussion of Results

The results obtained from the initial IEP assessment are consistent with those expected for a building of this age and construction type founded on Class E soils. The building was assumed to have been constructed in 1973 at the same time as the adjacent Porritt Park grandstand complex. The building has been assessed as an Importance Level 1 structure as it is used for storage and is unlikely to have people inside the building. The increase in the hazard factor for Christchurch to 0.3 further reduces the %NBS score.

Ground damage in the form of liquefaction and resulting settlement has a high probability of reoccurring on the site. Due to the nature of the concrete masonry structure (single storey, slab foundation), liquefaction and settlement are unlikely to cause premature collapse of this part of the building. However, it should be noted that lateral spreading around the concrete masonry building is likely to cause damage that may lead to premature collapse. The roof over the yard has already collapsed as a result of these factors. The site characteristics have been assessed as a 'significant' factor in terms of the Detailed Engineering Evaluation. This is in accordance with NZSEE guidelines.

10.5 Occupancy

The building has been assessed as being potentially Earthquake Prone. As a result, it is recommended that the building remain unoccupied pending further detailed assessment and strengthening if required, as per Christchurch City Council's policy regarding the occupancy of potentially Earthquake Prone buildings.

11. Initial Conclusions

The building has been assessed to have a seismic capacity in the order of 20% NBS and is therefore potentially Earthquake Prone.

12. Recommendations

The recent seismic activity in Christchurch has caused some structural damage to the building. The lightweight roof over the yard adjacent to the concrete masonry building has detached during the recent seismic activity.

The structure has been assessed as potentially Earthquake Prone, due to the assumed age of the building and the site ground conditions. As a result, we recommend that the building remain closed as per Christchurch City Council's Earthquake Prone Buildings policy until further detailed assessment of the structure and ground conditions is undertaken and if necessary, strengthening options explored.

Given the land damage observed, we recommend a series of location specific geotechnical assessments, including testing and investigation, be completed.

13. Limitations

13.1 General

This report has been prepared subject to the following limitations:

- Drawings of the building were unavailable. As a result the information contained in this report has been inferred from visual inspections of the building and site only.
- The interior of the building was unable to be inspected.
- The foundations of the building were unable to be inspected.
- No intrusive structural investigations have been undertaken.
- No level or verticality surveys have been undertaken.
- No material testing has been undertaken.
- No calculations, other than those included as part of the IEP in the CERA Building Evaluation Report, have been undertaken. No modelling of the building for structural analysis purposes has been performed.

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.

13.2 Geotechnical Limitations

This report presents the results of a geotechnical appraisal prepared for the purpose of this commission, and for prepared solely for the use of Christchurch City Council and their advisors. 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.

The advice tendered in this report is based on a visual geotechnical appraisal. No subsurface investigations have been conducted. An assessment of the topographical land features have been made based on this information. It is emphasised that Geotechnical conditions may vary substantially across the site from where observations have been made. Subsurface conditions, including groundwater levels can change in a limited distance or time. In evaluation of this report cognisance should be taken of the limitations of this type of investigation.

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

Photograph 1 **North-western face of the building with door opening.**

Photograph 2 **Sand and silt from liquefaction piled outside the building and adjacent covered yard with concrete retaining wall.**

Photograph 3 Detached lightweight roof over yard.

Photograph 4 South-western face of the building.

Photograph 5 South-eastern face of the building.

Photograph 6 Ground damage outside the front of the building.

