Wainoni Community Centre

Detailed Engineering Evaluation

Quantitative Assessment Report

BU 1264-001 EQ2

31 Hampshire Street
Aranui, Christchurch

Wainoni Aranui Community Centre
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Christchurch City Council

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Quantitative Assessment Report

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Wainoni Aranui Community Centre
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Detailed Engineering Evaluation
Stage One Quantitative Report - SUMMARY
Version 1 - Final
Detailed Engineering Evaluation

Background

This is a summary of the Stage 2 Quantitative Assessment for the building structure Wainoni Community Centre located at 31 Hampshire Street in Aranui, Christchurch and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on 15 March 2011, 29 September 2011, and available drawings.

Key Damage Observed

Key damage observed includes:

- Residual displacement of the portal frame columns in the hall
- Cracking to connections between neighbouring structural elements, and associated finishings throughout the building
- Vertical hairline cracking to the foundations, and some differential settlement between foundations of different depths, and external slabs
- Horizontal cracking above most door openings
- Evidence of liquefaction in the adjoining playing fields

Critical Structural Weaknesses

The following critical structural weaknesses have been identified:

- Liquefaction potential
- Insufficient reinforcement or supporting members for the reinforced concrete masonry walls to the hall
- There is no system for transferring the loads from the central portal frames to the end shear walls
- In the Community Rooms the hold down fixings of the timber frame are unlikely to be sufficient to transfer the loads.

Indicative Building Strength (from Initial Capacity Assessment)

Based on the information available and from undertaking a quantitative assessment, the building's capacity has been assessed to be in the order of 5% NBS. The structure is therefore classed as an earthquake prone building.

Recommendations

It is recommended that:

(a) It is recommended that the building not be occupied, given its earthquake prone building status and the elevated level of seismic risk in Christchurch.

(b) The masonry end walls to the main hall should be cordoned off in order to protect the public.

(c) A strengthening works scheme be developed to increase the seismic capacity of the building to at least 67% NBS. This will also need to consider compliance with accessibility and fire requirements.

(d) A quantity surveyor be engaged to determine the costs for either strengthening the building or demolishing and rebuilding.

(e) A verticality survey of the main structural elements should be undertaken to investigate the extent of foundation settlement and residual displacement.

Contents

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council (CCC) to undertake a detailed seismic assessment of the Wainoni Community Centre, located at 31 Hampshire Street in Aranui, Christchurch following the M6.3 Christchurch earthquake on 22 February 2011.

The purpose of the assessment is to determine if the building is classed as being earthquake prone in accordance with the Building Act 2004.

The seismic assessment and reporting have been undertaken based on the qualitative and quantitative procedures detailed in the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011.

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 to carry out a full structural survey before the building is re-occupied.

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

It is anticipated that a number of factors, including the following, will determine the extent of evaluation and strengthening level required:

1. The importance level and occupancy of the building.
2. The placard status and amount of damage.
3. The age and structural type of the building.
4. Consideration of any critical structural weaknesses.

Any building with a capacity of less than 33% of new building standard (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67% as required by the CCC Earthquake Prone Building Policy

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 the 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)) is satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'.

This is typically interpreted by CCC as being 67% of the strength of an equivalent new building. This is also the minimum level recommended by the New Zealand Society for Earthquake Engineering (NZSEE).

Section 121 – Dangerous Buildings

This section was extended by the Canterbury Earthquake (Building Act) Order 2010, and defines a building as dangerous if:

1. 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
2. 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
3. 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
4. There is a risk that other property could collapse or otherwise cause injury or death; or
5. 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 loads 33% of those 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 on 4th September 2010.

The 2010 amendment includes the following:

1. A process for identifying Earthquake Prone Buildings, commencing on 1 July 2012.
2. A strengthening target level of 67% of a new building.
3. A timeframe of 15-30 years for buildings to be strengthened.
4. 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.

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.

On 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

- 36% increase in the basic seismic design load for Christchurch (Z factor increased from 0.22 to 0.3);
- Increased serviceability requirements.

2.5 Institution of Professional Engineers New Zealand (IPENZ) Code of Ethics

One of the core ethical values of professional engineers in New Zealand is the protection of life and safeguarding of people. The IPENZ Code of Ethics requires that:

Members shall recognise the need to protect life and to safeguard people, and in their engineering activities shall act to address this need.

1.1 Giving Priority to the safety and well-being of the community and having regard to this principle in assessing obligations to clients, employers and colleagues.

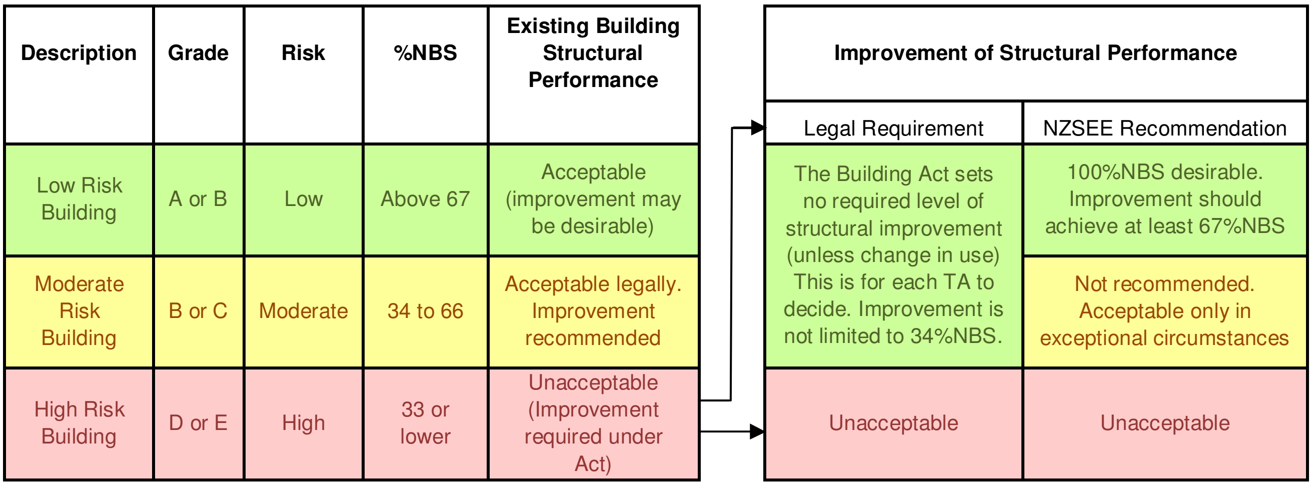
1.2 Ensuring that responsible steps are taken to minimise the risk of loss of life, injury or suffering which may result from your engineering activities, either directly or indirectly.

All recommendations on building occupancy and access must be made with these fundamental obligations in mind.

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 loadings are in accordance with the current earthquake loading standard NZS1170.5 [1].

