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Phillipstown Community Centre

BU 2336 – 001 EQ2

Detailed Engineering Evaluation Quantitative Report

Version FINAL

39 Nursery Road, Phillipstown

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

Phillipstown Community Centre

BU 2336 – 001 EQ2

Detailed Engineering Evaluation
Quantitative Report
Version FINAL

39 Nursery Road
Phillipstown
Christchurch

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Date
08th March 2013

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Phillipstown Community Centre

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

39 Nursery Road, Phillipstown

Background

This is a summary of the Quantitative report for the Phillipstown Community Centre, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011; NZS 3604:2011 Timber-Framed buildings; inspections of the building on 19th January 2012 and 26th September 2012; and a review of drawings and consent documents available.

Brief Description

The Phillipstown Community Centre building is located at 39 Nursery Road, Phillipstown. The building was constructed in 1997 and serves as a community centre. The site consists of the community centre building, a car park and a large garden area.

The building is a single storey timber framed structure on subfloor framing. The roof is pitched up to ridges and consists of lightweight metal cladding and timber sarking fixed to timber purlins. The purlins are fixed to the timber trusses which are supported by load bearing timber framed walls. All internal surfaces of walls are lined with plasterboard and exterior cladding is provided by a prefinished aluminium cladding system.The floor is timber on timber joists and bearers. The building's foundations consists of a reinforced concrete perimeter foundation wall and timber piles internally.

The building dimensions are approximately 18m long by 6.5m wide with an approximate total floor area of 120m^2. The overall height of the building is 4m with wall stud heights of 2.4m.

Key Damage Observed

Key damage noted includes:-

- Minor cracking to plasterboard linings above and below windows and doors
- Cracking to the perimeter strip footing
- Settlement of south-west corner of the building
- Significant ground damage to the property around the south-west corner of the building caused by liquefaction

Critical Structural Weaknesses

No Critical Structural Weaknesses were identified for the building.

Indicative Building Strength

Based on the Quantitative Analysis carried out on the structure using NZS 3604:2011 for Timber-Framed buildings and referencing the New Zealand Society for Earthquake Engineering (NZSEE) guidelines, the building has been assessed to be >100% NBS along the building and >100% NBS across. Based on this, the overall %NBS for the building is **>100%**.

Recommendations

The building has been assessed to have a seismic capacity of **>100%NBS**. As the building's capacity is assessed to be greater than 67%NBS, it is not considered to be either an Earthquake Prone or an Earthquake Risk building. In addition there are no immediate collapse hazards, or Critical Structural Weaknesses associated with the structure, therefore general occupancy of the building is permitted.

Repair work should be carried out on all cracking observed in the building. The building foundations have settled significantly due to liquefaction in previous seismic events and it is recommended that remedial works be carried out to re-level the building as necessary. Severe liquefaction can be expected under significant earthquakes. Future ground damage from earthquakes may lead to further foundation settlement or damage and as a result, foundation strengthening is recommended. Any remedial works to foundations should be undertaken in accordance with MBIE's guidelines for TC3 land, due to the high levels of estimated settlement.

1 Background

GHD has been engaged by the Christchurch City Council to undertake a Detailed Engineering Evaluation of the Phillipstown Community Centre.

This report is a Quantitative Assessment of the building structure, and is based in general on NZS 3604:2011 Timber Framed buildings and the New Zealand Society for Earthquake Engineering (NZSEE) guidelines.

A Quantitative Assessment involves a full site measure of the building which is used to determine bracing capacity in accordance with manufacturers' guidelines where available. When the manufacturers' guidelines are not available, values for material strengths are taken from Table 11.1 of the NZSEE guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes. The demand for the building is determined in accordance with NZS 3604:2011 and the percentage of New Building Standard (%NBS) is assessed.

At the time of this report, no modelling of the building structure had been carried out. The detailed analysis for the report consisted of an analysis of the bracing capacity of the structure. No further analysis or calculations other than those set out within this report were 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)

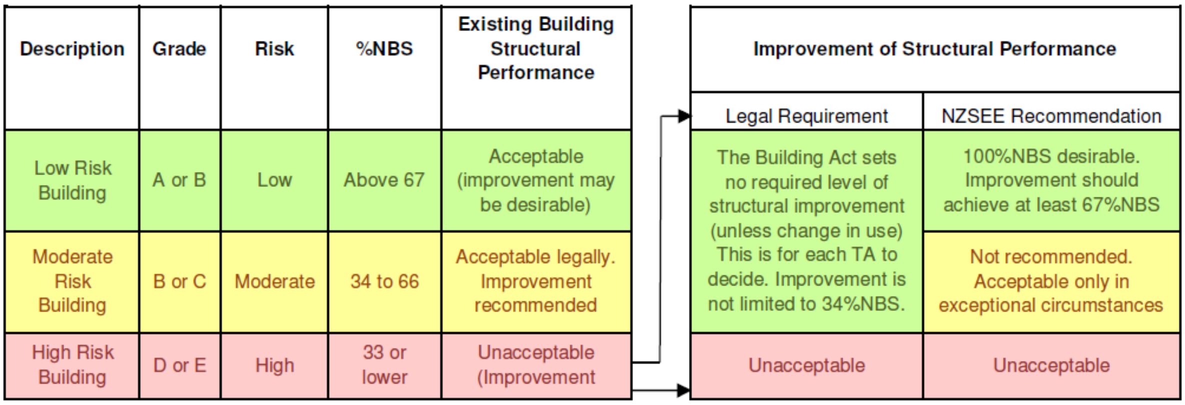
The increase in the above factors has resulted in a reduction in the level of compliance of an existing building relative to a new building despite the capacity of the existing building not changing.

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 building's 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.

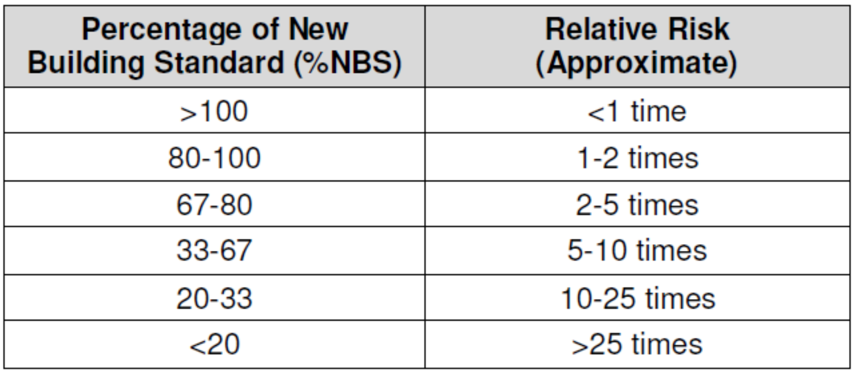


Description	Grade	Risk	%NBS	Existing Building Structural Performance
Low Risk Building	A or B	Low	Above 67	Acceptable (improvement may be desirable)
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally. Improvement recommended
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement

Improvement of Structural Performance	
Legal Requirement	NZSEE Recommendation
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
	Not recommended. Acceptable only in exceptional circumstances
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 Phillipstown Community Centre is located at 39 Nursery Road, Phillipstown. The site consists of the community centre building, a car park and a large garden area. The building was constructed in 1997 and serves as a community centre.

The building is a single storey timber framed structure on subfloor framing. The roof is pitched up to ridges and consists of lightweight metal cladding and timber sarking fixed to timber purlins. The purlins are fixed to the timber trusses which are supported by load bearing timber framed walls. All internal surfaces of walls are lined with plasterboard and exterior cladding is provided by a prefinished aluminium cladding system. The building has suspended timber flooring on timber joists and bearers supported by the foundations. The foundations consist of a reinforced concrete perimeter foundation wall and timber piles internally.

The building dimensions are approximately 18m long by 6.5m wide with an approximate total floor area of 120m^2. The overall height of the building is 4m with wall stud heights of 2.4m.

The nearest building is approximately 7m from the community centre building whilst the nearest waterway to the property is the Avon River, located approximately 1.5km to the north of the property.

A plan layout of the building is shown in Figure 2.

4.2 Gravity Load Resisting System

Gravity loads from the roof cladding are supported by timber purlins. These loads are then transferred from the purlins to the timber roof trusses which are at 900mm centres. Gravity loads from the trusses are then transferred to the load bearing timber framed external walls and then to the concrete perimeter foundation walls. Internal gravity loads are transferred through the suspended timber joists to bearers supported by timber pile foundations.

4.3 Lateral Load Resisting System

Lateral loads acting on the structure in both the long and short directions of the building are resisted by timber framed, plasterboard walls. Lateral forces acting on the roof structure are distributed to the walls through diaphragm action of the plasterboard lined ceiling. The walls are distributed throughout the building in both the long and short directions. The walls then transfer the lateral loads to the subfloor structure. The lateral loads to the sub-floor structure are then distributed by diaphragm action provided by the floor into the concrete perimeter foundation walls. The concrete perimeter foundation walls are expected to provide bracing for the subfloor structure.

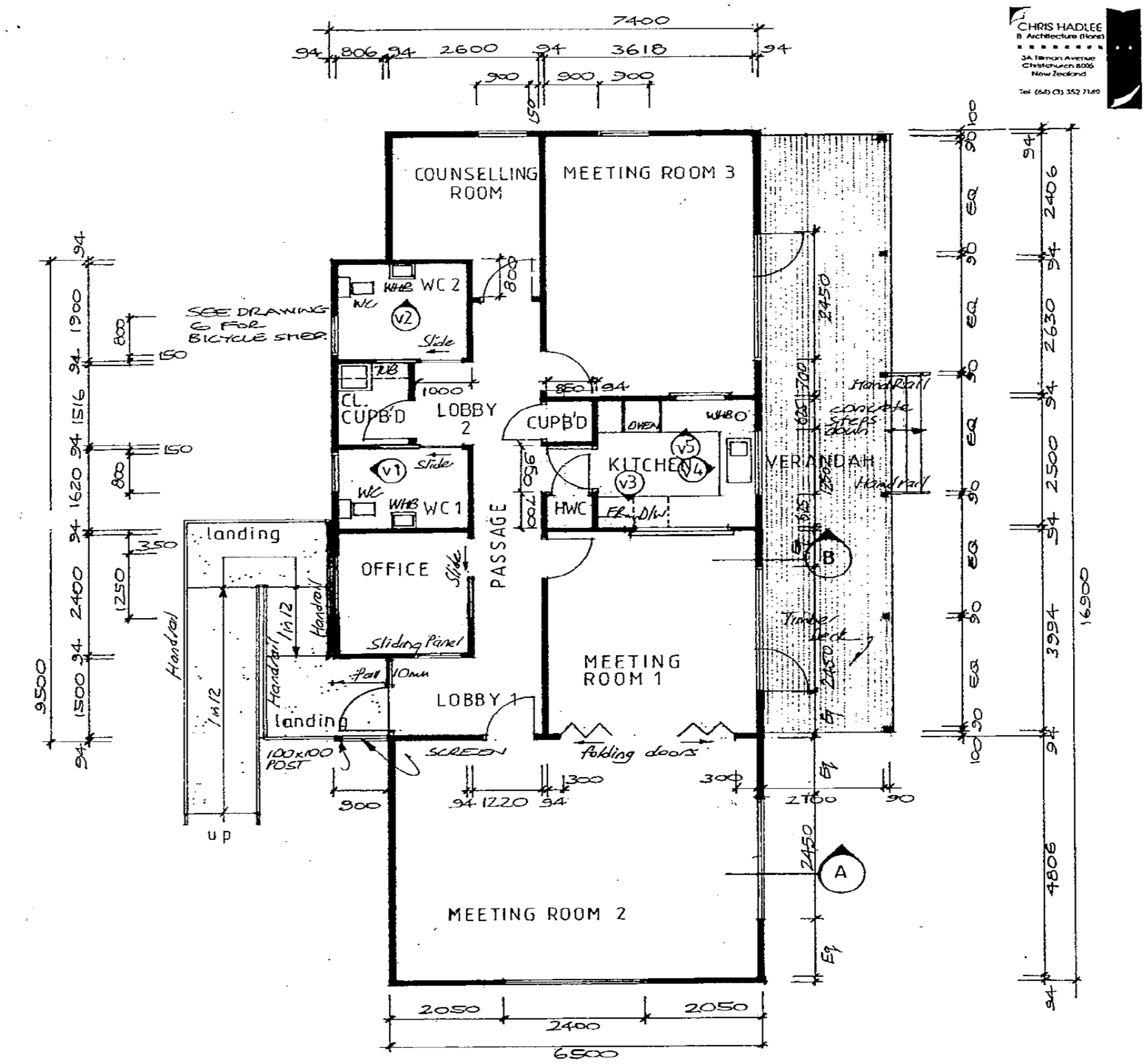


Figure 2 Plan layout of the building

5 Damage Assessment

5.1 Surrounding Buildings

No apparent damage was noted to the surrounding buildings or the adjoining properties.

5.2 Residual Displacements and General Observations

Cracking was noted to the internal plasterboard lining in several locations throughout the building, primarily above window and door openings (see Photograph 7 and Photograph 8). These cracks are not considered to be significant.

Cracking was noted at several locations on the building's perimeter foundation walls (see Photograph 4 to Photograph 6). The majority of the cracks appear to be cosmetic but it is evident that several of the cracks penetrate through the plaster finish and into the reinforced concrete foundation.

Residual displacements of the structure were noted during the inspection of the building. The south-west corner of the building appears to have settled. The difference in floor levels at the main entrance to the building and the south west corner was measured to be 38mm. It was evident from the site inspections that the west end of the building is sloping towards the south. Doors in the west of the building have a tendency to swing closed as a result of the displacement. It was also noted that the glass door to the timber decking area is askew and does not close smoothly, suggesting residual displacement in this area of the building.

An inspection of the sub-floor space was carried out on 26th September 2012. The visual inspection of the sub-floor space was conducted from the location of the access hatch only and it was not possible to carry out closer inspections of all the piles due to lack of access (see Photograph 10 to Photograph 12). No noticeable damage to the connections between the timber bearers and the piles were noted, however the piles near the south west corner of the building appear to be leaning slightly. Evidence of liquefaction of the ground under the building was also observed.

5.3 Ground Damage

There was evidence of liquefaction at the surface in the post-earthquake aerial photography (see Figure **3**). Ground damage to the property as a result of liquefaction was observed during the site visits and the tenant indicated that after the February 22nd earthquake, there was approximately 300mm deep layer of sediment covering the majority of the car park. Several areas of the paved car park have risen and fallen to create an uneven surface. A paved area of approximately 20m^2 at the south west of the building was severely damaged by liquefaction. Evidence of settlement of the site was observed in this area. The surrounding land has dropped approximately 100-120mm with a significant amount of fine sand sitting on the ground surface.

6 Survey

A floor level survey of the building was carried out and this has shown that there is differential settlement of the building. The south west corner of the building has appeared to have settled the most with the floor level at this end of the building being approximately 38mm lower than the floor level near the main entrance to the building.

Invasive structural investigations have not been undertaken for this building due to the low level of structural damage observed aside from the settlement of the foundations.

7 Geotechnical Investigation

Located on the site is a single level timber framed building with a suspended floor. The site is situated within the suburb of Phillipstown, 6.6km west of Pegasus Bay. The site is predominantly flat and approximately 1.2km from the Avon River and approximately 2km from the Heathcote River. The site is approximately 3m above mean sea level.

7.1 Published Information on Ground Conditions

7.1.1 Published Geology

The geological map of the area[1] indicates that the site is underlain by:

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

Figure 72 from Brown and Weeber indicates groundwater to be within 1 m of ground surface. Liquefaction susceptibility is indicated to be medium to high.

7.1.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that fourteen boreholes are located within a 200m radius of the site. Five of the boreholes were considered in this study (Table 2). The site geology described in these logs shows that the area is dominantly sand with varying amount of silt and clay with groundwater between 3.5 and 4.8m bgl.

