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Waltham Pool
Main Building Complex
PRO 1044-001

Detailed Engineering Evaluation

Qualitative Report

Final

30 – 40 Waltham Road, Waltham

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

Waltham Pool - Main Building Complex
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Detailed Engineering Evaluation
Qualitative Report
Final

30 – 40 Waltham Road, Waltham

Christchurch City Council

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Date
27 May 2013

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Waltham Pool Main Building Complex

PRO 1044-001

Detailed Engineering Evaluation

Qualitative Report - SUMMARY

Final

30 – 40 Waltham Road, Waltham

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, visual inspections on 18th January 2012.

Key Damage Observed

Key damage observed includes:-

- Masonry wall to wall separation and minor cracking in concrete masonry walls in the changing room areas of the building.

Critical Structural Weaknesses

The following potential critical structural weaknesses have been identified in the concrete frame and masonry wall structure.

- Plan irregularity caused by indirect masonry wall bracing connection and missing out of plane connection to masonry walls, constitutes **30% reduction to %NBS.**
- Site liquefaction potential assessed as significant; a **further 30% reduction to reduced %NBS.**

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 6% NBS and post-earthquake capacity also in the order of 6% NBS. The buildings post-earthquake capacity excluding critical structural weaknesses is in the order of 13% NBS.

Recommendations

It is recommended that a quantitative assessment of the building be undertaken to determine the seismic capacity and to develop potential strengthening concepts. The building should remain in its current unoccupied state until this has taken place.

1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of Waltham pool main building complex.

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 have been made available. The building description below is based on our visual inspections of the building and review of the drawings.

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.

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

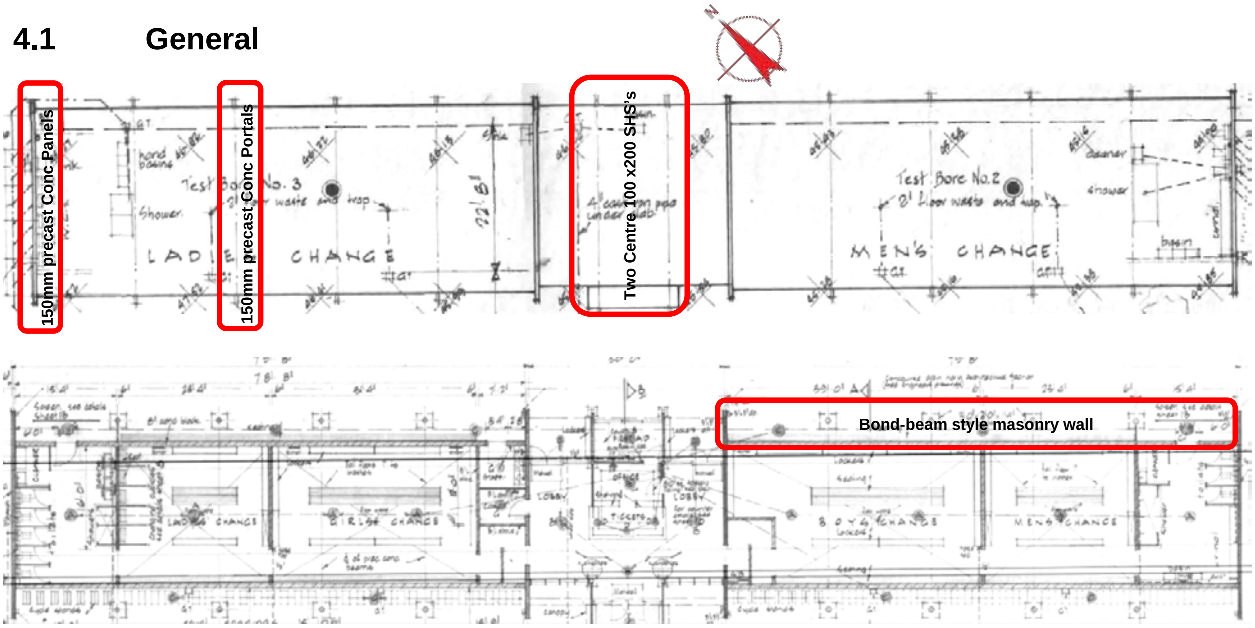


Figure 2 Plan Sketch Showing Key Structural Elements

The sketch above shows examples of typical structural elements of the building.