A generally accepted classification of earthquake risk for existing buildings in terms of %NBS that has been proposed by the NZSEE 2006 [2] is presented 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 required under Act)

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 Guidelines

Table 3.1 below compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% risk of exceedance in 50 years (i.e. 0.2% in the next year). It is noted that the current seismic risk in Christchurch results in a 6% risk of exceedance in the next year.

Table 3.1: %NBS compared to relative risk of failure

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

3.1 Minimum and Recommended Standards

Based on governing policy and recent observations, Opus makes the following general recommendations:

3.1.1 Occupancy

- The Canterbury Earthquake Order[i] in Council 16 September 2010, modified the meaning of "dangerous building" to include buildings that were identified as being EPB's. As a result of this, we would expect such a building would be issued with a Section 124 notice, by the Territorial Authority, or CERA acting on their behalf, once they are made aware of our assessment. Based on information received from CERA to date, this notice is likely to prohibit occupancy of the building (or parts thereof), until its seismic capacity is improved to the point that it is no longer considered an EPB.

3.1.2 Cordoning

- Where there is an overhead falling hazard, or potential collapse hazard of the building, the areas of concern should be cordoned off in accordance with current CERA/Christchurch City Council guidelines.

3.1.3 Strengthening

- Industry guidelines (NZSEE 2006 [2]) strongly recommend that every effort be made to achieve improvement to at least 67%NBS. A strengthening solution to anything less than 67%NBS would not provide an adequate reduction to the level of risk.

- It should be noted that full compliance with the current building code requires building strength of 100%NBS.

3.1.4 Our Ethical Obligation

- In accordance with the IPENZ code of ethics, we have a duty of care to the public. This obligation requires us to identify and inform CERA of potentially dangerous buildings; this would include earthquake prone buildings.

[1] This Order only applies to buildings within the Christchurch City, Selwyn District and Waimakariri District Councils authority

4 Background Information

4.1 Building Description

4.1.1 General

The Wainoni Community Centre was designed in 1976, and comprises an approximately T-shaped structure which can be divided into two discrete areas as follows:

- 'The Hall' on the south side of the building, which is approximately 26m long and 12m wide, is an asymmetrical steel portal frame structure with its long axis running approximately east/west (the top of the T), with a single storey timber framed store room annex to the west elevation. The store has been extended historically, for which no drawings are available. A single storey masonry extension block comprising external changing rooms and toilets has also been added historically to the south elevation (drawings dated 2007). The rooms in the extension to the south are accessed externally. Internally, there is a timber stage construction at the east end of the Hall.
- 'The Community Rooms' which extends from the north side of the Hall, is a single storey reinforced masonry and timber frame structure approximately 18m long and 11m wide, housing offices and other rooms, a kitchen and ablutions. It has a central corridor through which the Hall is accessed.

4.2 Gravity Load Resisting Systems

For the purpose of this report the structure will be described in its two main areas, 'the Hall' and 'the Community Rooms', which includes their associated extensions.

4.2.1 The Hall

The main footprint of the hall is approximately 21.6m x 12.0m and is made up of a perimeter of 190mm thick reinforced masonry (concrete block) walls of approximately 5m height to the north elevation and part of the east and west elevations. It remains at this height until it meets the roof line and slopes down to the south elevation where it is full height to the columns (3.5m). Above this height the exterior wall cladding is timber shiplap.

Inside the hall are 3 exposed welded steel portal fames of 305x102x28 UB sections spanning approximately north/south with a span of 12m and spaced at approx. 5.4m from the end walls and centre to centre with approximate column heights of 8m (north elevation) and 3.5m (south elevation).

The drawings show that the roof has 50x10mm flat steel diagonal bracing for the central three rafters of the roof, with each rafter divided into two bays, with a central 76x76x3.2 SHS strut for the full length of the Hall, fixed at mid span to the top of the rafters and to a 254x102x25 UB steel wind post in each of the east and west walls.

The wind posts span from the foundations to the central roof strut. The presence of the roof bracing and strut were verified by intrusive inspection.

The column and wind post loads are transferred to the foundations by steel base plates which are anchored to the foundation by holding down bolts (J-bolts).

Above the level of the masonry on the higher (north) long elevation there is 50x10mm flat steel diagonal wind bracing between the three central columns (Appendix 1, photograph 24).

The drawings detail a connection between each column and the top of the block walls at using two R16 tie bars welded in pairs to the front flange of the column, wrapping back into the RC filled void of the masonry, with anchor legs to be cast in to the concrete infill. There was no evidence of this connection. However, it was established during the intrusive investigation that in these locations a vertical steel plate is welded flat against the rear (wall side) flange of each column at these locations, with a connection into the vertical mortar joint of the blockwork. The lower connection is at about 2.7m above floor level for each column and the upper connection is in the second blockwork course from the top of the wall.

The profiled metal sheet roof cladding is supported on timber purlins at approx. 120mm centres which span between the UB rafters.

The original Hall annex, or changing facilities, to the west elevation is approximately 5.5m long and 2.5m wide and is of 100mm wide timber frame construction with a mono-pitch roof constructed as a 'lean-to' to the external wall. This has been extended historically to extend beyond the width of the Hall in a northerly direction, by means of a masonry wing wall which has been constructed as an extension to the west elevation of the Hall by approximately 3m.

In 2003 external changing facilities and toilets (for Wainoni Park) were constructed against the south wall of the hall and has a stepped plan but is approximately 20.0m long and 6.0m wide and is reinforced concrete blockwork construction, with a duo-pitch roof forming a 'lean-to' to the original external wall. These two parts of the building share a concealed gutter where they meet.

Structural drawings indicate the main hall foundations comprise a suspended wooden floor supported on a reinforced concrete perimeter strip footing with internal concrete piles (200mm by 200mm wide) embedded approximately 500mm below ground level (bgl). New changing rooms and public toilets were added onto the south western wall of the Main Hall in approximately 2007. The changing rooms and public toilet foundations comprise a 100-150mm thickened edge concrete floor slab integrated with 240mm wide concrete floor beams positioned under load bearing walls founded to 350mm below the ground level.

4.2.2 The Community Rooms

The original external walls of the single storey Community Rooms are predominantly 190mm thick reinforced masonry (concrete block) walls, other than the recessed section to the east elevation, which is of timber frame construction.

The main roof is a duo-pitch cut roof comprising timber rafters and purlins. The majority of the roof has a split ridge with high level glazing above the corridor, with a maximum ridge height of approximately 6m. The ridge height in all other sections is approx. 5m. On the west side of the building, the rafters appear to span east/west between the external walls and the internal timber frame spine walls either side of the corridor, with purlins spanning between the rafters. It was established that the rafters do not bear onto the timber studs of the partition wall, but instead are face fixed with only nominal skew nailed connections. To the east of the corridor, the purlins have been confirmed to span north/south between load-bearing timber stud partition walls and the reinforced blockwork walls.

