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Avondale Park Toilet
PRK 1283 BLDG 001

Detailed Engineering Evaluation
Quantitative Report
Version FINAL

Mervyn Drive, Avondale
Christchurch

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

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Mervyn Drive, Avondale
Christchurch

Christchurch City Council

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Date
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Avondale Park Toilet

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

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Background

This is a summary of the Quantitative report for the Avondale Park Toilet building located at Avondale, Christchurch, and is based in part on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on the 7th of August 2013, and seismic capacity calculations.

Building Construction

- Roof: Timber rafters and steel hip beams clad with lightweight roofing;
- Wall: 20 series solid filled reinforced masonry wall;
- Floor: 125mm thick concrete on-grade slab;
- Foundation: Perimeter concrete strip footings.

Key Damage Observed

There was no damage observed during the site inspection.

Critical Structural Weaknesses

No critical structural weaknesses have been identified when assessing the building.

Geotechnical Investigation

The geotechnical 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 is considered to be susceptible to significant liquefaction. A soil class of D (in accordance with NZS1170.5:2004) should be adopted for the site.

Quantitative Assessment Summary

The overall seismic capacity for the Avondale Park Toilet building assessed in accordance with NZSEE guidelines is **67%** NBS. The limiting element for this 67% NBS value is the over-turning capacity of the cantilevered partition walls. The in-plane seismic capacity of the building has been assessed as greater than 100% NBS in both the longitudinal and the transverse directions.

Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the building is considered a Low Earthquake Risk as it achieves greater than 67% NBS. The results obtained from the seismic capacity assessment are consistent with those expected for a building of this age and construction type, and combined with the increase in the hazard factor for Christchurch to 0.3, it would be expected that the building would not achieve 100% NBS.

Recommendations

The building has been assessed as a Low Risk Building, and no critical structural weaknesses have been identified. As a result of this assessment, general occupancy is premitted.

1. Background

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

This report is a Quantitative Assessment and is based in general on NZS 1170.5: 2004, NZS 4230:2004 and the New Zealand Society for Earthquake Engineering (NZSEE) guidelines.

This quantitative assessment of the building comprises of an investigation of the in-plane and out-of-plane strengths of the solid filled reinforced masonry walls. The investigation is based on analysis of the seismic loads that the structure is subjected to, analysis of the distribution of these forces throughout the structure and analysis of the capacity of the existing structural elements to resist the seismic forces applied to them. The capacity of the existing structural elements is compared to the demand placed on the elements to give the percentage of New Building Standard (%NBS) of each of the structural elements.

Electromagnetic scans have been carried out on site to confirm the extent of the reinforcement in the masonry walls.

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; and
- 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).

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 building is a single-storey rectangular structure, located on the north side of Avondale Park, Avondale. The original construction date of the toilet is unknown. However, the extension that is used as a storage room (comprising 80% of the current floor area) was built in 2007.

The building measures approximately 8.8m long by 6.2m wide by 3.5m height at apex. It is rectangular in plan, with a gross floor area of approximately 55m^2. The site location is shown in Figure 2 and the floor layout is shown in Figure 3.