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35/12154	4.57m	N/A	160m NW
M35/12155	4.57m	N/A	120m NW
M35/12156	6.1m	N/A	100m N
M35/2030	128m	3.5m	100m SW
M35/2081	126m	4.8m	150m S
M35/1989	126.7m	N/A	450m NW

It should be noted the quality of soil logging descriptions included on the boreholes is unknown and were likely written by the well driller and not a geotechnical professional or to a recognised geotechnical standard. In addition strength data is not recorded.

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[1] Brown, L. J. and Weeber, J.H. 1992: Geology of the Christchurch Urban Area. Institute of Geological and Nuclear Sciences

7.1.3 EQC Geotechnical Investigations

The Earthquake Commission has not undertaken geotechnical testing in the area of the site. The nearest EQC testing is over 250m from the site. The nearest CPT and borehole are in the Linwood region (CPT-LWD-22, M35_1989).

Table 3 EQC Geotechnical Investigation Summary Table

Bore Name	Grid Reference	Depth (m bgl)	Log Summary
CPT – LWD - 22	2482301.76 mE 5741339.21 mN	0 – 4.75	Soft sandy silt or clay

Initial observations of the CPT results indicate the soil is a soft sandy silt or clay, becoming a dense sand at 4.0m.

7.1.4 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.

Land in the CERA green zone has been divided into three technical categories. The technical categories – TC1 (grey), TC2 (yellow) and TC3 (blue) describe how the land is expected to perform in future earthquakes.

The site is classified as N/A - Urban Non-residential, however the nearby area is classified as TC2 (yellow) - minor to moderate land damage from liquefaction is possible in future significant earthquakes.

7.1.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows signs of significant liquefaction outside the building footprint and adjacent to the site, as shown in Figure **3.**

Figure 3 Post February 2011 Earthquake Aerial Photography

7.2 Seismicity

7.2.1 Nearby Faults

There are many faults within 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[2,3]

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	135 km	NW	~8.3	~300 years
Greendale (2010) Fault	27 km	W	7.1	~15,000 years
Hope Fault	100 km	NW	7.2~7.5	120~200 years
Kelly Fault	100 km	NW	7.2	150 years
Porters Pass Fault	60 km	NW	7.0	1100 years

The recent earthquakes since 4 September 2010 have identified the presence of a previously unmapped active fault system underneath the Canterbury Plains, including Christchurch City, and the Port Hills.

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

[3] GNS Active Faults Database

Research and published information on this system is in development and not generally available. Average recurrence intervals are yet to be estimated.

7.2.2 Ground Shaking Hazard

This seismic activity has produced earthquakes of Magnitude-6.3 with peak ground accelerations (PGA) up to twice the acceleration due to gravity (2g) in some parts of the city. This has resulted in widespread liquefaction throughout Christchurch.

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

7.3 Field Investigations

In order to further understand the ground conditions at the site, intrusive testing comprising one piezocone CPT investigations and two hand augers with scala penetrometer tests were conducted. Hand augers and scala penetrometer were conducted on 29 May 2012.

The locations of the tests are tabulated in Table 5.

Table 5 Coordinates of Investigation Locations

Investigation	Depth (m bgl)	Easting (NZMG)	Northing (NZMG)
CPT 01	25	2482274	5741064
HA 01	3.5	2482265	5741078
HA 02	3.5	2482261	5741089

The CPT investigations were undertaken by McMillans Drilling Ltd on 27 June 2012, typically to a target depth of 20m below ground level, however due to soft depositions it was extended 25m bgl.

Interpretation of output graphs[4] from the investigation showing Cone Tip Resistance (q_c), Friction Ratio (Fr), Inferred Lithology and Inferred Liquefaction Potential are presented in Table 7.

7.4 Ground Conditions Encountered

The two hand auger holes undertaken on 29 May 2012 are summarised in Table 6.

Table 6 Summary of Ground Investigation Results

Depth (m bgl)	Ground Conditions Encountered	Blows per 100mm
0.6	Topsoil/FILL	6 -12
0.6 – 3.2	SILT, with some clay; grey. Soft to firm; moist; low to medium plasticity.	1 - 12

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[4] McMillans Drilling CPT data plots, Appendix C.

Depth (m bgl)	Ground Conditions Encountered	Blows per 100mm
3.2 – 3.5	Silty SAND; grey. Dense to very dense, saturated.	14 - 23
3.5	End of Borehole - Collapsing	

Groundwater was encountered during the investigation at depths of 3.3m (HA01) and 3.2m (HA02) bgl.

7.4.1 Summary of CPT-Inferred Lithology

Interpretation of output graphs[4] from the investigation showing Cone Tip Resistance (q_c), Friction Ratio (Fr), Inferred Lithology and Inferred Liquefaction Potential are summarised in Table 7 and Table 8.

A summary of the lithology inferred from the CPT results is outlined in Table 7 below.

Table 7 Summary of CPT-Inferred Lithology

Depth (m)	Lithology [1]	Cone Tip Resistance q_c (MPa)	Friction Ratio Fr (%)	Relative Density Dr (%)
0 – 0.6	Pre-drilled			
0.6 – 3	SILT mixture	1 – 6	1 - 5	(Su = 80 – 200 kPa)
3 – 18.5	SANDS	2 – 30	0.5 – 2	>60
18.5 – 22	SAND mixture	1 – 22	0.5 – 3	(Su = 40 – 200 kPa)
22 – 25	SILT mixture	1 – 5	1	(Su = 40 kPa)

From the results above, the ground conditions at the site are understood to be predominantly silts to 3m, overlying sands to 22m, and silt mixtures to 25m.

This is considered consistent with the published geology and EQC investigations for the area, from the desktop information reviewed in Sections 7.1.1 and 7.1.2.

Please refer to Appendix C for further detail.

7.5 Liquefaction Analysis

7.5.1 Parameters used in Analysis

Assumptions made for the analysis process are as follows:

- D_{50} particle sizes for the site soil (sands) from CPT soil analysis
- Importance Category 2, post seismic event (50-year design life)
- PGA ULS 0.35g, SLS 0.13g

The following equation has been used to approximate soil unit weight from the CPT investigation data: [5]

$$\gamma = \frac{\gamma_w Gs}{2.65}\left(0.27\log Fr \ + \ 0.36\log\left(\frac{qc}{p_{atm}}\right) + 1.236\right)$$

This typically gave values ranging between 15 and 20 kN/m^3 (saturated).

The liquefaction analysis process has been conducted using the methodology from Stark & Olson[6], and from the NZGS Guidelines[7].

7.5.2 Results of Liquefaction Analysis

The results of the liquefaction analysis, as outlined in Table 8, indicate that depths of 2m to 4m, 5.2 m to 7m, 10.2m to 13m, and 17m to 22m are considered highly liquefiable.

Table 8 Summary of Liquefaction Susceptibility

Depth (m)	**Lithology**	**Triggering Factor F_L F_L**	**Liquefaction Susceptibility [8]**
0 – 0.6	Pre drilled	N/A	N/A
0.6 - 2	SILT Mixture	1.3 – 5	Low
2 – 4	SAND Mixtures	0.6 – 3	High
4 – 5.2	SANDS	2 – 5	Low
5.2 – 7	SANDS	0.8- 5	High
7 – 10.2	SANDS	1.2 – 5	Low
10.2 – 13	SANDS	0.3 – 1.8	High
13 – 17	SANDS	0.8 – 2.4	Moderate
17 – 22	SILT Mixtures	0.3 – 0.7	Severe

Settlement estimates for the CPT points are between 71mm for SLS conditions and 212mm for ULS conditions.

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[5] Robertson P.K., & Cabal K.L. 2010: *Estimating soil unit weight from CPT*. Gregg Drilling & Testing Inc.: Signal Hill, California, USA.

[6] Olson, S.M. & Stark, T.D. (2002). *Liquefied strength ratio from liquefaction flow failure case histories.* Canadian Geotechnical Journal, 39 (3), 629–647pp.

[7] Cubrinovski M., McManus K.J., Pender M.J., McVerry G., Sinclair T., Matuschka T., Simpson K., Clayton P., Jury R. 2010: *Geotechnical earthquake engineering practice: Module 1 – Guideline for the identification, assessment and mitigation of liquefaction hazards*. NZ Geotechnical Society

[8] Table 6.1, NZGS Guidelines Module 1 (2010)

7.6 Interpretation of Ground Conditions

7.6.1 Liquefaction Assessment

Overall, the site is considered to be highly susceptible to liquefaction. This is based on:

- Evidence of liquefaction at the surface in the post-earthquake aerial photography;
- Estimated settlements from the CPT results (71mm to 212mm) are in excess of the 100mm limit for TC2 classification, indicating the site should be considered in line with TC3 guidelines; and,
- The liquefaction assessment shows the layers between 2m to 4m, 5.2 m to 7m, 10.2m to 13m, and 17m to 22m indicated to be highly susceptible, as outlined in Table 8.

7.6.2 Slope Failure and/or Rockfall Potential

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

7.6.3 Foundation Recommendations

Based on the information presented above, we recommend the following for the subject site:

- Given the additional information considered in this report, it is now recommended that a soil class of **D** be adopted (in accordance with NZS 1170.5:2004).
- Any remedial works to foundations (or proposed new structures) be undertaken in accordance with MBIE's guidelines for TC3 land, due to the high levels of estimated settlement.

8 Seismic Capacity Assessment

8.1 Qualitative Assessment

An initial Qualitative Assessment has been completed by GHD for the Phillipstown Community Centre. This included a visual inspection of the building which was undertaken on 19th January 2012.

The Qualitative Assessment consisted of a visual inspection of the building's interior and exterior to determine the structural systems and likely behaviour of the building during an earthquake. The site was assessed for damage, including observations 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. A review of available drawings was also carried out.

The %NBS score determined for this building has been based on the Initial Evaluation Procedure (IEP) described by NZSEE and based on the information obtained from visual observation of the building and available drawings. Following the Qualitative Assessment, an initial capacity of the building was assessed to be 31% NBS taking into account the liquefaction potential of the site which was treated as a Critical Structural Weaknesses. Without factoring in any Critical Structural Weaknesses, the building capacity was assessed to be 84% NBS. The %NBS determined in the Qualitative Assessment is now superseded by the capacity of the building assessed through a more detailed Quantitative Assessment outlined below.

8.2 Quantitative Assessment

A Quantitative Assessment of the building was carried out using the information from the available drawings and visual inspections of the building carried out on 19th January 2012 and 26th September 2012. From this information, the building's bracing capacity was determined in accordance with NZS 3604:2011 and the NZSEE guidelines. The demand for the building was calculated in accordance with NZS 3604:2011 and the percentage of New Building Standard (%NBS) was assessed.

8.2.1 Building demand

The demand on the structure was determined in accordance with Section 5 of NZS 3604:2011. The bracing unit demand per square metre was determined from Table 5.8. In accordance with Table 5.8 of NZS 3604:2011 (for a single storey building with light roof, light single-storey cladding on heavy subfloor framing) a bracing demand of 17 BU/m^2 for the subfloor structure and 11 BU/m^2 for the single storey walls is taken. As the building is located in Christchurch (Earthquake Zone 2) on Class D soils, a multiplication factor of 0.8 is applied to reduce the demand in accordance with Table 5.8 of NZS 3604:2011. Therefore the total bracing demand for the building is;

$$\text{Single storey walls BU}_{\text{demand}} = (0.8 \text{ x } 11 \ \text{BU/m}^2 \ \text{ x } 117\text{m}^2)$$

$$= 1030 \text{ BU}$$

$$\text{Subfloor structure BU}_{\text{demand}} = (0.8 \text{ x } 17 \ \text{BU/m}^2 \ \text{ x } 117\text{m}^2)$$

$$= 1591 \text{ BU}$$

8.2.2 Wall bracing capacity

The building was constructed in 1997 which suggests that a bracing design of the whole building was undertaken in accordance with NZS 3604:1990, the current Code at the time. However no information was available with regards to the capacity of the bracing elements used in the building. Therefore the bracing capacity of the plasterboard linings was determined in accordance with Table 11.1 of the NZSEE guidelines and the “3604 Fix List Bracing Elements” publication by BRANZ in 1992.

For this purpose, the strength value of gypsum wall board given in Table 11.1 of the NZSEE guidelines (3kN/m each side) was converted to equivalent bracing units (1kN = 20BU) and then multiplied by the strength reduction factor of 0.7. This value was used for all walls with plasterboard lining on one side only. Therefore the bracing capacity for walls with plasterboard lining on only one side is taken as;

$$\mathrm{BU_{equivalent}} = \left(0.7 \text{ x} \frac{3\mathrm{kN}}{\mathrm{m}} \text{x} \frac{20\mathrm{BU}}{\mathrm{kN}} = 42\mathrm{BU/m} \text{ each side}\right)$$

For walls that are lined with plasterboard on both sides, the value calculated from Table 11.1 of NZSEE guidelines will be 84 BU/m. However this value is judged to be high considering modern wall bracing systems have lower bracing ratings. Therefore the bracing capacity for walls with plasterboard lining on both sides is taken as 60 BU/m from the “3604 Fix List Bracing Elements” publication by BRANZ in 1992.

Section 11.4 of the NZSEE guidelines states that shear panels can utilise their full bracing capacity for aspect ratios (height-to-width) up to 2:1. For aspect ratios greater than 2:1 and up to 3.5:1 a limiting factor can be applied in accordance with the NEHRP Recommended Provisions (BSSC, 2000) as follows;

$$\text{Aspect Ratio Factor} = \frac{2 \text{ x Width of Wall}}{\text{Wall Height}}$$

Any sections of wall with an aspect ratio greater than 3.5:1 were not included for the purpose of the bracing calculations. The walls in this building are 2.4m in height, and as such any wall less than 0.7m in length was not considered for the bracing calculations.

The subfloor bracing capacity is provided by the reinforced concrete perimeter foundation wall. The bracing capacity rating for this was determined as 300BUs/m in accordance with Table 5.11 of NZS 3604:2011. As the bracing capacity rating is very high, the bracing capacity will far exceed the bracing demand for the subfloor structure. As such no bracing analysis was carried out for the subfloor structure as the single-storey wall bracing capacity is more critical.

The calculated bracing capacities along and across the building are shown in Table 9.

Table 9 Bracing Units Provided

Direction	Bracing Units Provided
Along the building	1482 BUs
Across the building	1045 BUs

8.2.3 %NBS

The bracing capacity both along and across the building are compared to the demand to determine the critical direction, and therefore the overall %NBS for the building. The %NBS value is calculated as follows;

$$\%\mathrm{NBS} = \frac{\mathrm{BU_{provided}}}{\mathrm{BU_{demand}}} \text{ x } \%100$$

The calculated %NBS for both along and across the building is presented in Table 10.

Table 10 %NBS

Direction	%NBS
Along the building	144%
Across the building	102%

Following a detailed assessment the building has been assessed as having a seismic capacity **>100% NBS**. Under the NZSEE guidelines the building is not considered to be either an Earthquake Prone building or an Earthquake Risk as it achieves above 67% NBS.

8.3 Discussion of Results

The >100% NBS capacity obtained through the Quantitative Assessment was much higher than the initial Qualitative Assessment due to a more accurate bracing analysis performed to determine the capacity of the structure. Further, after a more detailed analysis of the structure, the liquefaction potential of the site was not considered to be a Critical Structural Weakness as any liquefaction induced settlement is not expected to cause a premature collapse of a single storey, light timber framed structure.

The building has a strength greater than 67% NBS and therefore is not deemed to be earthquake prone or earthquake risk.

8.4 Occupancy

As the building has been assessed to have a %NBS greater than 67% NBS, it is not considered to be an Earthquake Prone Building or an Earthquake Risk. In addition there are no immediate collapse hazards, or Critical Structural weaknesses associated with the structure, therefore general occupancy of the building is permitted.

9 Recommendations and Conclusions

The building has been assessed to have a seismic capacity of **>100%NBS**. As the building's capacity is assessed to be greater than 67%NBS, it is not considered to be either an Earthquake Prone or an Earthquake Risk building. In addition there are no immediate collapse hazards, or Critical Structural Weaknesses associated with the structure, therefore general occupancy of the building is permitted.

