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Woolston Crèche
BU 1985-002 EQ2

Detailed Engineering Evaluation
Quantitative Report

Version FINAL

52 Glenroy Street, Woolston

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BU 1985-002 EQ2

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52 Glenroy Street
Woolston
Christchurch

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Woolston Crèche

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

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52 Glenroy Street

Woolston

Christchurch

Background

This is a summary of the Quantitative report for the above building structure, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011; NZS 3604:2011 Timber-Framed buildings; a visual inspection and site measure up carried out on the 13th of August 2012; and a review of drawings and consent documents held by Christchurch City Council.

Brief Description

The Woolston Crèche building is located at 52 Glenroy Street, Woolston. The original structure was constructed in 1985 and served as an office and staff room for an adjoining garage/workshop. The building has undergone alterations in 1996 in the form of demolition of the existing garage/workshop, and extensions to the west and east of the original office space. These alterations were made to convert the garage/workshop with office space to a crèche.

The building is a single storey timber framed structure on slab-on-ground foundation. The roof is pitched and consists of lightweight metal cladding on timber purlins and trusses spanning between external walls. The exterior wall cladding is a lightweight pre-coated aluminium system. The internal wall linings consist of plasterboard to both the timber framed walls and ceilings.

The building is approximately 22m long by 9.5m wide with an internal wall height of 2.4m.

Key Damage Observed

Key damage noted includes:-

- Minor cracking to plasterboard linings around windows and doors.

Critical Structural Weaknesses

The site has liquefaction potential, however due to the nature of the structure (timber framed, single storey structure on slab-on-ground foundation), any settlement as a result of liquefaction is not expected to cause premature collapse of the building.

Indicative Building Strength (from DEE and CSW assessment)

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

As the building has been assessed to have a %NBS greater than 67% NBS, it is not considered to be either an Earthquake Prone or an Earthquake Risk Building. Therefore, based on the Christchurch City Council's policy for earthquake prone buildings no further action is required.

In addition there are no immediate collapse hazards, or any significant critical structural weaknesses associated with the structure, therefore general occupancy of the building is permitted.

1 Background

GHD has been engaged by the Christchurch City Council to undertake a Detailed Engineering Evaluation of the Woolston Crèche building.

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 intrusive site investigation or modelling of the building structure had been carried out. The detailed analysis consisted of a bracing calculation of the structure and a check of the adequacy of the roof bracing to act as a diaphragm. No further analysis or calculations 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	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

Woolston Crèche is located at 52 Glenroy Street, Woolston, Christchurch. The site consists of a crèche building and outdoor play area. The original structure was constructed in 1985 and served as an office and staff room for an adjoining garage/workshop. Demolition of the garage/workshop and extensions to the west and east of the original office space were completed in 1996.

The original structure consists of a single storey timber frame structure with plasterboard lined internal walls. External wall cladding is provided by a lightweight pre-coated aluminium system. The roof structure consists of lightweight timber trussed roof with corrugated external cladding. The entire building is supported by a concrete slab on-ground foundation. The alterations to the building made in 1996 are of similar construction. On the northern wall of the extensions there are significant areas of window and door openings which do not provide bracing. Figure 2 below shows the plan geometry of the building along with key structural elements.

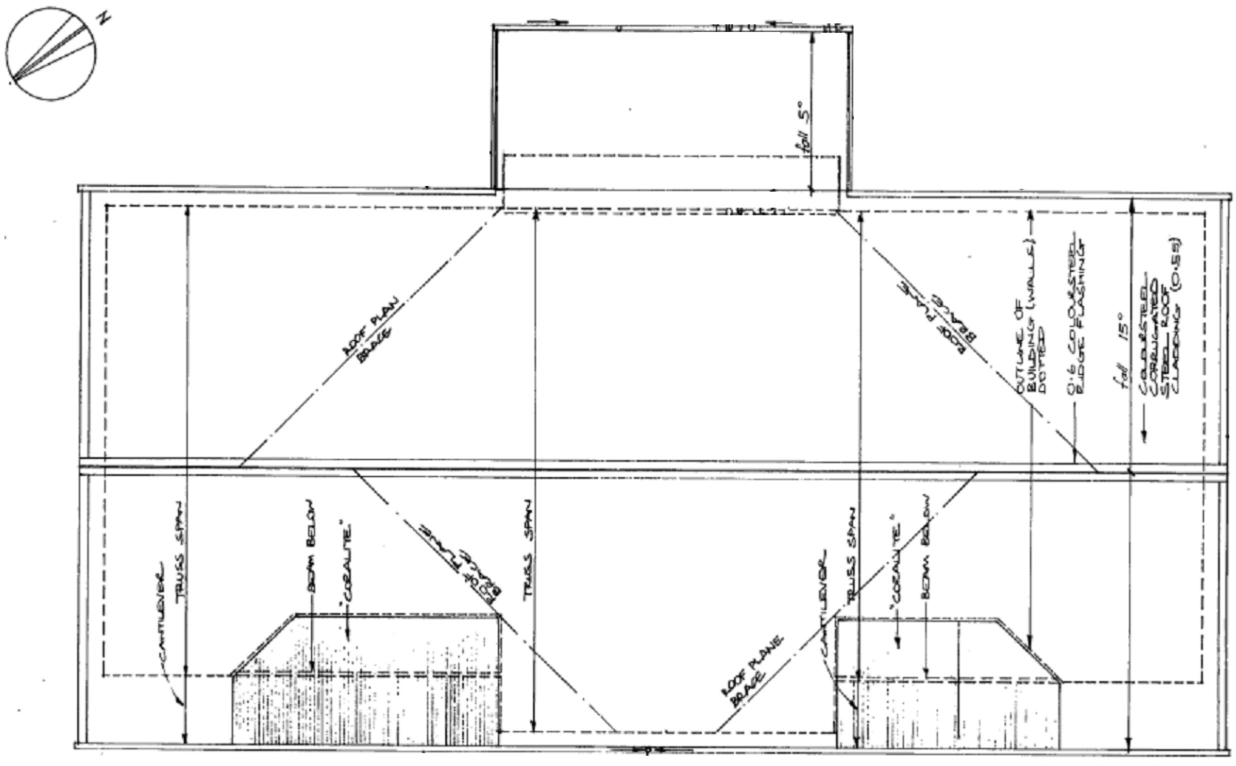


Figure 2 Plan sketch of building showing key structural elements

The dimensions of the crèche are approximately 22m long by 9.5m wide and 4m tall. The overall footprint of the building is approximately 210m^2. The nearest building to the crèche is the retail property approximately 3m to the west. Heathcote River is located approximately 200m to the south of the property. The site is predominantly flat with a gentle slope from the north of the building towards Glenroy Street.

4.2 Gravity Load Resisting System

The gravity loads in the structure are resisted by a timber frame system supporting the roof loads. The frame system supporting the roof consists of timber roof trusses, at 900mm centres, supporting timber purlins. Loads are transferred from the purlins to the roof trusses. Loads are mostly then transferred from the roof trusses to the supporting timber framed external walls. The north facing wall of the building has a high proportion of glazing. Roof loads are transferred through a timber lintel and post system in this wall. Gravity loads are then transferred from the walls and the frame system to foundations.

4.3 Lateral Load Resisting System

Lateral loads acting on the structure are resisted by the plasterboard lined timber framed walls. Lateral loads are transferred from the roof through the purlins and rafters to the internal and external plasterboard lined timber framed walls. The walls then transfer the load into the concrete strip foundations.

In the ceiling/roof plane bracing is provided by a number of mechanisms:

- In the central (original) area of the building the ceiling is a bracing diaphragm which carries the load between the roof structure and the walls and distributes the load to the wall bracing areas. This mechanism is effective for both longitudinal and transverse loads.
- In the eastern and western sections of the building the ceiling diaphragm is not directly connected to any bracing elements on the northern wall. In these areas the roof plane is braced with steel straps. Transverse loads from the roof are expected to be adequately transferred by the purlins through diaphragm action provided by the ceiling and roof linings into the overall bracing system. Longitudinal loads are expected to be transferred by the strap bracing in the roof plane. While the layout of the strap bracing as indicated in the drawings is not ideal we expect that the action of the strap bracing combined with the purlins, cladding and trusses will be sufficient to provide an adequately rigid roof structure to transfer longitudinal loads to the braced wall elements.