Figure 2 Site Location Plan

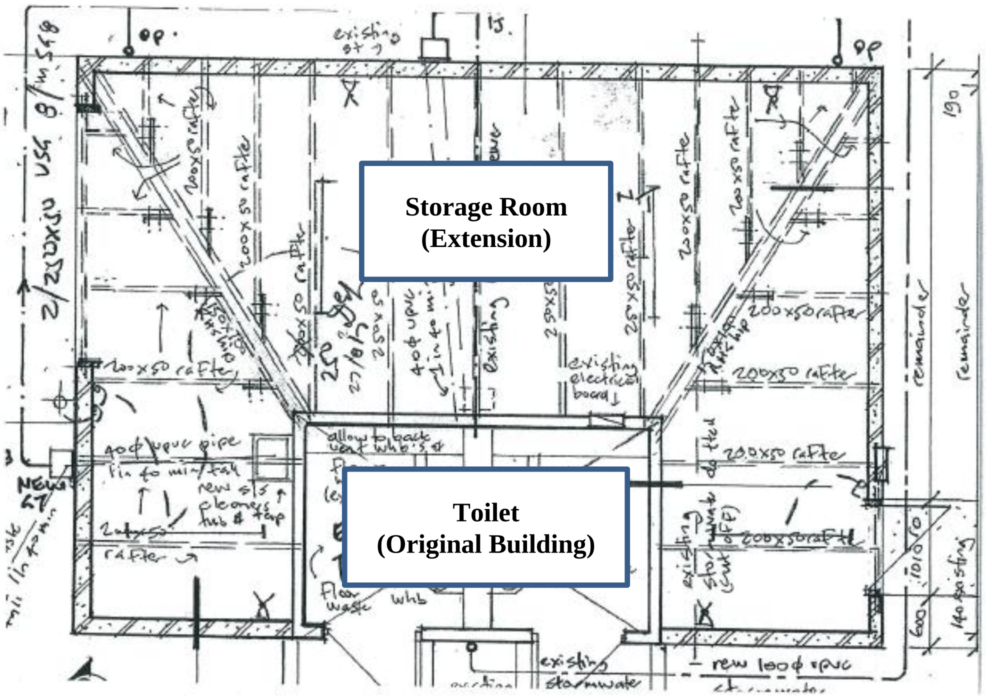

Figure 3 Floor Plan

The roof comprises timber rafters which are supported on steel hip beams spanning across the building, and is clad with lightweight roofing on timber purlins. Walls are 2.4 m high, constructed using 20 series solid filled reinforced masonry blockwork. The walls are supported on concrete strip foundations and the floor is 125mm thick reinforced concrete on-grade slab as per existing drawings.

4.2 Gravity Load Resisting System

The gravity support for the building is provided by the purlins, timber rafters, steel hip beams, masonry walls and strip foundations. The roof cladding is supported by timber purlins spanning between the timber rafters. These rafters transfer the roof load through steel hip beams to the masonry walls. The masonry walls then transfer the load to the concrete strip foundations.

4.3 Lateral Load Resisting System

Lateral loads acting on the structure are resisted by the solid filled reinforced masonry walls in both of the longitudinal and the transverse directions. These walls transfer the lateral seismic loading of the structure to the foundations.

5. Damage Assessment

An inspection of the building was undertaken on 7th of August, 2013. Both the interior and exterior of the building were inspected. The main structural components of the building were inspected. However, the foundations were unable to be viewed due to inaccessibility.

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

No floor level survey or verticality surveys have been undertaken by GHD for this building at this stage.

A Hilti PS 200 Ferroscan was used to determine and to confirm the position, depth and diameter of the reinforcement in the masonry walls. This scanning equipment uses electro-magnetic fields to determine the size and depth of the reinforcing steel in the building. In the case of conflicting results, the most conservative bar diameter has been chosen for the capacity calculations.

5.1 Surrounding Buildings

No obvious damage to surrounding structures was noted during site inspection.

5.2 Residual Displacements and General Observations

No residual displacements of the structure were noticed during our inspection of the building.

No damage was noted to the masonry walls, concrete on-grade slab and roof structures.

No significant changes in floor level were observed.

5.3 Ground Damage

There was no obvious sign of ground damage observed during site inspection.

6. Critical Structural Weakness

Short Columns

No short columns are present in the structure.

Lift Shaft

The building does not contain a lift shaft.

Roof

Roof bracing was not seen from the access point. Roof elements such as timber purlins and rafters were clearly visible and are expected to provide bracing to the roof structure. In particles the angled hip rafters provide significant bracing.

Staircases

The building does not contain a staircase.

Site Characteristics

Refer to geotechnical consideration section (Section 7) for details.

Plan Irregularity

The building is rectangular therefore no plan irregularity is present.

Vertical irregularity

The building is single-storey, with a constant ceiling height. No vertical irregularity is present.

Pounding effect

No adjacent buildings; pounding is not applicable.

Height difference offset

Not applicable.

7. Geotechnical Consideration

This desktop geotechnical study outlines the ground conditions, as indicated from sources quoted within, for inclusion in the subject structure's DEE Qualitative Assessment. This is a desktop study report and no site visit has been undertaken by GHD Geotechnical personnel.

This section is specific to the Avondale Park Toilet at Avondale, Christchurch. The site is surrounded by residential properties, and is owned by the Christchurch City Council.

7.1 Site Description

The site is situated in the suburb of Avondale, in east Christchurch. It is relatively flat at approximately 1 m above mean sea level. It is approximately 250 m south of the Avon River, and 3 km west of the coast (Pegasus Bay).

7.2 Published Information on Ground Conditions

7.2.1 Published Geology

Brown & Weeber, 1992[1] describes the site geology as:

- Dominantly alluvial sand and silt overbank deposits, of the Yaldhurst Member, sub-group of the Springston Formation, Holocene in age;
- Underlying sediments (younger than 6500 years) are surface alluvial silt and sand, subsurface marine sand and alluvial silt and sand, and some peat. No interbedded gravel;
- The Riccarton gravels are located approximately 31 m bgl; and
- Groundwater is 1 m below ground level.

7.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that fourteen boreholes with lithographic logs are located within 200 m of the site. Four ECan borehole logs have been summarised in Table 2.

