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Hoon Hay Park – Toilet/Pavilion
PRK 1492 BLDG 001 EQ2

Detailed Engineering Evaluation
Quantitative Report
Version FINAL

61 Mathers Road, Hoon Hay

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

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Quantitative Report
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16 Mathers Road, Hoon Hay

Christchurch City Council

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Hoon Hay Park – Toilet/Pavilion

PRK_1492_BLDG_001 EQ2

Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

61 Mathers Road, Hoon Hay

Background

This is a summary of the Quantitative report for the Hoon Hay Park Toilets and Pavilion, and is based in part on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011 and visual inspections on 28th June 2012 and 21st November 2012.

Building Description

The overall structure is a single level park facilities building and comprises of four pavilion rooms and toilets at the northern end. Roof and wall construction appears to be consistent throughout. The roof is formed by lightweight metal cladding supported by timber purlins, rafters and trusses, with ceiling linings to the pavilion area. The roof trusses are connected to the top of the unreinforced concrete masonry block walls. The floors are on-grade reinforced concrete slabs and strip footings are located under the masonry walls. Walls extend from the concrete strip footings to the eaves level are 190mm concrete masonry units.

Key Damage Observed

Key damage observed includes:-

- No key damage was observed.

Building Strength

Based on the information available, and using the NZSEE guidelines for a Quantitative Assessment, the building's baseline post-earthquake capacity (including critical structural weaknesses and earthquake damage) has been assessed to be the order of 19% NBS.

The site has a minor to moderate liquefaction potential, in particular where sands and/or silts are present, however this is not considered a critical structural weakness.

The building has been assessed to have a seismic capacity in the order of 19% NBS and is therefore considered to be Earthquake Prone.

Recommendations

The recent seismic activity in Christchurch has not caused any visible damage to the building. The building has achieved approximately 19% NBS following a Quantitative Detailed Engineering Evaluation. Further assessment is not required. GHD recommends significant wall strengthening or wall replacement options be explored and implemented to bring the %NBS of the building up to a minimum of 67% NBS in accordance the NZSEE guidelines.

1. Background

GHD has been engaged by Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the Hoon Hay Park Toilet/Pavilion block.

This report is a Quantitative Assessment and is based on NZS 1170.5: 2004 and NZS 4230: 2004.

The quantitative assessment of the building comprises an investigation of the in-plane and out-of-plane strength of the unreinforced masonry block walls. The investigation is based on the analysis of the seismic loads that the structure is subjected to, the analysis of the distribution of these forces throughout the structure and the analysis of the capacity of existing structural elements to resist the forces applied. 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 locate any reinforcement in the walls.

At the time of this report, no finite element modelling of the building structure has been carried out.

2. Compliance

This section contains a brief summary of the requirements of the various statutes and authorities that control activities in relation to buildings in Christchurch at present.

2.1 Canterbury Earthquake Recovery Authority (CERA)

CERA was established on 28 March 2011 to take control of the recovery of Christchurch using powers established by the Canterbury Earthquake Recovery Act enacted on 18 April 2011. This act gives the Chief Executive Officer of CERA wide powers in relation to building safety, demolition and repair. Two relevant sections are:

Section 38 – Works

This section outlines a process in which the chief executive can give notice that a building is to be demolished and if the owner does not carry out the demolition, the chief executive can commission the demolition and recover the costs from the owner or by placing a charge on the owners' land.

Section 51 – Requiring Structural Survey

This section enables the chief executive to require a building owner, insurer or mortgagee carry out a full structural survey before the building is re-occupied.

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

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

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

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

2.2 Building Act

Several sections of the Building Act are relevant when considering structural requirements:

Section 112 – Alterations

This section requires that an existing building complies with the relevant sections of the Building Code to at least the extent that it did prior to any alteration. This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

Section 115 – Change of Use

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

2.2.1 Section 121 – Dangerous Buildings

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

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

Section 122 – Earthquake Prone Buildings

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

Section 124 – Powers of Territorial Authorities

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

Section 131 – Earthquake Prone Building Policy

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

2.3 Christchurch City Council Policy

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

The 2010 amendment includes the following:

- A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
- A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;
- A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
- Repair works for buildings damaged by earthquakes will be required to comply with the above.

The council has stated their willingness to consider retrofit proposals on a case by case basis, considering the economic impact of such a retrofit.

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

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

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

2.4 Building Code

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

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

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

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

3. Earthquake Resistance Standards

For this assessment, the building's earthquake resistance is compared with the current New Zealand Building Code requirements for a new building constructed on the site. This is expressed as a percentage of new building standard (%NBS). The new building standard load requirements have been determined in accordance with the current earthquake loading standard (NZS 1170.5:2004 Structural design actions - Earthquake actions - New Zealand).

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

The New Zealand Society for Earthquake Engineering has proposed a way for classifying earthquake risk for existing buildings in terms of %NBS and this is shown in Figure 1 below.

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

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

Table 1 compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% risk of exceedance in 50 years (i.e. 0.2% in the next year). It is noted that the current seismic risk in Christchurch results in a 6% risk of exceedance in the next year.

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

Table 1 %NBS compared to relative risk of failure

4. Building Description

4.1 General

The Pavilion/Toilets Building is located in Hoon Hay Park on 61 Mathers Road, Hoon Hay. The building was constructed in 1966 and is used as a changing shed, clubroom and toilet facility. The toilet area internal walls at the northern end of the building has been rearranged and renovated. The toilets now consist of four separate toilets areas with a single door on the north and south sides of the building and two doors on the east side.

The eastern section of the building is used as a toilet block and has no ceiling linings on the roof framing leaving the trusses exposed. The roof consists of timber trusses to support the roof and to accommodate a skylight. The roof cladding is corrugated steel with corrugated plastic sheeting on the skylight.

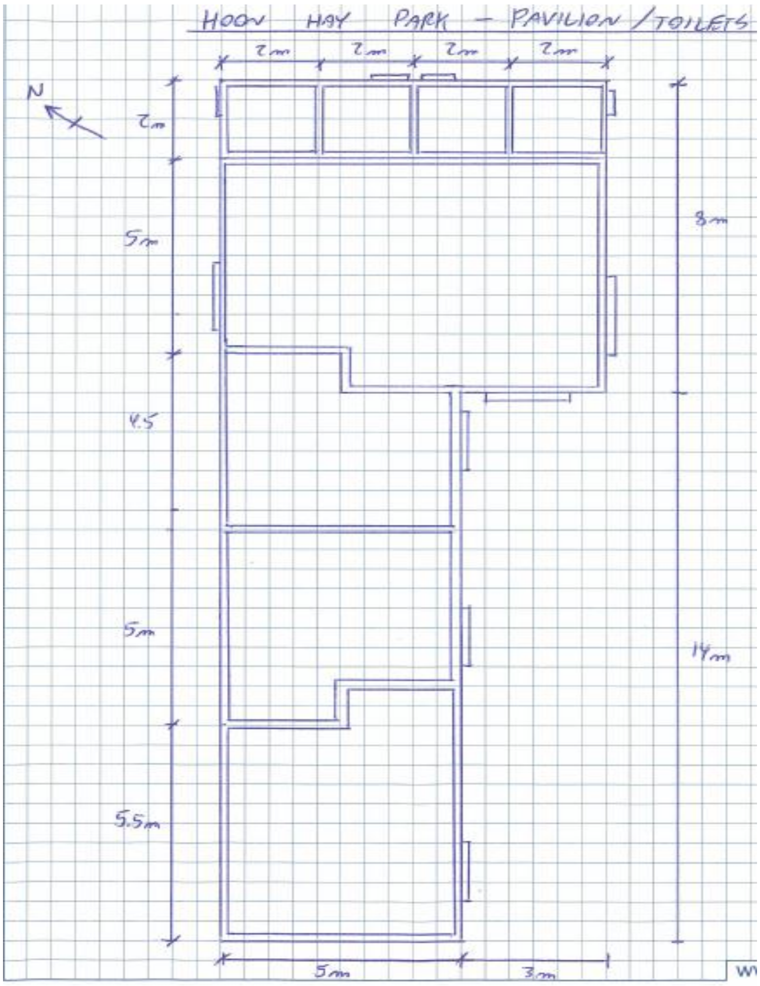


Figure 2 Plan Sketch Showing Key Structural Elements

The interior of the roof inside the western (longer) section of the building was unable to be viewed because of the presence of the fully clad ceilings. It is likely that this roof section is also supported by timber trusses.

The roof structure is supported by load bearing unreinforced concrete masonry walls. The walls appear to be partially grout filled. The connection between roof and walls was obscured and a timber top plate has been assumed to provide support to the roof. The interior of the northern section (toilets) is clad with ceramic tiles and plasterboard. All other walls have no lining. The building sits on a reinforced concrete slab on strip foundations.

The building is 22.80m long and is 4.88m and 7.54m wide at its southern and northern ends respectively. It has a height of 3.4m and a plan area of approximately 130m^2. It is built on a flat site and is relatively isolated with the nearest building being approximately 70m away. There are no waterways near.

Construction plans were made available however these were largely illegible and the layout of the toilets has been changed since the original construction. The results of the Hilti Ferroscans have indicated no reinforcement in any masonry block walls.

4.2 Gravity Load Resisting System

The roof gravity loads in the structure are supported by timber trusses across the structure. The steel roof cladding is supported by timber trusses which rest on the outer masonry walls. The roof loads are transferred from the timber trusses into the side walls. The walls then carry the loads into the concrete slab and footings.

4.3 Lateral Load Resisting System

The unreinforced masonry walls are the primary lateral load resistance system in this structure and serve to carry wall and roof seismic loads through to foundation level. The walls provide this function by in-plane panel action in shear and moment resistance. Upon reaching the foundations these lateral loads are dispersed into the founding soils via bearing and frictional resistance. The masonry walls are propped at the eaves level by the roof structure. The ceiling linings act as a diaphragm resisting lateral loads. The masonry walls are considered to be acting as simply supported walls connected to the foundations. In the absence of propping, there is a nominal level of horizontal spanning capability is present in the masonry, allowing lateral support from adjacent walls.

5. Damage Assessment

5.1 Surrounding Buildings

Some damage to surrounding buildings or structures was observed. There was evidence of significant and widespread liquefaction in the area.

5.2 Residual Displacements and General Observations

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

No damage was evident to the concrete masonry block walls or the ceiling linings.

