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Waltham Pool Plant Room
BU 1044-003 EQ2

Detailed Engineering Evaluation

Qualitative Report

FINAL

Waltham Park, 30-40 Waltham Road,
Christchurch

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

Waltham Pool Staff Room
BU 1044-002 EQ2

Detailed Engineering Evaluation
Qualitative Report
FINAL

Waltham Park, 30-40 Waltham Road, Christchurch

Christchurch City Council

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Date
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Waltham Pool Plant Room

BU 1044-003 EQ2

Detailed Engineering Evaluation

Qualitative Report - SUMMARY

FINAL

Waltham Park, 30-40 Waltham Road

Corner of Fifield Tce & Waltham Road, Christchurch

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 24 January 2012.

Key Damage Observed

The structure is roughly 50% damaged. Key damage observed includes the following:

- Differential settlement was noted in the middle area of the northeast wall.
- The floor slab inside the plant room has cracked in the middle area of the northeast wall, with cracks of roughly 10mm width evident.
- The northwest and northeast walls have suffered diagonal shear failure, with extensive large cracks to the mortar between concrete masonry blocks and separation of the unreinforced blockwork, up to 30mm in width.
- The foundation around the service pit area shows cracking, with cracks up to 5mm in width.
- The eastern end wall has settled.
- The steel roof joists have rolled to the side slightly and are not fully vertical.

Critical Structural Weaknesses

The lack of any significant attachment between the roof structure and the external block walls results in a lack of out-of-plane propping or in-plane transfer of seismic load to the external walls. This results in a vertical irregularity, which constitutes a critical structural weakness inherent in the original design of the structure.

Indicative Building Strength (from IEP and CSW assessment)

Based on the information available, and using the NZSEE Initial Evaluation Procedure, the buildings original capacity has been assessed to be in the order of 5% NBS, excluding critical structural weaknesses. The building's capacity including critical structural weaknesses is in the order of 3% NBS. Therefore the building is potentially Earthquake Prone.

Recommendations

It is recommended that:

- The classification of the building be revised to potentially Earthquake Prone.
- A quantitative assessment of the building, supported by intrusive investigations if required, be undertaken to determine the seismic capacity and to develop potential strengthening concepts.
- A geotechnical investigation be undertaken to investigate the differential settlement exhibited under the building and the potential for liquefaction at the site.
- We consider that barriers around the building are not necessary.

1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the Waltham Pool grounds. This report covers the Staff Building.

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 have been carried out. The building description is based on the visual inspection carried out on site and the building drawings made available.

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.

2.0 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

2.1 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.

2.1.1 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.

2.2 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.

2.3 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.

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

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

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.

Table 1 %NBS compared to relative risk of failure

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

4. Building Description

4.1 General

The Waltham Pool Plant Room was constructed in 1966, at roughly the same time as other structures at Waltham Park.

The structure is located at Waltham Park, which is located at 30-40 Waltham Road. The surrounding area consists of the park, which includes a pool, other staff grounds, and open space. The park is bordered to the south by Heathcote River, and on all other sides by residential dwellings.