These indicate the area is underlain by sand and gravel to 1.1 m bgl, underlain by sandy silt to 2.2 m bgl, underlain by wet sand and sandy silt to 2.6 m bgl. Varying amounts of clay is also indicated to be present from 1.2 m to 2.4 m bgl, 130 m west of the site.

Groundwater was not recorded on the logs. However the logs indicate that the soil becomes wet between 2.1 m and 2.35 m bgl.

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

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	From Site	Log Summary	
M35/14991	2.6 m	2.35 m	130 m W	0.0 to 0.1 m	Topsoil
				0.1 to 0.7 m	Sand
				0.7 to 1.1 m	Sand and gravel
				1.1 to 1.2 m	Sandy silt
				1.2 to 2.4 m	Clay and silt
				2.4 to 2.6 m	Sandy silt; wet
M35/14992	2.5 m	2.2 m	170 m NW	0.0 to 0.9 m	Sand
				0.9 to 2.2 m	Sandy silt
				2.2 to 2.5 m	Sandy silt; wet
M35/15001	2.55 m	2.1 m	150 m W	0.0 to 0.2 m	Topsoil
				0.2 to 1.0 m	Sand and gravel
				1.0 to 1.1 m	Topsoil
				1.1 to 1.2 m	Sand and silt
				1.2 to 2.1 m	Sandy silt
				2.1 to 2.6 m	Sand; wet
M35/15002	2.6 m	2.2 m	110 m W	0.0 to 0.5 m	Topsoil
				0.5 to 1.3 m	Sand
				1.3 to 2.2 m	Sandy silt
				2.2 to 2.6 m	Sand; wet

It should be noted that the logs may have been written by the well driller and not a geotechnical professional or to a standard. In addition strength data is not recorded.

7.2.3 EQC Geotechnical Investigations

The Earthquake Commission has undertaken geotechnical testing in the area of the site. Information pertaining to this investigation is included in the Tonkin & Taylor Report for Avondale[2]. Six investigation points were undertaken within 100 m of the site, two of which are summarised below in Table 3.

Table 3 EQC Geotechnical Investigation Summary Table

Bore Name	Orientation from Site	Depth (m bgl)	Log Summary[3]
CPT-AVD-42	40 m NW	0.0 – 1.2	Pre-drilled
		1.2 – 2.0	Silty CLAY; very stiff.
		2.0 – 3.0	Silty SAND; loose

[2] Tonkin & Taylor Ltd., 2011: Christchurch Earthquake Recovery, *Geotechnical Factual Report, Avondale.*

[3] Log Summary for CPT's interpreted from Soil Behavior Type Robertson *et al*. 2010

Bore Name	Orientation from Site	Depth (m bgl)	Log Summary[3]
		3.0 – 10.0	Silty SAND; medium dense
		10.0 – 10.6	Silty CLAY; firm
		10.6 – 28.7	SAND; medium dense to dense
		28.7 – 31.7	Sandy SILT; stiff
		31.7 – 32.3	Silty CLAY; stiff
		32.3 – 32.4	Silty SAND; dense
			(WT at 2.8 m bgl)
CPT-AVD-43	60 m S	0.0 – 1.2	Pre-drilled
		1.2 – 2.0	Silty CLAY; very stiff
		2.0 – 10.9	Silty SAND; loose to medium dense
		10.9 – 11.4	Silty CLAY; firm
		11.4 – 26.6	SAND; dense
		26.6 – 28.0	Silty SAND; very loose
		28.0 – 28.6	SAND; dense
		28.6 – 32.1	Silty SAND; very loose
		32.1 – 32.7	Silty CLAY; very stiff
		32.7 – 33.0	SAND; dense
			(WT at 2.5 m bgl)

Initial observations of the CPT results indicate the site is underlain by very stiff silty clay to 2.0 m bgl, underlain by loose silty sand to 11.4 m bgl, underlain by medium dense to dense sand to 26.6 m bgl, underlain by interbedded layers of very loose silty sand and dense sand to 31.7 m bgl. Both CPTs refused on dense sand and silty sand at 33.0 m bgl.

The CPT results also indicate occasional layers of firm to very stiff silty clay layers. Two prominent layers of silty clay, 600 mm thick, are present between 10.0 m and 11.4 m bgl and between 31.7 m and 32.7 m bgl.

An assumed ground water level of 2.5 m to 2.8 m bgl was indicated on the logs.

7.2.4 CERA Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has indicated the site is situated within the Green Zone, indicating that repair and rebuild may take place. However, some neighbouring residential properties have been red zoned.

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

The site has been categorised as “N/A” – Urban Non-residential”[4]. However, neighbouring residential properties have either been categorised as TC3 (blue), indicating moderate to severe land damage from

[4] CERA Landcheck website, http://cera.govt.nz/my-property

liquefaction is possible in future significant earthquakes, or as “Red zone”, indicating that land repair would be prolonged and uneconomic.