The un-braced walls of the building are lined internally with plasterboard which would also provide some additional resistance to the lateral loads in both directions.

5 Damage Assessment

5.1 Surrounding Buildings

Woolston Crèche is located in a semi residential/commercial area with a residential property adjacent to the site on the North West. To the West of the crèche property there are several commercial properties. An above ground water tank is situated on the property to the east of the crèche. The crèche is not connected to any of the buildings situated on the adjacent properties. During inspection of the crèche it was noted that there was extensive liquefaction on the property containing the water tank. There was no noticeable damage to the surrounding buildings.

5.2 Residual Displacements and General Observations

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

Minor cracking was noted to the internal plasterboard lining in several locations throughout the building, primarily above windows and doors. These cracks are not considered significant and have now been repaired.

5.3 Ground Damage

There was no evidence of ground damage on the property, however the tenants indicated that ground damage in the form of destruction of external concrete paving slabs had occurred but had been remediated. It was noted that concrete slabs on the crèche property had been damaged and been replaced prior to inspection. Discussions with the tenants indicated that no liquefaction occurred on the crèche property during the recent seismic events. Liquefaction on the adjacent water tank property to the east had seeped through the boundary fence onto the crèche property but this had been removed prior to inspection. Evidence of liquefaction on the water tank property was noted.

6 Survey

A level survey will not be required as there is no evidence of significant liquefaction or ground settlement. No invasive investigations were carried out on the structure.

7 Geotechnical Investigation

Woolston Crèche is located in Woolston, Christchurch and is accessed from Glenroy Street. The site is predominantly flat with a gentle slope from the north of the building towards Glenroy Street. The Heathcote River is located approximately 200m to the south of the property. 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 sand and silt overbank deposits of the Yaldhurst Member, sub-group of the Springston Formation.

7.1.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates eight boreholes are located within a 75m radius of the site. The logs indicate that the site is underlain by layers of sand, clay, sand and gravels. The logs also indicate that the groundwater is artesian.

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Distance & Direction from Site
M36/1045 & 1046	~79.8m	~22m NE
M36/1030	~134.1m	~30m NE
M36/1056	~135m	~25m NE
M36/9702	~66.4m	~42m NE
M36/1025	~98m	~27m S
M36/5838	~124m	~26m S
M36/5839	~79.5m	~37m S

It should be noted that 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.

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

7.1.3 EQC Geotechnical Investigations

The Earthquake Commission has undertaken geotechnical testing in the area of the site. Information pertaining to this investigation is included in Tonkin and Taylor Report[2]. Within 200 m of the property one investigation point was undertaken, the results of which are detailed below in Table 3.

Table 3 EQC Geotechnical Investigation Summary Table

Bore Name	Grid Reference	Log Summary
CPT – WSW - 45	2484590.2 mE – 5739783.2 mN	0 – 2.0 m Sandy Clay to silty Sand 2.0 – 5.1 m Medium dense coarse Sand 5.1 – 9.7 m Loose to medium dense fine Sand 9.7 – 15.4 m Medium dense coarse Sand 15.4 – 18.0 m Dense coarse Sand 18.0 – 20.0 m Sandy Clay

Initial observations of the CPT results indicate the soils are fine to coarse grained, and are loose to dense with depth.

7.1.4 Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has zoned the site as Green, indicating repair and rebuild may take place.

CERA has published areas showing the Green Zone Technical Category in relation to the risk of future liquefaction and how these areas are expected to perform in future earthquakes.

The site is classified as Technical Category 2 (TC2). The site is at risk from minor to moderate land damage from liquefaction in future significant earthquakes.

7.1.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake (Figure 3 below) shows signs of significant liquefaction outside the building footprint along the road frontage, as outlined in red.

[2] Tonkin and Taylor . September 2011: Christchurch Earthquake Recovery, Geotechnical Factual Report, Woolston

Figure 3 Post February 2011 Earthquake Aerial Photography [3]

7.1.6 Summary of Ground Conditions

The ground conditions as encountered from ECan boreholes and EQC CPT investigation undertaken in vicinity to the site show sandy clay to silty sand from 0.0 to 2.0 m with loose to medium dense material comprising fine to coarse sand from 2.0 m to 9.7 m bgl, becoming denser with depth.

7.2 Seismicity

7.2.1 Nearby Faults

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

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

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	150	8.3	~300 years	150
Greendale (2010) Fault	28	7.1	~15,000 years	28
Hope Fault	113	7.2~7.5	120~200 years	113

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

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

[5] GNS Active Faults Database

Kelly Fault	119	7.2	~150 years	119
Porters Pass Fault	82	7.0	~1100 years	82

Recent earthquakes since 22 February 2011 have identified the presence of a new active fault system / zone underneath Christchurch City and the Port Hills. Research and published information on this system is in development and not generally available. Average recurrence intervals are yet to be estimated.

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 two piezocone/seismic CPT investigations was conducted at the site on 05 April 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 18A	20.0	2491361	5737137
CPT 18B	20.0	2491361	5737137

The CPT investigation was undertaken by McMillans Drilling Ltd on 05 April 2012 to a target depth of 20m below ground level. Please refer to the attached CPT results for detail (Appendix A).

Interpretation of output graphs[6] from the investigation showing Cone Tip Resistance (q_c), Friction Ratio (Fr), Inferred Lithology and Inferred Liquefaction Potential are presented in Table 2.

[6] McMillans Drilling CPT data plots, Appendix A.

7.4 Ground Conditions Encountered

7.4.1 Summary of CPT-Inferred Lithology

Table 6 Summary of CPT-Inferred Lithology

CPT	Depth (m)	Lithology [1]	Cone Tip Resistance q_c (MPa)	Friction Ratio Fr (%)	Relative Density Dr (%)
18 A	0 - 2.5	Pre-drilled			
	2.5 - 3.5	SILT	~2.5	~1.2	(Su = 120 - 200 kPa)
	3.5- 6.8	SAND	~10.0	~0.7	60-90
	6.8 - 9.0	Sandy SILT to clayey SILT	~2.5	~2.0	(Su = 60 - 200 kPa)
	9.0 – 18.2	SAND	~10.0	~0.8	50-80
	18.2 – 20.0	Clayey SILT to silty CLAY	~2.0	~3.0	(Su = 80 - 200 kPa)
18 B	0 – 0.47	Pre-drilled			
	0.47 - 3.5	SILT	~2.5	~1.2	(Su = 120 - 200 kPa)
	3.5- 7.0	SAND*	~10.0	~0.7	60-90
	7.0 – 10.4	Sandy SILT to clayey SILT	~2.5	~2.0	(Su = 60 - 200 kPa)
	10.4 – 18.0	SAND	~10.0	~0.8	50-80
	18.0 – 20.0	Clayey SILT to silty CLAY	~2.0	~3.0	(Su = 80 - 200 kPa)

*one ~0.4m thick layer of clay occurred at 5m depth.

7.5 Interpretation of Ground Conditions

7.5.1 Liquefaction Assessment

Assumptions made for the analysis process are as follows:

- D50 particle sizes for the site soil (sands) from CPT soil analysis
- Hazard factor for Christchurch Z = 0.30
- Importance Level 2, post seismic event (50-year design life)- R = 1.0
- Spectral shape factor C = 1.12 (for class D, E)
- PGA a_h=Z· R· C = 0.30 · 1.0 ·1.12 = 0.336

The following equation has been used to approximate soil unit weight from the CPT investigation data: [7]

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

This obtained unit (saturated) unit weight is 17.0-19.5 kN/m^3 (saturated).

The liquefaction analysis process has been conducted using the methodology from Stark & Olson[8], and from the NZGS Guidelines[9].

7.5.2 Results of Liquefaction Analysis

The results of the liquefaction analysis, as outlined in Table 7, indicate that depths of 2.5-3.5m and 7.0-18.0 are considered low to severe liquefiable.