Minor cracks and movement was visible in the external concrete paved areas, but this damage does not appear to be significant (See photograph 5 & 6).

5.3 Ground Damage

There was some evidence of ground movement and liquefaction in areas of the park and properties adjacent to Hoon Hay Park. The liquefaction on site has been cleared since the significant aftershocks and is only evident in garden areas on the western side of the building.

6. Geotechnical Investigation

6.1 Site Description

The site is situated in the suburb of Hoon Hay, in south western Christchurch. It is relatively flat at approximately 12 m above mean sea level. It is approximately 700 m west of the Heathcote River, and 13 km west of the coast (Pegasus Bay).

6.2 Published Information on Ground Conditions

6.2.1 Published Geology

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

- Dominantly alluvial sand and silt overbank deposits, being alluvial soils of the Yaldhurst Member, sub-group of the Springston Formation, Holocene in age
- Underlying sediments (younger than 6500 years) are surface and subsurface alluvial silt and sand interbedded with gravel
- The Riccarton gravels are located approximately 5 m bgl
- Groundwater is likely within 3 m of ground level

6.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that three boreholes with lithographic logs are located within 200 m of the site. ECan boreholes with appropriate logs are summarised in Table 2.

These indicate the area is underlain by varying strata. Silt and clay deposits were encountered to between 2 m and 5.19 m depth to the north and to 200 m to the south of the site. However gravels were encountered at shallow depth to at least 4.5 m bgl 195 m north of the site. This may indicate presence of intermittent gravel layers in the area.

Groundwater was recorded between 0.8 m and 1.52 m bgl in the borehole logs.

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	From Site	Log Summary
M36/9042	2.06 m	1.52 m	40 m NW	0 to 0.7 m Topsoil 0.7 to 2.06 m Clayey SILT
M36/8417	4.5 m	N/A	195 m N	0 to 0.6 Sandy SILT 0.6 to 3.5 m Sandy fine to coarse GRAVEL 3.5 to 4.5 m Sandy GRAVEL with

[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.

Bore Name	Log Depth	Groundwater	From Site	Log Summary
				cobbles
M36/4740	28.2 m	1.5 m	200 m S	0 to 1 m Topsoil and clay
				1 to 5.19 m Silty CLAY and CLAY with trace vegetation
				5.19 to 8.2 m Sandy GRAVEL
				8.2 to 17.5 m CLAY with trace organic material
				17.5 to 28.2 m Sandy GRAVEL with trace clay

It should be noted that 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.

6.2.3 EQC Geotechnical Investigations

The Earthquake Commission has not undertaken geotechnical testing in the area of the site. However, the site is on the boundary of the Hoon Hay investigation area. Information pertaining to this investigation is included in the Tonkin & Taylor Report for Hoon Hay[2]. One investigation points was undertaken within 200 m of the site, as summarised below in Table 3.

Table 3 EQC Geotechnical Investigation Summary Table

Bore Name	Orientation from Site	Depth (m bgl)	Log Summary[3]
CPT HNH 14	200 m NE	0 – 1.2	Pre-drilled
		1.2 – 2.0	Interbedded Clayey SILT
		2.0 – 2.9	Silty SAND
		2.9 – 4.0	Silty CLAY
		4.0 – 4.8	Silty SAND
		4.8 – 5.6	SAND
		5.6 – 6.0	Sandy SILT
		6.0 – 6.5	SAND
			(WT assumed at 0.1 m bgl)

Initial observations of the CPT results indicate the soils are typically fine grained, and are loose to medium dense. This would infer that liquefaction is possible in a significant seismic event.

6.2.4 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.

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

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

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

The site has been categorised as “N/A” – Urban Non-residential”. However, neighbouring residential properties have been categorised as TC2 (yellow), indicating minor to moderate land damage from liquefaction is possible in future significant earthquakes.

6.2.5 Post Earthquake Land Observations

Aerial photography[4] taken following the 22 February 2011 earthquake shows minor signs of liquefaction outside the building footprint or adjacent to the site, as shown in Figure 3. All other aerial photography following the major earthquakes of the Canterbury earthquake sequence does not cover the location of the site.

Figure 3 Post February 2011 Earthquake Aerial Photography [5]

The Canterbury Geotechnical database shows there are no cracks in the area of the site[6].

6.2.6 Summary of Ground Conditions

These indicate the area is underlain by varying strata. Silt and clay deposits were encountered to between 2 m and 5.19 m depth to the north and to 200 m to the south of the site. Gravels were

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

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

[6] Canterbury Geotechnical Database (2012) "Observed Ground Crack Locations", Map Layer CGD0400 - 23 July 2012, retrieved [13/12/2012] from https://canterburygeotechnicaldatabase.projectorbit.com/

encountered at shallow depth to at least 4.5 m bgl 195 m north of the site. This may indicate presence of intermittent gravel layers in the area.

Groundwater is considered to vary between 0.8 m and 1.52 m bgl.

6.3 Seismicity

6.3.1 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[7,8]

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)	18 km	W	7.1	~15,000 years
Hope Fault	110 km	N	7.2~7.5	120~200 years
Kelly Fault	110 km	NW	7.2	~150 years
Porters Pass Fault	60 km	NW	7.0	~1100 years
Port Hills Fault (2011)	*6 km*	W	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.

6.3.2 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[9] indicate the PGA to be 0.25 g during the 4 September 2010 earthquake, 0.41 g on 22 February 2011, and 0.17 g on 13 June 2011.

[7] 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.

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

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

6.4 Slope Failure and/or Rockfall Potential

Given the site's location in Hoon Hay, global slope instability is considered negligible. However, any localised retaining structures or embankments should be further investigated to determine the site-specific slope instability potential.

6.5 Liquefaction Potential

The site is considered to be minor to moderately susceptible to liquefaction, due to the following reasons:

- Observations of sand boils in the post-earthquake aerial photography
- TC2 land surrounding the site
- Anticipated saturated sands and silts beneath the site

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

6.6 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 varying strata of silt and clay deposits and gravels at shallow depth, indicating that there may be intermittent gravel layers in the area.

Associated with this the site also has a minor to moderate 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. Scope and Limitations

6.7 Scope and Limits of this Assessment

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

7. Assessment

An inspection of the building was undertaken on the 16th July 2012. Further inspections of the building were carried out on 28th June 2012 and 21st November 2012. No placard sign was evident during the inspection, however based on the inspection carried out it would be expected to have a green placard. Both the interior and exterior of the building were inspected. Most of the main structural components of the building were able to be viewed due to the exposed simple construction of the building.

Electro-magnetic scanning of the concrete masonry blocks was undertaken and no reinforcement was located in the block walls. Construction plans were made available however these were largely illegible.

The inspection also 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.

7.1 Quantitative Assessment

The quantitative assessment of the building includes the investigation of in-plane and out-of-plane strength of the unreinforced masonry block walls. The investigation was based on the analysis of the seismic loads that the structure is subjected to, distribution of these forces throughout the structure and the analysis of the capacity of existing structural elements to resist the forces applied. A Hilti PS 200 Ferroscan was used to check for reinforcement in the walls. The capacity of the existing structural elements was compared to the demand placed on the elements to give the %NBS of each of the structural elements. A full methodology of the calculation process is attached in Appendix D.

7.2 Seismic Coefficient

The elastic site hazard spectrum for horizontal loading, C(T), for the building was derived from Equation 3.1(1) of NZS 1170:2004

$$C(T) = C_h\ Z\ R\ N(T.D)$$

Where

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

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

R = 1.0, the return period factor from Table 3.5 for an annual probability of exceedance of 1/500 for an Importance Level 2 building

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

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

$$S_P = 1.3 - 0.3\mu$$

Where μ is the structural ductility factor. A structural ductility factor of 1.25 has been taken for lateral loading across and along the building; this is due to the walls being constructed of reinforced, filled concrete blocks.

For T1 < 0.7s and soil class D, the seismic weight coefficient was determined in accordance with Cl 5.2.1.1 of NZS 1170.5: 2011. For the purposes of calculating the seismic weight coefficient a period, T1, of 0.4 was assumed for the in-plane masonry walls. The coefficient was then calculated using Equation 5.2(1);

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

Where

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

7.2.1 Shear Capacity of the Unreinforced Filled Masonry Walls

The shear capacity of the unreinforced filled concrete masonry walls 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. The strength reduction factor, ϕ, for shear and shear friction was taken as 0.75 in accordance with Cl 3.4.7. The overall shear capacity of the wall was calculated from Cl 10.3.2.1, Equation 10-4;

$$V_n = v_n b_W d \phi$$

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 (As no steel was present this was taken as 0).

b_w = the thickness of the wall

d = 0.8 times the length of the wall

7.2.2 In-Plane Capacity of the Unreinforced Unfilled Walls

The in-plane capacity of the unreinforced concrete masonry wall was determined using the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance (06/2006). The NZSEE guidelines recommend checks for 4 different in-plane response modes.

- Diagonal tension failure mode
- Bed-sliding failure mode
- Toe crushing failure mode
- Rocking failure mode

An analysis of each wall was carried out using the methods set out in Section 8 – In-Plane Wall Response, of the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance (06/2006).

7.2.3 In-plane Wall Shear Capacity of the Unreinforced Walls

The in-plane nominal shear capacity of a wall, pier or spandrel was taken as the minimum of the nominal capacity in the diagonal tension failure mode, V_{dt}, the rocking failure mode, V_r, the bed-joint sliding failure mode, V_s, and the toe crushing failure mode, V_{tc}.

$$\mathbf{V_n = min(V_{dt}, V_s, V_r, V_{tc})}$$

7.2.4 Out-of-Plane Capacity of the Unreinforced Walls

The % NBS for out-of-plane flexure of the filled and unfilled concrete masonry walls was determined using the methods set out in Section 10.3 of the NZSEE guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes.

7.3 Calculation of %NBS

The shear and moment capacity of the concrete masonry walls, the axial, bending and shear capacity of the concrete masonry as well as the bracing capacity of the walls both in the along and across directions were then compared to their respective demands to assess which were the most critical and thus determine the overall %NBS for the building.

8. Initial Capacity Assessment

8.1 % NBS Assessment of Walls

A summary of the results of the calculations detailed in this section of the report can be seen below in Tables 5 & 6. The %NBS for each unreinforced masonry wall was calculated by comparing the demand placed upon the individual element with the capacity of that element as derived from the current design code.