7.2.5 Historic Land Use

The Listed Land Use Register (LLUR)[5] indicates that no hazardous activities have occurred at the site.

The Black Maps[6] shows that the area was historically “swamp”.

The CCC historic landfill map[7] shows that shallow fill consisting of river dredgings is located on the site.

Historical aerial photography shows that the site was previously farm land (1946 and 1955).

7.2.6 Post-Earthquake Land Observations

Aerial photography[8] taken following the 22 February 2011 earthquake shows signs of severe liquefaction in Avondale Park in the form of sand boils, and on nearby streets, as shown in Figure 4. Ponding in the courts adjacent to the site also occurred following the 22 February 2011 earthquake. Aerial photography taken following the 4 September 2010 earthquake shows no signs of liquefaction, and aerial photography taken following the 13 June 2011 and 23 December 2011 earthquakes shows no new evidence of liquefaction.

Figure 4 Post February 2011 Earthquake Aerial Photography

The Canterbury Geotechnical database shows that cracks between 10 and 50 mm occurred within 150 m of the site[9].

[5] Environmental Canterbury Regional Council: *Listed Land Use Register,* retrieved 16/07/2013 from http://llur.ecan.govt.nz/

[6] Waterways, Swamps and Vegetation Cover in 1856 Compiled from "Black Maps", Source: Christchurch City Council retrieved 24 September 2013, http://resources.ccc.govt.nz/files/blackmap-environmentecology.pdf

[7] Christchurch City Council (1993): *Christchurch Landfill Sites* and accompanying key *Old Landfills within Christchurch*.

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

[9] Canterbury Geotechnical Database (2012) "Observed Ground Crack Locations", Map Layer CGD0400 - 23 July 2012, retrieved [24/09/2013] from https://canterburygeotechnicaldatabase.projectorbit.com/

7.2.7 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated to comprise sand, silt and clay to 2.6 m bgl, underlain by silty sand to 11.4 m bgl, underlain by underlain by sand to 26.6 m bgl, underlain by interbedded layers of sand and silty sand to 33.0 m bgl, with occasional layers of silty clay.

Groundwater is considered to vary between 2.1 m and 2.8 m bgl.

7.3 Seismicity

7.3.1 Nearby Faults

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

Table 4 Summary of Known Active Faults[10,11]

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	130 km	NW	~8.3	~300 years
Greendale Fault (2010)	25 km	W	7.1	~15,000 years
Hope Fault	100 km	N	7.2~7.5	120~200 years
Porters Pass Fault	65 km	NW	7.0	~1100 years
Port Hills Fault (2011)	8 km	S	6.3	Not Estimated

The recent earthquake sequence since 4 September 2010 has identified the presence of a previously unmapped active fault system underneath the Canterbury Plains; this includes the Greendale Fault and Port Hills Fault listed in Table 4 above. Research and published information on this system is in development and the average recurrence interval is yet to be established for the Port Hills Fault.

7.3.2 Ground Shaking Hazard

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

The recent seismic activity has produced earthquakes of Magnitude 6.3 with significant peak ground accelerations (PGA) across large parts of the city.

Conditional PGA's from the CGD[12] indicate the PGA to be 0.18 g during the 4 September 2010 earthquake, 0.36 g on 22 February 2011, and 0.25 g on 13 June 2011.

[10] 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.

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

[12] Canterbury Geotechnical Database (2012): "Conditional PGA for Liquefaction Assessment", Map Layer CGD5110 - 27 Sept 2012, retrieved 31/10/2012 from https://canterburygeotechnicaldatabase.projectorbit.com/

7.4 Global Land Movement

Given the site's proximity to the Avon River, and evidence from the recent earthquakes, the site may be susceptible to lateral spreading. In addition, any retaining structures or embankments nearby should be further investigated to determine the site-specific local slope instability potential.

According to table 12.3 in section 12.2.1 of the MBIE guidance[13], the site may be assumed to be susceptible to minor to moderate global lateral spread, as the edge of building is further than 200 m from the bank of the Avon River.

7.5 Liquefaction Potential

The site is considered to be susceptible to moderate to significant damage due to liquefaction, due to:

- Evidence of severe liquefaction in Avondale Park and on nearby streets;
- The site categorised as TC3;
- Layers of loose to medium dense silty sand in the top 10 m underlying the site; and
- The shallow ground water table at a depth of 2.1 m to 2.8 m bgl.

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

7.6 "Sufficiently Tested at SLS"

Site observations of recent earthquake damage can be correlated to the likely performance of the site at serviceability limit state (SLS) by comparing the PGA observed with design values. This methodology is outlined in the MBIE guidance on Liquefaction Methodology.