Repair work should be carried out on all cracking observed in the building. The building foundations have settled significantly due to liquefaction in previous seismic events and it is recommended that remedial works be carried out to re-level the building as necessary. Severe liquefaction can be expected under significant earthquakes. Future ground damage from earthquakes may lead to further foundation settlement or damage and as a result, foundation strengthening is recommended. Any remedial works to foundations should be undertaken in accordance with MBIE's guidelines for TC3 land, due to the high levels of estimated settlement.

10 Limitations

10.1 General

This report has been prepared subject to the following limitations:

- No intrusive structural investigations have been undertaken.
- No verticality survey has been undertaken.
- No material testing has been undertaken.
- No calculations, other than the wall bracing calculations included in this report, 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 Scope and Limitations of Geotechnical Investigation

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.

Where drill hole or test pit logs, cone tests, laboratory tests, geophysical tests and similar work have been performed and recorded by others under a separate commission, the data is included and used in the form provided by others. The responsibility for the accuracy of such data remains with the issuing authority, not with GHD.

The advice tendered in this report is based on information obtained from the desk study investigation location test points and sample points. It is not warranted in respect to the conditions that may be encountered across the site other than at these locations. It is emphasised that the actual characteristics of the subsurface materials may vary significantly between adjacent test points, sample intervals and at locations other than where observations, explorations and investigations have been made. Subsurface conditions, including groundwater levels and contaminant concentrations can change in a limited time. This should be borne in mind when assessing the data.

It should be noted that because of the inherent uncertainties in subsurface evaluations, changed or unanticipated subsurface conditions may occur that could affect total project cost and/or execution. GHD does not accept responsibility for the consequences of significant variances in the conditions and the requirements for execution of the work.

The subsurface and surface earthworks, excavations and foundations should be examined by a suitably qualified and experienced Engineer who shall judge whether the revealed conditions accord with both the assumptions in this report and/or the design of the works. If they do not accord, the Engineer shall modify advice in this report and/or design of the works to accord with the circumstances that are revealed.

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-east (front) elevation.

Photograph 2 View of the north side (rear) of the community centre.

Photograph 3 Area where pavement was destroyed and settlement of surrounding ground occurred as a result of liquefaction.

Photograph 4 Vertical cracking to perimeter footing in the south-west of the building.

Photograph 5 Cracking to strip footing in the south-east corner of the building.

Photograph 6 Cracking to the strip footing to the north of the building.

Photograph 7 Damage to GIB lining above doors.

Photograph 8 Damage to GIB lining above doors.

Photograph 9 Evidence of liquefaction and settlement of site to south of the building.

Photograph 10 Timber piles and bearers near the west of the building viewed from sub-floor space

Photograph 11 Foundation piles at the south-west of the building where building has settled most. Evidence of significant liquefaction was observed at location of arrow.

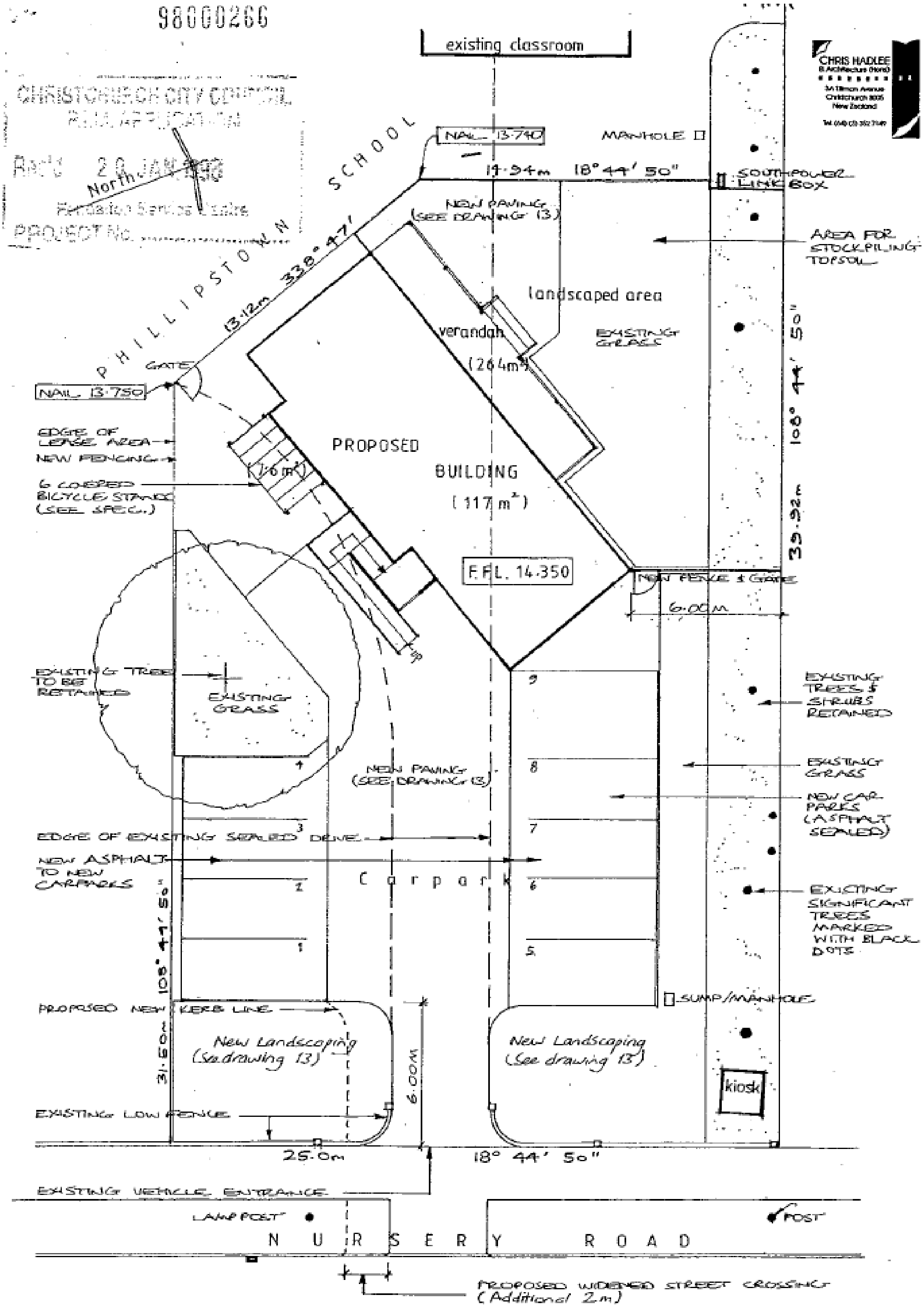
Photograph 12 Evidence of liquefaction of ground under the building.

Appendix B

Existing Drawings

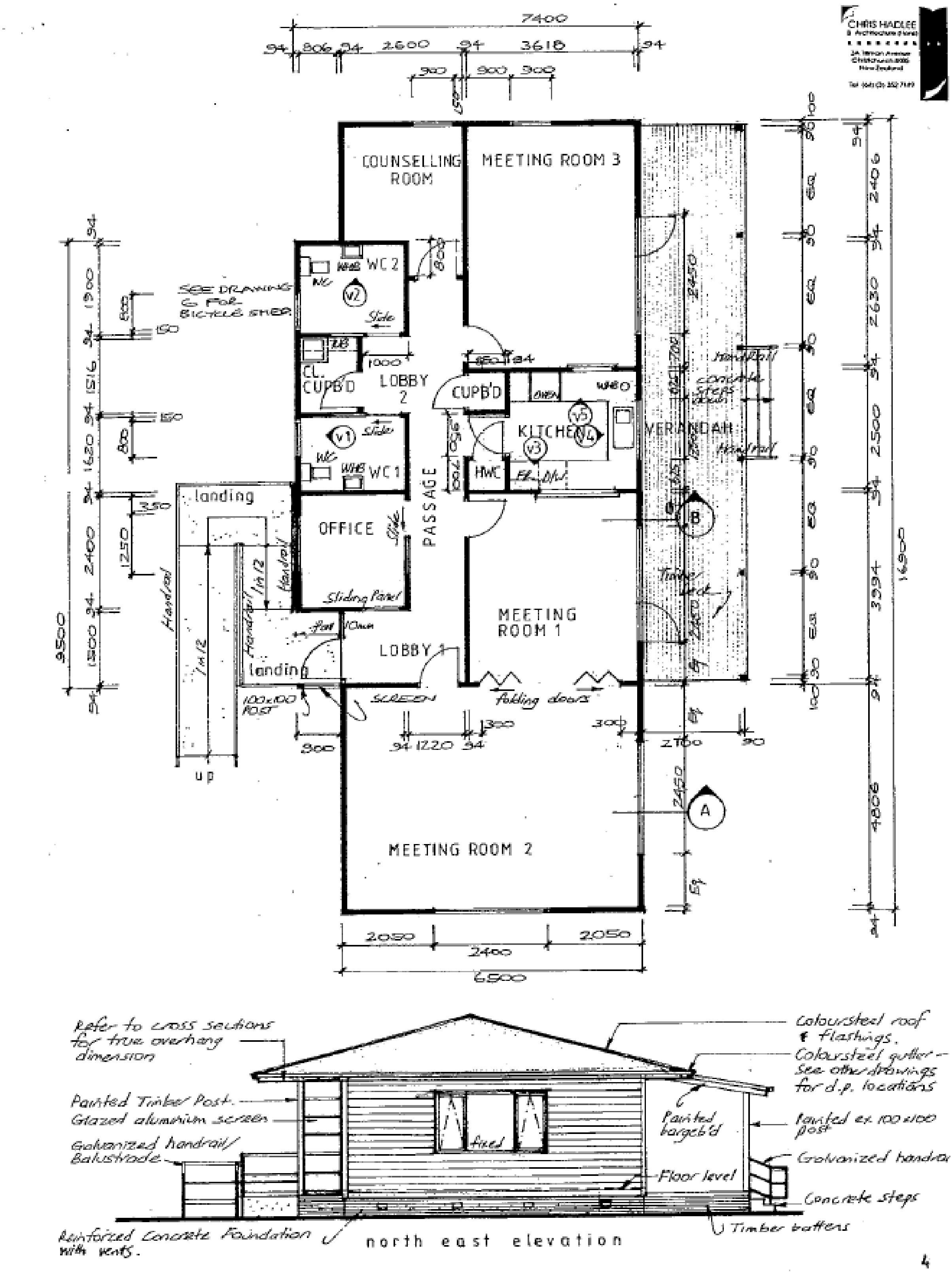
98000266

CHRISTCHURCH CITY COUNCIL
[illegible] APPLICATION
Rec'd 20 JAN 1998
Fendalton Service Centre
PROJECT No.