The position of each wall is indicated on Figure four below and each wall is numbered accordingly.

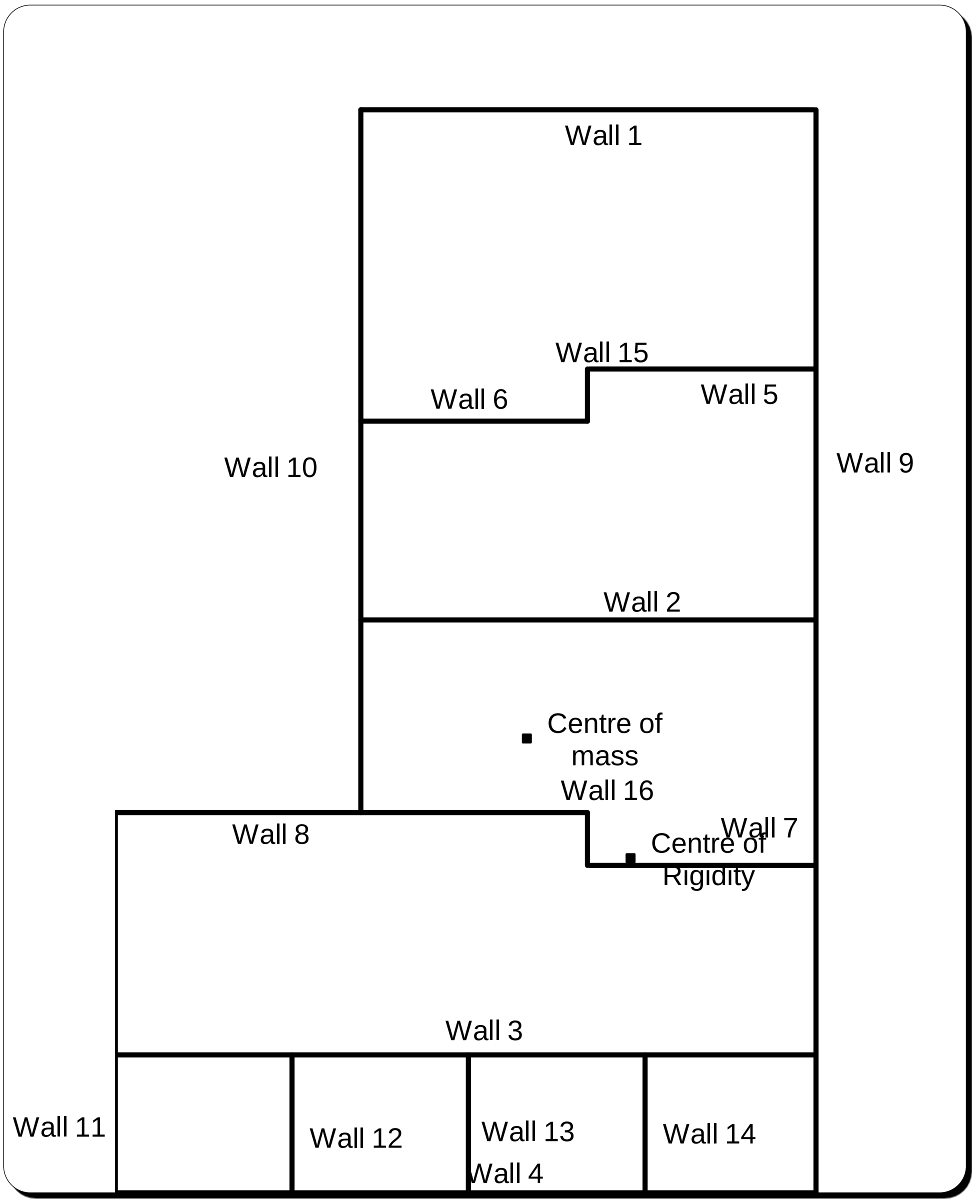


Figure 4 Plan Centres of Mass and Rigidity and Wall Locations

8.2 Hoon Hay Park Toilet/Pavilion Analysis Results

The results of the in plane analysis and subsequent earthquake designation under the NZSEE guidelines are listed below in Table 5.

Wall number	ϕV_n kN	V* kN	%NBS	Earthquake Status	ϕM_n kNm	M* kNm	%NBS	Earthquake Status
1	34.7	184.2	19%	**Prone**	110.5	478.8	23%	**Prone**
2	34.7	122.1	28%	**Prone**	110.5	317.5	35%	**Risk**
3	53.9	224.9	24%	**Prone**	263.7	584.8	45%	**Risk**
4	53.9	226.8	24%	**Prone**	263.7	589.8	45%	**Risk**
5	8.8	38.2	23%	**Prone**	27.6	99.2	28%	**Prone**
6	8.8	36.6	24%	**Prone**	27.6	95.1	29%	**Prone**
7	8.8	23.3	38%	**Risk**	27.6	60.5	46%	**Risk**
8	36.2	106.9	34%	**Risk**	119.7	277.9	43%	**Risk**
9	167.6	553.9	30%	**Prone**	2411.3	1440.2	>100%	**Not at Risk**
10	107.2	308.2	35%	**Risk**	1014.6	801.2	>100%	**Not at Risk**
11	57.3	116.0	49%	**Risk**	296.9	301.5	98%	**Not at Risk**
12	12.5	12.8	98%	**Not at Risk**	39.0	33.3	>100%	**Not at Risk**
13	12.5	10.4	>100%	**Not at Risk**	39.0	26.9	>100%	**Not at Risk**
14	12.5	8.2	>100%	**Not at Risk**	39.0	21.3	>100%	**Not at Risk**
15	1.8	1.3	>100%	**Not at Risk**	5.6	3.3	>100%	**Not at Risk**
16	1.8	1.3	>100%	**Not at Risk**	5.7	3.3	>100%	**Not at Risk**

Table 5 In Plane Analysis Results for URM Walls

The results of the out of plane displacement response capability analysis and subsequent earthquake designation under the NZSEE guidelines are listed in Table 6

Wall number	D_{ph} kN	Δ_m kN	%NBS	Earthquake Status
1	0.268	0.055	25%	**Prone**
2	0.268	0.055	25%	**Prone**
3	0.268	0.055	25%	**Prone**
4	0.268	0.055	25%	**Prone**
5	0.268	0.055	25%	**Prone**
6	0.268	0.055	25%	**Prone**
7	0.268	0.055	25%	**Prone**
8	0.268	0.055	25%	**Prone**
9	0.268	0.055	25%	**Prone**
10	0.268	0.055	25%	**Prone**
11	0.268	0.055	25%	**Prone**
12	0.268	0.055	25%	**Prone**

13	0.255	0.056	26%	**Prone**
14	0.255	0.056	26%	**Prone**
15	0.255	0.056	26%	**Prone**
16	0.255	0.056	26%	**Prone**

Table 6 Out Of Plane Analysis Results for URM Walls

8.3 Discussion of Results

The loading standards following the Christchurch earthquakes have been modified with increased seismic requirements. The additional requirements 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.

Following a detailed assessment, the toilet/pavilion block has been assessed as achieving 19 **%**NBS for the unreinforced masonry walls. Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines Hoon Hay Park Toilet/Pavilion is considered to be Earthquake Prone. Although masonry walls are unreinforced, there are no other critical structural weakness or potential collapse hazards identified in the building.

9. Recommendations

The recent seismic activity in Christchurch has not caused any visible damage to the building. However the building has achieved approximately 19% NBS following a Quantitative Detailed Engineering Evaluation. Further assessment is not required. GHD recommends significant wall strengthening or wall replacement options be explored and implemented to bring the %NBS of the building up to a minimum of 67% NBS in accordance the NZSEE guidelines.

10. Limitations

10.1 General

This report has been prepared subject to the following limitations:

- Drawings of the building were unavailable. As a result the information contained in this report has been inferred from visual inspections of the building and site only.
- No intrusive structural investigations have been undertaken. Electro-magnetic scanning of the walls was conducted to determine the levels of steel reinforcement present.
- No level or verticality surveys have been undertaken.
- No material testing has been undertaken.
- No calculations, other than those detailed in Section 8 have been carried out on the structure.

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.

10.2 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 South elevation.

Photograph 2 View of the toilet block end from the northeast.

Photograph 3 View of the pavilion section from the northwest.

Photograph 4 Tile wall lining and roof trusses in toilet areas.

Photograph 5 Cracks in paved area.

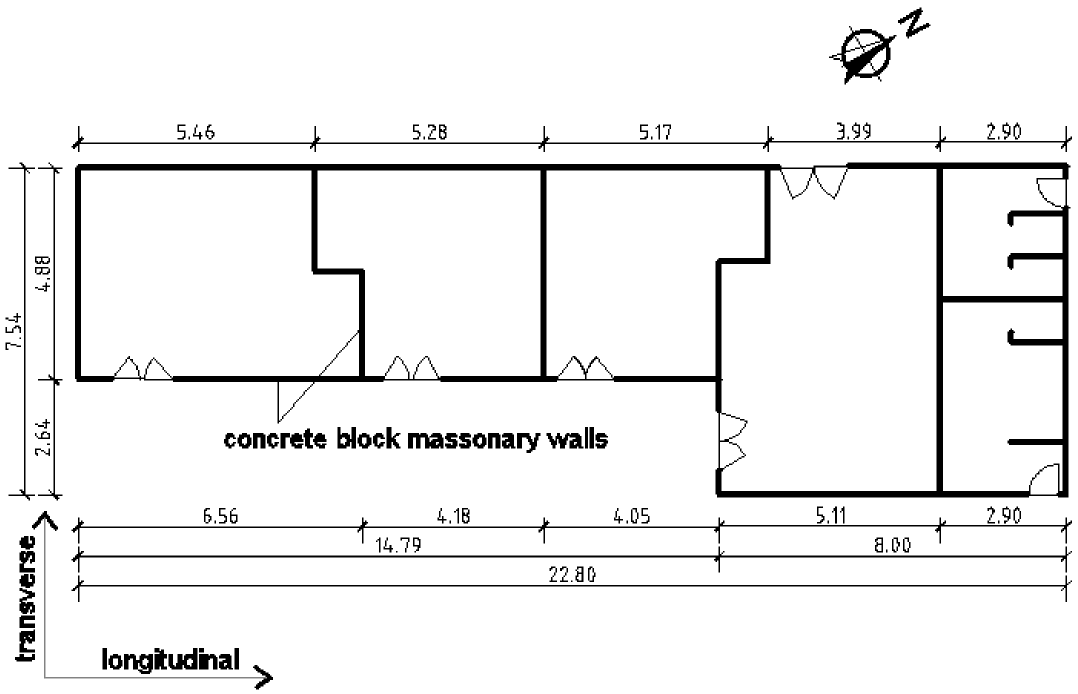
Photograph 6 Settlement at Southeast corner of paved area.