The dimensions of the building are approximately 48 m long by 10 m wide and 4 m tall. The complex is divided by four precast concrete panels into three main areas; equipment stores and women's changing areas wing to the west and bicycle lock up and men's change wing to the east. The central part of the complex houses office and administration facilities.

The building is constructed of 20 series equivalent masonry bond-beam style walls primarily running in the longitudinal direction, with mostly concrete portals and precast concrete panels running in the transverse direction. The roof is a light steel sheet profile on timber purlins. Foundations are concrete pads for the portals connected to strip footings for the masonry walls. The floor is a 100mm slab poured to compacted hardfill.

4.2 Gravity Load Resisting System

The roof is supported on timber purlins running longitudinally on top of the precast concrete beam portals. In the office/lobby central area, the purlins are instead supported by two 200x100 RHS rafters on SHS posts. The posts of these roof support rafters transfer the structure's gravity load to the ground via their concrete pad footings. The strip footings beneath the masonry and concrete panels transfer those walls' self-weight to the ground.

4.3 Lateral Load Resisting System

In the transverse direction, two different systems exist. There are two 150mm thick concrete reinforced panels in the centre of the building forming the lobby/office area and a similar concrete panel at each end of the complex. There are also eight concrete beam/column portals at around 4.7m centres. The portals were likely entirely precast and have unusual 75mm diameter solid steel bar knee and base connections. By check calculation, these connectors have a similar EIxx stiffness as the 203mm diameter columns that they are cast into, so allowing this arrangement to behave as a portal. The foot connector is cast into a concrete footing that is structurally connected into the main footings of the concrete masonry, thus giving some flexural resistance fixity to the base of the portal.

In the longitudinal direction, bond-beam style walls brace the structure which may be lightly reinforced but this could not be proven without intrusive investigation. The masonry walls also unfortunately have no direct connection to the portals or the roof. Consequently the bracing load path to the masonry is not direct and relies on roof connection load transfer to the four concrete panels, then the panels cantilevering out-of-plane over the top of the masonry to transfer that seismic load to the masonry. Out-of-plane the masonry walls are not restrained at their top edge, though some cantilever action may be achieved if reinforcement is present, as the wall footings are 600mm deep.

5. Assessment

An inspection of the building was undertaken on the 24th of January 2012. Both the interior and exterior of the building were inspected and the building had a yellow placard in place, visible at public entrance front door. Most above ground structural members could be viewed due to the unlined nature of the construction. The bicycle lockup area was not accessible as the key could not be located; we believe though that access would not have changed the contents of this report. Inspection of the foundations of the structure was extremely limited to external masonry strip footings and only their top edge could be seen above ground and in most cases was plastered.

The inspection consisted of observing the building to determine the structural systems and likely behaviour of the building during an earthquake. The building was inspected for structural and non-structural damage, and nearby ground conditions were also observed.

The %NBS score determined for this structure has been based on the IEP procedure described by the NZSEE using construction drawing information and from visual observation. The lack of connection from top of masonry walls to the building structure constitutes a plan irregularity resulting in a Critical Structural Weakness (CSW) in the longitudinal direction of the building and has been accounted for in the IEP calculations by reducing the %NBS by 30%. Geotechnical desktop study assessment also showed that significant liquefaction potential for the site reduces the resulting %NBS by a further 30%, giving an overall deduction to the original %NBS of around half. These reductions are in line with NZSEE guidelines for accounting for CSW severity of a building deemed to be 'significant' in an IEP.

6. Damage Assessment

6.1 Surrounding Buildings

Though there are other structures and buildings on this site that sustained damage, notably the plant room, picnic area and masonry fences, none of these structures are of significant proximity to the main complex so no damage was sustained from other buildings and future risk of this occurring is negligible.

6.2 Residual Displacements and General Observations

Minor residual displacements of the structure were observed in some of the external masonry walls on the pool side of the building. These were apparent in an outward top of wall displacement of 3-5mm.

Most structural damage occurred during differential seismic movement of masonry walls forming 90 degree internal corners. These walls now have visible separation cracks at the corners, an indication of sparse or absent steel reinforcement continuity where the masonry meets at the corners.