Table 7 Summary of Liquefaction Susceptibility

Depth (m)	Lithology	Triggering Factor F_L	Liquefaction Susceptibility [10]
0 – 2.5		5.0	Not Liquefiable
2.5 – 3.5	SILT	0.50-0.73	Severe
3.5 ~ 7.0	SAND*	~1.0	Low
7.0 ~ 10.0	Sandy SILT to clayey SILT	0.37-0.53	Severe
10.0 – 18.0	SAND	0.46-1.19	Moderate to severe
18.0 – 20.0	Clayey SILT to silty CLAY	5.0	Not Liquefiable

7.5.3 Interpretation of Analysis

Overall, the site is considered to be severely susceptible to liquefaction based on:

- The presence of sand in vicinity to the site
- Post-earthquake aerial photography evidence
- Liquefaction assessment indicating that the strata between 2.5m to 3.5 m and 7.0m to 10.0m is severely liquefiable.
- TC2 Classification

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

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

[9] 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

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

- Given the site's proximity to two watercourses it is considered possible that lateral spreading could occur.

Please refer to Appendix C for further detail.

7.5.4 Slope Failure and/or Rockfall Potential

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

7.5.5 Foundation Recommendations

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

- The soil class of D (in accordance with NZS 1170.5:2004) recommended in Section 8 of the Qualitative DEE is still believed to be appropriate; and,
- Any remedial works to foundations (or proposed new structures) be undertaken in accordance with DBH and CERA guidelines; and,
- All repairs to and proposed new foundations be specifically-designed by a suitably qualified and experienced geotechnical engineer.

8 Seismic Capacity Assessment

8.1 Qualitative Assessment

An initial qualitative assessment has been completed by GHD for the crèche building. This included a visual inspection of the building which was undertaken on 19th January 2012. Both the interior and exterior of the building were inspected. The main structural components of the roof of the building were all able to be viewed due to the exposed nature of the structure. No inspection of the foundations of the structure was able to be undertaken.

The visual inspection consisted of scrutinising the building 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 and building documentation 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. The capacity of the building was assessed to be 34% NBS taking into account critical structural weaknesses in the form of 'Plan Irregularity' and 'Liquefaction Potential.' Without factoring in these critical structural weaknesses, the building capacity was assessed to be 69% NBS.

8.2 Quantitative Assessment

A Quantitative Assessment of the building was carried out using the information gathered from a full site measure of the building on the 13th of August 2012. Relevant information from the drawings, specifications and the bracing design analysis included in the building consent documents submitted in 1995 for the conversion of the crèche was also used. 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 5.10. In accordance with Table 5.10 of NZS 3604:2011 for a light roof, light wall cladding and slab-on-ground foundation a bracing demand of 6 BU/m^2 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 5.10 of NZS 3604:2011. Therefore the total bracing demand for the building is;

$$\mathrm{BU_{demand}} = (0.8 \times 6\ \mathrm{BU/m^2} \times 207\mathrm{m^2})$$

$$= 994\ \mathrm{BU}$$

8.2.2 Wall bracing capacity

A bracing design analysis has been carried out and submitted with the consent documents in 1995 for the extensions to the building. Therefore the bracing provided by the existing wall linings that have been specifically detailed as bracing walls has been determined in accordance with the GIB Bracing Systems product manual published in 1994.

Walls that are not specifically detailed as bracing walls in the 1996 extensions to the building have been included in the calculation of bracing capacity using strength values from Table 11.1 of the NZSEE guidelines. For this purpose, the strength value of gypsum wall board (3kN/m) was converted to equivalent bracing units (1kN = 20BU) and then multiplied by the strength reduction factor of 0.7. Therefore the bracing capacity for internal walls not detailed as bracing walls is taken to be;

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

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 greather 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;

$$\text{Aspect ratio factor} = \frac{\text{2x Width}}{\text{Height}}$$

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 bracing capacities along and across the building are shown in Table 8.

Table 8 Bracing Units Provided

Direction	Bracing Units Provided
Along the building	1057BU's
Across the building	1549BU's

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;

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

The %NBS for both along and across the building is presented in Table 9.

Table 9 %NBS

Direction	%NBS
Along the building	>100%
Across the building	>100%

Following a detailed assessment the building has been assessed as having a seismic capacity >100% New Building Standard (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 34%NBS value determined in the initial Qualitative Assessment. This is due to a more accurate bracing design analysis performed to determine the capacity of the structure. Further, after a more detailed analysis of the structure, it was determined that the Critical Structural Weaknesses identified in the initial Qualitative Assessment are not significant. It was determined that the high proportion of glazing on the northern face of the building would not present a plan irregularity as the roof bracing provided would adequately transfer the lateral loads to the braced perimeter walls in the middle of the building's northern face. The liquefaction potential was also considered insignificant as any liquefaction induced settlement is not expected to cause a premature collapse of a single storey, timber framed structure on slab-on-ground foundation.

The building has a strength greater than 67% NBS and therefore is not deemed to be earthquake prone or earthquake risk. Therefore, based on the Christchurch City Council's policy for earthquake prone buildings no further action is required.

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

As the building has been assessed to have a %NBS greater than 67%NBS, it is not considered to be either an Earthquake Prone building or an Earthquake Risk. Therefore, based on the Christchurch City Council's policy for earthquake prone buildings no further action is required.

In addition there are no immediate collapse hazards, or critical structural weaknesses associated with the structure, therefore general occupancy of the building is permitted.

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 North-east elevation.

Photograph 2 North-west elevation.

Photograph 3 Cracking at door frame of staff room

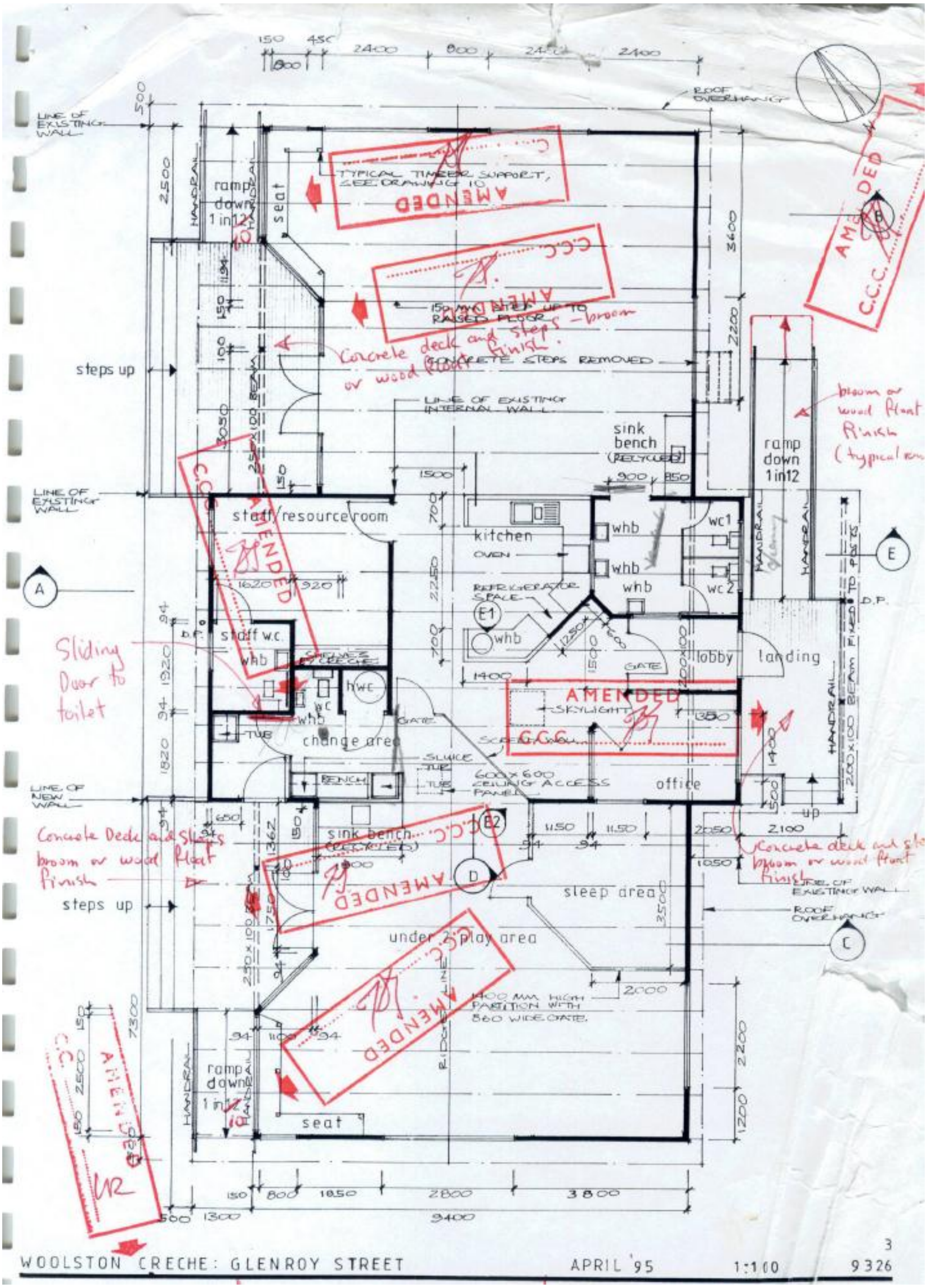
Photograph 4 Exposed timber roof trusses at eastern end of building.