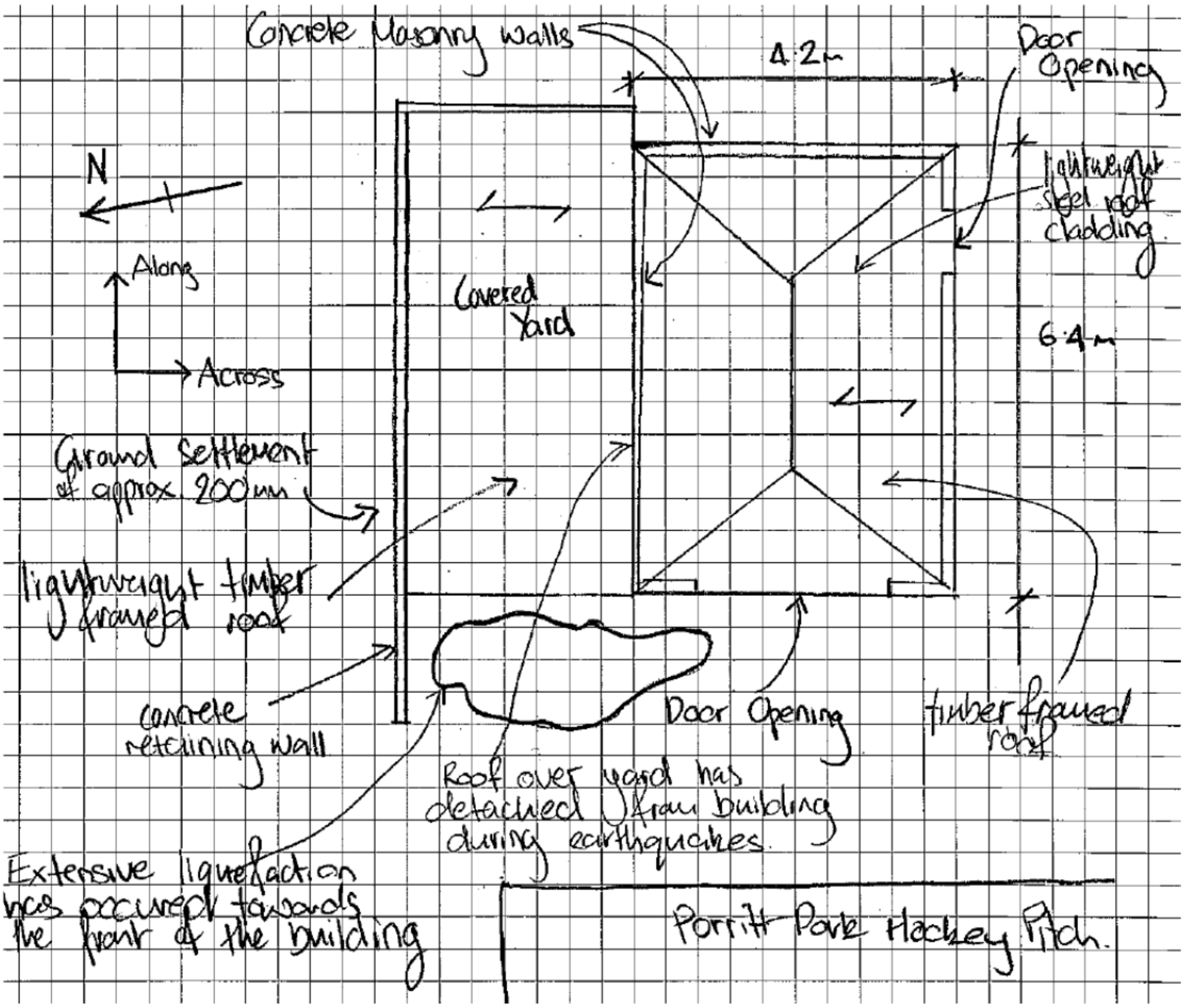
Photograph 7 Settlement of the retained ground to the north of the building.

Photograph 8 Ground cracks near the river to the south of the garage.

Appendix B

Existing Drawings / Sketches

No structural or architectural drawings have been made available for this building. Shown below is a marked up plan of the building showing key structural elements. A site plan is also included.