6.3 Ground Damage

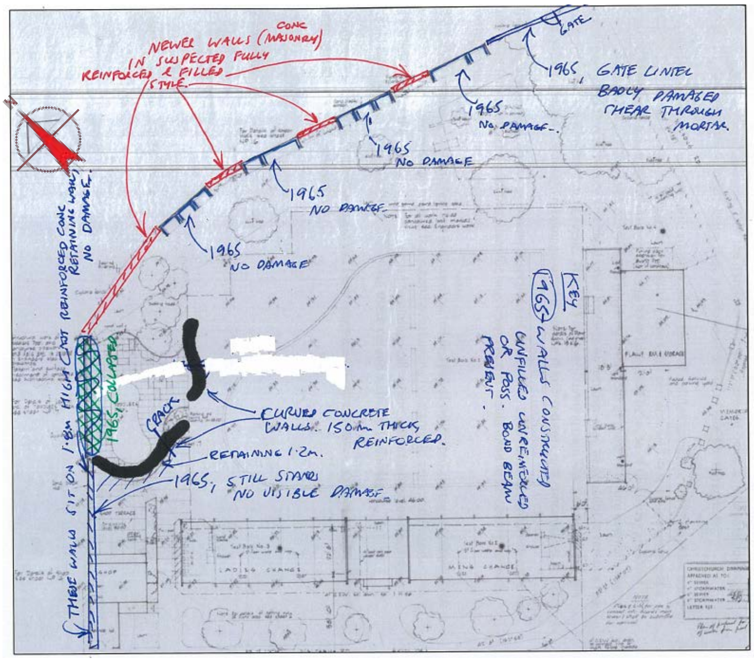
Paving at the building front shows damage that may be caused by minor lateral spread movement of the raised garden. Though geotechnical desktop study confirms there is significant potential for liquefaction, there was no evidence of liquefaction occurrence in this seismic event.

6.4 Perimeter Masonry Fences

The parameter fences were primarily constructed in 1965 likely of unfilled or bond-beam style unfilled masonry. This style of masonry does not perform well in seismic events and a large section of the parameter fences on this site have collapsed due to this (refer site sketch plan below, shaded green). The likelihood of the 1965 constructed masonry falling out of plane in future events is high given their unreinforced nature and likely small footings so removal would be prudent to remove the hazard in view of a possible future event. The 1965 masonry running NW side of the site sits atop an undamaged 1.8m high retaining wall of probable reinforced insitu concrete construction.

There are sections newer perimeter wall that have been constructed after original 1965 masonry which could be fully filled and fully reinforced with adequate footing, however this would need to be confirmed with further investigation and equipment. These walls are shown in red in the sketch below and appeared to be undamaged.

The curved feature retaining wall, marked in bold black below, constructed in 1965 of 150mm thick reinforced concrete are of an advantageous shape, giving them good overturning resistance to seismic demand. There is a shear crack at the short end of the curved retaining wall where overturning leverage shear would have been at its most critical; otherwise these curved walls are undamaged.


GATE
1965, GATE LINTEL BADLY DAMAGED SHEAR THROUGH MORTAR.
CONC NEWEL WALLS (MASONRY) IN SUSPECTED FULLY REINFORCED & FILLED STYLE.
1965 NO DAMAGE.
1965 NO DAMAGE
1965 NO DAMAGE.
1965 NO DAMAGE
REINFORCED CONC RETAINING WALL, NO DAMAGE.
THESE WALLS SIT ON 1.8m HIGH CAST REINFORCED CONC RETAINING WALL
1965, COLLAPSED
CRACK
CURVED CONCRETE WALLS. 150mm THICK, REINFORCED.
RETAINING 1.2m.
1965, STILL STAND NO VISIBLE DAMAGE
KEY
1965 - WALLS CONSTRUCTED UNFILLED UNREINFORCED OR POSS. BOND BEAM PRESENT.
LADIES CHANGE
MENS CHANGE
PLANT BUILDING

7. Critical Structural Weakness

7.1 Short Columns

The columns in the building are not subject to short column effects.

7.2 Lift Shaft

The building does not contain a lift shaft.

7.3 Roof

Possible bracing could not be observed in the roof of this building due to the presence of a ceiling lining. Bracing across relies on concrete portals at regular centres so in plane roof bracing could not be considered essential. However, the structure would benefit from roof braces if present to distribute more load to the end concrete panels, stiffening the structure. Opening the ceiling would be beneficial to prove or disprove the existence of roof plane bracing.

Roof bracing or lack thereof has therefore not been included in the IEP as a Critical Structural Weakness.

7.4 Staircases

The building does not contain a staircase.