Photograph 5 View of roof structure concealed by ceiling.

Photograph 6 Area of paving being replaced at south side of building.

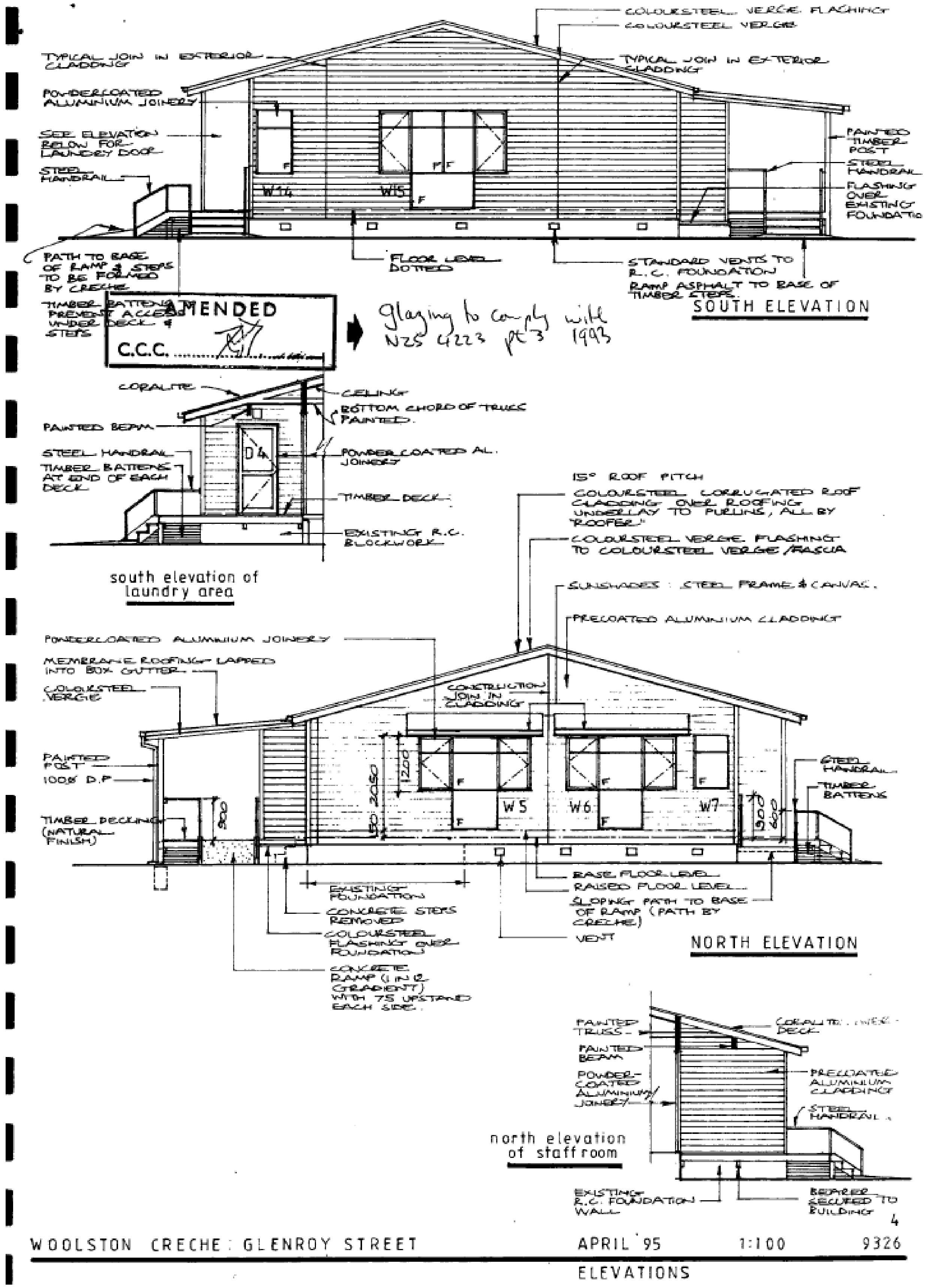
Appendix B

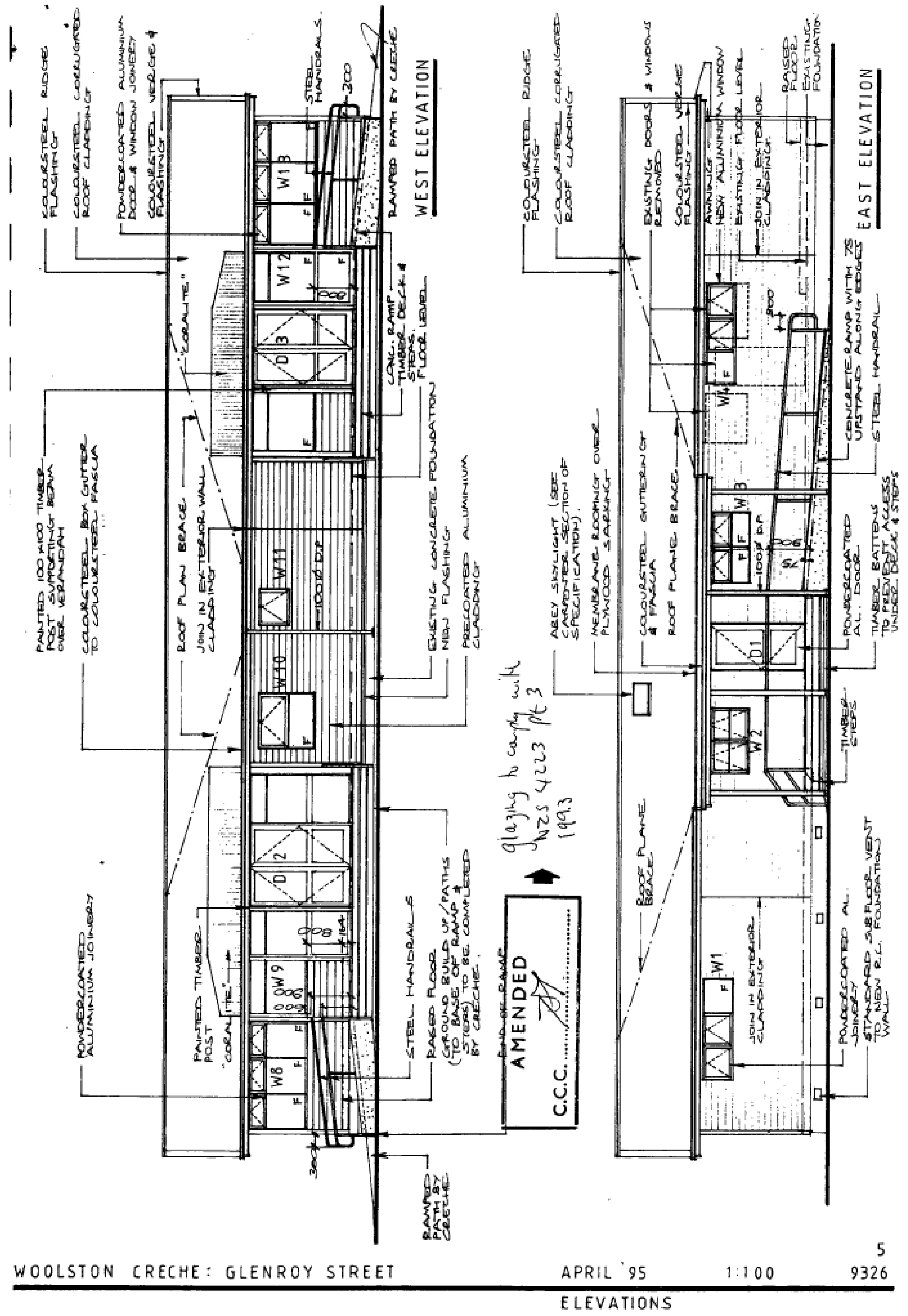
Existing Drawings



Detailed Engineering Evaluation

BU 1985-002 EQ2 Woolston Creche DEE Quantitative Report Final


COLOURSTEEL VERGE FLASHING
COLOURSTEEL VERGE
TYPICAL JOIN IN EXTERIOR CLADDING
TYPICAL JOIN IN EXTERIOR CLADDING
POWDERCOATED ALUMINIUM JOINERY
SEE ELEVATION BELOW FOR LAUNDRY DOOR
STEEL HANDRAIL
PAINTED TIMBER POST
STEEL HANDRAIL
FLASHING OVER EXISTING FOUNDATION
W14
W15
PATH TO BASE OF RAMP & STEPS TO BE FORMED BY CRECHE
FLOOR LEVEL DOTTED
STANDARD VENTS TO R.C. FOUNDATION
RAMP ASPHALT TO BASE OF TIMBER STEPS
TIMBER BATTENS PREVENT ACCESS UNDER DECK & STEPS
AMENDED
C.C.C.
glazing to comply with NZS 4223 pt 3 1993
SOUTH ELEVATION
CORALITE
CEILING
BOTTOM CHORD OF TRUSS PAINTED
PAINTED BEAM
STEEL HANDRAIL
TIMBER BATTENS AT END OF EACH DECK
D4
POWDER COATED AL. JOINERY
TIMBER DECK
EXISTING R.C. BLOCKWORK
south elevation of laundry area
15° ROOF PITCH
COLOURSTEEL CORRUGATED ROOF CLADDING OVER ROOFING UNDERLAY TO PURLINS, ALL BY "ROOFER"