Since the PGA for 22 February exceeds 170% of the SLS value, the site can be considered "sufficiently tested at SLS". As a result, the ground damage during a future moderate earthquake (SLS) is likely to be similar or less than that observed in the 22 February 2011 earthquake.

7.7 Summary & Recommendations

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

The site appears to be situated on stratified alluvial deposits, comprising sand and silt with some clay. Associated with this the site also has a moderate to significant liquefaction potential, in particular where sands and/or silts are present.

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

Should a more comprehensive liquefaction and/or ground condition assessment be required, it is recommended that intrusive investigation be conducted.

[13] Ministry of Business, Innovation & Employment – Building & Housing (2012): Repairing and Rebuilding Houses affected by the Canterbury Earthquakes; Version 3, Dec 2012. MBIE: Wellington, NZ.

8. Seismic Capacity Assessment

8.1 Seismic Parameters

The seismic parameters have been determined in accordance with NZS1170.5. A full detailed calculation has been included into Appendix C.

8.1.1 Shear Capacity of the Reinforced Masonry Walls

The in-plane shear capacity of the solid filled reinforced masonry wall was provided by the vertical reinforcing and the strength was determined in accordance with NZS 4230:2004. The out-of-plane shear was generally not critical and the design shear stress should be less than the out-of-plane strength of the masonry alone. The strength reduction factor, ϕ, for shear was taken as 0.75 in accordance with Cl3.4.7.

8.1.2 Moment Capacity of the Reinforced Masonry Walls

The moment capacity of the reinforced masonry wall (i.e. in-plane and out-of-plane) was determined in accordance with NZS 4230:2004. The strength reduction factor, ϕ, for flexural with or without axial tension or compression was taken as 0.85 in accordance with Cl3.4.7.

8.2 Quantitative Assessment Procedure

The seismic capacity was calculated in accordance with NZS 4230:2004 and the NZSEE guidelines[14] and based on the information obtained from visual observation and site measurements of the building. The demand for the structure was calculated in accordance with NZS 1170.5:2004, and the percentage of New Building Standard capacity (%NBS) was assessed.

The building was modelled as in-plane and out-of-plane shear walls. For further details on the assessment methodology, please refer to Appendix C.

8.2.1 %NBS

The shear capacity of the walls and the out-of-plane moment capacities were then compared to their respective demands to assess which was the most critical and thus determine the overall %NBS for the structure as follows:

$$\%\mathrm{NBS} = \frac{\emptyset \mathrm{S_n}\ (\mathrm{Capacity})}{\mathrm{S^*}\ (\mathrm{Demand})} \times 100\%$$

8.3 % NBS Assessment

As part of the Quantitative assessment, a more detailed seismic capacity assessment was carried out for the shear capacity of the building, with the results summarised below.

A summary of the different seismic capacities assessed is presented in Table 5 on the following page.

[14] New Zealand Society for Earthquake Engineering (2006): *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes*. Recommendations of a NZSEE Study Group on Earthquake Risk Buildings, June 2006. NZSEE

Table 5 Assessment Summary

Direction	%NBS
Longitudinal Direction	67%
Transverse Direction	67%
Critical %NBS	**67%**

The overall seismic capacity for the Avondale Park Toilet building assessed in accordance with NZSEE guidelines is **67%** NBS. The limiting element for this value is the overturning resistance capacity of existing concrete strip foundations.

As there was no damage observed during the site inspection, no further reduction is applied to the assessment result. Therefore, the overall seismic capacity for this building remains at 67% NBS.

8.4 Discussion of Results

Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the building is considered as a Low Risk Building as it achieves 67% NBS. The results obtained from the seismic capacity assessment are consistent with those expected for a building of this age and construction type, and combined with the increase in the hazard factor for Christchurch to 0.3, it would be expected that the building would not achieve 100% NBS. Due to the lack of any Critical Structural Weaknesses and the presence of vertical reinforcing steel in the masonry walls, it is reasonable to expect the building to be classified as a Low Risk Building.

8.5 Occupancy

The building does not pose an immediate risk to users and occupants as no critical structural weaknesses have been identified. Furthermore, the building has been classified as a Low Risk Building and therefore general occupancy is permitted.

9. Survey

No floor level survey or verticality survey have been undertaken by GHD for this building at this stage.

10. Conclusions and Recommendations

The building has been assessed to have a seismic capacity in the order of **67%** NBS and therefore it has been classified as a Low Risk Building.

Improvement to the seismic capacity of the building is not required since the building has an overall seismic capacity of 67% NBS. In addition, there was no damage or CSWs observed on site, therefore general occupancy is permitted.