The section of roof adjacent to the Hall and over the lobby, which extends out to form the canopy over the main entrance, comprises timber framing spanning between the north wall of the Hall to the south (blockwork) wall forming the toilets. Externally, the framing is supported on 100x100 timber posts which extend down to ground level.

There is an extension to the north of masonry (concrete block) wall construction, assumed to be reinforced, with a mono-pitch roof construction dipping to the north. It is assumed that timber rafters span between the north elevation of the extension and the flank wall to the south. The flank wall is along the former north elevation line, and this wall has been extended above the original roof level in timber framed construction to support the roof. The wall has also been extended towards the west in a similar manner with blockwork up to approximately 2.40m and timber framed construction above.

4.3 Seismic Load Resisting Systems

4.3.1 The Hall

North-South direction

Seismic loads in the north-south direction are resisted by the moment connection between the columns and rafters of the portal frames of the Hall and by the reinforced masonry end walls acting as in-plane shear walls and transferring loads to the foundations.

With very limited structural shear connection between these load resisting systems, these elements would tend to act in an independent manner supporting their own tributary seismic mass.

The portal frames are assumed to provide restraint to the reinforced masonry shear walls between the columns, which are subject to the out of plane seismic action of their own self weight and any tributary parts of the roof which they support. A typical frame has been modelled and its capacity to resist these loads assessed.

The original roof construction is assumed to have provided insufficient diaphragm action to transfer seismic loads between the portal frames and between the end portal frames and the reinforced masonry end walls. The inner ceiling lining was damaged and has been removed.

The west and south elevation annex structures resist seismic loads by in-plane shear action of the masonry walls running north/south and by any diaphragm action that may be provided by sheathing of the mono-pitch roof construction. These would transfer loads to the foundations and/or the reinforced masonry walls of the Hall respectively. The roof diaphragm and the shear walls have been assessed.

East/West Direction

Seismic loads are considered to be mainly resisted by the reinforced masonry walls at lower level running east/west acting as in-plane shear walls.

This resistance is also assisted by the portal frames bending about their minor axes, in conjunction with their connection to the longitudinal shear walls. Load sharing between portal frames is given through the cross bracing of the central columns at high level and of the central rafters, the central SHS rafter strut and by the diaphragm action of the roof construction.

Both these systems act to transfer forces to the foundations. The presence of two points of connection between each column of the portal frames and the reinforced blockwork walls has been confirmed by intrusive investigation and therefore these elements are assumed to act together to resist seismic loads in this direction. The roof diaphragm and the shear walls have been assessed.

The west and south elevation annex structures resist seismic loads through in-plane shear of the masonry walls running east/west and any diaphragm action that may be provided by sheathing of the mono-pitch roof construction. These would transfer loads to the foundations and/or the reinforced masonry walls of the Hall respectively. The roof diaphragm and the shear walls have been assessed.

4.3.2 Community Rooms - North/South Direction:

North-South direction

Seismic loads are resisted by the shear resistance the reinforced masonry external walls and by plasterboard sheathed load-bearing timber partitions, running north/south, acting as in-plane shear walls. Seismic loads are assumed to be transferred to these shear walls by diaphragm action of the timber cut roof construction, with plasterboard lining, transferring loads to the foundations.

East/West Direction

Seismic loads are resisted by the shear resistance of the reinforced masonry external wall and by plasterboard sheathed load-bearing timber partitions, running east/west, acting as in-plane shear walls. Seismic loads are assumed to be transferred to these shear walls by diaphragm action of the timber cut roof construction, with plasterboard lining, transferring loads to the foundations.

The original north external wall of the meeting room has a large opening in it, to the resource interview room, a modern extension, such that both these rooms can be opened up to form one. A PFC section steel portal frame (goalpost) within the depth of the wall forms this opening. The wall above this has been re-constructed in timber frame, but the ends of the purlins of the original roof remain largely unsupported along the line of the partition. This frame will provide some in-plane stability, transferring loads to the foundations, and has been considered to be part of the seismic resisting system.

4.4 Survey

4.4.1 Post 22 February 2011 Rapid Assessment

Post February Earthquake the site was posted with a green placard, but there is restricted entry to the Hall due to the condition of the false ceiling to the main hall.

A damage assessment of the structure was undertaken on 15th March 2011 by Opus International Consultants. This inspection included external and internal visual inspections of all structural elements, without the benefit of opening up works.

4.4.2 Further Inspections

Once the construction drawings had been made available, a further inspection was carried out on 29th September 2011 by Opus International Consultants, with the benefit of limited opening up works and trial pits at five locations to the west elevation, and a limited ground investigation was carried out.

4.5 Qualitative Assessment

A Qualitative Assessment Report (Rev A) was produced by Opus International Consultants Ltd on 19 October 2011, following an intrusive survey and geotechnical investigation. This report, without the benefit of calculation, concluded that the building was not earthquake prone, but was at earthquake risk, with an estimated seismic capacity of 36%. Critical structural weaknesses were identified and observed damage detailed, along with photographs, and the inclusion of construction drawings which were available at the time.

5 Structural Damage

The details of the damage observed are provided with the benefit of photographic evidence in the Opus International Consultants Limited Quantitative Assessment (Rev A) of 19 October 2011.

5.1 Surrounding Buildings

This building stands alone and is sufficiently far away from other structures so as not to be affected by them.

5.2 Residual Displacements and Damage

An initial check on the verticality of the portal frame columns and masonry in the Hall indicated that there is some residual displacement of the sway of the portal frames in their direction of span. It is recommended that this is investigated further, by means of a verticality survey, as part of the detailed design.

5.3 General Observations

The Opus International Consultants Qualitative Assessment (Rev A) dated 19 October 2011 details findings from the surveys, and the damage observed.

Generally, there has been cracking to connections between neighbouring structural elements, and associated finishes throughout the building. The reinforced blockwork of the Hall is generally cracked vertically at supports (either side of the portal frames). The ceiling finishes were damaged and have been removed.

Floor levels have been affected particularly in the Hall where there is a difference in level across the floor being lower against the external walls and tending to hog in the middle. The maximum difference in level is approximately 70mm.

There is vertical hairline cracking to the foundations, and some differential settlement visible between foundations of different depths, and external slabs, up to 65mm.

There is horizontal cracking above most door openings.

The gap between the portal frame columns, and the adjoining masonry varies with height, suggesting that there is a residual displacement resulting from seismic induced sway of the portal frames.

There is evidence of liquefaction in the adjoining playing fields.

5.4 Reported Damage

A Geotechnical walkover survey was completed by an Opus Geotechnical Engineer on 3 August 2011. The walkover survey identified ejected sand and sinkholes from liquefaction in Wainoni Park 30m to the west of the building, resultant from both the 22 February 2011 and 13 June 2011 earthquakes.

In addition, minor cracking up to 20mm wide in paved areas and cracking between the entrance foyer and the main building was observed.