COLOURSTEEL VERGE FLASHING TO COLOURSTEEL VERGE/FASCIA
SUNSHADES: STEEL FRAME & CANVAS.
PRECOATED ALUMINIUM CLADDING
POWDERCOATED ALUMINIUM JOINERY
MEMBRANE ROOFING LAPPED INTO BOX GUTTER
COLOURSTEEL VERGE
CONSTRUCTION JOIN IN CLADDING
PAINTED POST
100Ø D.P.
TIMBER DECKING (NATURAL FINISH)
STEEL HANDRAIL
TIMBER BATTENS
1200
2050
900
W5
W6
W7
EXISTING FOUNDATION
CONCRETE STEPS REMOVED
COLOURSTEEL FLASHING OVER FOUNDATION
CONCRETE RAMP (1 IN 12 GRADIENT) WITH 75 UPSTAND EACH SIDE.
BASE FLOOR LEVEL
RAISED FLOOR LEVEL
SLOPING PATH TO BASE OF RAMP (PATH BY CRECHE)
VENT
NORTH ELEVATION
PAINTED TRUSS
PAINTED BEAM
POWDER-COATED ALUMINIUM JOINERY
CORALITE OVER DECK
PRECOATED ALUMINIUM CLADDING
STEEL HANDRAIL
north elevation of staff room
EXISTING R.C. FOUNDATION WALL
BEARER SECURED TO BUILDING
4
WOOLSTON CRECHE : GLENROY STREET
APRIL '95
1:100
9326
ELEVATIONS


WEST ELEVATION
EAST ELEVATION
AMENDED
C.C.C.
WOOLSTON CRECHE: GLENROY STREET
APRIL '95
1:100
5
9326
ELEVATIONS

Appendix C

Geotechnical Information

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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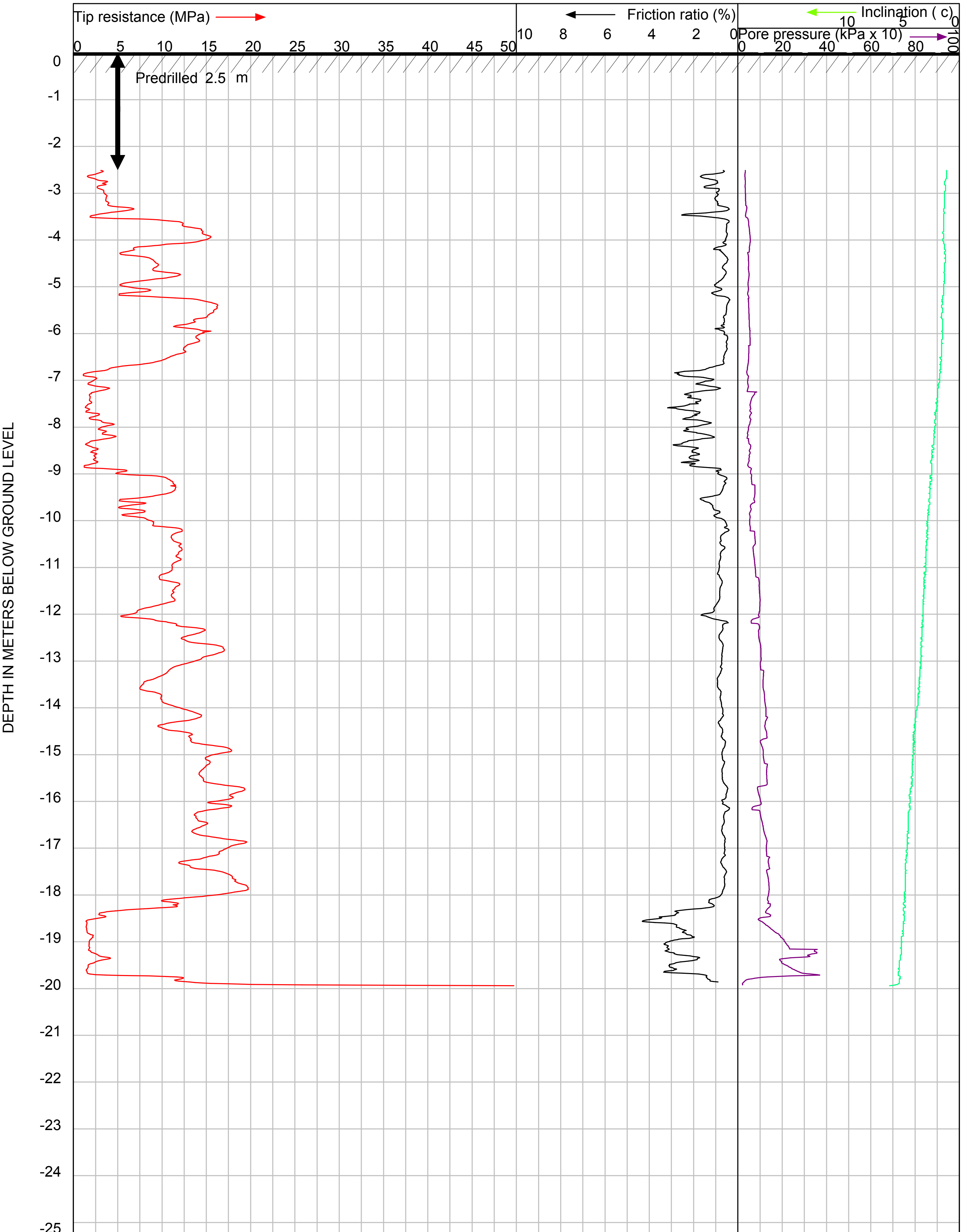
DRILLING SERVICES