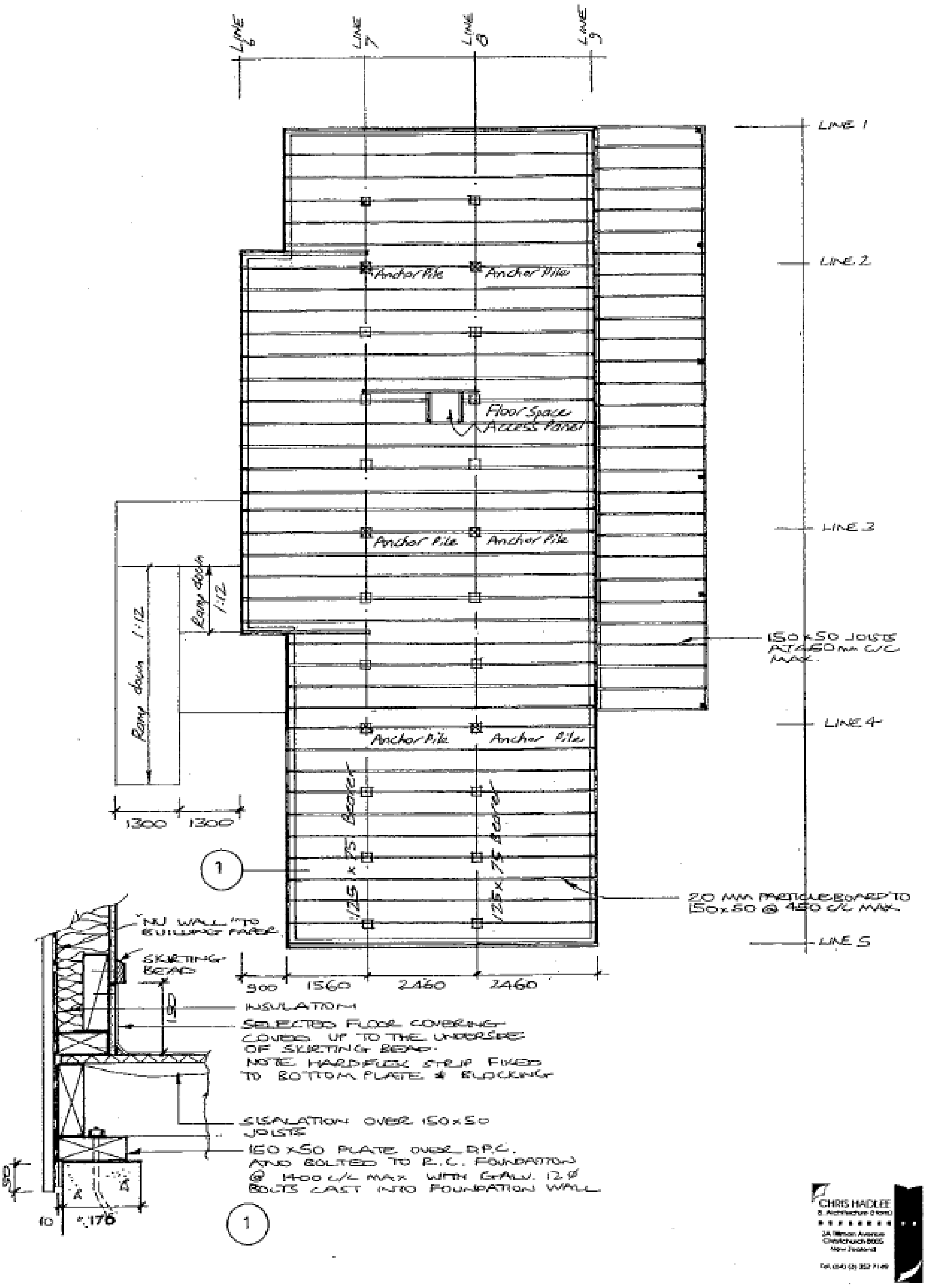
SITE AREA 956m²

PROPOSED PHILLIPSTOWN COMMUNITY CENTRE SITE PLAN 1:200 21·5·97 1 9710



4

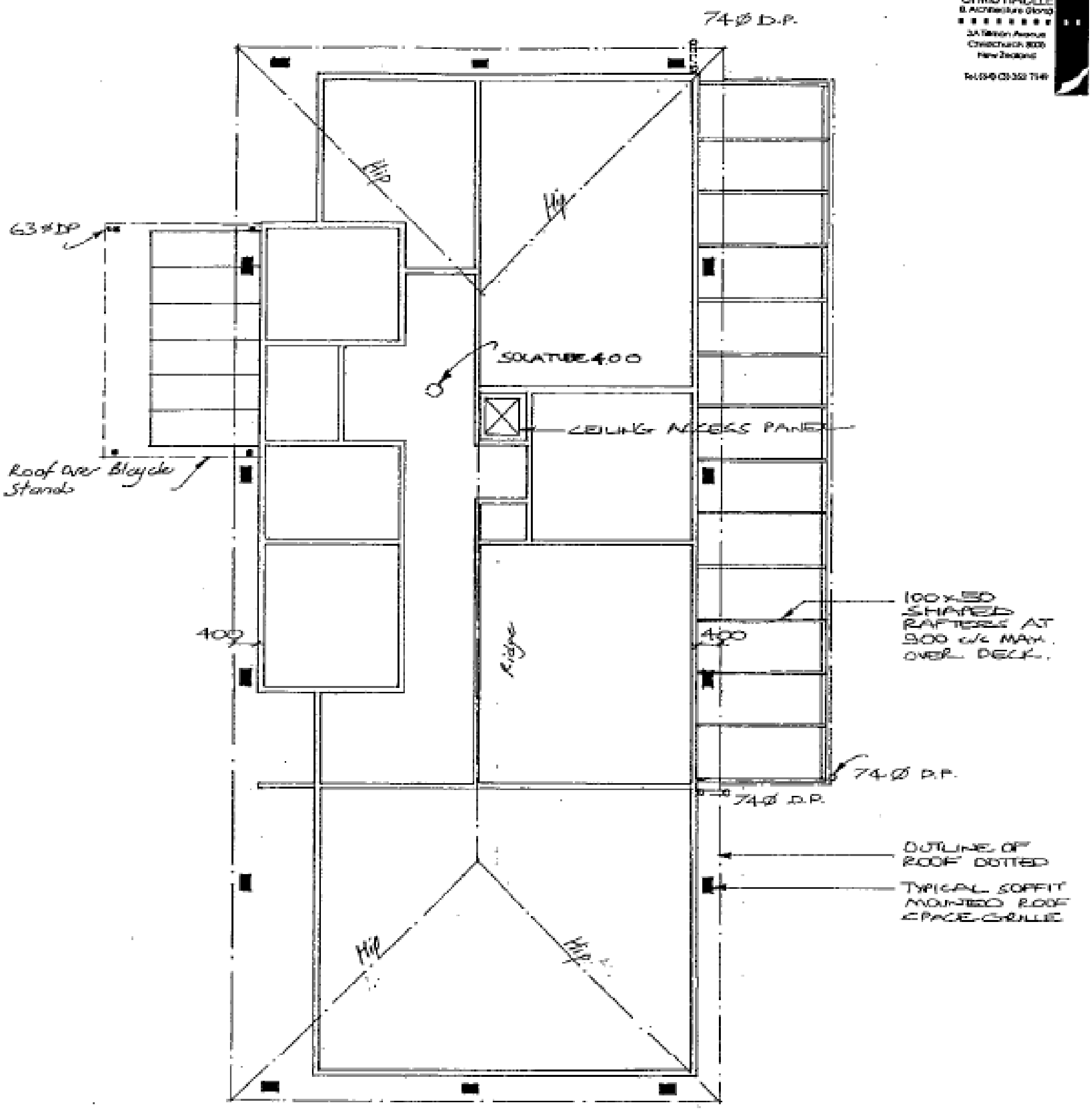
PROPOSED PHILLIPSTOWN COMMUNITY CENTRE: FLOOR PLAN 1:100 21-5-97 9710

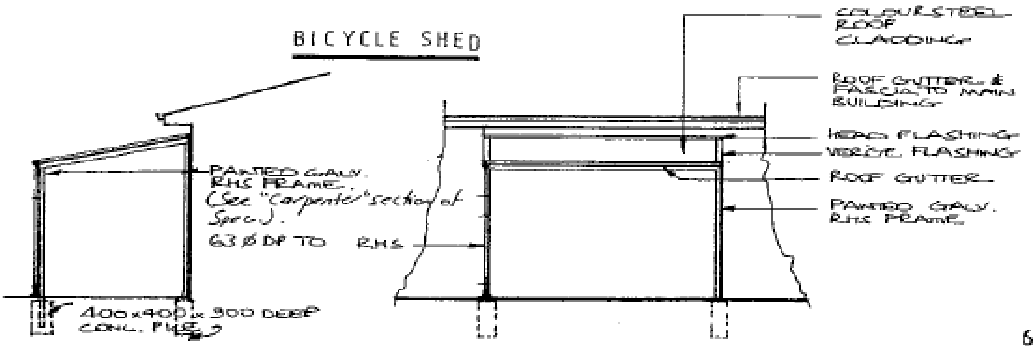


CHRIS HADLEE
B. Architecture (Hons)
New Zealand

PHILLIPSTOWN COMMUNITY CENTRE, NURSERY ROAD: FLOOR STRUCTURE 1:100 6.6.97 5 9710

CHRIS HADLEE
B. Architecture (Hons)
3A Tillman Avenue
Christchurch 8005
New Zealand
Tel (03) 352 7149

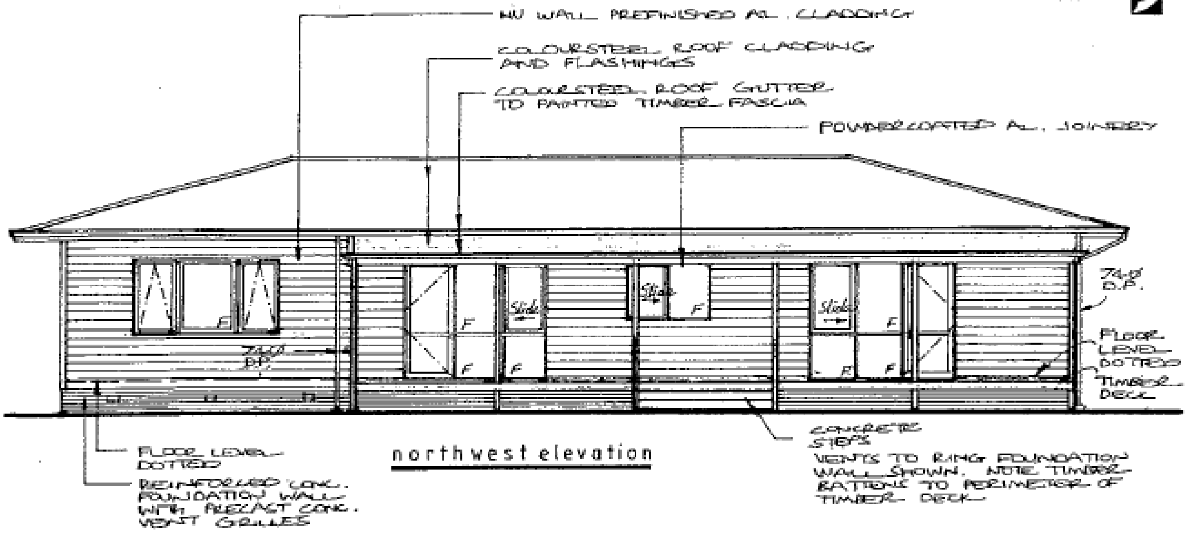




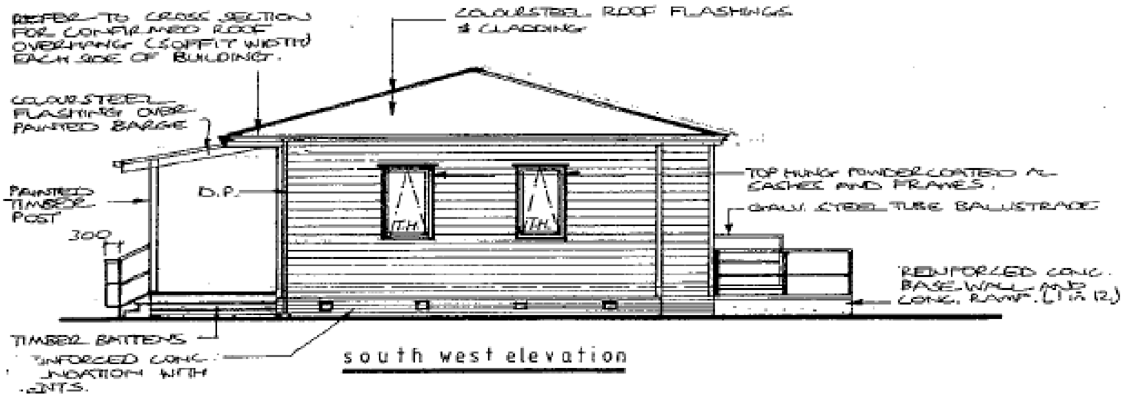
6

PHILLIPSTOWN COMMUNITY CENTRE, NURSERY ROAD: ROOF 1:100, 6.6.97, 9710

CHRIS HADLEE
B Architecture (Hons)
3A [illegible] Avenue
Christchurch [illegible]
New Zealand
Tel. (64) (3) 352 7149



northwest elevation

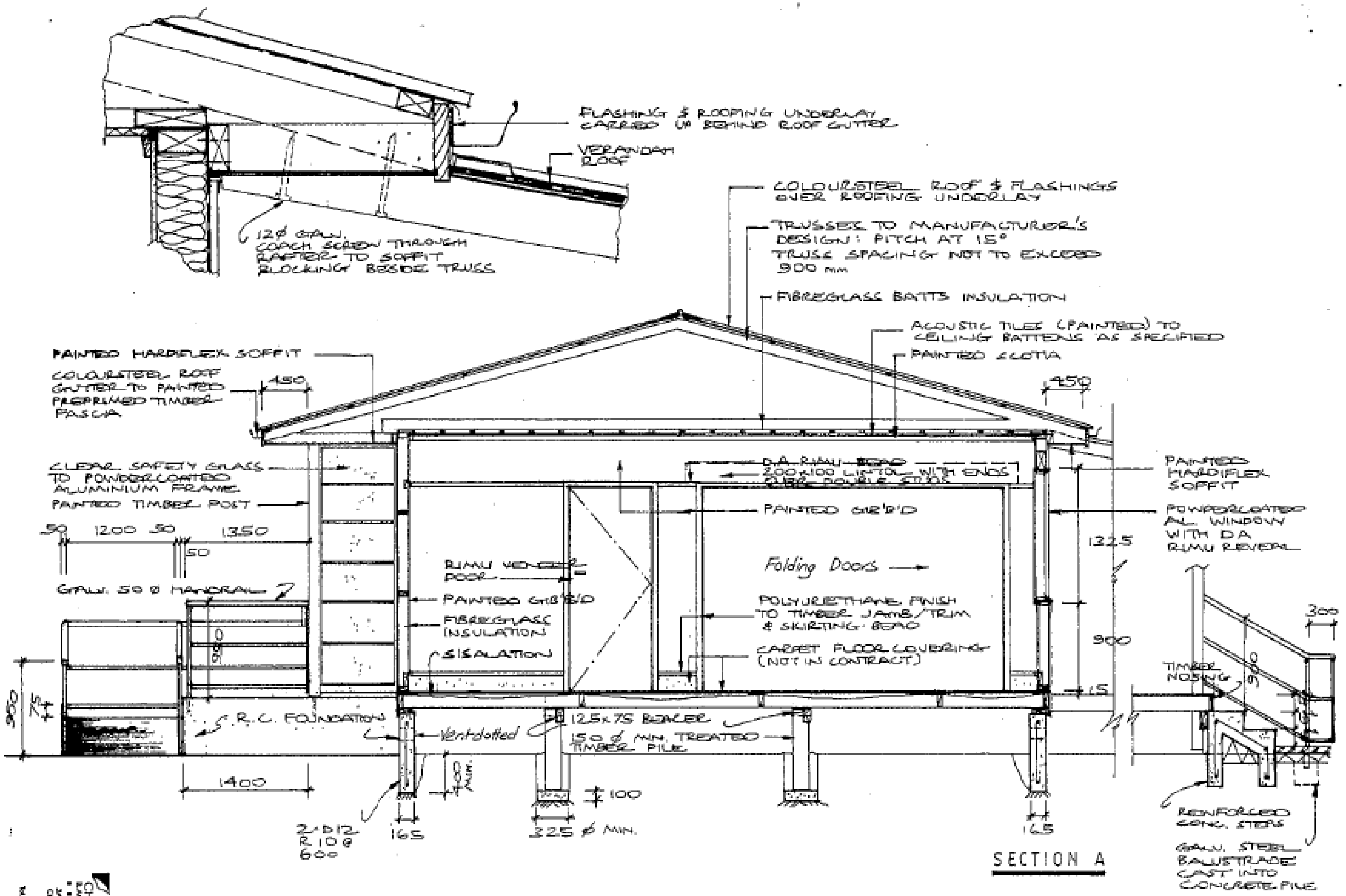


south west elevation

south east elevation

NB. Refer to Specification & Drawing 6 for bicycle "shed" (covered bicycle stands)

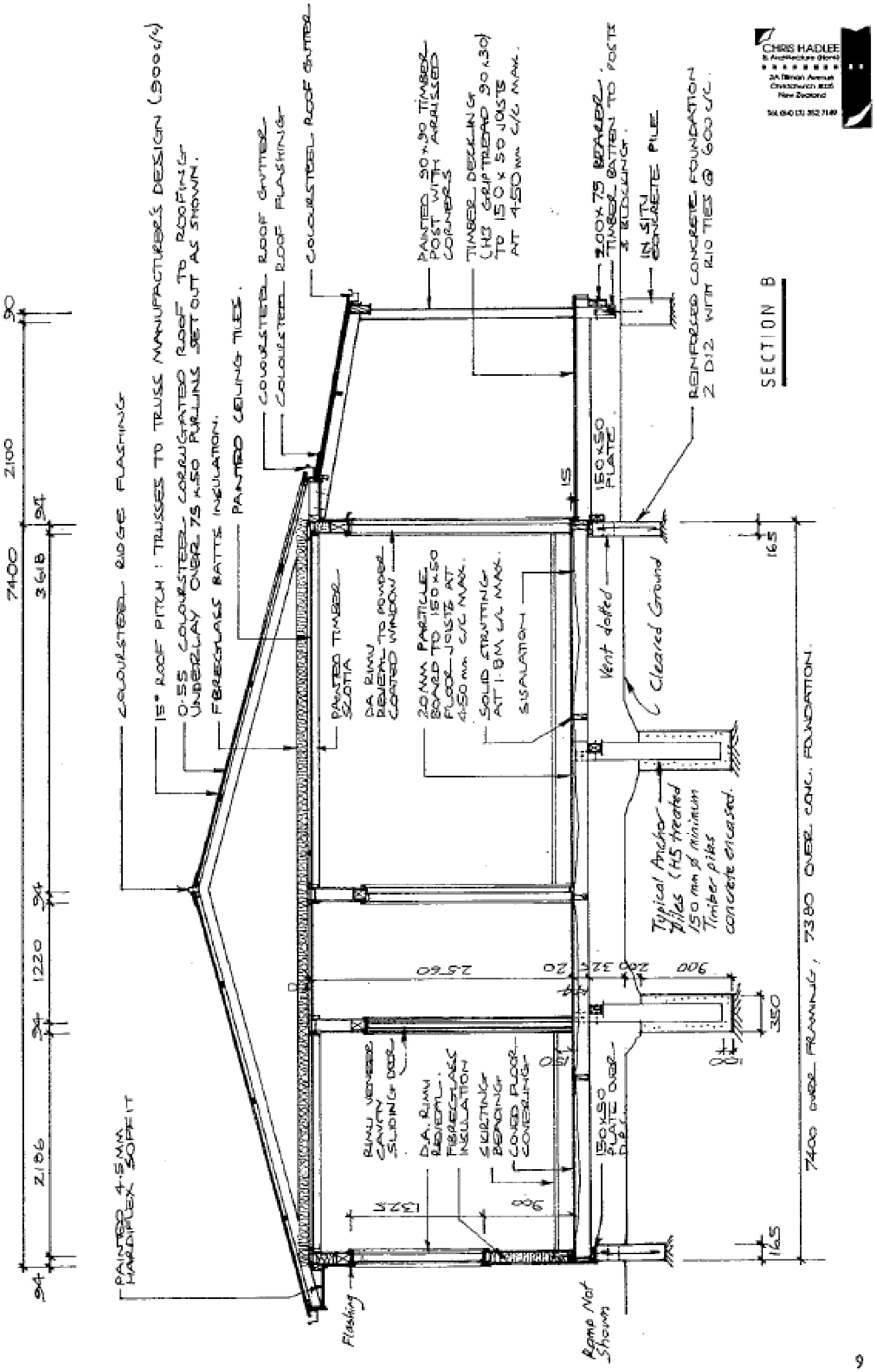
PROPOSED PHILLIPSTOWN COMMUNITY CENTRE FLOOR PLAN 1:100 21-5-97 9710 7



SECTION A

PHILLIPSTOWN COMMUNITY CENTRE, NURSERY ROAD

1:50 6 697 9710 8



PHILLIPSTOWN COMMUNITY CENTRE, NURSERY ROAD 1:50 6697 9710

9

Appendix C

Geotechnical Investigation

Bore or Well No: M35/2081
Well Name: NURSERY ROAD
Owner: CHRISTCHURCH CITY COUNCIL