Level Survey readings of the floor were taken on 28 June 2011. The readings indicate up to 70mm of differential settlement has occurred over a distance of 6m (slope of 1.2% or 1 in 85) across the Main Hall floor.

6 Detailed Seismic Assessment

6.1 Critical Structural Weaknesses

The term Critical Structural Weakness (CSW) refers to a component of a building that could contribute to increased levels of damage or cause premature collapse of a building. The following potential CSW's have been identified for each of the buildings and have been considered in the analysis:

The following critical structural weaknesses have been identified:

a) Liquefaction potential

b) Insufficient flexural strength of the reinforced concrete masonry walls to the Hall

c) No effective ceiling diaphragm in the Hall for transferring the loads from the central portal frames to the end shear walls

d) In the Community Rooms the hold down fixings of the timber frame are unlikely to be sufficient to transfer the loads.

6.2 Quantitative Assessment Methodology

A brief summary of the quantitative assessment methodology is as follows:

- A 2D model of a typical portal frame of the Hall has been created in Microstran, which is a structural analysis program, to ascertain the effects of the combined actions on the portal frames, including the actions imposed on them by the masonry panels.
- The capacity of the frames has been assessed using the output data from Ultimate Limit State (ULS) load combinations from the model analysis.
- The reinforced masonry of the Hall has been assessed under in-plane (along their length) and out-of-plane ULS seismic actions.
- The Community Rooms have been assessed under ULS seismic actions in order to ascertain the Bracing Units required of the shear walls.
- The end connections of the rafters and roof beams have been assessed on the assumption that these are simple nailed connections.
- An assumption has been made that the connections between shear walls and diaphragm are satisfactory and that the diaphragm action of the roof construction is provided. This will need to be checked prior to detailed design and improvements carried out as required.

6.3 Seismic Coefficient Parameters

The seismic coefficient parameters used in the assessment are as follows:

- Site subsoil class: D (Deep or soft soil sites)
- Hazard factor: Z = 0.3
- Importance Level: 2

6.4 Expected Ductility Factors

The expected ductility factor throughout in both N/S and E/W directions:

- $\mu = 1.25$

6.5 Detailed Seismic Assessment Results

A summary of the structural performance of the building is shown in the following table.

Table 3: Summary of Seismic Performance

Structural Element/System	Failure mode and description of limiting criteria	Critical Structural Weakness and Collapse Hazard	% NBS based on calculated capacity
Masonry walls to ends and sides of Hall - out of plane flexural failure.	The walls have insufficient reinforcement to span horizontally between the restraints at portal frame locations and end walls under the out of plane seismic loads. The elastic sway of the portal frame columns imposes out of plane loads in the walls which exceed the permissible SLS deflections, causing cracking and compression failure.	Yes	14%
Portal Frames to Hall	Buckling of the rafters/columns due to the permissible bending moments being exceeded.	Yes	9%
Roof Diaphragm to Hall	Failure in the distribution of loads between the portal frames and the end shear walls	No	(Failed in previous seismic event)
Timber Roof Member Connections to Community Rooms	Pull out failure of nailed connections, resulting in collapse of roof.	No	Details unknown so no capacity can be calculated
Concrete Piles to Foundations	Rotational failure of piles to foundations below structural walls due to seismic lateral loads and insufficient level of restraint	Yes	Details unknown so no capacity can be calculated

Structural Element/System	Failure mode and description of limiting criteria	Critical Structural Weakness and Collapse Hazard	% NBS based on calculated capacity
Differential Settlement as a result of liquefaction	Liquefaction potential has been assessed, and there is found to be a risk of differential settlement of up to 160mm along the length of the hall, potentially overstressing key structural elements.	Yes	N/A
Timber frame shear walls to Community Rooms	Pull-out failure of holding down fixings or insufficient fixings or diaphragm action to transfer diaphragm loads into these walls.	No	5%

Note that the values given represent the worst performing elements in the building, as these effectively define the building's capacity. Other elements within the building may have significantly greater capacity when compared with the governing elements (for example the extension added to the original building in 2007). This will be considered further when developing the strengthening options.

6.6 Discussion

- Main Hall: Out of plane flexural damage of reinforced concrete masonry shear walls to the Hall has occurred, evidenced by vertical cracking in the vicinity of the steel portal frames. The walls have been found to contain insufficient reinforcement and are inadequately supported to resist the actions of their seismic mass in a design event and therefore are considered a CSW. Their failure and collapse could have potentially fatal consequences in a future seismic event. The rafters of the portal frames are currently undersized and the bending moments induced in a design seismic event will result in their failure in buckling. The sway of the portal frames would induce further out of plane actions to the reinforced masonry walls, and thus further increase the risk of collapse. However the likelihood that the portal frames themselves collapse is lower than that for the masonry walls, due to their ductility and the ability for energy dissipation through permanent deformation and/or hinge formation.
- Hall roof: The original roof construction of the Hall was not capable of distributing the loads between the portal frames and the shear walls by diaphragm action and hence partially failed during the February earthquake and the ceiling boarding has since been removed. Although this in itself does not constitute a CSW, the provision of a construction to allow for the distribution of seismic loads to the portal frames would increase the performance of the frames in a seismic event.
- Community Rooms: This part of the building can generally be expected to behave better in a design level seismic event, although the structural detailing is poor and connections between timber members, the roof diaphragm and both timber and reinforced masonry shear walls have been assessed to potentially fail in a design level seismic event and have hence been identified as a CSW. Assessment of the connections between the shear walls and the roof diaphragm are based on

assumptions. Although the %NBS is low, improvement by the provision of suitable connections can be relatively easily implemented, and it is to be expected that a %NBS greater than 67% can be relatively easily achieved.

- Changing rooms: These were designed in 2007 and would therefore be expected to have a %NBS greater than 67%. The assessment confirms this for the superstructure, however, the shallow strip foundations increase the risk of potential further settlement in excess of acceptable levels.
- The store rooms are of lightweight construction, and due to their small size and low importance are not considered to pose a significant risk to life should their construction fail in a seismic event. As the construction is relatively ductile, their performance will not significantly influence the seismic performance of the Hall.

The building has an overall seismic capacity of less than 33% NBS and is therefore considered to be earthquake prone in accordance with the Building Act 2004. It is recommended that this building not be occupied and that the masonry walls of the Main Hall be cordoned off to protect the public.

6.7 Limitations and Assumptions in Results

Our analysis and assessment is based on an assessment of the building in its undamaged state. Therefore the current capacity of the building will be lower than that stated.

The results have been reported as a %NBS and the stated value is that obtained from our analysis and assessment. Despite the use of best national and international practice in this analysis and assessment, this value contains uncertainty due to the many assumptions and simplifications which are made during the assessment. These include:

- Simplifications made in the analysis, including boundary conditions such as foundation fixity.
- Assessments of material strengths based on limited drawings, specifications and site inspections
- The normal variation in material properties which change from batch to batch.
- Approximations made in the assessment of the capacity of each element, especially when considering the post-yield behaviour.