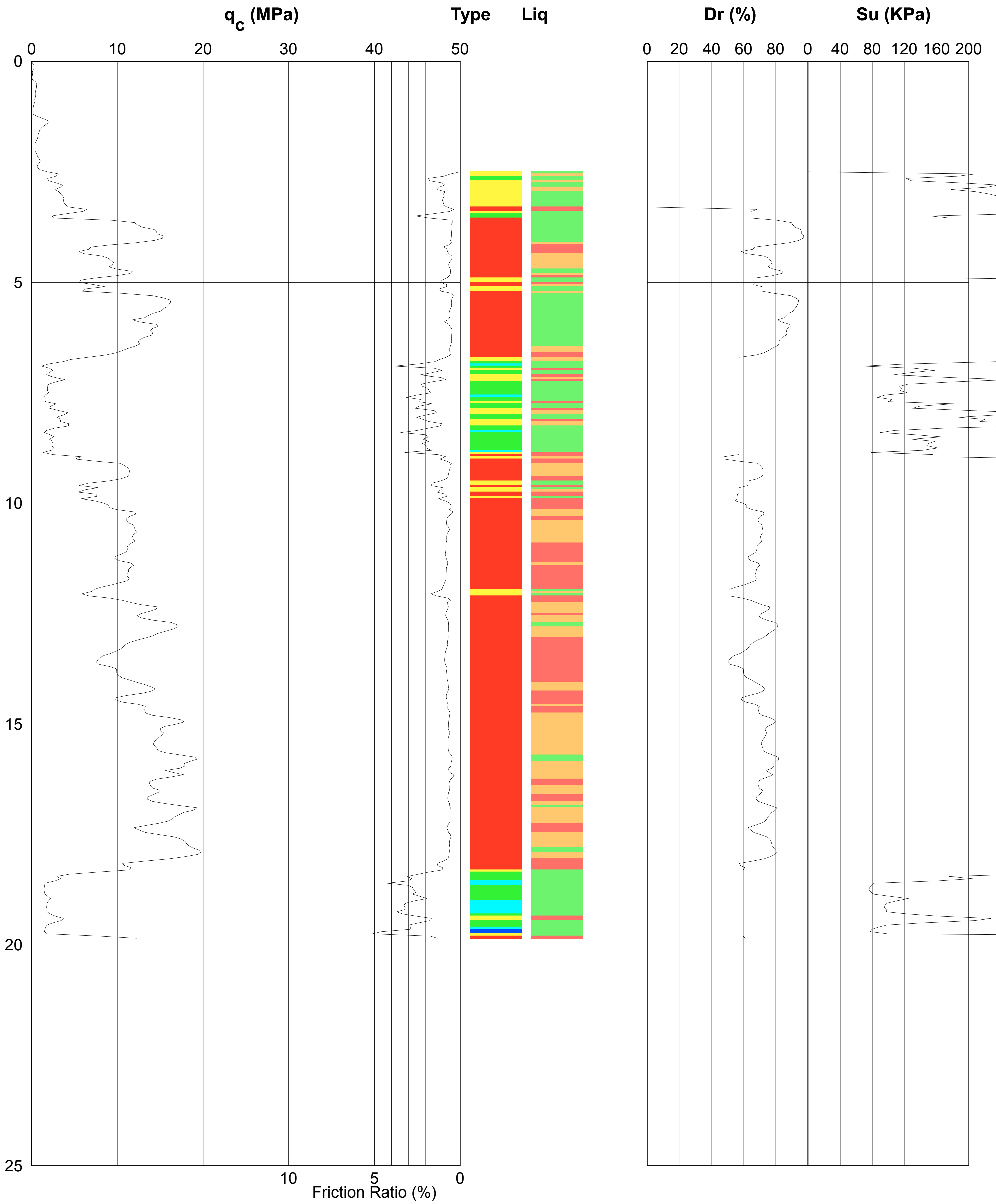


CLIENT : GHD
LOCATION : Christchurch Various (CCC Properties)
DATE : 5-4-2012
OPERATOR : C. Nee
REMARK 1 : CPTu18/A
REMARK 2 :

JOB # : 10386
TEST # : 18

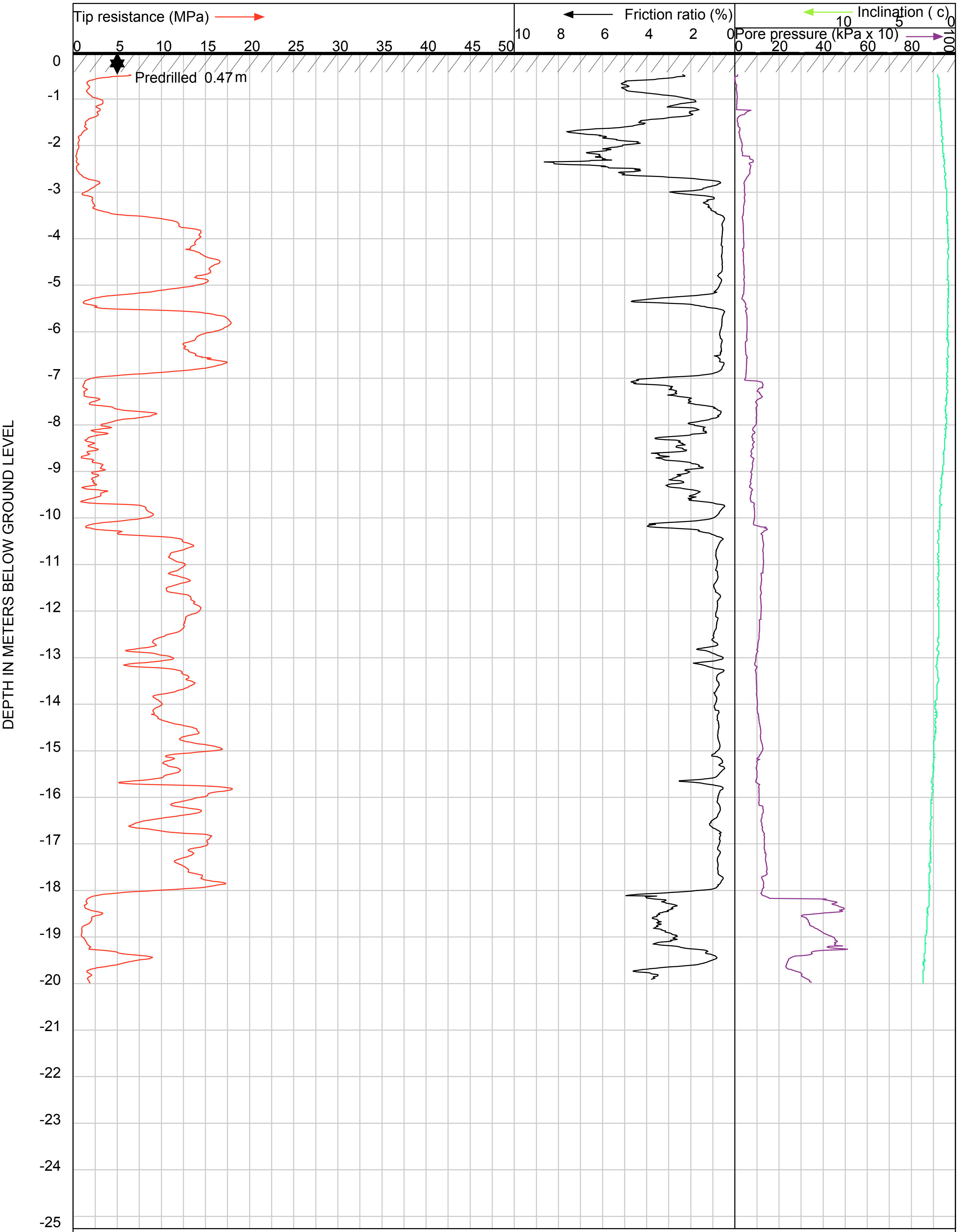
McMILLAN DRILLING SERVICES
120 High St Southbridge CANTERBURY NZ
Ph +64 3 324 2571 Fax +64 3 324 2431
www.drilling.co.nz

PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



Job No:	10386	Date:	05/04/2012
CPT No:	CPTu 18/A	Operator:	C. Nee
Project:	GHD	Remark:	Effective Refusal
Location:	Christchurch Various (CCC Properties)		

McMILLAN DRILLING SERVICES



CLIENT : GHD
LOCATION : Christchurch Various (CCC Properties)
DATE : 11-4-2012
OPERATOR : H. Pardoe
REMARK 1 : CPTu18/B
REMARK 2 : Target Depth