7.5 Plan Irregularity

Plan irregularity is present due to the structural bracing load path to the masonry walls being indirect; therefore an uneven distribution of seismic load demand is possible. Rows of windows separate the masonry walls from the roof so end concrete panels are relied on for out of plane transfer of building seismic loads to the masonry. This plan irregularity has been considered as “significant” in the IEP for this report and represents a reduction to the %NBS score in accordance with the NZSEE guidelines.

7.6 Liquefaction

Liquefaction potential has been identified in the geotechnical desktop study for this site and is confirmed to be “significant” in regards to IEP grading, further reducing the %NBS score.

8. Geotechnical Consideration

8.1 Introduction & Site Description

This report outlines the ground conditions, as indicated from sources quoted within. This is a desktop report and no site visit has been undertaken by Geotechnical personnel.

This report is specific to the Waltham Main Pool, 30 Waltham Road only. The pool is bound to the north by Waltham Park and south by Waltham Road and Fifield Terrace. The property is owned and maintained by the Christchurch City Council.

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

8.2.1 Published Geology

The geological map of the area 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).

8.2.2 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 Bore Log Summary Table

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.

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 Tonkin and Taylor Report. 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

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

8.2.5 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.

Figure 3 Post February 2011 Earthquake Aerial Photography[1]

8.2.6 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.

[1] 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 Summary of Known Active Faults[2,3]

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

[2] 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.

[3] GNS Active Faults Database

that lateral spreading has occurred within or adjacent to the site. In future seismic events the site is 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.

8.8 Scope and Limits of this Assessment

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.

9. Survey

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

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 building's 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.

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

Item	%NBS
Building excluding CSW's	13
Plan Irregularity (30% Reduction) - Significant bracing discontinuity to masonry	**6**
Site Liquefaction potential significant (30% Reduction)	

Following an IEP assessment, the building has been assessed as achieving 6% New Building Standard (NBS). The structure is therefore considered potentially Earthquake Prone as it achieves less than 34% NBS. This score has not been adjusted when considering damage to the structure as all damage observed was relatively minor in non-loadbearing concrete masonry partitions and considered unlikely to adversely affect the load carrying capacity of the structural systems.

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: 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
- Return period factor R_u = 1.0, NZS 1170.5:2004, Table 3.5, Importance Level 2 structure with a 50 year design life.

Several key seismic parameters that have influenced the %NBS score obtained from the IEP assessment. The building has been assessed as an Importance Level 2 building due to the general use of the building. An increased Z factor of 0.3 for Christchurch has been used in line with recommendations from the Department of Building and Housing recommendations resulting in a reduced % NBS score.

10.3 Expected Structural Ductility Factor

A structural ductility factor of 1.25 has been assumed for the building based on the partially filled masonry bracing system observed for the longitudinal direction and the date of construction. The concrete portal frames are expected to accommodate some rotation at their solid steel bar joints, allowing energy to be dissipated during an earthquake and a higher ductility factor of 2.0 assumed, in accordance with NZSEE guidelines for highest ductility selection. For the purposes of this report and overall assessment of the building performance, the governing ductility of 1.25 is the basis.

Plans were available for the building so the reinforcement in the portal elements is known, though no reinforcement information was detailed for the masonry walls. It is possible that the ductility factor assigned to the transverse direction for the concrete portals could be increased, though this would mean that current standard fully ductile reinforcement provisions would need to be present, something that would be assessed in a detailed analysis of the structure. NZSEE prohibits a selection of higher structural ductility factors for this engineering evaluation report.

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 D soils. The original building was constructed in February 1965 so is more likely to have been designed for only 0.1g lateral seismic force rather than the provisions of NZS1900:1965.ch8. The design loads that were used are therefore likely to have been less than those required by the current loading standard and the detailing requirements for ductile seismic behaviour that are present in the current codes would not have been considered in the design. As a result, it would be expected that the building would not achieve 100% NBS. When combined with the increase in the hazard factor for Christchurch to 0.3 and presence of critical structural weaknesses in the form of plan irregularity and liquefaction, it is reasonable to expect the building to be classified as Earthquake Prone.

10.5 Occupancy

The age and consequent building style of the structure combined with the critical structural weakness of indirect bracing connection to masonry walls renders the building as being potentially Earthquake Prone. As a result, it is recommended that the building is unoccupied pending further detailed assessment and strengthening if required, as per Christchurch City Council's policy regarding occupancy of potentially Earthquake Prone buildings.