7 Summary of Geotechnical Appraisal

7.1 General

CERA has published residential rebuilding zones:

- Green (Go Zone): repair / rebuild process can begin

- Orange (Hold Zone): further assessment required
- Red (No Go Zone): land repair would be prolonged and uneconomic
- White (Unzoned): CBD or hillside land where geotechnical mapping and further assessment currently underway

The Department guidance breaks the Green Zone into three technical categories. Foundation requirements differ from category to category. For a quick guide see below:

- Technical Category 1 (TC1) – future land damage from liquefaction unlikely.
- Technical Category 2 (TC2) – minor to moderate land damage from liquefaction is possible in future large earthquakes.
- Technical Category 3 (TC3) – moderate to significant land damage from liquefaction is possible in future large earthquakes.

31 Hampshire Street, Aranui is located in an area designated as unmapped, but bounds TC2 to the east and TC3 to the west.

The site is indicated to have moderate potential for liquefaction in the ECAN study with subsidence in the order of 100 mm to 300mm expected in a design level earthquake event.

There is in situ testing data available for the site from one borehole in 1986 that can be used to evaluate liquefaction potential.

The sand, and silty sand, around the basements and possibly under part of the original building could liquefy if they are loose to medium dense. The annexe building is on both medium dense gravel and sand that would be liquefiable in a design level earthquake.

- Subsidence of up to 200mm might be expected to occur in the design level seismic event (PGA of 0.44g) and 100mm of this could occur differentially.
- General differential settlement to the foundations has occurred due to liquefaction, and where vertical foundation loads are highest, further settlement of up to 160mm may be expected in a future seismic event. Investigation of the foundations may reveal that the lateral restraint of the piles is insufficient, and these may overturn in a seismic event. Should this be the case for load bearing wall foundations, this is considered to be a CSW.

LiDAR data for the site suggests less than 100mm of settlement has occurred in the recent earthquake events.

7.2 Summary

A series of shallow and deep geotechnical investigations were completed at the Centre to determine the susceptibility of underlying soils to liquefaction, confirm the static bearing capacity of the surficial soils, and inspect the existing perimeter footings for structural damage. The investigations comprised three Cone Penetrometer Tests completed to a depth of 20m below ground level, and three test pits with hand augers and Scala Penetrometers to a depth of 3.0m.

Surficial soils at the site comprise sandy SILT to 0.5m depth, underlain by a relatively thick layer of SAND of varying densities down to GRAVEL (RICCARTON) at 37m below ground level.

Scala Penetrometer tests of the shallow soils indicate the allowable static bearing capacity of the sand layer underlying the existing foundations is 95 – 180 kPa.

Liquefaction analysis identified two potentially liquefiable layers between depths of 1.3m to 3.0m and 6.0m to 20m for the site. Liquefaction induced settlement in the order of 15mm (SLS) and 130mm (ULS) is predicted in future earthquake events at the Site. In a ULS earthquake event, 60-130mm of differential settlement is possible.

Liquefaction analysis indicates that the site could be considered a “Green/Blue-TC3”, in accordance with the Department of Building and Housing zonation (DBH, 2011). For the extent of reported damage sustained to the hall foundations, the DBH guidance document indicates the observed differential settlement exceeds the tolerance for serviceability and a foundation re-level is required.

8 Remedial Options

The building requires repair and strengthening, with a target of increasing the seismic performance to as near as practicable to 100%NBS, and at least 67%NBS. Our concept strengthening scheme to achieve this may include, but would not be restricted to:

(a) Improvement of the performance of the reinforced masonry shear walls, which may include the provision of additional intermediate steel members between the portal frames, the application of resin bonded steel straps to either side of the blockwork, and improvement of the connection between wall panels and the portal frames.

(b) Strengthening of portal frame sections and the provision of additional steel members to distribute loads in the plane of the roof, particularly between rafters.

(c) Fitting of nail plate or similar fixings to timber connections and timber frame wall panels of the Community Rooms, and provision of hold down fixings at the base of timber frame panels.

(d) Improvement of the ceiling diaphragm in the Community Rooms.

(e) Crack repairs to reinforced masonry

(f) Demolition of the Screen wall at the main entrance

(g) Potential options to remediate the settlement of the Main Hall Foundations include shallow foundation solutions comprising re-levelling or replacement with a ribraft foundation, replacement or integrating deep piles with the existing foundation and ground improvement. Collapse of the Main Hall is considered unlikely in a ULS earthquake event regardless of which option is adopted.

9 Conclusions

(a) The seismic performance of the Hall is governed by the reinforced concrete blockwork shear walls, which have an assessed resistance to out of plane failure of 14%NBS, and are potentially a risk to human life in the event of collapse. The building is therefore considered to be earthquake prone in accordance with the Building Act 2004.

(b) The seismic performance of the Community Rooms is governed by the capacity of the timber connections, especially the hold down fixings. This connection has been estimated to have a capacity of 5% NBS. The building is therefore considered to be earthquake prone in accordance with the Building Act 2004. The strength of this connection should be improved relatively cost effectively and simply.

(c) The free-standing and unreinforced hollow core concrete block screen wall at the entrance has not suffered damage, but is by inspection earthquake prone, and should be demolished.

(d) The seismic performance generally is reduced by the liquefaction potential of the ground conditions, which indicate that over the length of the building a differential settlement of up to 130mm can be expected in a design level seismic event.

(e) Although the roof diaphragm of the Community Rooms, which functions as a shear collector element, does not limit the building capacity in this area, improvements should be carried out to ensure that the shear walls could be utilised to their full capacity i.e. the roof construction is improved to provide enhanced diaphragm action and additional connections between the roof construction and sheer walls are provided. Similarly, horizontal perpendicular connections between shear walls and the walls which provide restraint to them should be enhanced.

(f) In a future seismic event further differential settlement may occur between sections of the building which are founded at different depths. Shallow foundations of the store and changing room annexes may settle significantly more that the predicted 130mm of the deeper foundations and this may have to be accepted.

(g) The lateral restraint of the piles below structural walls has not been investigated, but may require improvement to prevent these being displaced in a seismic event.

(h) Occupancy of the building should be prevented given its earthquake prone building status.

(i) The masonry end walls to the main hall should be cordoned off in order to protect the public.

10 Recommendations

(a) It is recommended that the building not be occupied, given its earthquake prone building status and the elevated level of seismic risk in Christchurch.

(b) The masonry end walls to the main hall should be cordoned off in order to protect the public.

(c) A strengthening works scheme be developed to increase the seismic capacity of the building to at least 67% NBS, this will also need to consider compliance with accessibility and fire requirements.

(d) A quantity surveyor be engaged to determine the costs for either strengthening the building or demolishing and rebuilding.