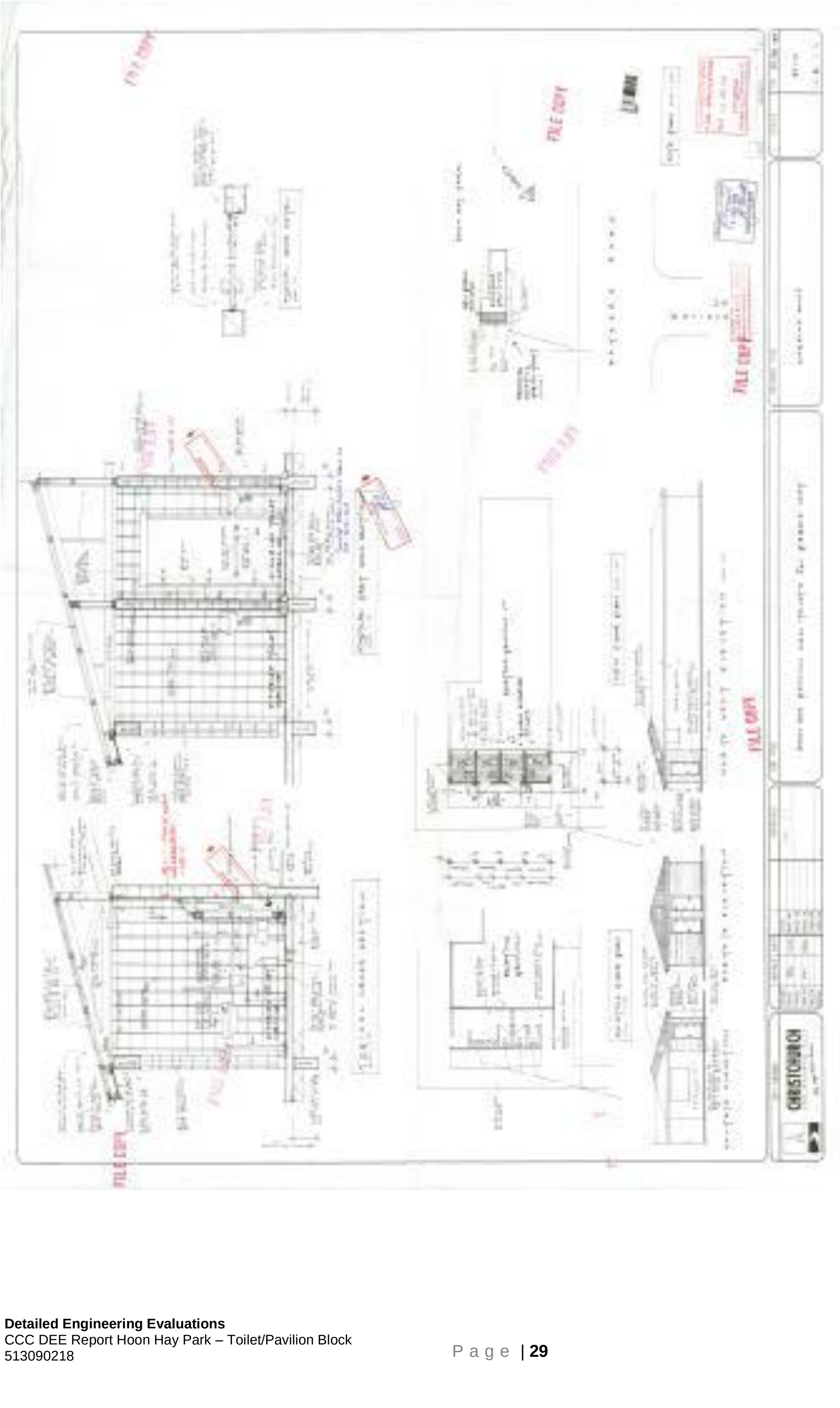
Photograph 7 Fully lined ceiling in Pavilion areas of structure.

Photograph 8 One area of the Eastern wall that was scanned for reinforcement.

Appendix B

Existing Drawings





Appendix C

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data

V1.11

Location

Building Name:	Pavilion/Toilets - Hoon Hay Park			Reviewer:	David Lee
	Unit	No:	Street	CPEng No:	112052
Building Address:	Hoon Hay Park	61	Mathers Road	Company:	GHD
Legal Description:	PRK_1492_BLDG_001 EQ2			Company project number:	51/30902/18
				Company phone number:	04 472 0799
	Degrees	Min	Sec		
GPS south:				Date of submission:	15/02/2013
GPS east:				Inspection Date:	21/11/2012
				Revision:	
Building Unique Identifier (CCC):	PRK_1492_BLDG_001 EQ2			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	mixed	Soil Profile (if available):	
Site Class (to NZS1170.5):	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):	
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	
Building height (m):	3.40		height from ground to level of uppermost seismic mass (for IEP only) (m):	3.4
Floor footprint area (approx):	130			
Age of Building (years):	46		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):	Park Toilet & Pavilion			
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber truss	truss depth, purlin type and cladding	
Floors:	concrete flat slab	slab thickness (mm)	
Beams:	none	overall depth x width (mm x mm)	None
Columns:			None
Walls:	partially filled concrete masonry	thickness (mm)	

Lateral load resisting structure

Lateral system along:	unreinforced masonry bearing wall - brick	**Note: Define along and across in detailed report!**		190 Series Block
Ductility assumed, μ:	1.25		note wall thickness and cavity	
Period along:	0.40	0.40 from parameters in sheet	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	unreinforced masonry bearing wall - brick			190 Series Block
Ductility assumed, μ:	1.25		note wall thickness and cavity	
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:

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

Non-structural elements

Stairs:			No stairs
Wall cladding:	other light	describe	Toilet walls have ceramic tiles
Roof Cladding:	Metal	describe	Corrugated Iron
Glazing:	timber frames		
Ceilings:	plaster, fixed		Some walls
Services(list):	Water, Sewage, Power		

Available documentation

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

Damage

Site:

(refer DEE Table 4-2)

Site performance:	Good	Describe damage:	None Apparent
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:			
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):			
Across	Damage ratio:	0%		
	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:	Strengthening to 67% NBS
	Building Consent required:	yes	Describe:	
	Interim occupancy recommendations:		Describe:	
Along	Assessed %NBS before e'quakes:	19% ##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Quantitative Assessment
	Assessed %NBS after e'quakes:	19%		
Across	Assessed %NBS before e'quakes:	19% ##### %NBS from IEP below		
	Assessed %NBS after e'quakes:	19%		

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

Period of design of building (from above): 1965-1976

h_n from above: 3.4m

Seismic Zone, if designed between 1965 and 1992: B

not required for this age of building

not required for this age of building

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

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

Note 2: for RC buildings designed between 1976-1984, use 1.2

Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)

	along	across
Final (%NBS)$_{nom}$:	0%	0%

2.2 Near Fault Scaling Factor

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

	along	across
Near Fault scaling factor (1/N(T,D), **Factor A**:	#DIV/0!	#DIV/0!

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:

Z_{1992}, from NZS4203:1992

Hazard scaling factor, **Factor B**: #DIV/0!

		along	across
2.4 Return Period Scaling Factor	Building Importance level (from above):		
	Return Period Scaling factor from Table 3.1, **Factor C**:		
2.5 Ductility Scaling Factor	Assessed ductility (less than max in Table 3.2)		
	Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:		
	Ductiity Scaling Factor, **Factor D**:	0.00	0.00
2.6 Structural Performance Scaling Factor:	Sp:		
	Structural Performance Scaling Factor **Factor E**:	#DIV/0!	#DIV/0!
2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom} \times A \times B \times C \times D \times E$	**$\%NBS_b$:**	#DIV/0!	#DIV/0!

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

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

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

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

		Along	Across
3.6. Other factors, Factor F	For ≤ 3 storeys, max value =2.5, otherwise max value =1.5, no minimum		
	Rationale for choice of F factor, if not 1		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

3.7. Overall Performance Achievement ratio (PAR)		0.00	0.00
4.3 PAR x (%NBS)b:	PAR x Baseline %NBS:	#DIV/0!	#DIV/0!
4.4 Percentage New Building Standard (%NBS), (before)			#DIV/0!

Appendix D

Assessment Methodology

A. Seismic Coefficient

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

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

Where

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

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

R = 1.0, the return period factor from Table 3.5 for an annual probability of exceedance of 1/500 for an Importance Level 2 building

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

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

$$\mathrm{S_P = 1.30 - 0.3\mu}$$

A structural ductility factor μ of 1.25 has been taken for lateral loading both across and along the building.

For T_1 < 0.7s and soil class D, the seismic weight coefficient was determined in accordance with Cl 5.2.1.1 of NZS 1170.5: 2011. For the purposes of calculating the seismic weight coefficient a period, T_1, of 0.4 was assumed for the in-plane masonry walls. The coefficient was then calculated using Equation 5.2(1);

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

Where

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

Horizontal Design Actions on Parts

Horizontal Design Actions on Parts $F_{ph,}$ was determined using Equation 8.5(1):

$$F_{ph} = C_p(T_p)C_{ph}R_pW_p \leq 3.6W_p$$

Where

$C_p(T_p)$ = the horizontal design coefficient of the part, determined from Clause 8.2

$C_p(T_p) = C(0)C_{hi}C_i(T_p)$

Where

$C(0) = C_h(T)ZRN(T.D)$

$C_h(T)$ =1.12 the spectral shape factor determined from CL 3.1.2

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

R = 1.0, the return period factor from Table 3.5 for an annual probability of exceedance of 1/500 for an Importance Level 2 building

N(T,D) = 1.0 the near-fault scaling factor from CL 3.1.6

C_{hi} = (1 + h_i/6) from Equation 8.3(1) CL8.3 the floor acceleration coefficient at level i

Where

h_i = height of the attachment of the part

h_n = height from the base of the structure to the uppermost seismic weight or mass

$C_i(T_p)$ = 2.0 for $T_p \leq 0.75$s

C_{ph} = the part horizontal response factor determined from Clause 8.6

R_p = the part risk factor as given by Table 8.1

W_p = the weight of the part

The mean value of the seismic coefficient was calculated using horizontal coefficient for the top and bottom of the wall.