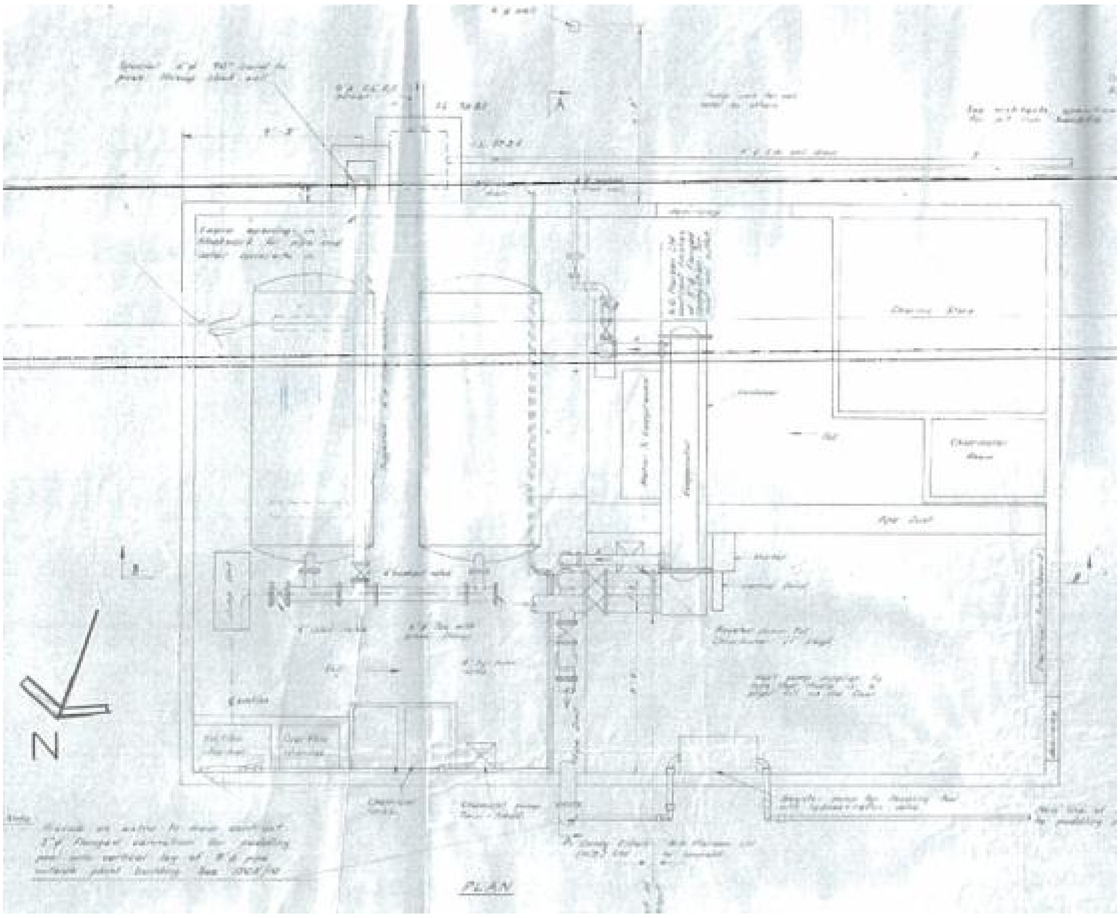
The site is situated on the outskirts of a recreational reserve, within the residential suburb of Waltham in southeast Christchurch. It is relatively flat at approximately 6m above mean sea level. It is approximately 80m northwest of the Heathcote River, 4.5km west of the estuary and 8.5km west of the coast (Pegasus Bay).

The general structure consists of rolled steel joist rafters and timber roof purlins, with unfilled, unreinforced concrete masonry walls and a floor slab on-grade. The walls sit atop an edge beam which is supported by concrete strip footings. There are high-level windows between the top of the external lateral walls and the roof structure. The external walls have no internal lining. The roof members are exposed internally with no ceiling lining.

There is a small waterslide pumproom addition which is not shown on the original construction drawings. It is likely to be of the same 1998 construction date as the waterslide and therefore of better structural style, likely fully reinforced, fully filled concrete masonry.

The dimensions of the building are approximately 13.8m long, 9.0m wide and 3.6m in height.

Figure 2 Plan Showing Key Structural Elements



4.2 Gravity Load Resisting System

The gravity loads in this simple structure are resisted by the external concrete masonry walls, which are unfilled and likely unreinforced. Roof loads are transferred to the walls via timber roof purlins and rolled steel joist rafters, and through the external walls into the concrete edge beams and strip footings. Floor loads are supported directly by the concrete slab on-grade.