11. Limitations

11.1 General

This report has been prepared subject to the following limitations:

- No intrusive structural investigations have been undertaken;
- No intrusive geotechnical investigations have been undertaken;
- No inspection of the floor slab or foundations could be undertaken due to inaccessibility;
- No floor level or verticality surveys have been undertaken;
- No material testing has been undertaken; and
- 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 reportrite a specific limitations section.

11.2 Geotechnical Limitations

This report presents the results of a geotechnical appraisal prepared for the purpose of this commission, and for prepared solely for the use of Christchurch City Council and their advisors. The data and advice provided herein relate only to the project and structures described herein and must be reviewed by a competent geotechnical engineer before being used for any other purpose. GHD Limited (GHD) accepts no responsibility for other use of the data.

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

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

Appendix A

Photographs

Photograph 1: North Elevation

Photograph 2: East elevation

Photograph 3: West Elevation

Photograph 4: South Elevation

Photograph 5: Interior of the Storage Room

Photograph 6: Interior of the Storage Room

Photograph 7: Interior of Toilet

Appendix B

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Avondale Park Toilet		Reviewer:	D. Lee
	10 No:	Street	CPEng No:	112052
Building Address:		Mervyn Drive, Avondale	Company:	GHD Ltd
Legal Description:	Lot 57, DP 59056		Company project number:	51 30766 00
			Company phone number:	03 3780900

	Degrees	Min	Sec		
GPS south:	43	50	42.00	Date of submission:	
GPS east:	172	59	2.00	Inspection Date:	24-Sep-13
				Revision:	FINAL
Building Unique Identifier (CCC):	PRK 1283 BLDG 001			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	silty sand	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	1.00

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	1.00
Ground floor split?	no		Ground floor elevation above ground (m):	1.20
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	Subfloor & concrete slab
Building height (m):	3.50		height from ground to level of uppermost seismic mass (for IEP only) (m):	7
Floor footprint area (approx):	55			
Age of Building (years):	45		Date of design:	1965-1976
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):	public toliet and storage room			
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	250x50 timber rafters, steel RHS hip beams
Floors:	other (note)	describe sytem	125mm thick concrete on-grade slab
Beams:	timber	type	
Columns:			
Walls:	fully filled concrete masonry	#N/A	

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	other (note)		describe system	solid filled masorny walls
Ductility assumed, μ:	1.25			
Period along:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	other (note)		describe system	solid filled masorny walls
Ductility assumed, μ:	1.25			
Period across:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

leave blank if not relevant

north (mm):	
east (mm):	
south (mm):	
west (mm):	

Non-structural elements

Stairs:			
Wall cladding:	exposed structure	describe	reinforced masorny
Roof Cladding:	Metal	describe	Galv corrugated iron
Glazing:			
Ceilings:			
Services(list):			

Available documentation

Architectural	none	original designer name/date	
Structural	partial	original designer name/date	City Solutions Aug 2007
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:	good	Describe damage:	
Settlement:	more than 200mm	notes (if applicable):	
Differential settlement:	1:250-1:150	notes (if applicable):	
Liquefaction:	0-2 m³/100m²	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	moderate to substantial (1 in 5)	notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
Diaphragms	Damage?:		Describe:	
CSWs:	Damage?:		Describe:	
Pounding:	Damage?:		Describe:	
Non-structural:	Damage?:		Describe:	

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

Recommendations

	Level of repair/strengthening required:	none		Describe:	no damage observed on site
	Building Consent required:	no		Describe:	
	Interim occupancy recommendations:	full occupancy		Describe:	
Along	Assessed %NBS before e'quakes:	67%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	quantitative analysis
	Assessed %NBS after e'quakes:	67%			
Across	Assessed %NBS before e'quakes:	67%	##### %NBS from IEP below		
	Assessed %NBS after e'quakes:	67%			

Appendix C

Quantitative Assessment Methodology

C1. Building Seismic Demand

The demand on the structure was determined in accordance with NZS 1170.5:2004, which uses the equivalent static method. The structure is located in Christchurch, on class D soils.

An Importance Level of 2 was used for the calculations. This results in the Return Period Factor, as given by Table 3.5 of NZS 1170.5: 2004 and as prescribed by Table 3.3 of AS/NZS 1170.0:2002, for the structure as 1.0.

C2. Seismic Weight Coefficient

The elastic site hazard spectrum for horizontal loading, C(T), for the building was derived from Equation 3.1(1), NZS 1170.5:2004;

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

Where

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

Z = the hazard factor from Clause 3.1.4, and subsequent amendments issued by DBH, which increased the hazard factor to 0.30 for Christchurch;

R = the return period factor from Table 3.5 for an annual probability of exceedance of 1/500 (earthquake action for an Importance Level 2 building); and,

N(T,D) = the near-fault scaling factor from Clause 3.1.6.