B. Shear Capacity of the Unreinforced filled Masonry Walls

The shear capacity of the unreinforced filled 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. The strength reduction factor, ϕ, for shear and shear friction was taken as 0.75 in accordance with Cl 3.4.7. The overall shear capacity of the wall was calculated from Cl 10.3.2.1, Equation 10-4;

$$V_n = v_n b_W d \phi$$

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. (As no steel was present this was taken as 0).

b_w = the thickness of the wall

d = 0.8 times the length of the wall

C. In-Plane Capacity of the Unreinforced Walls

The in-plane capacity of the unreinforced concrete masonry wall was determined using the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance (06/2006). The NZSEE guidelines recommend checks for 4 different in-plane response modes.

- Diagonal tension failure mode
- Bed-sliding failure mode
- Toe crushing failure mode
- Rocking failure mode

An analysis of each wall was carried out using the methods set out in Section 8 – In-Plane Wall Response, of the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance (06/2006).

D. In-plane Wall Properties of the Unreinforced Walls

Properties of in-plane loaded URM walls, piers or spandrels for use in the calculation of nominal in-plane shear capacity were as follows:

- **Unit Weight of Masonry**

2.10 kN/m^2 was adapted for the unit weight of 20-series concrete hollow block masonry with standard aggregate (see Table A2 from NZS 1170.1:2002).

- **Weight of Wall**

The weight of the wall, W_w, was calculated in accordance with the equation.

$$W_w = 1.82 \times l_w \times h$$

Where: l_w is the total wall length and h is the wall height.

- **Normal Force at Base of Wall**

The normal force acting on the cross section of the base of the wall, N_b, was calculated in accordance with the equation.

$$\mathrm{N_b = W_w + N_t}$$

Where: Values for weight of the wall, W_w, and axial load above the wall, N_t.

- **Diagonal Tension Strength**

The diagonal tension strength of masonry, f_{dt}, was calculated in accordance with the equation below for walls, piers and spandrels.

$$\mathrm{f_{dt}} = \frac{1}{2}\left(\mathrm{c} + \frac{\mathrm{N_t}}{\mathrm{A_w}} 0.8\mu_\mathrm{f}\right)$$

Where: Values for cohesion, c, and coefficient of friction, μ_f, were given in Section 2.5.5 of NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance. The factor of 0.8 is to account for vertical accelerations and other dynamic effects.

- **Distance to Centre of Inertia of Wall**

Distance to the centre of inertia of the wall from the compression toe, a_i, was calculated in accordance with the equation for walls with no flanges:

$$\mathrm{a_i = 0.5 \times l_w}$$

- **Average Compressive Stress**

Average compressive stress acting on the wall, σ_{ave}, was calculated in accordance with the equation

$$\sigma_{\mathrm{avg}} = \frac{\mathrm{N_t}}{\mathrm{l_w . b_w}}$$

Where: Value for width of the block shell, b_w which was equivalent to 0.45 of the block width. This reduced value of b_w was calculated by multiplying the actual width by a modification factor based on the difference between the unit density of the block compared to the unit density of concrete.

$$\mathrm{b_w} = \text{ width x } \frac{\rho_{block}}{\rho_{concrete}}$$

E. In-plane Wall Shear Capacity of the Unreinforced Walls

The in-plane nominal shear capacity of a wall, pier or spandrel was taken as the minimum of the nominal capacity in the diagonal tension failure mode, V_{dt}, the rocking failure mode, V_r, the bed-joint sliding failure mode, V_s, and the toe crushing failure mode, V_{tc}.

$$\mathrm{V_n} = \min(\mathrm{V_{dt}}, \mathrm{V_s}, \mathrm{V_r}, \mathrm{V_{tc}})$$

Nominal capacity of each failure mode was derived as following:

- **Capacity in Diagonal Tension Failure Mode, V_{dt}**

Nominal shear capacity corresponding to diagonal tension failure, V_{dt}, was calculated in accordance with the equation below for walls where no perpendicular flanges are present

$$V_{dt} = 0.54.b_w.l_w.\zeta.f_{dt}.\sqrt{\left(1+\frac{\sigma_{avg}}{f_{dt}}\right)}$$

Where: ζ was a factor to correct for nonlinear stress distribution (See Table)

	ζ
Slender walls, where $h/l_w > 2$	1.5
Stout walls, where $h/l_w < 0.5$	1.0
Linear interpolation may be used for values of h/l_w	

Table 3 Shear stress factor for inclusion in diagonal tension failure mode equation

- **Capacity in Rocking Failure Mode, V_r**

Nominal shear capacity corresponding to the rocking failure mode, V_r, was calculated in accordance with the equation;

$$V_r = \frac{N_b}{h}.\left[a_i - \frac{l_{er}}{3}\right]$$

Where: l_{er} was the effective length of the wall in rocking, taken as 0.1 x l_w.

- **Capacity in Bed-joint Sliding Failure Mode, V_s**

Bed-joint sliding failure was not an expected behaviour of URM walls subjected to seismic loading. The bed-joint sliding capacity of an in-plane loaded wall needed only be assessed when conditions suited the initiation of bed-joint sliding, specifically, when either or both the brick compressive strength and mortar compressive strength fell in the bounds of “soft”.

Ultimate shear capacity corresponding to bed-joint sliding failure, V_s, was calculated in accordance with the equation

$$V_s = l_w . b_w . c + 0.8 . \mu_f . N_t$$

Where: Values for cohesion, c, and coefficient of friction, μ_f, were given in Section 2.5.5 of NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance. The factor of 0.8 is to account for vertical accelerations and other dynamic effects.

- **Capacity in Toe Crushing Failure Mode, V_{tc}**

Nominal shear capacity corresponding to toe crushing failure, V_{tc}, was calculated in accordance with the below equation for walls where perpendicular flanges were present:

$$V_{tc} = \frac{N_b}{h} . \left[\frac{1}{2} . l_w - \frac{1}{3} . l_{etc} \right]$$

Where the effective length of wall was calculated as:

$$l_{etc} = \frac{2 . N_b}{1.3 . f'_m . b_w}$$

F. Out-of-Plane Capacity of the Unreinforced Walls

The % NBS for out-of-plane flexure of the concrete masonry walls was determined using the methods set out in NZSEE guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes Section 10.3. The following steps were those required to assess the displacement response capability and the displacement demand, from which the adequacy of the walls can be determined.

The wall panel was assumed to form hinge lines at the points where effective horizontal restraint was assumed to be applied. The centre of compression on each of these hinge lines was assumed to form a pivot point. The height between these pivot points was the effective panel height h. At mid-height between these pivots, a third pivot point is assumed to form.

Step 1

The wall panel was divided into two parts, a top part bounded by the upper pivot and the mid-height between the top and bottom pivots, and a bottom part bounded by the mid-height pivot and the bottom pivot.

Step 2

The weight of the wall parts, W_b of the bottom part and W_t of the top part, and the weight acting at the top of the storey, P were calculated.

Step 3

From the nominal thickness of the wall, t_{nom}, the effective thickness, t was calculated as follows:

$$t = t_{nom} \left(0.975 - 0.025 \frac{P}{W} \right)$$

Step 4

The eccentricity values e_p, e_b, e_t and e_o were calculated . Usually, the eccentricities e_b and e_p will each vary between 0 and *t/2* (where *t* is the effective thickness of the wall). Exceptionally they may be negative.

Where,

e_p = eccentricity of the P measured from the centroid of W_t

e_t = eccentricity of the mid-height pivot measure from the centroid of W_t

e_b = eccentricity of the pivot at the bottom of the panel measured from the centroid of W_b

e_o = eccentricity of the mid-height pivot measured from the centroid of W_b

Step 5

The mid-height deflection, Δ_i was calculated, which would cause instability under static conditions. The following formula was used to calculate this deflection.

$$\Delta_i = \frac{b\mathrm{h}}{2a}$$

Where

$$b = W_b e_b + W_t(e_0 + e_b + e_t) + P\left(e_0 + e_b + e_t + e_p\right) - \Psi(W_b y_b + W_t y_t)$$

And

$$a = W_b y_b + W_t\left(\frac{h}{2} + y_t\right) + Ph$$

And

$$\Psi = Initial\ slope\ of\ wall$$

Step 6

The maximum usable deflection, Δ_m was calculated as 0.6 Δ_i.

Step 7

The period of the wall, T_p, was four times the duration for the wall to return from a displaced position measured by Δ_m to the vertical. The period was calculated from the following equation:

$$T_p = 6.27\sqrt{\frac{J}{a}}$$

Where J was the rotational inertia of the masses associated with W_b, W_t and *P* and any ancillary masses, and was given by the following equation.

$$J = J_{bo} + J_{to} + \frac{1}{g}\left\{W_b[{e_b}^2 + {y_b}^2] + W_t[(e_0 + e_b + e_t)^2 + {y_t}^2] + P\left[\left(e_0 + e_b + e_t + e_p\right)^2\right]\right\} + J_{ancillary}$$

Where;

$$J_{bo} = J_{to} = \frac{\left\{\left(\frac{W}{h}\right)[h^2 + 16t^2] + 4Pt^2\right\}}{g}$$

Where y_t was the distant from the top of the wall to the centroid of the top wall and y_b was the distant from the bottom of the wall to the centroid of the bottom wall.

Step 8

The seismic coefficient ($C_p(T_p)$) for an elastically responding part (μ_p = 1) with this period (T_p), was calculated as follows:

$$C_p(T_p) = C(0)C_{Hi}C_i(T_p)$$

Where

C(0) = the site hazard coefficient for T = 0 determined from NZS 1170.5 Section 3.1, using the values for the modal response spectrum method and numerical integration time history methods

C_{Hi} = the floor height coefficient for level I, from NZS 1170.5 Section 8.3.