Street of Well: NURSERY ROAD RESERVE
Locality: PHILLIPSTOWN
NZGM Grid Reference: M35:8231-4093 QAR 3
NZGM X-Y: 2482310 - 5740930
Location Description:
ECan Monitoring: ECan Recorder Network
Well Status: Not Used

File No: CO6C/03276
Allocation Zone: Christchurch/West Melton
Uses: Water Level Observation

Drill Date: 07 Nov 1930
Well Depth: 125.80m -GL
Initial Water Depth: 8.20m -MP
Diameter: 76mm

Measuring Point Ait: 5.16m MSD QAR 1
GL Around Well: -0.05m -MP
MP Description: TOC concrete base

Driller: Job Osborne (& Co/Ltd)
Drilling Method: Hydraulic/Percussion
Casing Material: STEEL
Pump Type: Unknown
Yield: 3 l/s
Drawdown: 5 m
Specific Capacity: 0.59 l/s/m

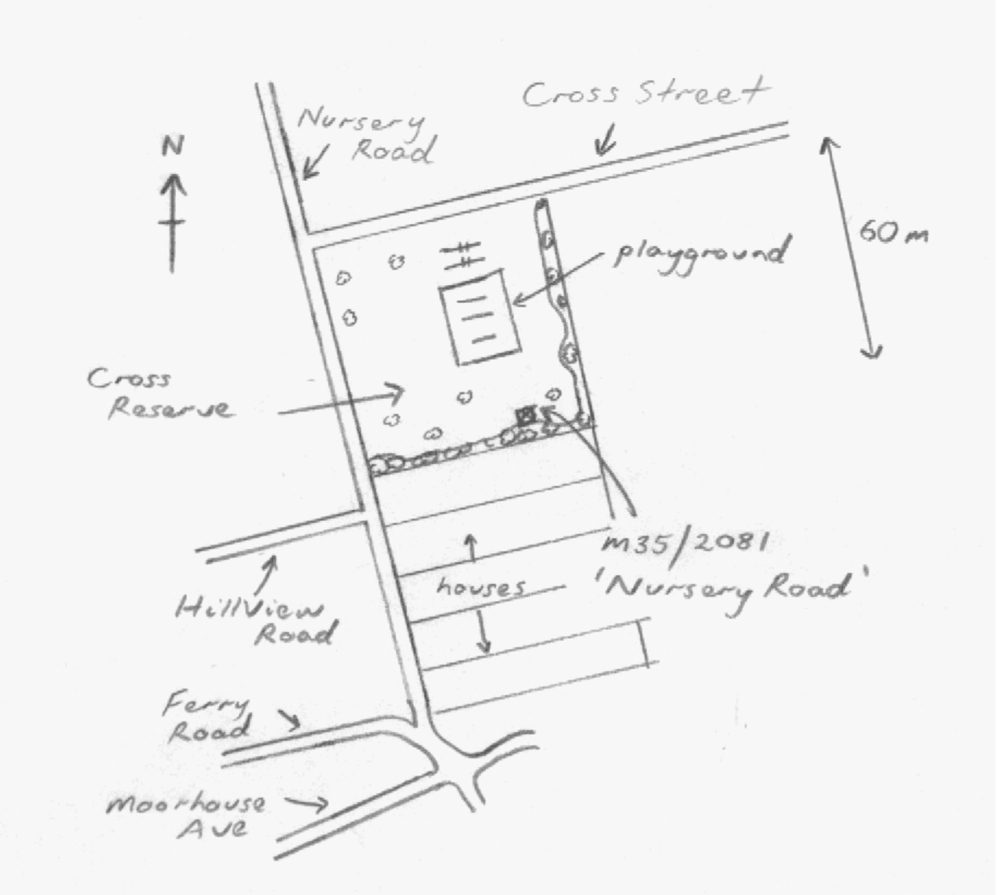
Aquifer Type: Flowing Artesian
Aquifer Name: Wainoni Gravel

Water Level Count: 1010
Strata Layers: 20
Aquifer Tests: 1
Isotope Data: 0
Yield/Drawdown Tests: 1

Highest GW Level: 8.40m from MP
Lowest GW Level: 3.73m from MP
First Reading: 11 Oct 1984
Last Reading: 21 Dec 2011
Calc. Min. GWL: 4.81m -MP
Last Updated: 27 Oct 2009
Last Field Check: 21 Dec 2011

Screens:
Screen Type:
Top GL:
Bottom GL:

Date	Comments
	Previous owner Phillipstown Baths.
11 Jan 2001	Free flow test data entered for Yield and Drawdown (see aquifer test)
24 Feb 2006	Re-levelled Feb 2006 ref. LB395/57 (139mm higher)
27 Oct 2009	Changed description reference from "TOC" to "TOC concrete base" after information received from network Review


Cross Street
Nursery Road
N
60 m
playground
Cross Reserve
M35/2081
'Nursery Road'
houses
Hillview Road
Ferry Road
Moorhouse Ave

Borelog for well M35/2081 page 1 of 2

Gridref: M35:8231-4093 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 5.11 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -125.9m Drill Date : 7/11/1930

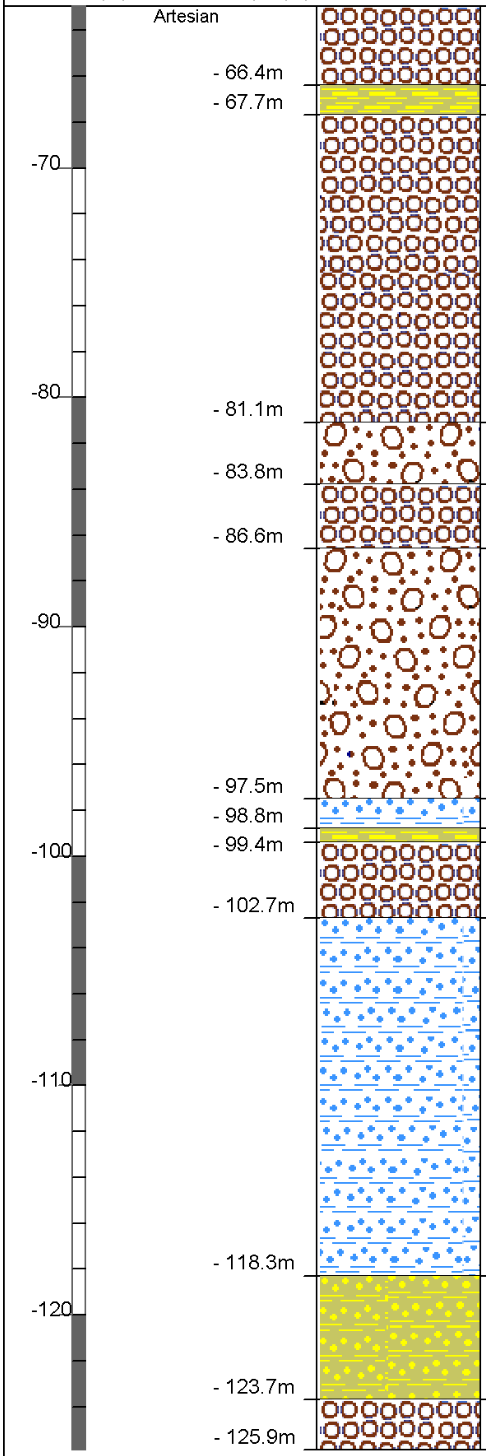
Environment Canterbury Regional Council

Scale(m): -10, -20, -30, -40, -50, -60

Water Level	Depth(m)	Full Drillers Description	Formation Code
Artesian	- 15.2m	Sand & clay	sp?
	- 28.9m	Blue sand & clay	ch
	- 33.8m	Blue shingle	ri
	- 39.0m	Brown shingle	ri
	- 41.4m	Blue sand & clay	br
	- 56.6m	Yellow sand	br
	- 59.4m	Blue sand & clay	br
	- 60.3m	Yellow clay	br
	- 63.0m	Brown shingle (Water level 1.5m above surface)	li

Borelog for well M35/2081 page 2 of 2

Gridref: M35:8231-4093 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 5.11 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -125.9m Drill Date : 7/11/1930



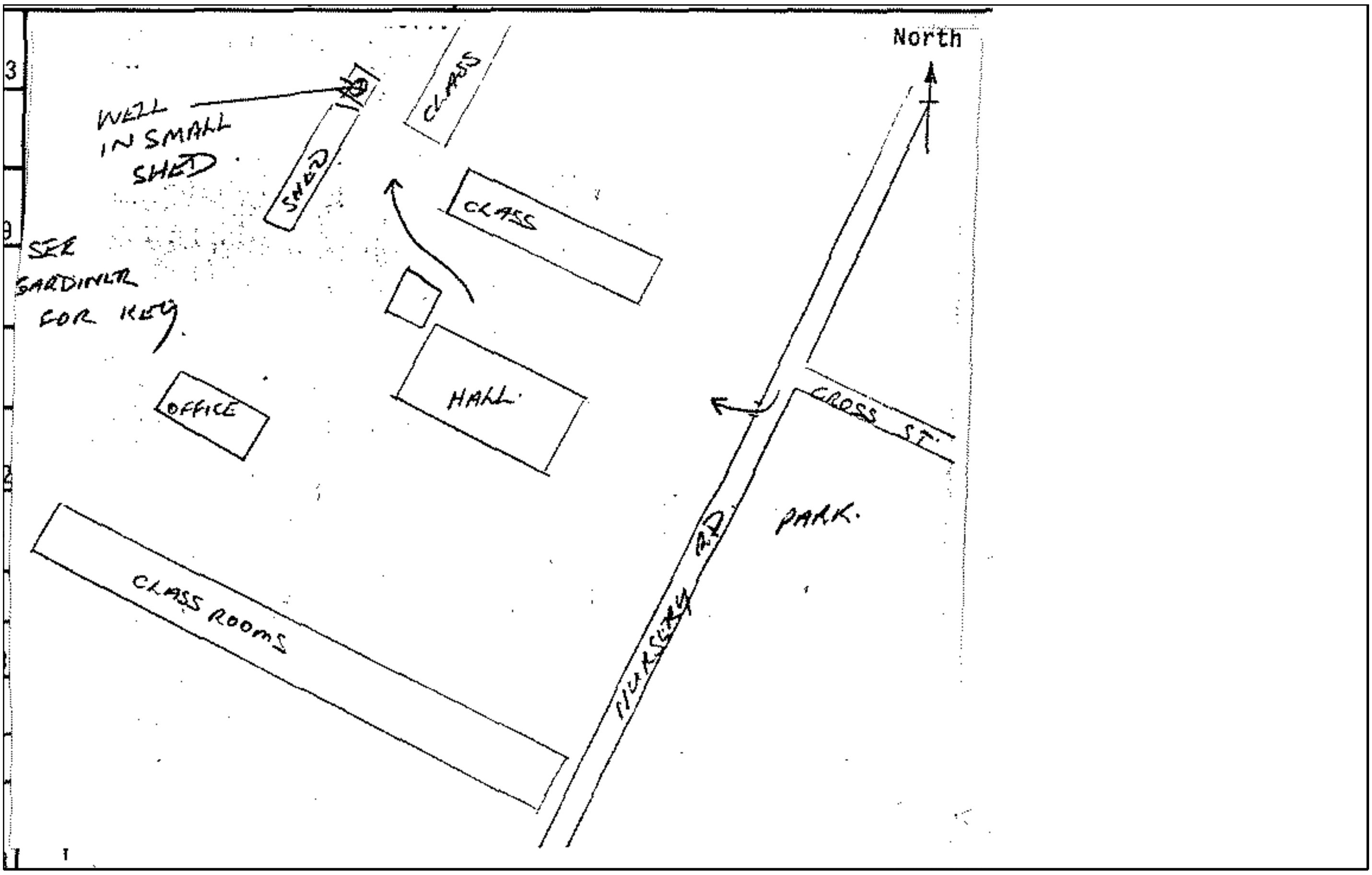
Water Level	Depth(m)	Full Drillers Description	Formation Code
Artesian	- 66.4m	Brown shingle (Water level 1.5m above surface)	li
	- 67.7m	Yellow clay	li
	- 81.1m	Brown shingle	li
	- 83.8m	Brown sand & gravel	li
	- 86.6m	Brown shingle	li
	- 97.5m	Brown sand & gravel	he
	- 98.8m	Blue sand & clay	he
	- 99.4m	Yellow clay	he
	- 102.7m	Brown shingle	bu
	- 118.3m	Blue sand & clay	sh
	- 123.7m	Yellow sand & clay	sh
	- 125.9m	Brown shingle	wa

Bore or Well No: M35/2030
Well Name:
Owner: EDUCATION BRD

Street of Well:	PHILLIPSTOWN SCHOOL	**File No:**	
Locality:	LINWOOD	**Allocation Zone:**	Christchurch/West Melton
NZGM Grid Reference:	M35:822-410 QAR 4		
NZGM X-Y:	2482200 - 5741000		
Location Description:		**Uses:**	
ECan Monitoring:			
Well Status:	Not Used		

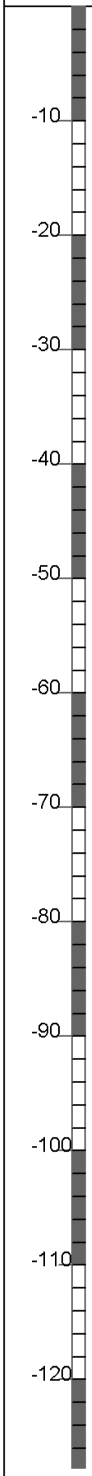
Drill Date:	06 Dec 1921	**Water Level Count:**	0
Well Depth:	126.10m -GL	**Strata Layers:**	14
Initial Water Depth:	8.50m -MP	**Aquifer Tests:**	0
Diameter:	64mm	**Isotope Data:**	0
		Yield/Drawdown Tests:	0
Measuring Point Ait:	5.12m MSD QAR 1	**Highest GW Level:**	
GL Around Well:	0.00m -MP	**Lowest GW Level:**	
MP Description:		**First Reading:**	
		Last Reading:	
Driller:	Job Osborne (& Co/Ltd)	**Calc. Min. GWL:**	3.50m -MP
Drilling Method:	Hydraulic/Percussion	**Last Updated:**	01 Dec 2009
Casing Material:	STEEL	**Last Field Check:**	
Pump Type:	Unknown		
Yield:	0 l/s	**Screens:**	
Drawdown:	0 m	**Screen Type:**	
Specific Capacity:		**Top GL:**	
		Bottom GL:	
Aquifer Type:	Flowing Artesian		
Aquifer Name:	Wainoni Gravel		

Date	Comments
01 Dec 2009	level ref LB395/36


North
WELL IN SMALL SHED
SHED
CLASS
CLASS
SEE GARDINER FOR KEY
OFFICE
HALL
CROSS ST
PARK
NURSERY RD
CLASS ROOMS

Borelog for well M35/2030

Gridref: M35:822-410 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 5.12 +MSD
Driller   : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -126.2m   Drill Date : 6/12/1921



Water Level	Depth(m)	Full Drillers Description	Formation Code
Artesian	- 28.0m	Sand and clay	sp-ch
	- 40.2m	Brown shingle	ri
	- 49.9m	Blue sand	br
	- 58.5m	Clay and peat	br
	- 67.9m	Brown shingle	li
	- 71.9m	Clay and peat	li
	- 79.2m	Brown shingle	li
	- 84.1m	Brown sand	li
	- 93.6m	Brown shingle	li
	- 98.1m	Yellow sand & clay	he
	- 102.4m	Brown shingle	bu
	- 120.1m	Blue sand & clay	sh
	- 124.4m	Yellow sand & clay	sh
	- 127.8m	Brown shingle	wa