JOB # : 10386
TEST # : 118

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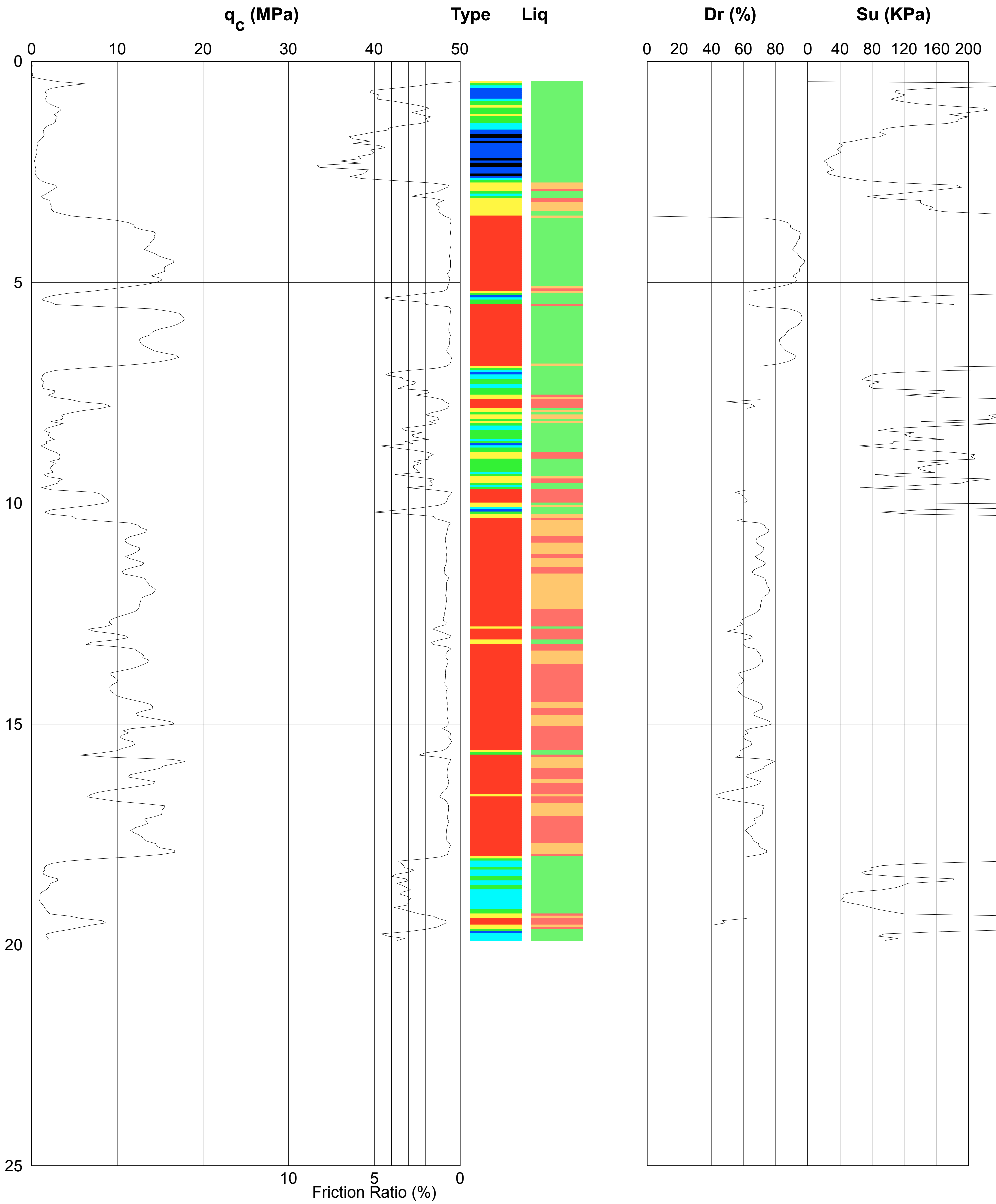
PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



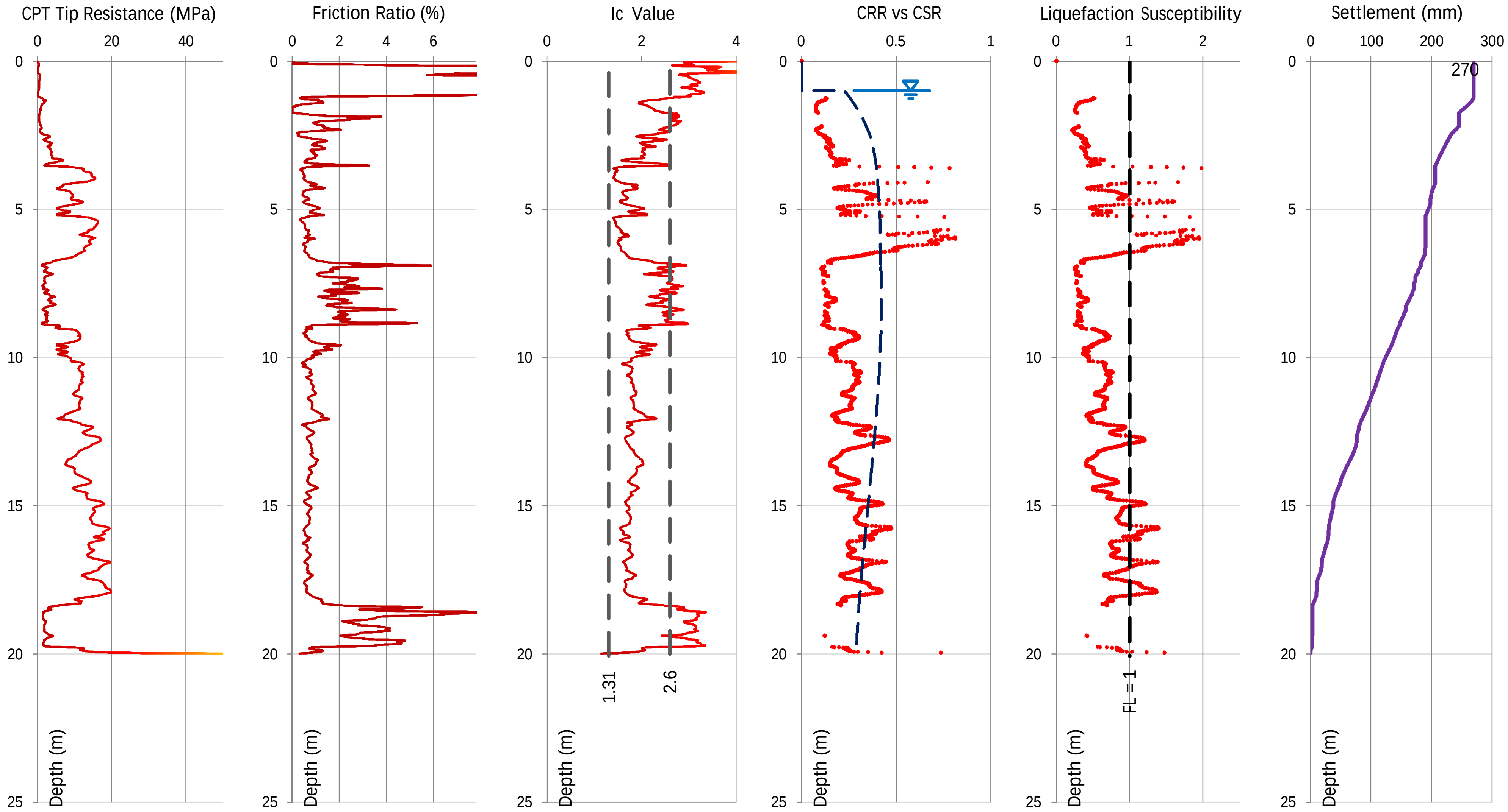
Job No:	10386	Date:	11-4-2012
CPT No:	CPTu18/B	Operator:	H. Pardoe
Project:	GHD	Remark:	Target Depth
Location:	Christchurch Various (CCC Properties)		