4.3 Lateral Load Resisting System

The external concrete masonry walls, which are unfilled and unreinforced, are the main building component to resist imposed lateral loads. We note that the roof structure is not well-attached to the external walls, as there are high-level windows or simple timber framing between the top of the walls and the roof structure. This design diminishes the load transfer of the seismic weight of the roof onto the external walls, and results in a lack of adequate out-of-plane propping to these walls. The steel roof joists are attached to the top of the external walls and therefore provide some propping to these walls, but the attachment is inadequate and full propping is therefore not achieved. There was no ceiling diaphragm present, but the roof sarking boards were installed diagonally across the purlins, which aids in lateral load bracing across the roof structure.

5. Assessment

A visual inspection of the building was undertaken on 24 January 2012. Both the interior and exterior of the building were inspected. There was no placard observed in place on the plant room structure. The main structural components of the building were all able to be viewed due to the exposed nature of the structure. No inspection of the foundations of the structure was able to be undertaken.

The visual 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, which is determined using the IEP procedure described by the NZSEE, is based on the information obtained from visual observation of the building. The observed lack of out-of-plane propping of the external longitudinal concrete walls constitutes a critical structural weakness inherent in the design of the building, and reduces the overall %NBS score accordingly.

6. Damage Assessment

6.1 Surrounding Buildings

The Waltham Pool grounds are located in Waltham Park, which is surrounded on most sides by residential dwellings and to the south by Heathcote River. During the inspection there was no apparent damage to the park or the surrounding properties.

Some of the pool grounds displayed signs of damage. The nearby Staff Building showed some minor cracking to its unfilled block external walls. The steel water slide ladder had several welded connection failures.

6.2 Residual Displacements and General Observations

The differential settlement and seismic damage has resulted in residual internal displacements to the structure.

Differential settlement was noted in the middle area of the northeast wall. This has caused cracks to develop in the floor slab inside the plant room, of roughly 10mm width, and in the northeast wall, where large cracks show evidence of this settlement and of lesser lateral movement. The northeastern wall has settled overall, in the order of 100mm to 200mm change in level.

The differential settlement and seismic damage has caused diagonal shear failure in the northwest, northeast, and southeast walls, with extensive large cracks to the mortar between concrete masonry blocks and separation of the unreinforced blockwork, up to 30mm in width. Daylight is visible through several of these cracks, as shown in the photographs in Appendix A of this report.

Differential settlement has induced cracking in the foundation around the service pit area, with cracks up to 5mm in width. These cracks occur just below the southeastern wall section where shear cracking is evident.

The steel roof joists have rolled slightly on their side due to lateral movement of the roof structure, which is not adequately secured to the block walls below. This likely occurred during recent seismic events.

In addition, the nearby perimeter block wall showed some moderate to significant cracking damage, though this damage did not occur at any locations immediately near the Plant Room. The Detailed Engineering Evaluation of the Main Building at Waltham Pool contains more information on the perimeter block wall.

6.3 Ground Damage

Apart from the differential settlement underneath the Plant Room structure, no ground damage was observed during the inspection of the site.

7. Critical Structural Weakness

7.1 Critical Structural Weaknesses

As mentioned above, the lack of any significant attachment between the roof structure and the external block walls results in a lack of out-of-plane propping or bracing of these walls. This results in a plan irregularity, which constitutes a critical structural weakness inherent in the original design of the structure, and should be addressed directly if any strengthening measures to the structure are undertaken in the future.

7.2 Short Columns

The building does not contain any short columns.

7.3 Lift Shaft

The building does not contain a lift shaft.

7.4 Roof

No critical structural weaknesses were observed in the roof structure proper. See section 7.0 above.

7.5 Staircases

The building does not contain a staircase.

7.6 Plan Irregularity

Since there is no significant attachment between the roof structure and the external block walls, the structure has no out-of-plane propping or bracing of these walls. This results in a plan irregularity, which constitutes a critical structural weakness inherent in the original design of the structure.

7.7 Liquefaction

No liquefaction was observed at the site. However, the geotechnical desktop review has noted the possibility of liquefaction at the site. The IEP score accordingly reflects this potential liquefaction in the "Site Characteristics" section.