Unknown No: M35/12156

Well Name: CCC BorelogID 154

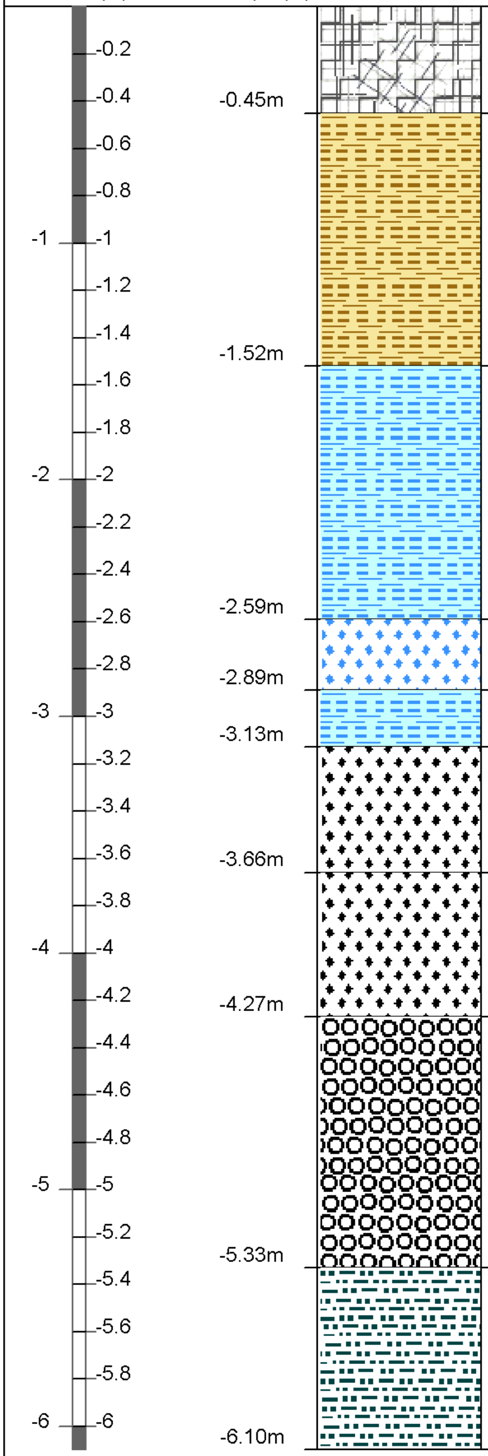
Owner: CCC borelog

Street of Well:	St Asaph St -	**File No:**	
Locality:		**Allocation Zone:**	Christchurch/West Melton
NZGM Grid Reference:	M35:82315-41173 QAR 3		
NZGM X-Y:	2482315 - 5741173		
Location Description:	St Asaph St - 18m west of Nursery Rd	**Uses:**	Foundation/Investigation Bore
ECan Monitoring:			
Well Status:	Filled in		

Drill Date:	12 Nov 1973	**Water Level Count:**	0
Well Depth:	6.10m -GL	**Strata Layers:**	9
Initial Water Depth:	-1.72m -MP	**Aquifer Tests:**	0
Diameter:		**Isotope Data:**	0
		Yield/Drawdown Tests:	0
Measuring Point Ait:	4.30m 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 M35/12156

Gridref: M35:82315-41173 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 4.3 +MSD
Well name : CCC BorelogID 154
Drill Method : Not Recorded
Drill Depth : -6.1m Drill Date : 12/11/1973



Water Level	Depth(m)	Full Drillers Description	Formation Code
	-0.45m	road metal	
	-1.52m	light brown dry clayey silt	
	-2.59m	blue saturated clayey silt	
	-2.89m	blue saturated sand	
	-3.13m	blue wet clayey silt	
	-3.66m	saturated sand	
	-4.27m	saturated running sand	
	-5.33m	saturated gravel	
	-6.10m	saturated sandy silt	

Unknown No: M35/12155

Well Name: CCC BorelogID 153

Owner: CCC borelog

Environment Canterbury
Your regional council

Street of Well: St Asaph St -

File No:

Locality:

Allocation Zone: Christchurch/West Melton

NZGM Grid Reference: M35:82214-41179 QAR 3

NZGM X-Y: 2482214 - 5741179

Location Description: St Asaph St - 122m west of Nursery Rd opposite #462 - north side

Uses: Foundation/Investigation Bore

ECan Monitoring:

Well Status: Filled in

Drill Date: 12 Nov 1973

Water Level Count: 0

Well Depth: 4.57m -GL

Strata Layers: 6

Initial Water Depth: -1.60m -MP

Aquifer Tests: 0

Diameter:

Isotope Data: 0

Yield/Drawdown Tests: 0

Measuring Point Ait: 4.30m 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:

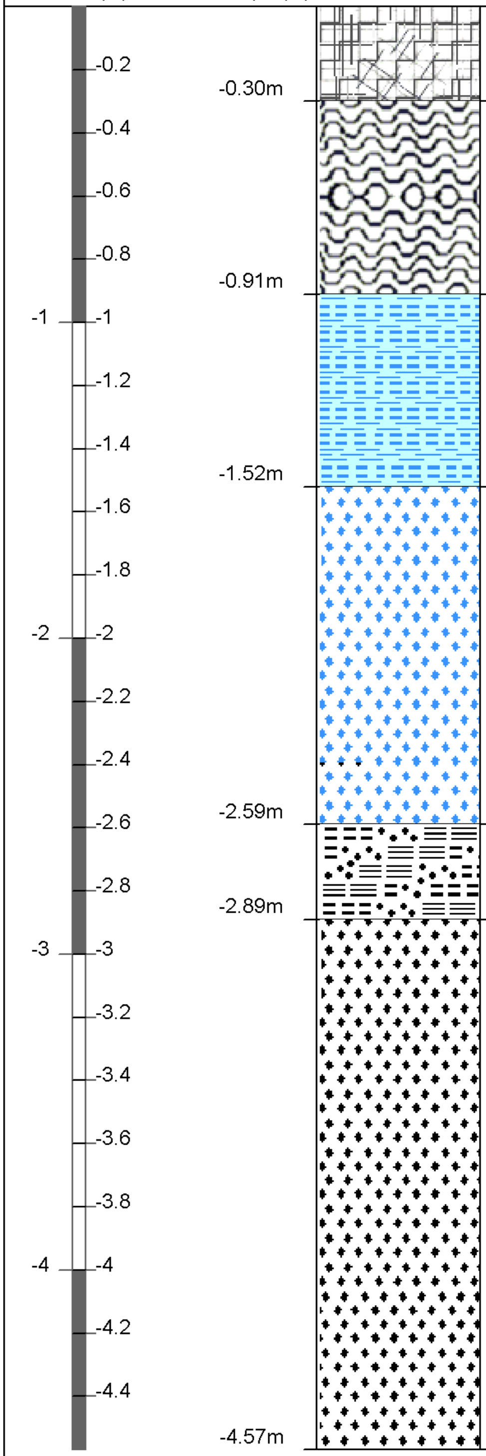
Aquifer Type:

Aquifer Name:

Borelog for well M35/12155

Gridref: M35:82214-41179 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 4.3 +MSD
Well name : CCC BorelogID 153
Drill Method : Not Recorded
Drill Depth : -4.57m Drill Date : 12/11/1973

Environment Canterbury Regional Council



Water Level	Depth(m)	Full Drillers Description	Formation Code
	-0.30m	road metal	
	-0.91m	topsoil	
	-1.52m	blue moist silty clay	
	-2.59m	blue saturated running sand	
	-2.89m	blue wet clayey silt with some sand	
	-4.57m	saturated running sand	

Unknown No: M35/12154
Well Name: CCC BorelogID 152
Owner: CCC borelog

Environment Canterbury
Your regional council

Street of Well: St Asaph St -
Locality:
NZGM Grid Reference: M35:82152-41177 QAR 3
NZGM X-Y: 2482152 - 5741177
Location Description: St Asaph St - opposite #446 - north side
ECan Monitoring:
Well Status: Filled in

File No:
Allocation Zone: Christchurch/West Melton
Uses: Foundation/Investigation Bore

Drill Date: 12 Nov 1973
Well Depth: 4.57m -GL
Initial Water Depth: -1.83m -MP
Diameter:

Measuring Point Ait: 4.30m MSD QAR 3
GL Around Well: 0.00m -MP
MP Description: ToC

Driller:
Drilling Method:
Casing Material:
Pump Type:
Yield:
Drawdown:
Specific Capacity:

Aquifer Type:
Aquifer Name:

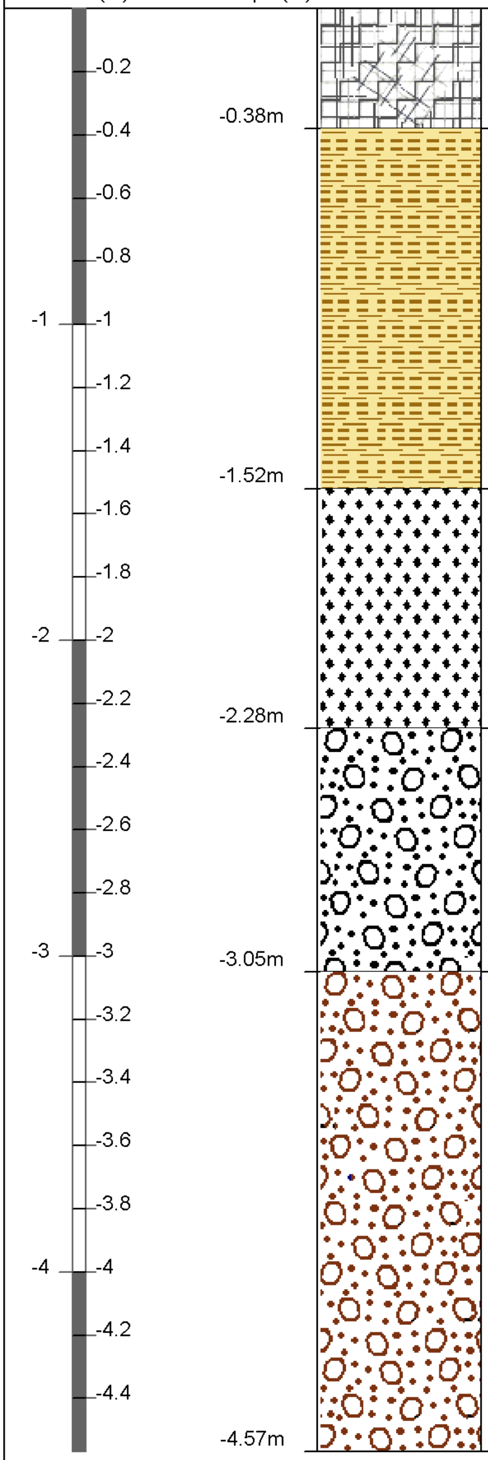
Water Level Count: 0
Strata Layers: 5
Aquifer Tests: 0
Isotope Data: 0
Yield/Drawdown Tests: 0

Highest GW Level:
Lowest GW Level:
First Reading:
Last Reading:
Calc. Min. GWL:
Last Updated: 27 Mar 2008
Last Field Check:

Screens:
Screen Type:
Top GL:
Bottom GL:

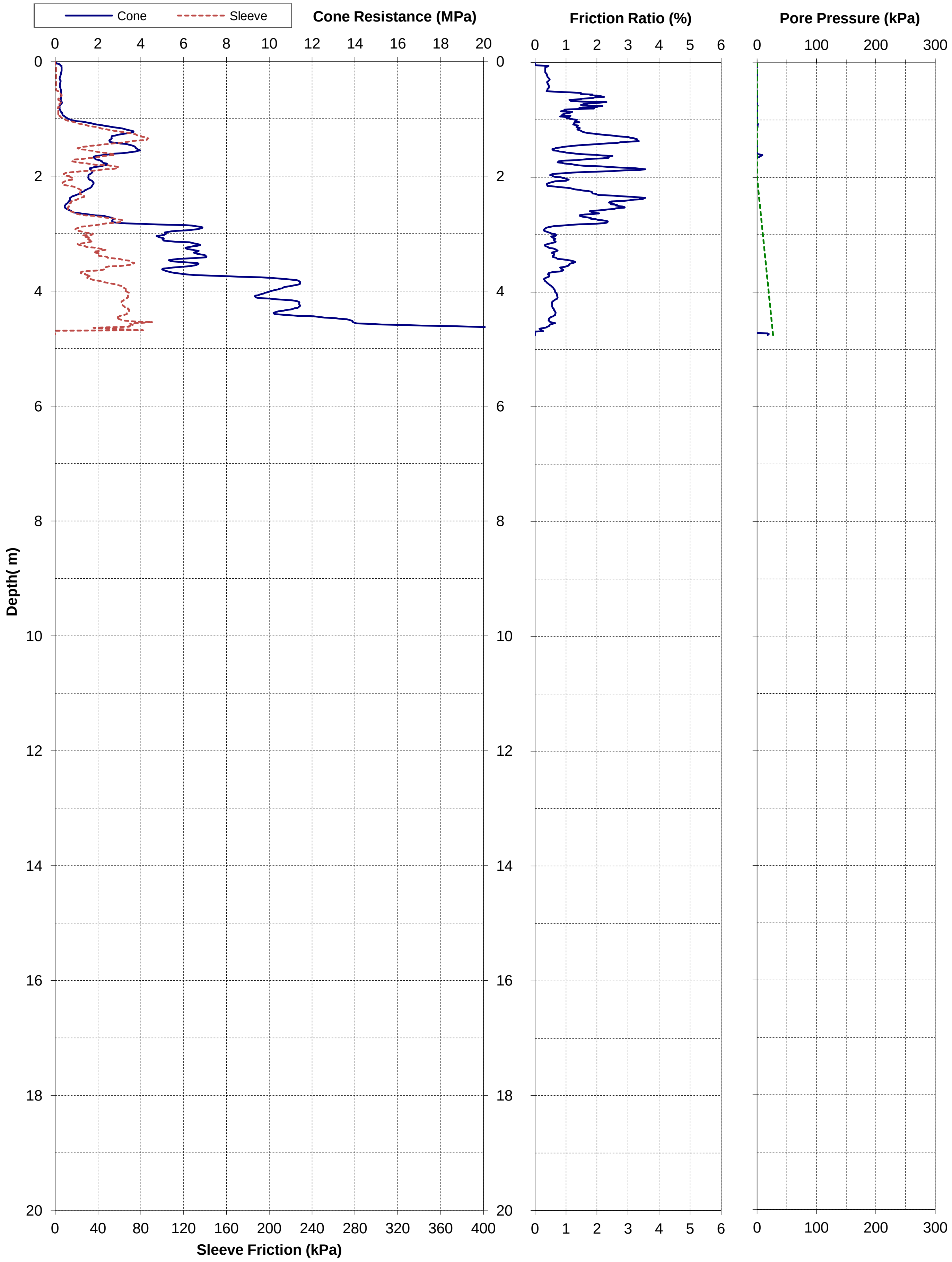
Borelog for well M35/12154

Gridref: M35:82152-41177 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 4.3 +MSD
Well name : CCC BorelogID 152
Drill Method : Not Recorded
Drill Depth : -4.57m Drill Date : 12/11/1973



Depth(m)	Full Drillers Description	Formation Code
-0.38m	road metal	
-1.52m	light brown dry clayey silt	
-2.28m	blue grey wet sand	
-3.05m	saturated sand and gravel	
-4.57m	brown saturated running sand with some gravel	

Project:	Christchurch 2011 Earthquake - EQC Ground Investigations		**Page:**	1 of 1	**CPT-LWD-22**
Test Date:	19-May-2011	**Location:** Linwood	**Operator:**	Opus	
Pre-Drill:	1.2m	**Assumed GWL:** 2mBGL	**Located By:**	Survey GPS	
Position:	2482301.8mE 5741339.2mN	3.46mRL	**Coord. System:**	NZMG & MSL	
Other Tests:			**Comments:**		



CPT ANALYSIS NOTES

Soil Type

Interpretation using chart of Robertson & Campanella (1983). This is a simple but well proven interpretation using cone tip resistance (q_C) and friction ratio (f_R) only. No normalisation for overburden stress is applied. Cone tip resistance measured with the piezocone is corrected with measured pore pressure (u_C).