$C_i(T_p)$ = the part spectral shape factor at level I, from NZS 1170.5 Section 8.4

Step 9

The participation factor, γ for the rocking system was taken as:

$$\gamma = \frac{(W_b y_b + W_t y_t)h}{2Jg}$$

Step 10

From $C_p(T_p)$, T_p, R_p *and* γ, the displacement response, D_{ph} was obtained from;

$$D_{ph} = \gamma\left(\frac{T_p}{2\pi}\right)^2 \times C_p(T_p) \times R_p \times g$$

Where R_p was from NZS 1170.5 Table 8.1

G. Calculation of %NBS

The capacities from the concrete masonry walls were compared to their respective demands to assess which were the most critical and thus determine the overall %NBS for the building.

Appendix E

Geotech Investigation

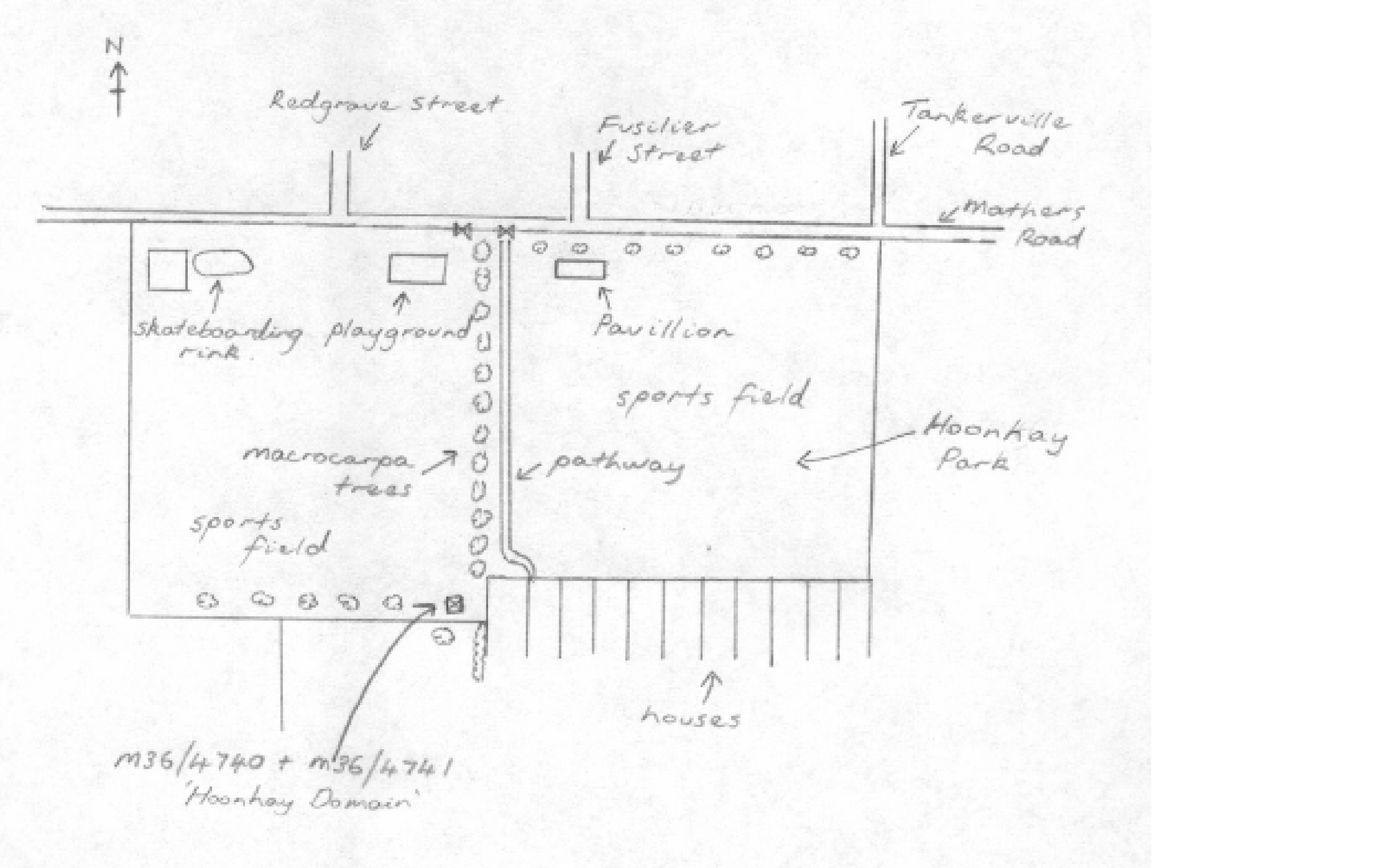
Well Cluster No: M36/4740
Well Name: HOONHAY DOMAIN
Owner: CANTERBURY REGIONAL COUNCIL

Environment Canterbury
Your regional council

Street of Well:	MATHERS ROAD	**File No:**	CO6C/04607
Locality:	HOON HAY	**Allocation Zone:**	Christchurch/West Melton
NZGM Grid Reference:	M36:77245-38120 QAR 2		
NZGM X-Y:	2477245 - 5738120		
Location Description:	UNDER RECORDER BOX IN THE DOMAIN	**Uses:**	Water Level Observation
ECan Monitoring:	ECan Recorder Network		
Well Status:	Active (exist, present)		

Drill Date:	01 Jun 1994	**Water Level Count:**	899
Well Depth:	27.20m -GL	**Strata Layers:**	8
Initial Water Depth:	1.50m -MP	**Aquifer Tests:**	0
Diameter:	38mm	**Isotope Data:**	1
		Yield/Drawdown Tests:	0
Measuring Point Ait:	13.28m MSD QAR 1	**Highest GW Level:**	0.59m below MP
GL Around Well:	-2.32m -MP	**Lowest GW Level:**	1.81m below MP
MP Description:	Top of PVC Pipe	**First Reading:**	10 Apr 1995
		Last Reading:	30 Mar 2012
Driller:	Borewell Drilling	**Calc. Min. GWL:**	0.68m -MP
Drilling Method:	Cable Tool	**Last Updated:**	07 Apr 2011
Casing Material:		**Last Field Check:**	30 Mar 2012
Pump Type:	None Installed		
Yield:		**Screens:**	
Drawdown:		**Screen Type:**	Slotted PVC
Specific Capacity:		**Top GL:**	24.00m
		Bottom GL:	24.50m
Aquifer Type:	Flowing Artesian		
Aquifer Name:	Riccarton Gravel		

Date	Comments
	1 OF 2 PIEZOMETERS FOR WATER LEVEL OBSERVATIONS see M36/4741 see also filetrack MO5C-491
12 May 2006	MP re-levelled May 2006 ref.LB395/63 (old level 11.05)
13 Apr 2010	Measuring point changed from 10.97 to 13.285 (increased by 2.315m), water levels all updated accordingly
15 Apr 2010	All tideda water levels changed to new measuring point 15 April 2010.
29 Oct 2010	Well depth checked following a 7.1 magnitude earthquake on 4 September 2010. Measured at 27.0 metres on 22 October 2010.
13 Dec 2010	10-12-2010 Well flushed and test pumped to ensure good connection with the aquifer.
07 Apr 2011	Well flushed and test pumped on 7th April to remediate any effects of the earthquake on 22 February 2011.Total depth measured at 27.2m.


N
Redgrove Street
Fusilier Street
Tankerville Road
Mathers Road
skateboarding rink.
playground
Pavillion
sports field
Hoonkay Park
macrocarpa trees
pathway
sports field
houses
M36/4740 + M36/4741
'Hoonkay Domain'

Borelog for well M36/4740

Gridref: M36:77245-38120 Accuracy : 2 (1=high, 5=low)
Ground Level Altitude : 10.97 +MSD
Driller : Borewell Drilling
Drill Method : Cable Tool
Drill Depth : -28.2m Drill Date : 1/06/1994

Environment Canterbury

Water Level: Artesian

Depth(m)	Full Drillers Description	Formation Code
-1.00m	Topsoil and clay	sp
-1.50m	Silty clay	sp
-5.19m	Blue Grey clay with traces of vegetation	sp
-7.80m	Sandy Brown stained Grey gravels	sp
-8.19m	Clean Grey gravels with <1% sand	sp
- 17.5m	Blue Grey clay with fragments of wood and roots	sp
- 25.5m	Sandy Grey Brown stained gravel with traces of clay in places	ri
- 28.2m	Brown stained claybound sandy gravel with thin bands of clay	ri

Scale(m): -5, -10, -15, -20, -25

Unknown No: M36/9042
Well Name: CCC BorelogID 2356
Owner: CCC borelog

Environment Canterbury
Your regional council

Street of Well:	Mathers Rd /	**File No:**	
Locality:		**Allocation Zone:**	Christchurch/West Melton
NZGM Grid Reference:	M36:77147-38336 QAR 3		
NZGM X-Y:	2477147 - 5738336		
Location Description:	Mathers Rd / Fusilier St - at M.H	**Uses:**	Foundation/Investigation Bore
ECan Monitoring:			
Well Status:	Filled in		

Drill Date:	01 Jan 1961	**Water Level Count:**	0
Well Depth:	2.06m -GL	**Strata Layers:**	2
Initial Water Depth:	-1.52m -MP	**Aquifer Tests:**	0
Diameter:		**Isotope Data:**	0
		Yield/Drawdown Tests:	0
Measuring Point Ait:	10.90m MSD QAR 3	**Highest GW Level:**	
GL Around Well:	0.00m -MP	**Lowest GW Level:**	
MP Description:	ToC	**First Reading:**	
		Last Reading:	
Driller:		**Calc. Min. GWL:**	
Drilling Method:		**Last Updated:**	27 Mar 2008
Casing Material:		**Last Field Check:**	
Pump Type:			
Yield:		**Screens:**	
Drawdown:		**Screen Type:**	
Specific Capacity:		**Top GL:**	
		Bottom GL:	
Aquifer Type:			
Aquifer Name:			

Borelog for well M36/9042

Gridref: M36:77147-38336 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 10.9 +MSD
Well name : CCC BorelogID 2356
Drill Method : Not Recorded
Drill Depth : -2.06m Drill Date : 1/01/1961

Scale(m)	Water Level	Depth(m)	Full Drillers Description	Formation Code
			topsoil with some silt	
		-0.69m	blue grey clayey silt	
		-2.06m		