8. Geotechnical Consideration

8.1 Site Description

The site is situated on the outskirts of a recreational reserve, within the residential suburb of Waltham in southeast Christchurch. It is relatively flat at approximately 6m above mean sea level. It is approximately 80m northwest of the Heathcote River, 4.5km west of the estuary and 8.5km west of the coast (Pegasus Bay).

8.2 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 alluvial soils of the Yaldhurst Member, sub-group of the Springston Formation, comprising alluvial sand and silt overbank deposits.

The map also indicates that the site is situated on an old stream bed (Jacksons Creek).

8.1.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that seven boreholes are located within a 120m radius of the site with four >2m depth. Of these boreholes, one was drilled at the pool site and it has an adequate lithographic log. The site geology described in this log indicates the area is predominantly layers of sand and clay to a depth of ~23mbgl. Varying amounts of gravel and silt are also indicated to be present.

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M36/1194	~34.1m	~1.3m bgl	0m N/A
M36/9705	~3.5m	N/A	~23m S
M36/9334	~3.71m	N/A	~30m SE
M36/9335	~2.44m	N/A	~61m S

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 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.

[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.

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 Tonkin and Taylor Report[2]. Two investigation points were undertaken within close proximity of the site, the results of which are summarised below in Table 3.

Table 3 EQC Geotechnical Investigation ECan Bore Log Summary Table

Bore Name	Grid Reference	Log Summary	
CPT-WTM-21 (WT at 3.0m bgl)	2481726 mE 5739636 mN	0 – 5.0m	SILT and SAND mixtures
		5.0 – 7.8m	Silty CLAY
		7.8 – 18.5m	Fine to coarse SAND; dense to very dense
		18.5 – 26.8m	SILT, sandy SILT and clayey SILT
CPT-STM-11 (WT at 0.5m bgl)	2481703 mE 5739201 mN	0 – 15.0m	Layers of clayey SILT, sandy SILT and silty SAND
		15.0 – 20.0m	Dense SAND and silty SAND

8.1.4 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 classified as not applicable (N/A). This means the site is non-residential property that has not been given a technical category.

8.1.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows no signs of liquefaction outside the building footprint or adjacent to the site.

[2] Tonkin and Taylor. September 2011: Christchurch Earthquake Recovery, Geotechnical Factual Report, Waltham & St Martins

Figure 3 Post February 2011 Earthquake Aerial Photography [3]

8.1.6 Summary of Ground Conditions

From information on ECan borehole logs and EQC CPT data subsoils at the site are anticipated to be layers of sands (with some gravel) and silts (with some sand and clay). These soils are consistent with the Springston formation (Yaldhurst member), being stratified alluvial deposits of predominantly sand and silt overbank deposits.

It is anticipated that the site is situated on an old stream bed. Associated with this is an increased potential for subsoil liquefaction beneath the site.

8.3 Seismicity

8.3.1 Nearby Faults

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

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

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

Known Active Fault	Distance from Site (km)	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	130	8.3	~300 years
Greendale (2010) Fault	24	7.1	~15,000 years
Hope Fault	110	7.2~7.5	120~200 years
Kelly Fault	110	7.2	~150 years
Porters Pass Fault	60	7.0	~1100 years

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. 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.

In addition, anticipation of Holocene alluvial soils of the Yaldhurst Member, sub-group of the Springston Formation, comprising alluvial gravel, sand, and silt of historic river flood channels, and a 475-year PGA (peak ground acceleration) of ~0.4 (Stirling et al, 2002). However, bedrock is anticipated to be in excess of 500m deep, and hence ground shaking is expected to be moderate to high.

8.4 Slope Failure and/or Rockfall Potential

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

8.5 Liquefaction Potential

Due to the presence of alluvial sand and silt deposits, it is considered possible that liquefaction will occur where sands and silts are present. However, there is no evidence of liquefaction from the post-earthquake aerial photography. Given the site's proximity to the Heathcote River, it is considered likely that lateral spreading has occurred within or adjacent to the site. In future seismic events the site is

[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, June 2002, pp 1878-1903.