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

$$\mathbf{S_P = 1.3 - 0.3\mu}$$

Where the μ is the structural ductility factor.

The seismic weight coefficient, $C_d(T_1)$, was then calculated in accordance with Clause 5.2.1.1 of NZS 1170.5:2004. For the purposes of calculating the seismic weight coefficient a period, T_1, for the building. The coefficient was then calculated using Equation 5.2(1);

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

Where

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

Expected Structural Ductility Factor

A structural ductility factor, μ, of 1.25 has been assumed based on the reinforced masonry structural system observed and the likely date of construction.

Fundamental Period of Building

A fundamental period of oscillation, T_1, of 0.4 seconds has been adopted for this assessment. This is based on the stiff, rigid nature of the reinforced masonry construction.

C3. Induced Shear Forces to Walls

The lateral forces induced on the walls of the building in a seismic event include the direct seismic shear and any torsional forces caused by the centre of mass and centre of rigidity of the building being offset.

NZS 1170.5 makes allowance for accidental eccentricity of ± 0.1 times b, the plan dimension of the structure at right angles to the direction of loading. That is, the force is applied at ±0.1b from the centre of mass. This results in a torsional action about the centre of resistance of the building, and induces forces in the lateral force resisting (in-plane) walls in addition to direct shear.

Clause 5.3.1.2 of NZS 1170.5 also requires that for brittle and nominally ductile structures, the forces are to be applied in such a way that 100% of the force is applied in one direction while 30% of that force is applied simultaneously in the orthogonal direction.

The induced shear force plus the direct shear is what must be designed for and the magnitude of the forces distributed into the walls is relative to their in-plane stiffness.

Moment demands were calculated by multiplying the shear forces by the effective seismic mass height. This effective height comes from the weighted average of the heights of all seismic weights. This is typically approximately half the structure's height for a single-storey structure.

C4. Wall Shear Capacity

The shear capacity of the solid filled reinforced masonry wall was determined using NZS 4230:2004. As there are no details as to the level of supervision during the construction stage, the Observation Type was classed in accordance with Table 3.1, and considered to be Type B. The overall shear capacity of the wall was calculated from Clause 10.3.2.1, Equation 10-4;

$$\mathbf{\emptyset V_n = \emptyset\, v_n b_w\, d}$$

Where

v_n = the total shear stress which consists of the contribution of the masonry, v_m, the axial load, v_p and the contribution of the shear reinforcement, v_s (zero as no horizontal reinforcing detected);

b_w = the thickness of the wall;

d = depth from compression end of wall to centre of reinforcing, approximated as 0.8 times the wall length (NZS 4230:2004); and

∅ = strength reduction factor, 0.75 for concrete masonry in shear (Clause 3.4.7).

C5. Out-of-Plane Capacity

Due to the observation from the site inspection, out-of-plane moments have been resisted by the cantilever action of the solid filled reinforced masonry walls. The out-of-plane flexural capacity of walls is determined in accordance with NZS 4230:2004