MCMILLAN DRILLING SERVICES

SOIL LIQUEFACTION SUSCEPTIBILITY ASSESSMENT

LOCATION :	CPT 18A	SHEET :	1
PROJECT :	Woolston Creche	CALCULATED :	DBS
JOB NO :	51 30596 18	CHECKED BY :	
TEST DATE :	5 Apr 2012	DATE :	23/8/2012

PGA (a_{max}):	0.35 g	Groundwater Level (m bgl):	1.0	Bore depth (m):	19.98	Total Estimated Settlement (mm)
EQ Magnitude:	7.5	Atmospheric Pressure (kPa):	101	Test data step (m):	0.01	270



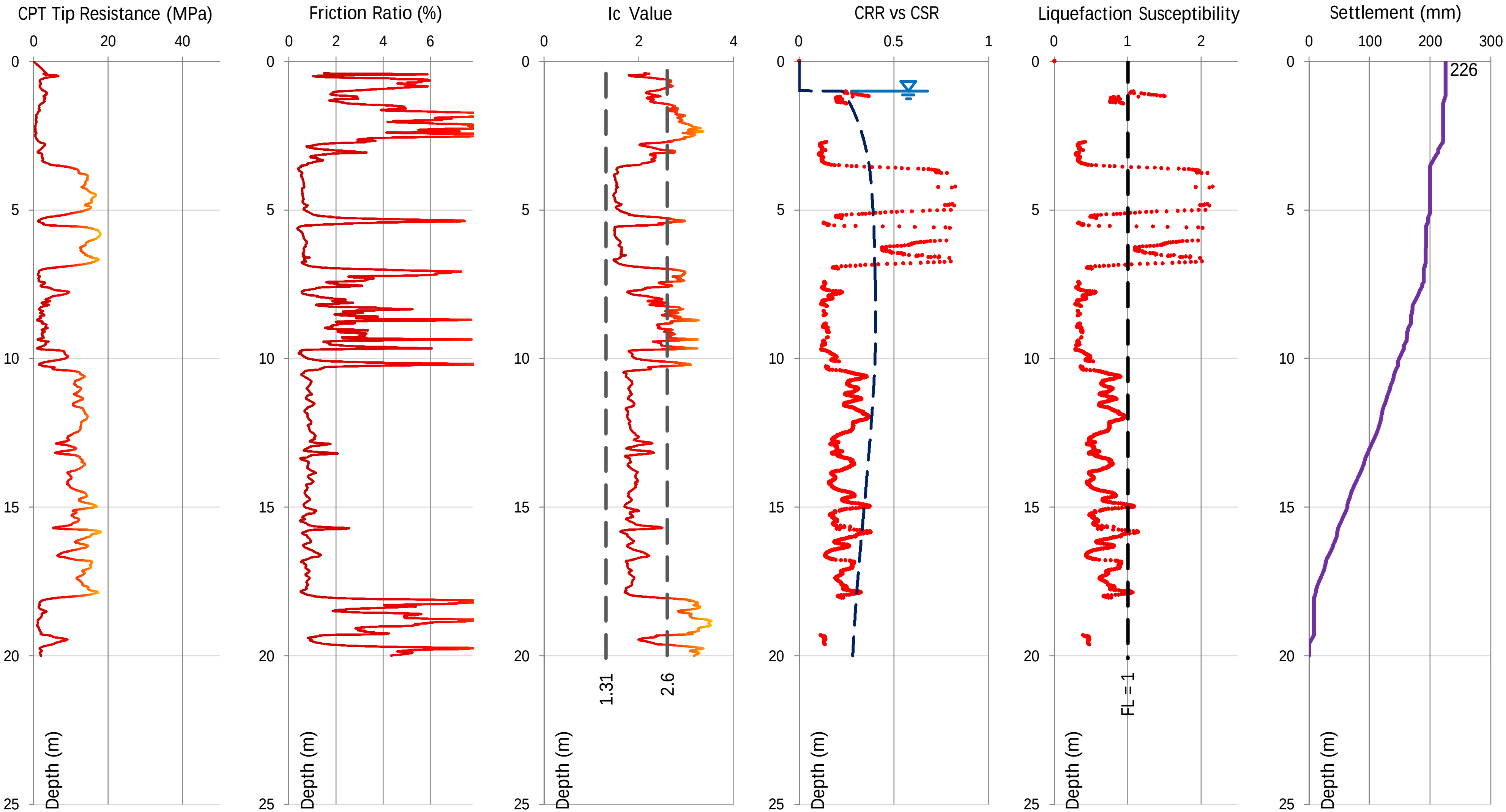
SOIL LIQUEFACTION SUSCEPTIBILITY ASSESSMENT

LOCATION :	CPT 18B	SHEET :	2
PROJECT :	Woolston Creche	CALCULATED :	DBS
JOB NO :	51 30596 18	CHECKED BY :	
TEST DATE :	11 Apr 2012	DATE :	23/8/2012

PGA (a_{max}):	0.35 g	Groundwater Level (m bgl):	1.0	Bore depth (m):	20
EQ Magnitude:	7.5	Atmospheric Pressure (kPa):	101	Test data step (m):	0.01

Total Estimated Settlement (mm)

226



Appendix D

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Woolston Creche	Reviewer:	Derek Chinn
Building Address: Unit / No: / Street	52 Glenroy Street	CPEng No:	177243
Legal Description:	Lot1, D.P. 63343	Company:	GHD
		Company project number:	513059618
		Company phone number:	(03) 378 0900
GPS south: (Degrees / Min / Sec)	42 / 20 / 54.69	Date of submission:	
GPS east: (Degrees / Min / Sec)	172 / 40 / 59.58	Inspection Date:	19/01/12
		Revision:	
Building Unique Identifier (CCC):	BU 1985-002 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	silty sand	Soil Profile (if available):	Aluvial sand over silt overbank
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	N/A
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	3.00

Building

No. of storeys above ground:	1	single storey = 1	
		Ground floor elevation (Absolute) (m):	3.23
Ground floor split?	no	Ground floor elevation above ground (m):	0.23
Storeys below ground	0		
Foundation type:	mat slab	if Foundation type is other, describe:	
Building height (m):	5.00	height from ground to level of uppermost seismic mass (for IEP only) (m):	4
Floor footprint area (approx):	200		
Age of Building (years):	27	Date of design:	1992-2004
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):	Creche		
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	frame system		
Roof:	timber truss	truss depth, purlin type and cladding	
Floors:	concrete flat slab	slab thickness (mm)	100
Beams:	timber	type	roof supposrts over glazing walls
Columns:	timber	typical dimensions (mm x mm)	
Walls:	non-load bearing	0	

Lateral load resisting structure

Lateral system along:	multi-level tilt panel	**Note: Define along and across in detailed report!**	note total length of wall at ground (m):	7.3
Ductility assumed, μ:	2.00		wall thickness (m):	
Period along:	0.10	##### enter height above at H31	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	lightweight timber framed walls		note typical wall length (m)	9.4
Ductility assumed, μ:	2.00			
Period across:	0.10	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:			
Wall cladding:	plaster system	describe	GIB board on timber frame
Roof Cladding:	Metal	describe	Lightweight metal
Glazing:	other (specify)		PVC
Ceilings:	heavy tiles		Roof trusses with lightweight cladding
Services(list):			

Available documentation

Architectural	full	original designer name/date	Designer Unknown, April 1995
Structural	partial	original designer name/date	Unknown, April 1995
Mechanical	none	original designer name/date	
Electrical	partial	original designer name/date	Unknown, April 1995
Geotech report	partial	original designer name/date	Ian McCahon, 12 December, 1991

Damage

Site:	Site performance:		Describe damage:	damage to exterior concrete slab
(refer DEE Table 4-2)				
	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:	slight	notes (if applicable):	

Building:				
	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):	Minor non-structural		
Across	Damage ratio:	0%		
	Describe (summary):	Minor non-structural		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	yes	Describe:	Slight cracking to plasterboard lining

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

Recommendations

	Level of repair/strengthening required:	none	Describe:	
	Building Consent required:	no	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	
Along	Assessed %NBS before:	100%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology: Bracing schedule as detailed in report
	Assessed %NBS after:	100%		
Across	Assessed %NBS before:	100%	##### %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		07/12/12