[5] GNS Active Faults Database, http://maps.gns.cri.nz/website/af/viewer.htm

considered at risk of lateral spreading. Therefore, until intrusive testing suitable for liquefaction analysis is carried out the overall liquefaction potential should be considered to be moderate.

8.6 Recommendations

If a more detailed assessment is required, intrusive investigation comprising one piezocone CPT test to 20m bgl should be undertaken. This will allow a numerical liquefaction analysis to be carried out.

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, predominantly comprising sand and silt. Associated with this the site also has a moderate potential for liquefaction including the potential for lateral spreading.

Should a more comprehensive liquefaction and/or ground condition assessment be required, it is recommended that an intrusive investigation comprising of one piezocone CPT be conducted. From this, a numerical liquefaction analysis may be undertaken.

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

9. Survey

No level or verticality surveys have been undertaken for this building at this stage.

10. Initial Capacity Assessment

10.0 % NBS Assessment

Following an IEP assessment, the building has been assessed as achieving in the order of 3% New Building Standard (NBS). Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the building is considered potentially Earthquake Prone as it does not achieve greater than 33% NBS. The critical structural weakness in the building lowered the IEP assessment score from 5% to 3% NBS. This score has not been adjusted when considering damage to the structure as the low score already urgently necessitates further assessment and action.

10.1 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: D, 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 (in accordance with recommendations from the Department of Building and Housing recommendations)
- Return period factor R_u = 1.0, NZS 1170.5:2004, Table 3.5, Importance Level 2 structure with a 50 year design life.

10.2 Expected Structural Ductility Factor

Based on the unfilled, unreinforced concrete masonry structure and date of construction, it is expected that the building would likely perform in a mostly brittle fashion under lateral loading. A structural ductility factor of 1.25 has therefore been assumed.

10.3 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 D soils with liquefaction potential. This building would have been designed to the standards at the time. The design loads used in this code will have been significantly less than those required by the current loading standard and detailing requirements for ductile seismic behaviour that are present in the current standards. When combined with the increase in the hazard factor for Christchurch to 0.3, it is reasonable to expect the building to be classified as Earthquake Prone.

11. Initial Conclusions

The building has been assessed to have a seismic capacity in the order of 3% NBS and is therefore potentially Earthquake Prone. As a result, the occupancy of the building should be revised under the CCCs policy to close earthquake prone buildings.

A number of Critical Structural Weaknesses have been identified. These will need to be further investigated to confirm the building capacity. However, it is probable that strengthening work will be required to address these weaknesses. The building should undergo a detailed quantitative seismic analysis to determine whether strengthening is necessary.

12. Recommendations

It is recommended that:

- The classification of the building be revised to potentially Earthquake Prone.
- A quantitative assessment of the building, supported by intrusive investigations if required, be undertaken to determine the seismic capacity and to develop potential strengthening concepts.
- A geotechnical investigation be undertaken to investigate the differential settlement exhibited under the building and the potential for liquefaction at the site.

13. Limitations

This report has been prepared subject to the following limitations:

- No inspection of the bracing in in the timber framed walls could be undertaken.
- 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.
- 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.

13.0 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: Northwestern exterior wall shear damage.

Photograph 2: Northwestern exterior wall elevation (damage at left).

Photograph 3: Northeastern exterior wall and addition – note settlement and lateral movement.

Photograph 4: External view of northeastern exterior wall damage (inside addition).

Photograph 5: Damage to northeastern exterior wall (inside addition).

Photograph 6: Damage to external northeastern wall.

Photograph 7: Floor cracking at northeastern external wall.

Photograph 8: Settlement damage at southern side of northeastern external wall.

Photograph 9: Southeastern external wall elevation.

Photograph 10: Shear damage to southeastern external wall.

Photograph 11: Internal view of damage to northwestern external wall.

Photograph 12: Damage to wall and foundation at service pit.