Appendix D

Existing Drawings

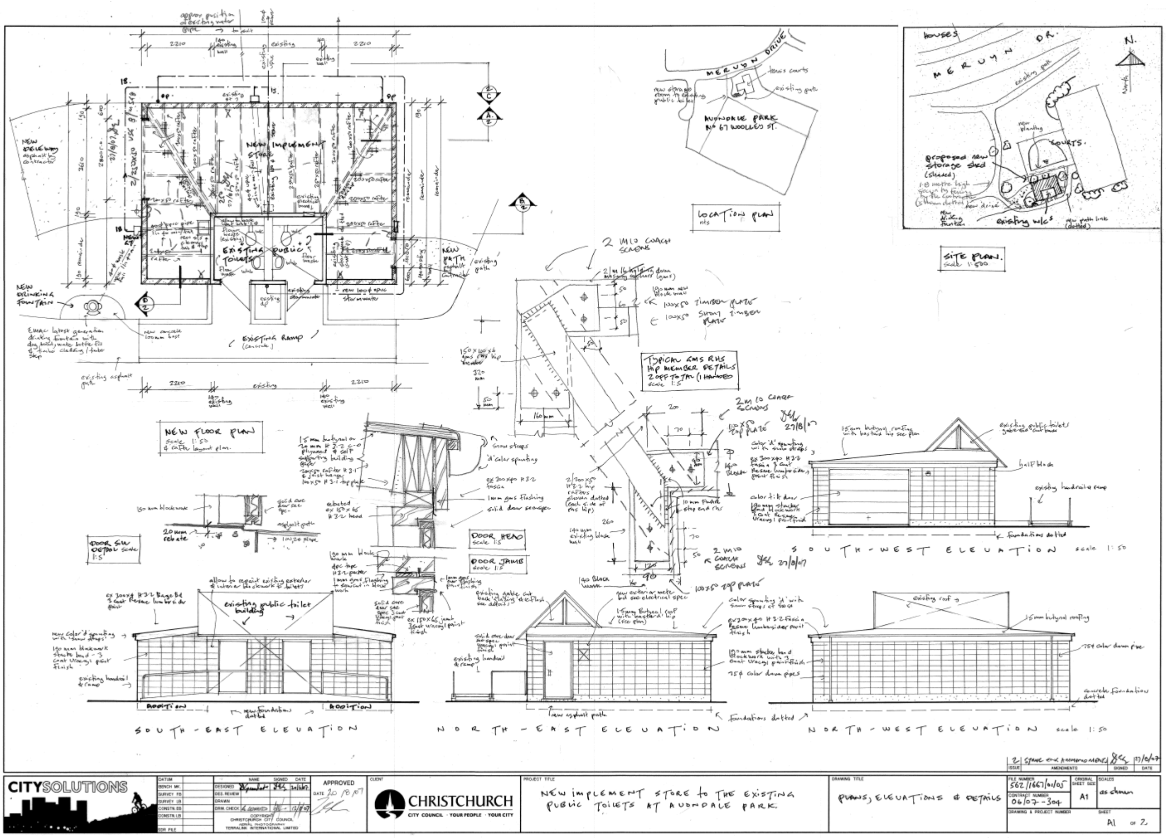

Figure 5 Existing Drawing

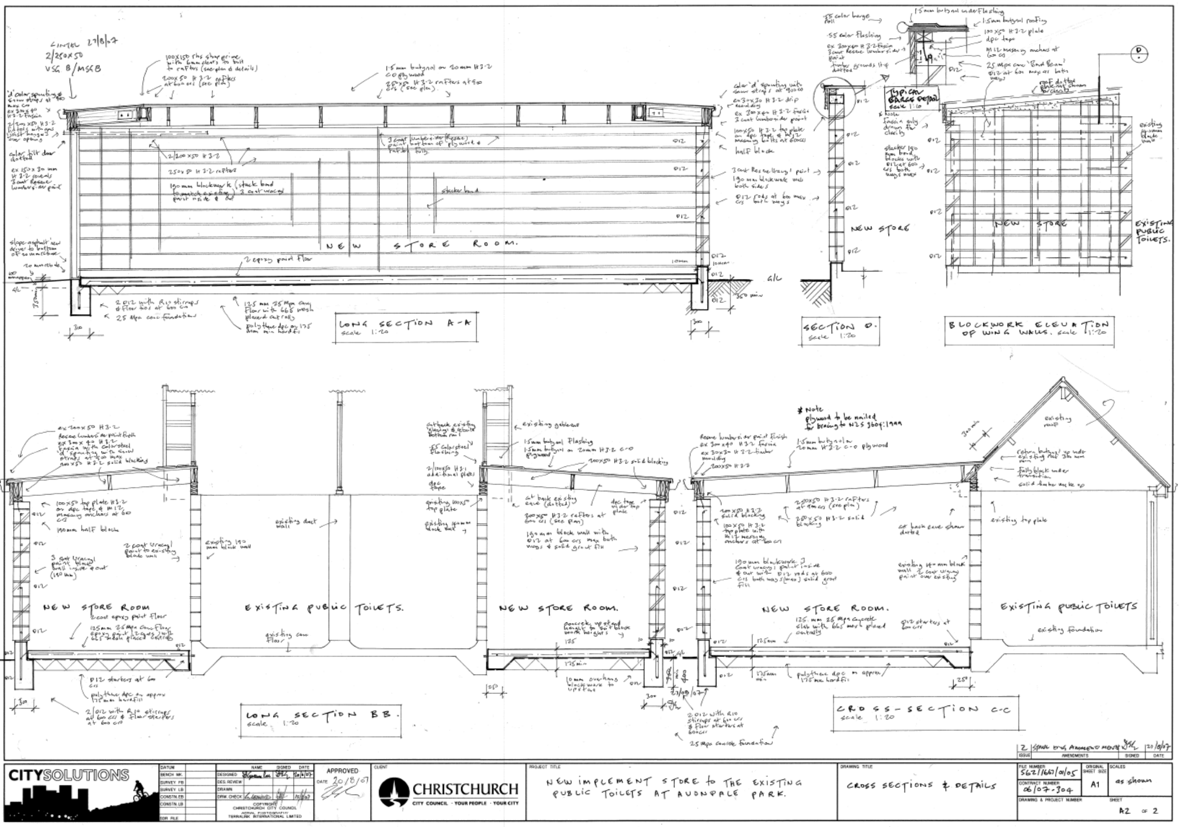

Figure 6 Existing Drawing

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Document Status

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