(e) A verticality survey of the main structural elements should be undertaken to investigate the extent of foundation settlement and residual displacement.

(f) Shallow foundation repair options are recommended for the site, although they will not remediate or reduce the susceptibility of soils at the site to liquefy, and it must be accepted by the Client that settlements in the order of 15mm and 130mm are likely in a respective SLS or ULS seismic event, and that the probability of such events occurring is elevated at present and will be for some time.

11 Limitations

(a) This report is based on an inspection of the structure of the buildings and focuses on the structural damage resulting from the 22 February 2011 Canterbury Earthquake and aftershocks only. Some non-structural damage is described but this is not intended to be a complete list of damage to non-structural items.

(b) Our inspections have been visual and limited-intrusive, with linings or finishes removed only locally to expose key structural elements. Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practicing in this field at this time.

(c) This report is prepared for CCC to assist with assessing the remedial works required for council buildings and facilities. It is not intended for any other party or purpose.

12 References

[1] NZS 1170.5: 2004, *Structural design actions, Part 5 Earthquake actions,* Standards New Zealand.

[2] NZSEE: 2006, *Assessment and improvement of the structural performance of buildings in earthquakes,* New Zealand Society for Earthquake Engineering.

[3] Engineering Advisory Group, *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure*, Draft Prepared by the Engineering Advisory Group, Revision 5, 19 July 2011.

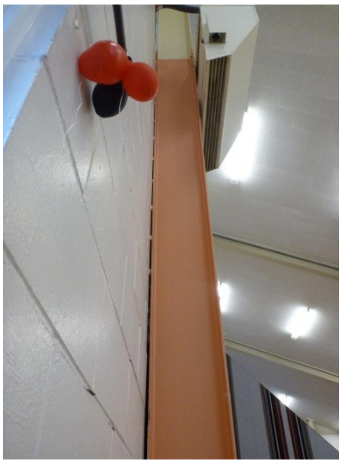
[4] Engineering Advisory Group, *Guidance on Detailed Engineering Evaluation of Non-residential buildings, Part 3 Technical Guidance*, Draft Prepared by the Engineering Advisory Group, 13 December 2011.

[5] SESOC, *Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes*, Structural Engineering Society of New Zealand, 21 December 2011.

Appendix A – Photographs

Wainoni Aranui Community Centre		
No.	**Item description**	**Photo**
General		
1.	General view of the centre from the north east.	
2.	External toilets and storage rooms	

3.	Close up of separation between toilet/storage and Hall	
4.	Separation of ramp slab to concrete landing at side entrance	
5.	Vertical crack on blockwork adjacent to column on south wall	

6.	North wall of Hall showing high level wind bracing between portal frame columns	
7.	Looking to the top of column. Blockwork and column area in contact at high level.	
8.	Trial pit adjacent to Community Rooms Extension (west elevation)	

9.	Trial pit at north west corner of store extension to Hall (west elevation) showing shallow founding depth	
10.	Face-nailed rafter connection to load bearing timber wall partition to Community Rooms corridor	

Appendix B – Geotechnical Appraisal

Wainoni Aranui Community Centre
31 Hampshire Street

Geotechnical Assessment of Earthquake Damage

Wainoni Aranui Community Centre
31 Hampshire Street

Geotechnical Assessment of Earthquake Damage

Prepared By

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Date:	9 May 2012
Reference:	6-QUCCC.19/045GC
Status:	Final

Executive Summary

The Wainoni Aranui Community Centre was damaged during the 22 February 2011 earthquake, with further damage occurring following the Magnitude 6.3 earthquake on 13 June 2011.

Opus International Consultants Ltd (OPUS) has been engaged by Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the Wainoni Aranui Community Centre at 31 Hampshire Street.

A series of shallow and deep geotechnical investigations were completed at the Centre to determine the susceptibility of underlying soils to liquefaction, confirm the static bearing capacity of the surficial soils, and inspect the existing perimeter footings for structural damage. The investigations comprised three Cone Penetrometer Tests completed to a depth of 20m below ground level, and three test pits with hand augers and Scala Penetrometers to a depth of 3.0m.

Surficial soils at the site comprise sandy SILT to 0.5m depth, underlain by a relatively thick layer of SAND of varying densities down to GRAVEL (RICCARTON) at 37m below ground level.

Scala Penetrometer tests of the shallow soils indicate the allowable static bearing capacity of the sand layer underlying the existing foundations is 95 – 180 kPa.

Liquefaction analysis identified two potentially liquefiable layers between depths of 1.3m to 3.0m and 6.0m to 20m for the site. Liquefaction induced settlement in the order of 15mm (SLS) and 130mm (ULS) is predicted in future earthquake events at the Site. In a ULS earthquake event, 60-130mm of differential settlement is possible.

Liquefaction analysis indicates that the site could be considered a “Green/Blue-TC3”, in accordance with the Department of Building and Housing zonation (DBH, 2011). For the extent of reported damage sustained to the hall foundations, the DBH guidance document indicates the observed differential settlement exceeds the tolerance for serviceability and a foundation re-level is required.

Potential options to remediate the settlement of the Main Hall Foundations include shallow foundation solutions comprising re-levelling or replacement with a ribraft foundation, replacement or integrating deep piles with the existing foundation and ground improvement. Collapse of the Main Hall is considered unlikely in a ULS earthquake event regardless of which option is adopted.

There is currently a significant risk of liquefaction and differential settlement occurring at the site due to seismic activity. Ground improvement and/or a deep piled foundation replacement could reduce or mitigate the effects of liquefaction induced settlement at the site. However, due to the relatively limited extent of damage sustained to the Main Hall in the recent earthquakes, these options may not be warranted. The cost of mitigation is considered relatively large verses the cost of re-levelling should similar damage occur in a future earthquake.

Shallow foundation repair options are recommended for the site, although they will not remediate or reduce the susceptibility of soils at the site to liquefy, and it must be accepted by the Client that settlements in the order of 15mm and 130mm are likely in a respective SLS or ULS seismic event, and that the probability of such events occurring is elevated at present and will be for some time.

Contents

List of Tables

Appendices

1 Introduction

Between 4 September 2010 and 13 June 2011, a series of earthquakes has occurred in the Canterbury region causing damage to infrastructure and buildings. Notably, the Magnitude 6.3 Earthquake on 22 February 2011 and the Magnitude 6.3 aftershock on 13 June 2011 have caused significant ground damage in the suburbs of Wainoni and Aranui.

Opus International Consultants Ltd (OPUS) has been engaged by Christchurch City Council (CCC) to undertake a detailed engineering evaluation (DEE) of the Wainoni Aranui Community Centre at 31 Hampshire Street.