Photograph 13: Steel rafter rollover damage.

Appendix B

Existing Drawings

Appendix C

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Waltham Pool Plant Room			Reviewer:	Stephen Lee
	Unit	No:	Street	CPEng No:	1006840
Building Address:	3	30	Waltham Road	Company:	GHD
Legal Description:	Pt Lot 1 DP 8016			Company project number:	5130596/05
				Company phone number:	04 472 0799
	Degrees	Min	Sec		
GPS south:	43	33	10.69	Date of submission:	5/17/2013
GPS east:	172	38	59.37	Inspection Date:	24-Jan-12
				Revision:	Final
Building Unique Identifier (CCC):	PRO 1044 003			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):	D		
Proximity to waterway (m, if <100m):	35	If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	6.00
Ground floor split?	no		Ground floor elevation above ground (m):	0.00
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	
Building height (m):	3.60		height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	124			
Age of Building (years):	46		Date of design:	1935-1965
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):	Plant Room, limited access			
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	other (note)	describe system	Timber purlins on Steel rafters
Floors:	concrete flat slab	slab thickness (mm)	125
Beams:	none	overall depth x width (mm x mm)	200 thk lintel beams
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**	note total length of wall at ground (m):	27.4

Ductility assumed, μ:	1.25	**detailed report!**	wall thickness (m):	0.2
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):	18
Ductility assumed, μ:	1.25		wall thickness (m):	0.2
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:			none
Wall cladding:			none
Roof Cladding:	Metal	describe	
Glazing:	aluminium frames		
Ceilings:	strapped or direct fixed		
Services(list):			

Available documentation

Architectural	partial	original designer name/date	Sheets 1305/10 & 13
Structural	partial	original designer name/date	Sheets 15 & 16
Mechanical		original designer name/date	none available
Electrical		original designer name/date	none available
Geotech report		original designer name/date	none available

Damage

Site:
(refer DEE Table 4-2)

Site performance:		Describe damage:	
Settlement:	100-200mm	notes (if applicable):	
Differential settlement:	0-1:350	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:	yellow		
Along	Damage ratio:	29%	Describe how damage ratio arrived at:	
	Describe (summary):	Settlement, Cracking to slab, diagonal shear of walls, roof joists displaced		
Across	Damage ratio:	29%		
	Describe (summary):	Settlement, Cracking to slab, diagonal shear of walls, roof joists displaced		

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

Diaphragms	Damage?: no	Describe:	
CSWs:	Damage?: yes	Describe:	Shear cracking to NE & NW walls, slab
Pounding:	Damage?: no	Describe:	
Non-structural:	Damage?: no	Describe:	

Recommendations

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

Along	Assessed %NBS before:	3%	3% %NBS from IEP below	If IEP not used, please detail assessment methodology:
	Assessed %NBS after:	3%		
Across	Assessed %NBS before:	3%	3% %NBS from IEP below	
	Assessed %NBS after:	3%		

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): 1935-1965

h_n from above: m

Seismic Zone, if designed between 1965 and 1992:

not required for this age of building: D soft soil

not required for this age of building:

	along	across
Period (from above):	0.4	0.4
(%NBS)nom from Fig 3.3:	3.0%	3.0%

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 $(\%NBS)_{nom}$:	**3%**	**3%**

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):	2
Return Period Scaling factor from Table 3.1, **Factor C**:	1.00

		along	across
2.5 Ductility Scaling Factor	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:	1.14	1.14
	Ductiity Scaling Factor, **Factor D**:	1.14	1.14
2.6 Structural Performance Scaling Factor:	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	**$\%NBS_b$:**	**12%**	**12%**

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	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

		Along	Across
3.6. Other factors, Factor F	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)		**0.28**	**0.28**
4.3 PAR x (%NBS)b:	PAR x Baselline %NBS:	**3%**	**3%**
4.4 Percentage New Building Standard (%NBS), (before)			**3%**

Official Use only:

Accepted By:

Date:

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Document Status

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