11. Initial Conclusions

The Waltham pool main complex has been assessed to have a seismic capacity in the order of 6% NBS and is therefore potentially Earthquake Prone. As per CCC policy, the current closed status of the building should remain as such and the yellow placard status of the building appears to be the correct current access designation.

12. Recommendations

The damage to the building during recent seismic activity in Christchurch has caused minor structural damage only, with minor cracking in concrete masonry walls the only damage that could be observed.

The structure has however been assessed as potentially Earthquake Prone, partially due to the critical structural weakness connection to masonry walls. As a result, we recommend that the building remain closed as per CCC EPB policy until further detailed assessment of the structure is undertaken and if necessary, strengthening options explored.

There is limited information in the archive drawings in relation to roof bracing and steel content in the masonry and precast walls; this should be intrusively investigated to assist in any such future assessment and strengthening.

13. Limitations

This report has been prepared subject to the following limitations:

- No inspection inside the bicycle lockup was undertaken due to no access.
- 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.

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.

13.1 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 Pool side elevation of the main complex.

Photograph 2 Typical masonry wall separation.

Photograph 3 Ceiling showing lining and concrete portal support.

Photograph 4 Typical concrete portal – note the solid steel connectors.

Photograph 5 Roadside south-east side.

Photograph 6 Roadside south-west side .

Photograph 7 Cracking in asphalt paving roadside, possible lateral spread.

Photograph 8 North-west end precast concrete panel.

Appendix B

Existing Drawings / Sketches

Appendix C

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Waltham Pool Main Building Complex	Reviewer:	Stephen Lee
	Unit / No: 30 -40 / Street: Waltham Road	CPEng No:	1006840
Building Address:	30 -40 Waltham Road	Company:	GHD
Legal Description:	PRO 1044-001	Company project number:	513059605
		Company phone number:	
	Degrees / Min / Sec		
GPS south:		Date of submission:	
GPS east:		Inspection Date:	24-Jan-12
		Revision:	
Building Unique Identifier (CCC):	PRO 1044-001	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):		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):	
Ground floor split?	no		Ground floor elevation above ground (m):	0.10
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	Pads for portal bases
Building height (m):	4.00		height from ground to level of uppermost seismic mass (for IEP only) (m):	4
Floor footprint area (approx):	577			
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):	public		Brief strengthening description:	
Use (upper floors):				
Use notes (if required):				
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	250x50 Purlins on concrete portals
Floors:	concrete flat slab	slab thickness (mm)	100
Beams:	precast concrete	overall depth (mm)	450 - 550
Columns:	precast concrete	typical dimensions (mm x mm)	203 diameter
Walls:	partially filled concrete masonry	thickness (mm)	203

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):	6
Ductility assumed, μ:	1.25		wall thickness (m):	0.203
Period along:	0.40	0.40 from parameters in sheet	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	ductile concrete moment frame		note typical bay length (m)	5
Ductility assumed, μ:	2.00			
Period across:	0.40	0.00	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

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

Non-structural elements

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

Available documentation

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

Damage

Site:
(refer DEE Table 4-2)

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

Building:

	Current Placard Status:	yellow		
Along	Damage ratio:	5%	Describe how damage ratio arrived at:	Observation
	Describe (summary):			
Across	Damage ratio:	5%		
	Describe (summary):			
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

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

Recommendations

	Level of repair/strengthening required:	significant structural and strengthening	Describe:	
	Building Consent required:		Describe:	
	Interim occupancy recommendations:	do not occupy	Describe:	
Along	Assessed %NBS before:	6%	6% %NBS from IEP below	If IEP not used, please detail assessment methodology:
	Assessed %NBS after:	6%		
Across	Assessed %NBS before:	14%	14% %NBS from IEP below	
	Assessed %NBS after:	13%		

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: 4m

Seismic Zone, if designed between 1965 and 1992:

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:	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	1.25
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:	**4%**	**4%**

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**:	1.00

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)	1.00	2.00
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:	1.00	1.57
Ductiity Scaling Factor, **Factor D**:	1.00	1.57

2.6 Structural Performance Scaling Factor:

	along	across
Sp:	1.000	0.700
Structural Performance Scaling Factor **Factor E**:	1	1.428571429

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

	along	across
$\%NBS_b$:	**13%**	**28%**

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: 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:	**6%**	**14%**
4.4 Percentage New Building Standard (%NBS), (before)		**6%**

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