As part of the DEE process, a Geotechnical Desk Study of the Site issued on 19 August 2011 by OPUS was completed. The Study recommended site investigations be undertaken to enable a quantitative assessment of the liquefaction hazard of the site, determine the static bearing capacity of the shallow soils, inspect the structural integrity of the perimeter strip footing, and provide geotechnical information for conceptual foundation design.

Accordingly, shallow and deep site investigations were completed in September and October 2012 which included three Cone Penetrometer Tests to a depth of 20m below ground and three Test Pits with hand auger and Scala Penetrometers to a depth of 3.0m below ground level.

The following report presents an appraisal of the ground conditions based on the site investigation results, an assessment of liquefaction risk and provides options for remediation of the settled Wainoni Aranui Community Centre building foundations in accordance with the detailed engineering evaluation procedure as provided by the Canterbury Earthquake Authority (CERA) in their guidance document dated 19 July 2011.

2 Site Location

The Wainoni Aranui Community Centre comprises two main parts; a main hall including a toilet/changing area and a kitchen/staff room. The overall structure is of reinforced masonry block, with the main hall constructed using steel portal framing and the kitchen/staff room constructed using light weight timber framing.

The site is bound by Hampshire Street to the east, a carpark area to the north and Wainoni Park to the south and west (refer Appendix D).

The ground profile is relatively flat, low lying and is typically level with Hampshire Street. All external areas are paved with asphalt or grassed.

Following the recent strong earthquakes in Canterbury, the Department of Building and Housing has sub-divided the “Green” zone land on the flat in Christchurch into technical categories depending on likely ground performance in future large earthquakes. The three technical categories are summarised in table 1 which has been adapted from The Revised Guidance on Repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence (DBH, 2011).

Foundation Technical Category	**Future land performance expected from liquefaction**	**Expected SLS land settlement**	**Expected ULS land settlement**
TC 1	Negligible land deformations expected in a future small to medium sized earthquake and up to minor land deformations in a future moderate to large earthquake.	0-15 mm	0-25 mm
TC 2	Minor land deformations possible in a future small to medium sized earthquake and up to moderate land deformations in a future moderate to large earthquake.	0-50 mm	0-100 mm
TC 3	Moderate land deformations possible in a future small to medium sized earthquake and significant land deformations in a future moderate to large earthquake.	>50 mm	>100 mm

Table 1: Technical categories based on expected future land performance

The Wainoni Aranui Community Centre is surrounded by residential properties which have been zoned as TC 2 to the east, south and west; and TC 3 to the north.

3 Existing Foundations

Structural Drawings indicate the Main Hall foundations comprise a suspended wooden floor supported on a reinforced concrete perimeter strip footing with internal concrete piles (200mm by 200mm wide) embedded approximately 500mm below ground level (bgl). New changing rooms and public toilets were added onto the south western wall of the Main Hall in approximately 2007. The changing rooms and public toilet foundations comprise a 100-150mm thickened edge concrete floor slab integrated with 240mm wide concrete floor beams positioned under load bearing walls founded to 350mm below the ground level.

4 Reported Damage

A Geotechnical walkover survey was completed by Emily Hodgkinson, an OPUS Geotechnical Engineer on 3 August 2011 (refer Appendix E). The walkover survey identified ejected sand and sinkholes from liquefaction in Wainoni Park 30m to the west of the building, resultant from both the 22 February 2011 and 13 June 2011 earthquakes.

In addition, minor cracking up to 20mm wide in paved areas and cracking between the entrance foyer and the main building was observed.

Level Survey readings of the floor were taken on 28 June 2011. The readings indicate up to 70mm of differential settlement has occurred over a distance of 6m (slope of 1.2% or 1 in 85) across the Main Hall floor (refer Appendix F).

5 Ground Conditions

The results of the shallow and deep site investigations carried out by Opus in September and October 2011 are summarised in table 1 below. The investigation reports are included in Appendix B. CPT's indicated the groundwater level was typically 1.3m below ground level (bgl). The following ground conditions are interpreted for the site:

Layer	**Stratigraphy**	**Thickness (m)**	**Depth Encountered from (m) bgl**	**Comment**
U1	Sandy SILT with trace fine gravel; dark brown. “Firm”; dry; low plasticity.	0.1-0.5m	0.0m	Topsoil
U2	SAND with trace silt; brown. “Loose”; moist. CPT qc = 2-4 MPa $q_{allowable}$ = 95-180kPa	0.3 – 2.5m	0.1-0.5m	Soils in this layer expected to liquefy when below the groundwater table at 1.3m bgl.
U3a	Grey fine to medium SAND, “dense to very dense”; moist. CPT qc = 10-19 MPa $q_{allowable}$ = 95-270 kPa	2.4-4m	2-3m	Dense sand layer not predicted to liquefy. HA 1, 2, 3 terminated at 3m.
U3b	Grey fine to medium SAND, “medium dense to dense”; moist. CPT qc = 10-26 MPa $q_{allowable}$ = 95-270 kPa	~32m	5.4-6m	All three CPT’s reached target depth (20m).
U4	GRAVEL (Riccarton)	-	37m	Identified in two ECan BH’s located approx. 300m south of the site.

Table 2: Interpreted Ground Conditions

6 Analysis and Interpretation

6.1 Liquefaction Potential

A liquefaction assessment has been completed using Liquefy Pro (Version 5.5b). Cone Penetration Tests (CPT 001, CPT 002, and CPT 003) form the basis for prediction of liquefaction potential (using the modified Robinson method), with the water table set at 1.3 m depth and a Magnitude 7.5 earthquake considered.

The analysis includes a correlation of the land damage observed in the 22 February - 13 June earthquakes with the predicted potential liquefaction and subsidence in future seismic events. Results from the analysis are summarised in Table 3. Refer to Appendix G for the analysis plots.

Two liquefiable layers were identified within the upper 20m of the ground profile; between depths of 1.3m to 3.0m and 6.0m to 20m.

Event	**Magnitude**	**Peak Ground Acceleration (PGA)**	**CPT 001 LiquefyPro Predicted settlement**	**CPT 002 LiquefyPro Predicted settlement**	**CPT 003 LiquefyPro Predicted settlement**	**Liquefaction Observed, Estimated Settlement**
Darfield Earthquake 4 Sept 2010	7.0	0.24g(1)	30mm	10mm	30mm	Minor
22 Feb 2011 Earthquake	6.3	1.25g(1)	270mm	180mm	220mm	Up to 70mm Differential Settlement Recorded (total settlement unknown)
13 June 2011 Earthquake	6.3	0.59g(1)	150mm	70mm	110mm	
23 December 2011 Earthquake	5.8	0.37g(1)	25mm	10mm	30mm	Yes (total settlement unknown)
Ultimate Limit State (ULS)	7.5	0.35g	130mm	60mm	100mm	N/A
Serviceability Limit State (SLS)	7.5	0.13g	<10mm	<10mm	15mm	N/A

Notes:
(1) Peak Ground Acceleration for actual seismic events recorded is the maximum acceleration taken in either the vertical or horizontal orientation, whichever is greatest, averaged between the two nearest seismograph recorders located 1.1km northeast of the site at the Hulverstone Drive Pumping Station, and 2km southwest of the site at the Pages Road Pumping Station (Cousins, 2012).
(2) The Main Hall is designated in terms of AS/NZS 1170 as Importance Level 2.