Porritt Park Garage

Appendix C

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Porritt Park Garage			Reviewer:	Stephen Lee
	Unit	No:	Street	CPEng No:	1006840
Building Address:		845	Avonside Drive	Company:	GHD
Legal Description:				Company project number:	51/30596/23
				Company phone number:	04 472 0799
	Degrees	Min	Sec		
GPS south:				Date of submission:	
GPS east:				Inspection Date:	1/5/2012
				Revision:	final
Building Unique Identifier (CCC):	BU 0706-003 EQ2			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	mixed	Soil Profile (if available):	
Site Class (to NZS1170.5):	E		
Proximity to waterway (m, if <100m):	50	If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	2.00

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	
Ground floor split?	no		Ground floor elevation above ground (m):	
Storeys below ground	0			
Foundation type:	mat slab		if Foundation type is other, describe:	
Building height (m):	3.50		height from ground to level of uppermost seismic mass (for IEP only) (m):	3.5
Floor footprint area (approx):	28			
Age of Building (years):	39		Date of design:	1965-1976
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	other (specify)		Brief strengthening description:	
Use (upper floors):	other (specify)			
Use notes (if required):	Storage Garage			
Importance level (to NZS1170.5):	IL1			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	
Floors:	concrete flat slab	slab thickness (mm)	
Beams:	cast-insitu concrete	overall depth x width (mm x mm)	Beams over openings
Columns:			
Walls:	partially filled concrete masonry	thickness (mm)	200

Lateral load resisting structure

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

Non-structural elements

Stairs:			
Wall cladding:	exposed structure	describe	
Roof Cladding:	Metal	describe	
Glazing:			
Ceilings:	none		assumed.
Services(list):			

Available documentation

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

Damage

Site:
(refer DEE Table 4-2)

Site performance:	Poor	Describe damage:	Extensive and severe ground damage
Settlement:	25-100m	notes (if applicable):	
Differential settlement:	0-1:350	notes (if applicable):	
Liquefaction:	5-10 m²/100m³	notes (if applicable):	
Lateral Spread:	250-500mm	notes (if applicable):	
Differential lateral spread:	0-1:400	notes (if applicable):	
Ground cracks:	more than 200mm/20m	notes (if applicable):	
Damage to area:	moderate to substantial (1 in 5)	notes (if applicable):	

Building:

	Current Placard Status:	red		
Along	Damage ratio:	10%	Describe how damage ratio arrived at:	Estimated.
	Describe (summary):	10% reduction for damage to roof.		
Across	Damage ratio:	10%		
	Describe (summary):	10% reduction for damage to roof.		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	yes	Describe:	Roof over yard has collapsed

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

Recommendations

	Level of repair/strengthening required:	minor structural	Describe:	
	Building Consent required:	no	Describe:	
	Interim occupancy recommendations:	do not occupy	Describe:	
Along	Assessed %NBS before:	22%	22% %NBS from IEP below	If IEP not used, please detail assessment methodology:
	Assessed %NBS after:	20%		
Across	Assessed %NBS before:	22%	22% %NBS from IEP below	
	Assessed %NBS after:	20%		

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): 1965-1976

h_n from above: 3.5m

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):	0.4	0.4
(%NBS)nom from Fig 3.3:	5.0%	5.0%

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	1.00
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 $(\%NBS)_{nom}$:	**5%**	**5%**

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	
Hazard scaling factor, **Factor B:**	3.333333333

2.4 Return Period Scaling Factor

Building Importance level (from above):	1
Return Period Scaling factor from Table 3.1, **Factor C**:	2.00

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)	1.25	1.25
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:	1.25	1.25
Ductiity Scaling Factor, **Factor D**:	1.25	1.25

2.6 Structural Performance Scaling Factor:

	along	across
Sp:	0.925	0.925
Structural Performance Scaling Factor **Factor E**:	1.081081081	1.081081081

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

	along	across
$\%NBS_b$:	**45%**	**45%**

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

3.1. Plan Irregularity, factor A:	significant	0.7
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	significant	0.7

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: Large opening on western face. 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)	**0.49**	**0.49**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	**22%**	**22%**

4.4 Percentage New Building Standard (%NBS), (before) **22%**

GHD

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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
DRAFT	Alex Baylis	Jenny Stevenson		Stephen Lee		29/5/12
FINAL	Alex Baylis	Jenny Stevenson		Donna Bridgman		12/12/13