Scale: -0.2, -0.4, -0.6, -0.8, -1, -1.2, -1.4, -1.6, -1.8, -2

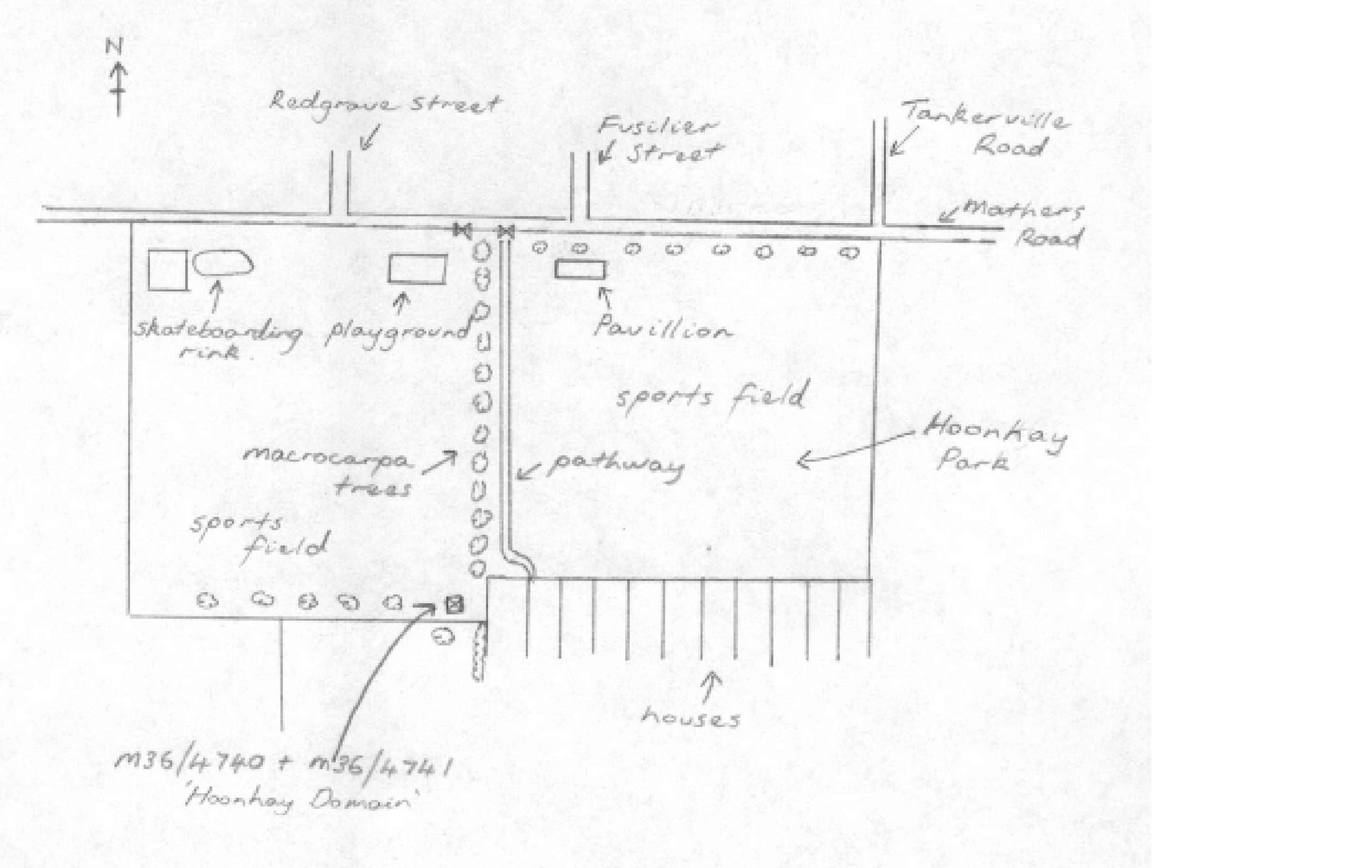
Well Cluster No: M36/4741
Well Name: HOONHAY DOMAIN
Owner: CANTERBURY REGIONAL COUNCIL

Environment Canterbury
Your regional council

Street of Well:	MATHERS ROAD	**File No:**	CO6C/04607
Locality:	HOON HAY	**Allocation Zone:**	Christchurch/West Melton
NZGM Grid Reference:	M36:77245-38120 QAR 2		
NZGM X-Y:	2477245 - 5738120		
Location Description:	UNDER RECORDER BOX IN THE DOMAIN	**Uses:**	Water Level Observation
ECan Monitoring:	ECan Recorder Network		
Well Status:	Active (exist, present)		

Drill Date:	01 Jun 1994	**Water Level Count:**	944
Well Depth:	12.40m -GL	**Strata Layers:**	8
Initial Water Depth:	-0.80m -MP	**Aquifer Tests:**	0
Diameter:	38mm	**Isotope Data:**	0
		Yield/Drawdown Tests:	0
Measuring Point Ait:	13.28m MSD QAR 1	**Highest GW Level:**	1.10m below MP
GL Around Well:	-2.32m -MP	**Lowest GW Level:**	2.59m below MP
MP Description:	Top of PVC Pipe	**First Reading:**	10 Apr 1995
		Last Reading:	30 Mar 2012
Driller:	Borewell Drilling	**Calc. Min. GWL:**	0.27m -MP
Drilling Method:	Cable Tool	**Last Updated:**	07 Apr 2011
Casing Material:	PVC	**Last Field Check:**	30 Mar 2012
Pump Type:	None Installed		
Yield:		**Screens:**	
Drawdown:		**Screen Type:**	Slotted PVC
Specific Capacity:		**Top GL:**	9.00m
		Bottom GL:	9.50m
Aquifer Type:	Non-Flowing Artesian		
Aquifer Name:	Springston Formation		

Date	Comments
	1 OF 2 PIEZOMETERS FOR WATER LEVEL OBSERVATIONS see M36/4740 see also Filetrack MO5C-491
12 May 2006	MP re-levelled May 2006 ref. LB395/63 Old level 11.05).
13 Apr 2010	Measuring point changed from 10.97 to 13.285 (increased by 2.315m), water levels all updated accordingly
15 Apr 2010	All tideda water levels corrected to new measuring point.15 April 2010
29 Oct 2010	Well depth checked following a 7.1 magnitude earthquake on 4 September 2010. Measured at 9.20 metres on 22 October 2010.
13 Dec 2010	10-12-2010 well flushed to remove silt build up as a result of earthquake.Well test pumped to ensure good connection with the aquifer.
07 Apr 2011	Well flushed and test pumped on 7th April to remediate any effects of the eartquake on 22nd February 2011. Total depth recorded at 12.4m.


N
Redgrave Street
Fusilier Street
Tankerville Road
Mathers Road
skateboarding rink.
playground
Pavillion
sports field
Hoonkay Park
macrocarpa trees
pathway
sports field
houses
M36/4740 + M36/4741
'Hoonkay Domain'

Borelog for well M36/4741

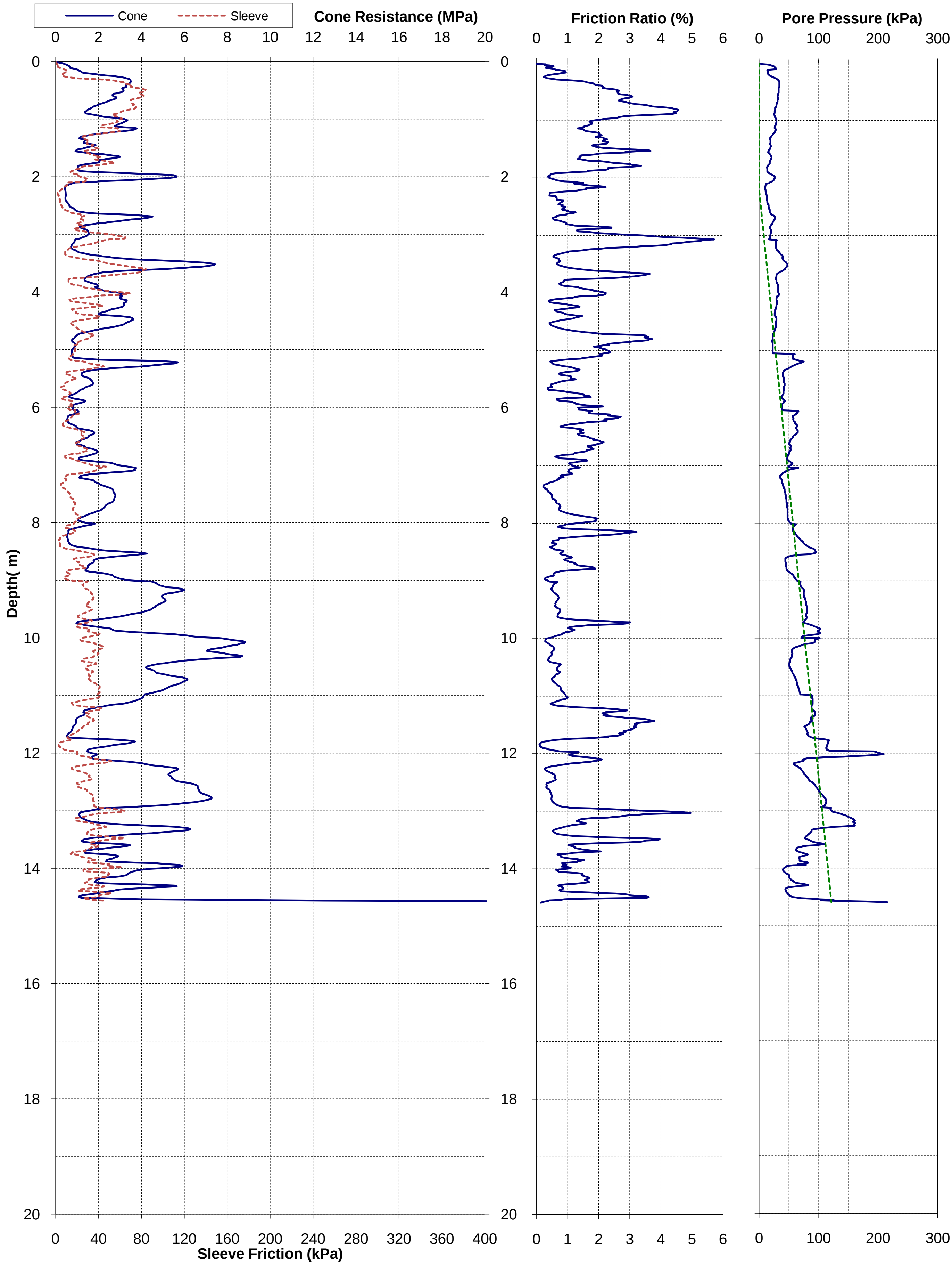
Gridref: M36:77245-38120 Accuracy : 2 (1=high, 5=low)
Ground Level Altitude : 10.97 +MSD
Driller : Borewell Drilling
Drill Method : Cable Tool
Drill Depth : -28.2m Drill Date : 1/06/1994