- 🟥 sand (and gravel)
- 🟨 silt-sand
- 🟩 silt
- 🟦 clay-silt
- 🟦 clay
- ⬛ peat

Liquefaction Screening

The purpose of the screening is to highlight susceptible soils, that is sand and silt-sand in a relatively loose condition. This is not a full liquefaction risk assessment which requires knowledge of the particular earthquake risk at a site and additional analysis. The screening is based on the chart of Shibata and Teparaksa (1988).

- 🟥 high susceptibility
- 🟧 medium susceptibility
- 🟩 low susceptibility

High susceptibility is here defined as requiring a shear stress ratio of 0.2 to cause liquefaction with D_{50} for sands assumed to be 0.25 mm and for silty sands to be 0.05 mm.

Medium susceptibility is here defined as requiring a shear stress ratio of 0.4 to cause liquefaction with D_{50} for sands assumed to be 0.25 mm and for silty sands to be 0.05 mm.

Low susceptibility is all other cases.

Relative Density (D_R)

Based on the method of Baldi et. al. (1986) from data on normally consolidated sand.

Undrained Shear Strength (S_U)

Derived from the bearing capacity equation using $S_U = (q_C - \sigma_{VO})/15$.

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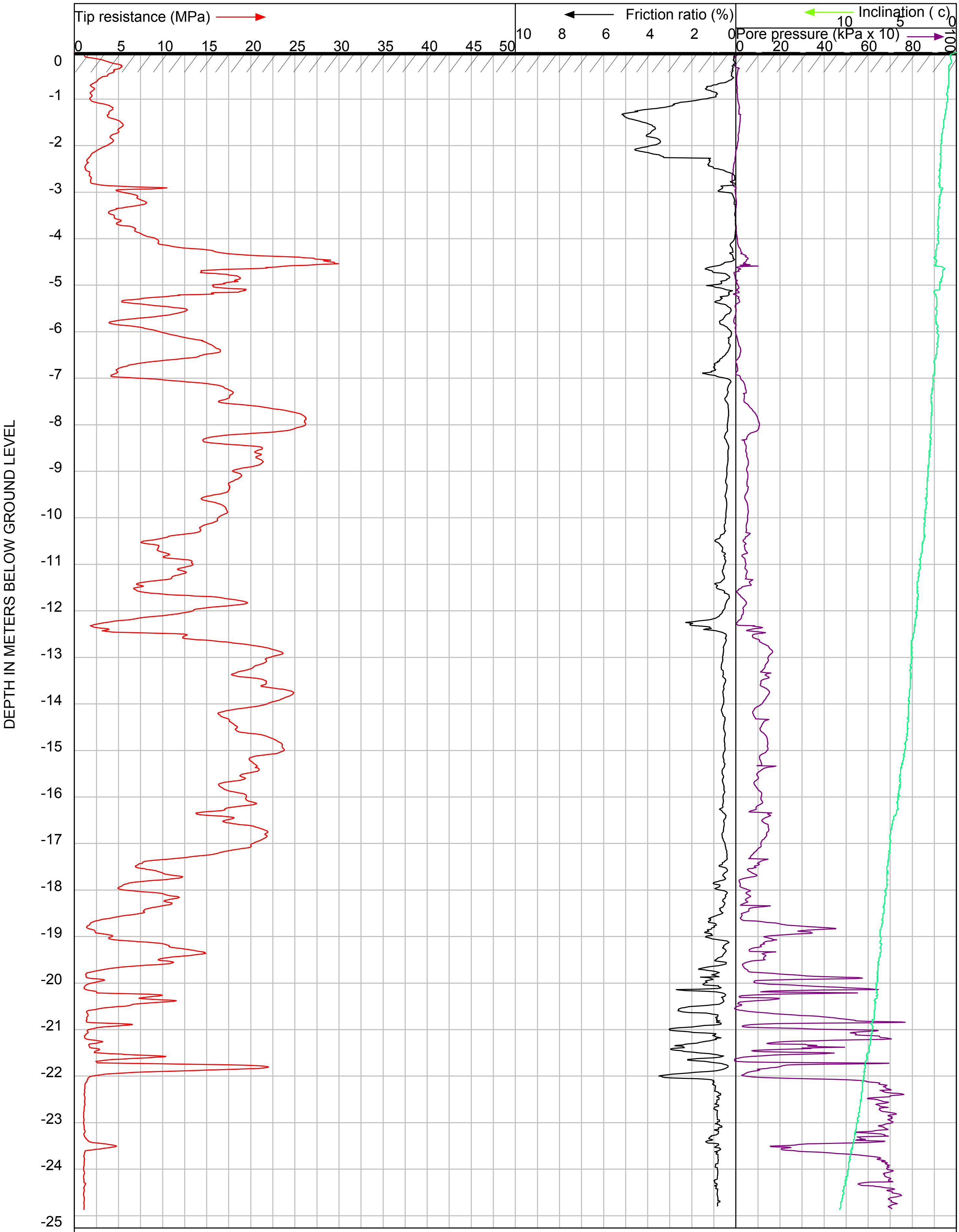
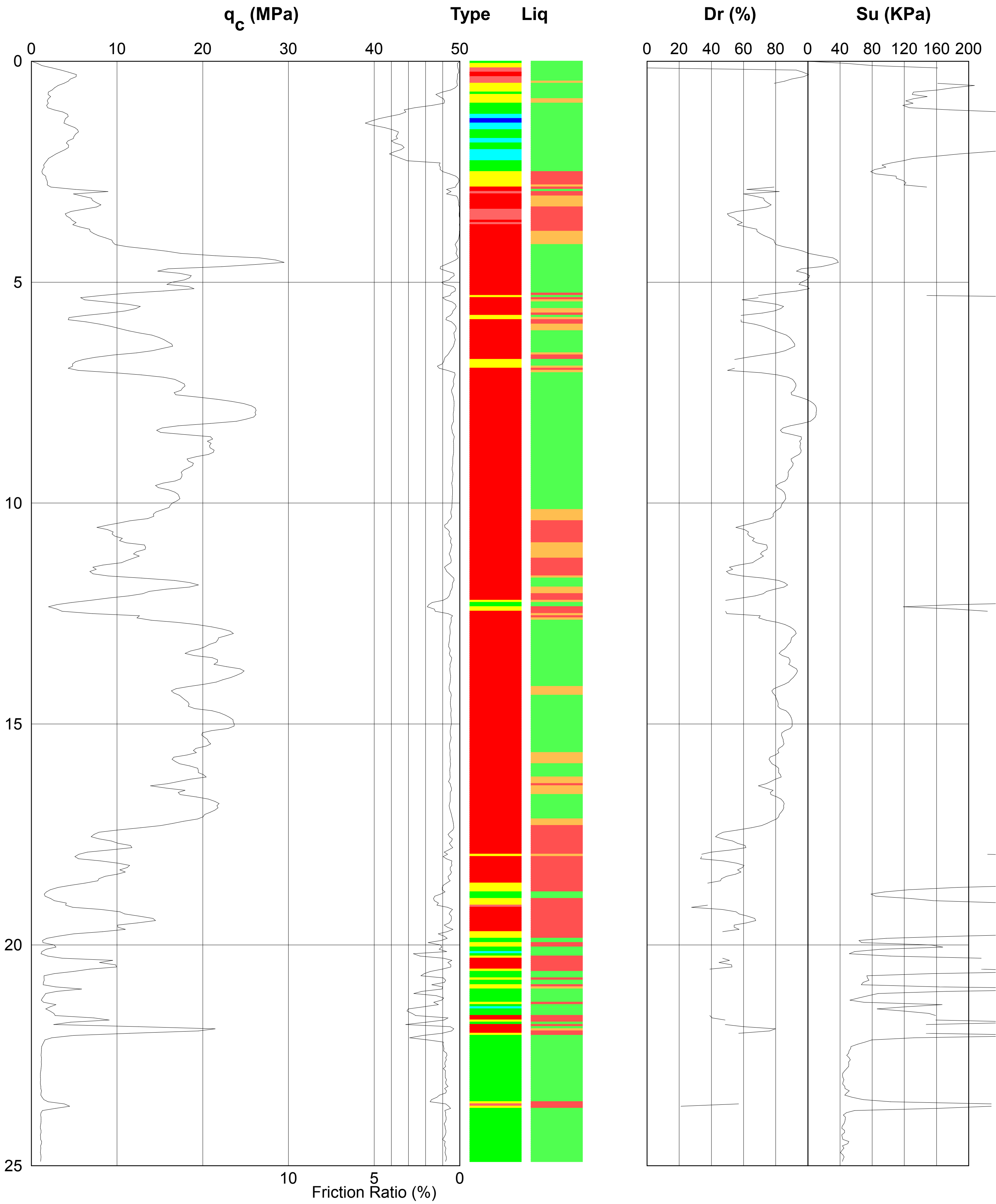


CLIENT : GHD NZ Ltd
LOCATION : 39 Nursery Road, Christchurch
DATE : 27-6-2012
OPERATOR : S.Cardona
REMARK 1 : CPTu001
REMARK 2 : Target Depth

JOB # : 10676
TEST # : 1
CONE TYPE/SERIAL #: I-CFXYP20-10/ 071017T

McMILLAN
DRILLING SERVICES
120 High St Southbridge CANTERBURY NZ
Ph +64 3 324 2571 Fax +64 3 324 2431
www.drilling.co.nz

PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



Job No: 10676

CPT No: CPTu001

Project: GHD NZ Ltd

Location: 39 Nursery Road, Christchurch

Date: 27-6-2012

Operator: S.Cardona

Remark: Target Depth

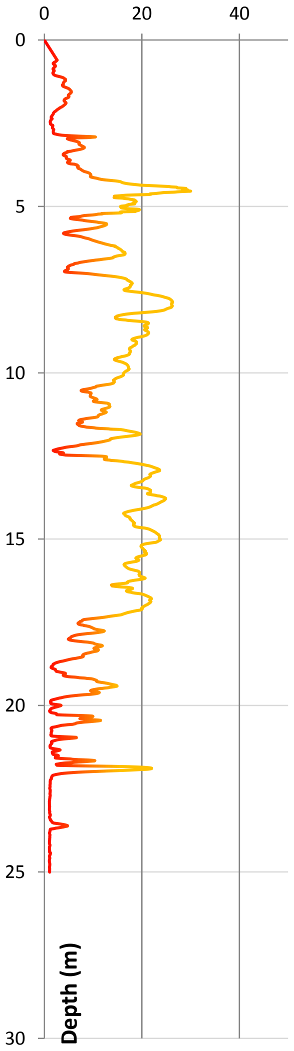
McMILLAN
DRILLING SERVICES

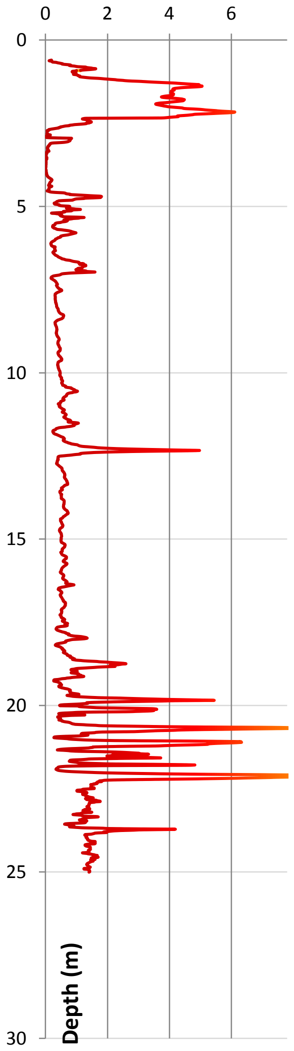
SOIL LIQUEFACTION SUSCEPTIBILITY ASSESSMENT

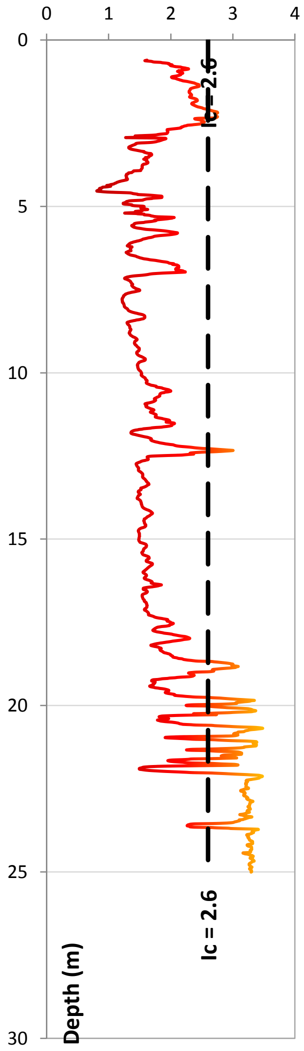
LOCATION :	**CPT 01**	SHEET :	**1 (ULS)**
PROJECT :	**Philpstown Community Centre - 39 Nursery Road**	CALCULATED :	**DW/HNN**
JOB NO :	**51/30596/22**	CHECKED BY :	**JR**
TEST DATE :	**27 Jun 2012**	DATE :	**2 Aug 2012**

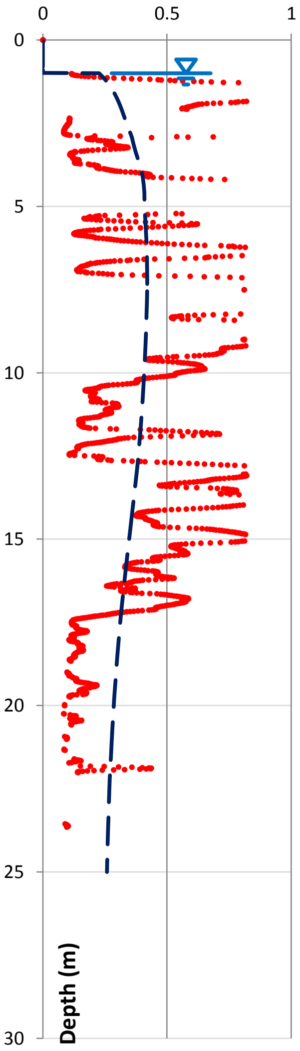
PGA (a_{max}): **0.35 g**
EQ Magnitude: **7.5**
Groundwater Level (m bgl): **1.0**
Atmospheric Pressure (kPa): **101**
Bore depth (m): **25**
Test data step (m): **0.01**

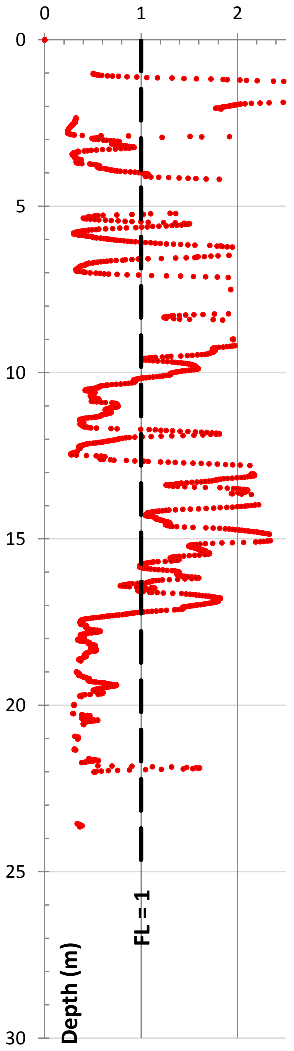
Total Estimated Settlement (mm)
212

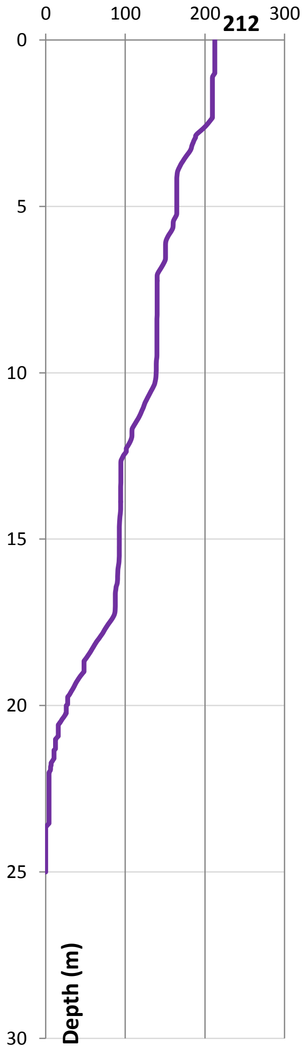

CPT Tip Resistance (MPa)