Water Level: Artesian

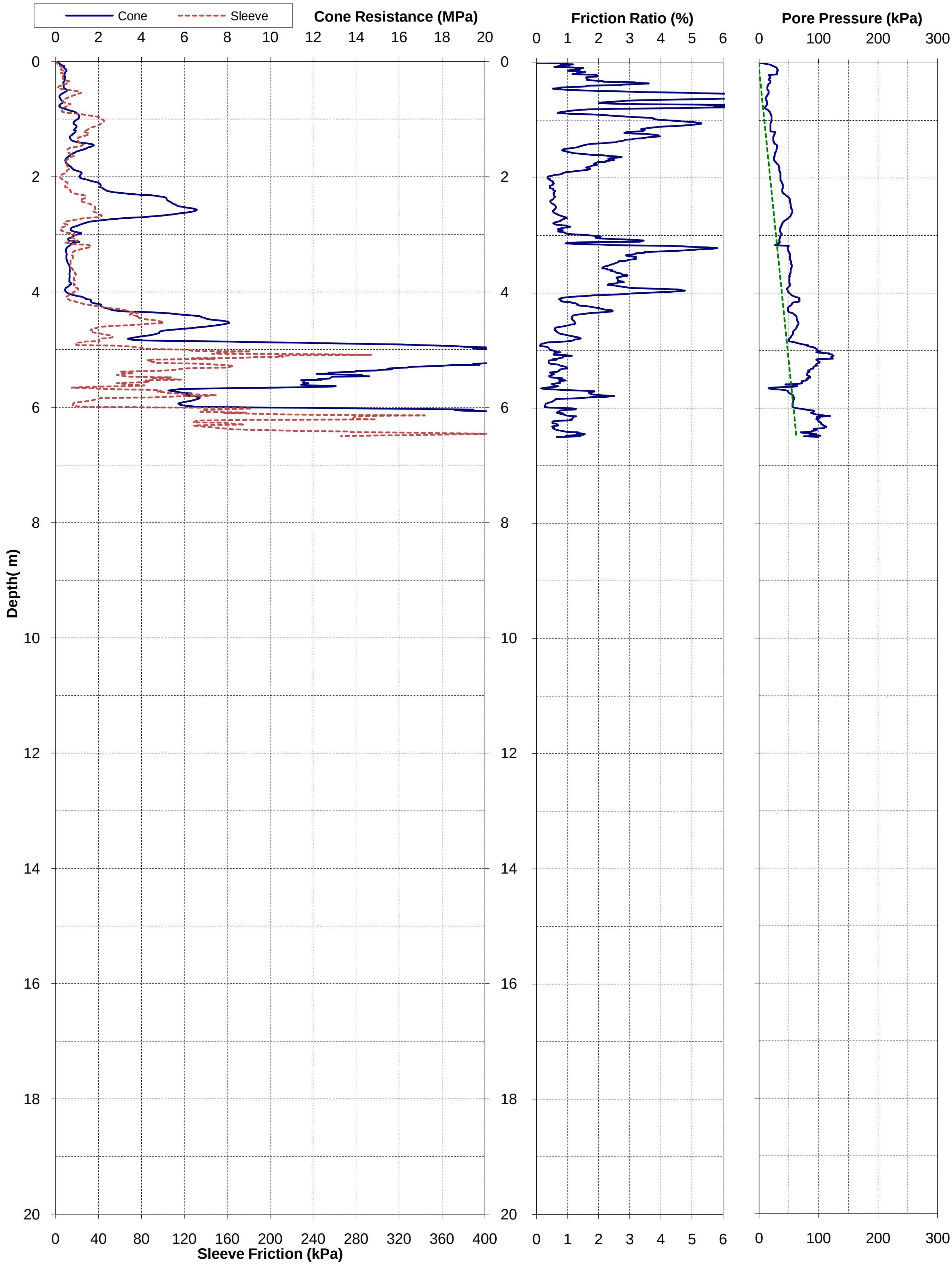
Depth(m)	Full Drillers Description	Formation Code
-1.00m	Topsoil and clay	sp
-1.50m	Silty clay	sp
-5.19m	Blue Grey clay with traces of vegetation	sp
-7.80m	Sandy, Brown stained gravels	sp
-8.19m	Clean Grey gravels with <1% sand	sp
- 17.5m	Blue Grey clay with fragments of wood and roots	sp
- 25.5m	Sandy Grey Brown stained gravels with traces of clay in places	ri
- 28.2m	Brown stained claybound sandy gravel with thin bands of clay	ri

Scale(m): -5, -10, -15, -20, -25

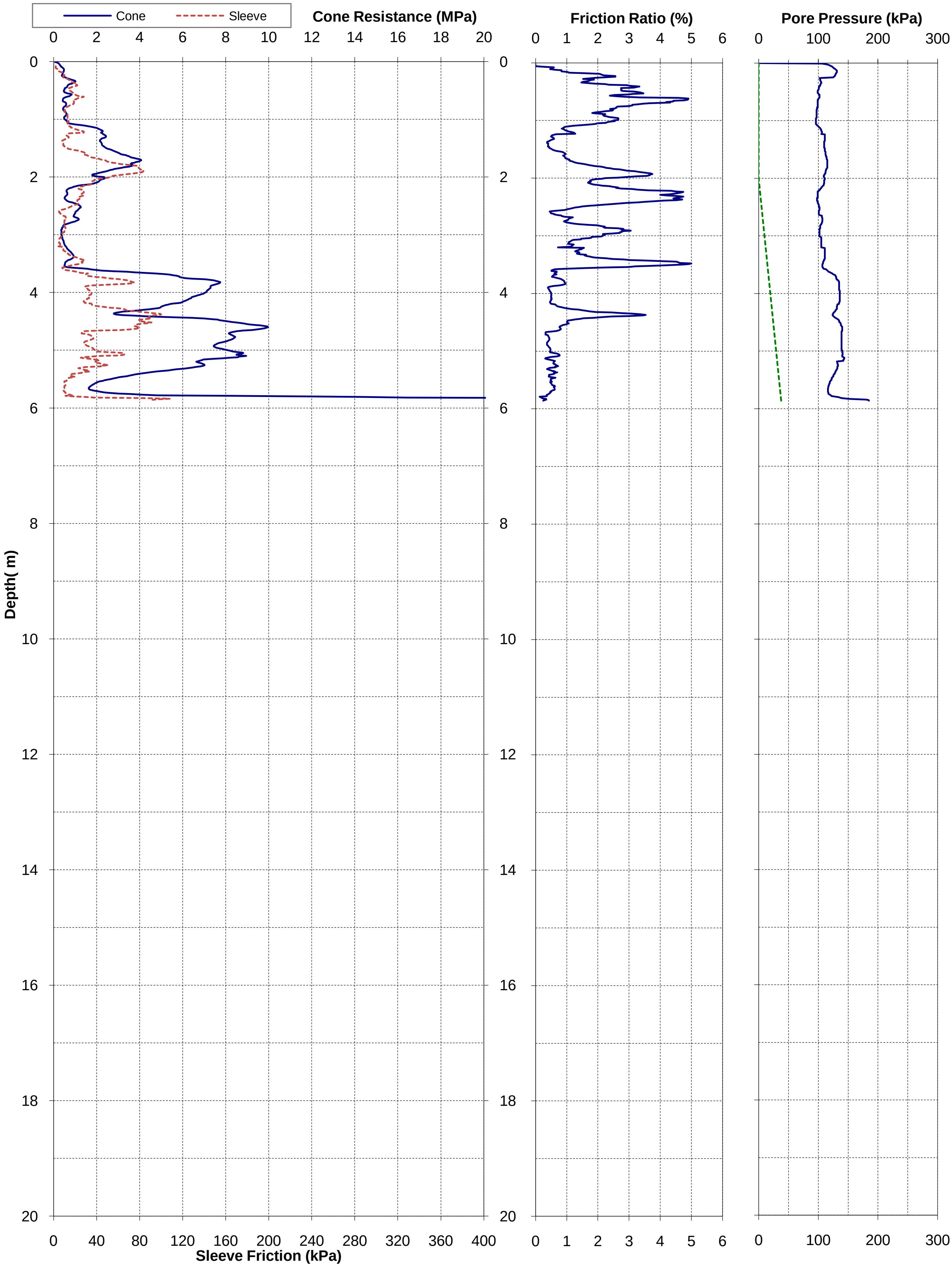
Project:	Christchurch 2011 Earthquake - EQC Ground Investigations			Page: 1 of 1		CPT-HNH-18
Test Date:	2-May-2011	Location:	Hoon Hay	Operator:	McMillan	
Pre-Drill:	1.2m	Assumed GWL:	2.2mBGL	Located By:	Survey GPS	
Position:	2476745.8mE	5738124.1mN	11.279mRL	Coord. System:	NZMG & MSL	
Other Tests:				Comments:		



Project:	Christchurch 2011 Earthquake - EQC Ground Investigations			Page:	1 of 1	CPT-HNH-14
Test Date:	2-May-2011	Location:	Hoon Hay	Operator:	McMillan	
Pre-Drill:	1.2m	Assumed GWL:	0.1mBGL	Located By:	Survey GPS	
Position:	2477297.8mE	5738449.9mN	11.728mRL	Coord. System:	NZMG & MSL	
Other Tests:				Comments:		



Project:	Christchurch 2011 Earthquake - EQC Ground Investigations			Page:	1 of 1	CPT-HNH-13
Test Date:	2-May-2011	Location:	Hoon Hay	Operator:	McMillan	
Pre-Drill:	1.2m	Assumed GWL:	2mBGL	Located By:	Survey GPS	
Position:	2476958.9mE	5738409.9mN	11.935mRL	Coord. System:	NZMG & MSL	
Other Tests:				Comments:		



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