Friction Ratio (%)


Ic Value


CRR vs CSR


Liquefaction Susceptibility


Settlement (mm)

N:\NZ\Wellington\Projects\51\30596\22 Phillipstown Community Centre\Investigation\CPT Results\Liquefaction and Settlement Analysis CPT 01

SOIL LIQUEFACTION SUSCEPTIBILITY ASSESSMENT

LOCATION : **CPT 01**
PROJECT : **Philipstown Community Centre - 39 N**
JOB NO : **51/30596/22**
TEST DATE : **27 Jun 2012**

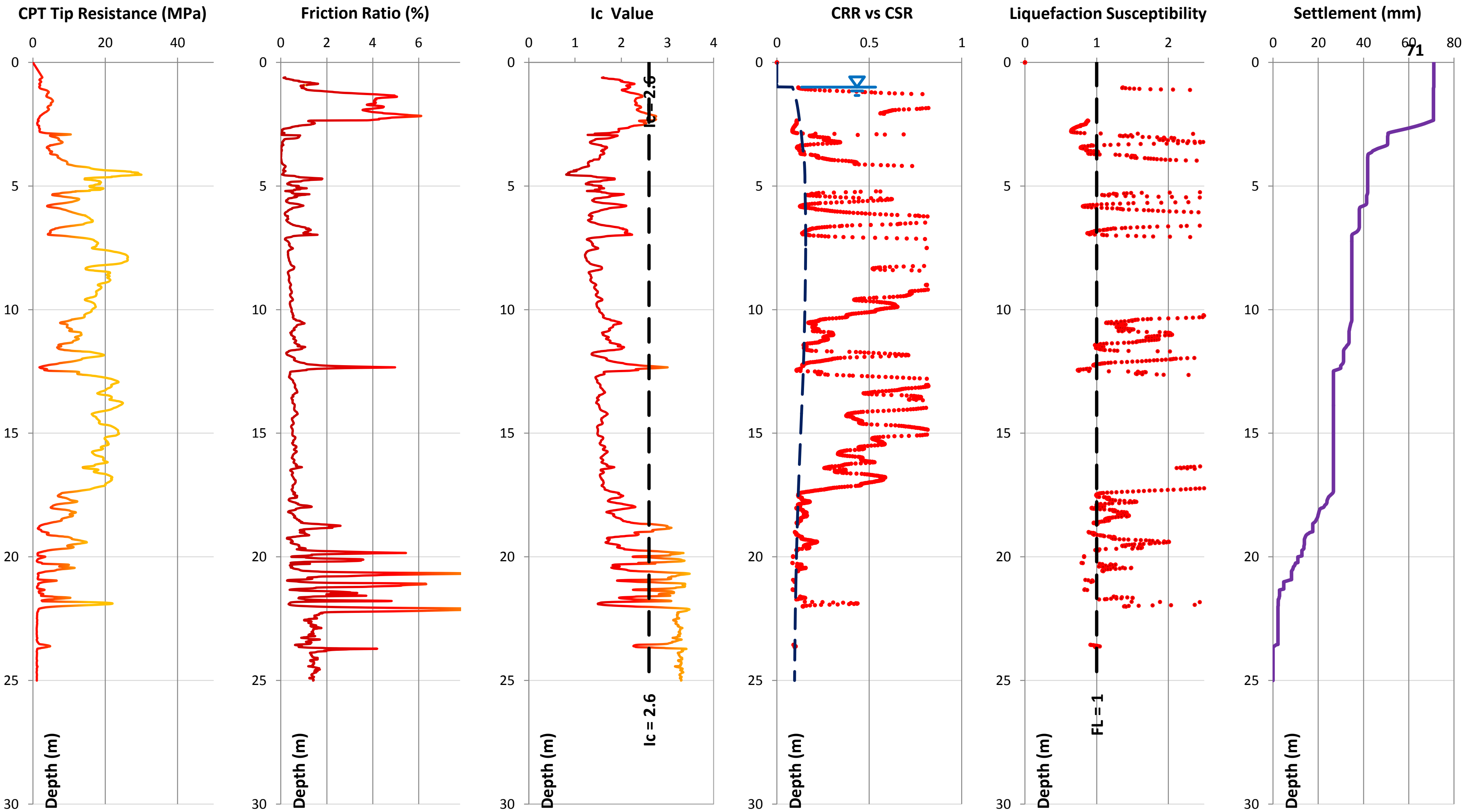
SHEET : **2 (SLS)**
CALCULATED : **DW**
CHECKED BY : **HNN**
DATE : **17 Jul 2012**

PGA (a_{max}): **0.13 g**
EQ Magnitude: **7.5**

Groundwater Level (m bgl): **1.0**
Atmospheric Pressure (kPa): **101**

Bore depth (m): **25**
Test data step (m): **0.01**

Total Estimated Settlement (mm)
71



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GHD Limited

PO Box 13468
Christchurch 8141

HAND AUGER LOG

Site Identification: **HA01**

Sheet 1 of

Project: Phillipstown Community Centre
Client: Christchurch City Council
Site: Phillipstown Community Centre
Job No.: 51/30596/22

Coordinates: **E** 2482 265, **N** 5741 078
Surface RL (m): +3.0m
Commenced: 29-May-12
Completed: 29-May-12
Contractor:

Datum: NZMG
Total Depth: 3.5m

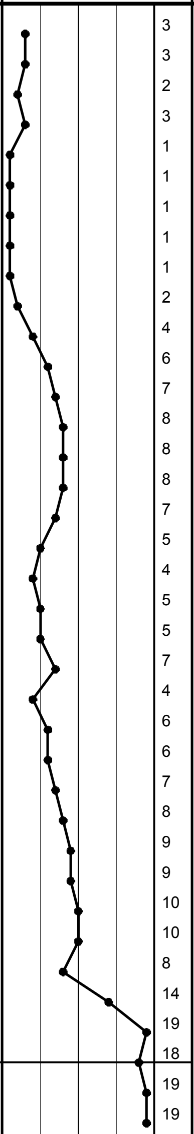
Equipment: 50 mm hand auger
Shear Vane:
Hole Diameter (mm):

Logged:	DBS/DW
Processed:	DBS
Checked:	HN

Depth (m)	Water	Depth (m)/ [Elev.]	Geological Unit	Graphic Log	Classification	SOIL DESCRIPTION: (Soil Code), Soil Name [minor MAJOR], colour, structure [zoning, defects, cementing], plasticity or grain size, secondary components, structure. (Geological Formation)	Moisture Condition	Consistency/ Relative Density	Sample Type & Depth	Sample No.	Sample/ Test Records & Comments	Depth Scale (m)
						Fine SAND with some gravel and traces of bricks; light grey. Moist; gravels, fine to medium, rounded to subrounded, greywacke. FILL	M	'L'				
		0.45 [+2.6]			OL	PEAT, Buried Topsoil; black. Soft; moist.	M	'S'				
		0.65 [+2.4]			ML	SILT; brown. Soft; moist; low to medium plasticity.	M	'S'				
1		1.15 [+1.9]			ML	SILT; light grey. Stiff; moist; low plasticity.	M	'St'				1
2		2.30 [+0.7]			CI	SILT with some clay; light brown. Firm; moist; low to medium plasticity.	M	'F'				2
		2.50 [+0.5]			ML	SILT; grey. Stiff; wet; low plasticity.	W	'St'				
3	▼	3.30 [-0.3]			SM	Silty, medium to coarse SAND; grey. Dense; saturated.	W	D				3
		3.50 [-0.5]				Termination Depth = 3.5m (Collapse)						
4												4
5												5

Test Results

(blows per 100mm)



Blowcounts: 3, 3, 2, 3, 1, 1, 1, 1, 1, 2, 4, 6, 7, 8, 8, 8, 7, 5, 4, 5, 5, 7, 4, 6, 6, 7, 8, 9, 9, 10, 10, 8, 14, 19, 18, 19, 19

GHD Limited
PO Box 13468
Christchurch 8141

HAND AUGER LOG

Site Identification: **HA02**
Sheet 1 of

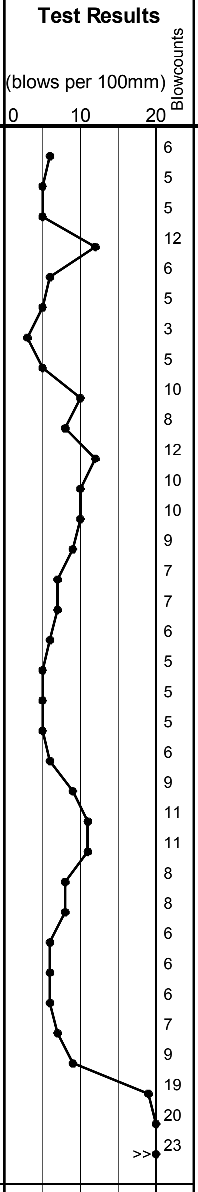
Project: Phillipstown Community Centre
Client: Christchurch City Council
Site: Phillipstown Community Centre
Job No.: 51/30596/22

Coordinates: **E** 2482 261, **N** 5741 089
Surface RL (m): +3.0m
Commenced: 29-May-12
Completed: 29-May-12
Contractor:
Datum: NZMG
Total Depth: 3.5m

Equipment: 50 mm hand auger
Shear Vane:
Hole Diameter (mm):

Logged:	DBS/DW
Processed:	DBS
Checked:	HN

Depth (m)	Water	Depth (m)/ [Elev.]	Geological Unit	Graphic Log	Classification	SOIL DESCRIPTION: (Soil Code), Soil Name [minor MAJOR], colour, structure [zoning, defects, cementing], plasticity or grain size, secondary components, structure. (Geological Formation)	Moisture Condition	Consistency/ Relative Density	Sample Type & Depth	Sample No.	Sample/ Test Records & Comments
					OL	Topsoil; dark brown.	M				
		0.55 [+2.5]			CL	Silty CLAY; light brown. Firm to stiff; moist; medium plasticity.	M	'F'			
1											
2											
		2.50 [+0.5]			ML	SILT, with organic rootlets; dark grey. Stiff; moist; medium plasticity.	M	'St'			
3											
	▼	3.20 [-0.2]			SP	Fine to medium SAND; dark grey. Very dense; saturated. Becomes wet at 3.5m.	W	'VD'			
		3.50 [-0.5]				Termination Depth = 3.5m (Collapse)					
4											
5											



Depth Scale (m): 1, 2, 3, 4, 5

BOREHOLE LOG NZ ALT PCC GINT LOG.GPJ NZ GINT DATA TEMPLATE VER 1.3.GDT 10/4/12

Appendix D

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name: Phillipstown Community Centre

	Unit	No:	Street
Building Address:		39	Nursery Road

Legal Description: Lot 93 DP 38

	Degrees	Min	Sec
GPS south:	43	32	14.19
GPS east:	172	39	24.77

Building Unique Identifier (CCC): BU 2336-001 EQ2

Reviewer: Derek Chinn
CPEng No: 177243
Company: GHD
Company project number: 513059622
Company phone number: (03) 3780900

Date of submission: 8/03/2013
Inspection Date: 19/01/12 and 26/09/2012
Revision: FINAL
Is there a full report with this summary? yes

Site

Site slope: flat
Soil type: silty sand
Site Class (to NZS1170.5): D
Proximity to waterway (m, if <100m):
Proximity to clifftop (m, if < 100m):
Proximity to cliff base (m,if <100m):

Max retaining height (m): 0
Soil Profile (if available):
If Ground improvement on site, describe:
Approx site elevation (m): 13.66

Building

No. of storeys above ground: 1 single storey = 1
Ground floor split? no
Storeys below ground 0
Foundation type: other (describe)
Building height (m): 4.50
Floor footprint area (approx): 117
Age of Building (years): 15

Ground floor elevation (Absolute) (m): 14.35
Ground floor elevation above ground (m): 0.69
if Foundation type is other, describe: Reinforced concrete strip footing to the perimeter of the building with timber anchor piles supporting the floors internally
height from ground to level of uppermost seismic mass (for IEP only) (m): 4.25
Date of design: 1992-2004

Strengthening present? no

If so, when (year)?
And what load level (%g)?
Brief strengthening description:

Use (ground floor): public
Use (upper floors):
Use notes (if required): Used as a community centre
Importance level (to NZS1170.5): IL2

Gravity Structure

Gravity System: load bearing walls
Roof: timber truss
Floors: timber
Beams: none
Columns:
Walls: load bearing timber frame

truss depth, purlin type and cladding 15 Degree Pitch. Trusses @ 900mm c/c, 0.55 coloursteel corrugated roof to roofing underlay over 75 x 50mm purlins
joist depth and spacing (mm) 150*50mm @ 450 c/c
overall depth x width (mm x mm)
thickness (mm) 100

Lateral load resisting structure

Lateral system along: lightweight timber framed walls
Ductility assumed, μ: 0.25
Period along: 0.10 0.00
Total deflection (ULS) (mm):
maximum interstorey deflection (ULS) (mm):

Note: Define along and across in detailed report!

note typical wall length (m) 17
estimate or calculation? estimated
estimate or calculation?
estimate or calculation?

Lateral system across: lightweight timber framed walls
Ductility assumed, μ: 3.00
Period across: 0.40 0.00
Total deflection (ULS) (mm):
maximum interstorey deflection (ULS) (mm):

note typical wall length (m) 6.5
estimate or calculation? estimated
estimate or calculation?
estimate or calculation?

Separations:

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

leave blank if not relevant

Non-structural elements

Stairs:
Wall cladding: other light
Roof Cladding: Metal
Glazing: aluminium frames
Ceilings: fibrous plaster, fixed
Services(list): Electric & Water

describe Lightweight Metal
describe Lightweight Metal

Available documentation

Architectural	partial	original designer name/date	Christopher W. Hadlee, June 1997
Structural	none	original designer name/date	
Mechanical	none	original designer name/date	
Electrical	none	original designer name/date	
Geotech report	partial	original designer name/date	Unknown, Mar 2008

Damage

Site:

(refer DEE Table 4-2)

Site performance:
Describe damage:

Settlement:	25-100mm	notes (if applicable):	
Differential settlement:	0-1:350	notes (if applicable):	South West corner of building settled.
Liquefaction:	5-10 m²/100m³	notes (if applicable):	Severe liquafaction observed at the south west corner.
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:	slight	notes (if applicable):	

Building:

Current Placard Status: green

Along
Damage ratio: 0%
Describe (summary): Settlement of foundations

Describe how damage ratio arrived at:

Across
Damage ratio: 0%
Describe (summary): Settlement of foundations

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

Diaphragms	Damage?: no	Describe:	
CSWs:	Damage?: yes	Describe:	Severe Liquefaction
Pounding:	Damage?: no	Describe:	
Non-structural:	Damage?: no	Describe:	

Recommendations

Level of repair/strengthening required: significant structural and strengthening
Building Consent required: yes
Interim occupancy recommendations: full occupancy

Describe: Re-leveling of the building and strengthening of the foundations
Describe:
Describe:

Along
Assessed %NBS before: 100% 0% %NBS from IEP below
Assessed %NBS after: 100%

If IEP not used, please detail assessment methodology: Bracing analysis (outlined in report)

Across
Assessed %NBS before: 100% 0% %NBS from IEP below
Assessed %NBS after: 100%

GHD

GHD Building
226 Antigua Street, Christchurch 8013
T: 64 3 378 0900 F: 64 3 377 8575 E: chcmail@ghd.com

Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
FINAL	Shashank Kumar	Derek Chinn		Nick Waddington		08/03/2013