Table 3: Liquefaction Analysis for Wainoni Aranui Community Centre

The peak ground accelerations (PGA) for the Ultimate Limit State (ULS) and Serviceability Limit State (SLS) seismic events at the site are based upon extensive probabilistic modelling by GNS Science and observations of land and building damage caused during the Canterbury Earthquake Sequence, as recommended in the Department of Building and Housing guidance document (DBH, 2012). The values in the table are based on a Class D soil type (deep or soft soils), which are considered appropriate for the site, and a design life of 50 years for the structure.

In the liquefaction analysis, a non-liquefiable layer below the groundwater table (layer U3a in table 2) was identified. This layer comprises dense to very dense sand approximately 2.4-4m thick. The presence of this layer is likely to reduce the potential for differential settlement and ground surface damage at the site.

Liquefaction induced settlement in the order of 15mm (SLS) and 130mm (ULS) is predicted in future earthquake events at the Site. In a ULS earthquake event, 60-130mm of differential settlement is possible. Observations from recent earthquake events indicate that a significant proportion of this subsidence may occur as differential settlement. This could cause significant land and building damage, as previously experienced in the 22 February 2011 and 13 June 2011 event.

GNS Science indicates an elevated risk of seismic activity is expected in the Canterbury region as a result of the earthquake sequence following the 4 September 2010 earthquake. Recent advice (Geonet, 2012) indicates there is a 15% probability of another Magnitude 6 or greater earthquake occurring in the next 12 months in the Canterbury region. This event may cause liquefaction induced land damage at the site similar to that experienced, however it is dependent on the location of the earthquakes epicentre. This confirms that there is currently a significant risk of liquefaction and differential settlement occurring at the site.

It is expected that the probability of occurrence is likely to decrease with time following periods of reduced seismic activity.

6.2 Lateral Spread

Lateral spreading occurs where differences in ground level or soil consistency allow liquefied soils to flow laterally toward a low point such as a stream or river where there is no lateral support to the soils. Lateral spreading displacements are typically greatest at the stream banks and become less with distance from the stream. The nearest waterway to the site is the Avon River, which is over 1km to the north and west. In addition, the site is on flat land. Accordingly, the site is evaluated as having a low lateral spread risk.

6.3 Bearing Capacity

Scala Penetrometer tests of the shallow soils indicate the allowable static bearing capacity of the footings on the underlying Sand layer is 95 – 180 kPa (Stockwell, 1977).

Comprehensive foundation analysis under seismic loading has not been completed as the existing foundations have performed adequately in the 22 February 2011 and 13 June 2011 seismic events. Specific assessment should be carried out for any ground or foundation improvement options, taking into account footing dimension and load eccentricity.

7 Foundation Remediation Options

The Department of Building and Housing New Zealand released a Revised Guidance Document on repairing and rebuilding houses in land zoned as Technical Category TC1 & TC2 affected by the Canterbury earthquake sequence (November 2011). Although the hall building is not a residential house, the foundation type and loadings which the structures are subjected to are not dissimilar to a residential house, and the guidance document provides a good indication as to the limits of foundation deformations. For the extent of reported damage sustained to the hall foundations, the guidance document indicates the observed differential settlement exceeds the tolerance for serviceability (variation in floor level is between 50 and 100mm) and a foundation re-level is required (DBH, 2011).

Potential options to remediate the settlement of the Main Hall Foundations are described below. It is acknowledged that any one or a combination of the options could be adopted to remediate the current damage and mitigate the effect of future seismic events.

7.1 Option 1 – Do Minimum

The existing perimeter strip footings appear to have settled by up to 70mm relative to the internal piles following the 22 February 2011 and 13 June 2011 earthquakes. In all three trial pit locations no evidence of significant structural damage to the concrete footings was reported. One remedial solution being considered involves leaving the perimeter footing at its current elevation and packing out/trimming the internal concrete piles to make the timber floor level. Localised repairs to the perimeter footing (including linking the footings together with new ground beams or similar) and replacement of any dislodged/tilted piles as necessary will be completed within this option.

This option minimises the disruption to the building and will expedite the return of use to the community, however total settlements of 60-130mm across the building would be expected to occur in a future ULS seismic event.

7.2 Option 2 – Re-level

In this option, the existing perimeter strip footing would be re-levelled with the internal concrete piles packed out to make the timber floor level. Localised repairs to the perimeter footing and replacement of any dislodged/tilted piles as necessary will be completed within this option.

Three alternative methodologies for re-levelling the existing perimeter strip footings are considered. These include: jacking of the building off shallow pads, jacking off screw piles and using low mobility grout injection.

Further invasive structural inspections of the concrete perimeter footings would be required to enable Structural Engineers to determine the acceptable spacing of jack points for re-levelling.

6.2.1 Shallow Pads

Shallow concrete pads constructed beneath the existing perimeter footing could be installed once the building and existing perimeter footings are jacked up, and lowered onto the re-levelled footing. The shallow investigations completed have confirmed there is sufficient bearing capacity in the surficial soils to support temporary loads on shallow pads. The size and spacing of the pads will be subject to detailed design.

6.2.2 Screw Piles

Screw piles are relatively quick to install and can be installed in low head room situations. Screw piles would be installed adjacent the existing footings to a depth of 3-4m below ground into the dense to very dense sand layers identified in the CPT’s. The screw piles would then be tied into the existing footing to allow transfer of loading.

In addition to installing screw piles around the perimeter footing, they will also need to be installed beneath internal columns and load bearing walls which have settled.

6.2.3 Grout or Resin Injection

The perimeter strip footing may be able to be raised using low mobility grout or resin injection systems. In this option, the existing perimeter footing would be mechanically jacked up, with low mobility grout or resin pumped into the void formed. The risks associated with this methodology are higher than other re-levelling options due to the uncertainty of ground conditions and the relatively limited track record of this type of re-levelling methodology in the Christchurch area. Due to the underlying thick sand layer, further investigations and assessment would be required to determine the viability of this option.

7.3 Option 3 – Ribraft Foundation

A ribraft foundation could be utilised to limit the damage due to differential settlement in a future seismic event. This option encompasses removing the existing timber floor and jacking up the building from its foundations to allow it to be shifted/ slid away from existing footprint. The existing concrete footings would be broken up and removed from site, with a new concrete ribraft foundation poured, and the building lowered back upon the new foundations.

7.4 Option 4 - Piled Foundations

Deep Piles can be used to reduce liquefaction induced total and differential settlement. This option encompasses removing the existing timber floor and jacking up the building from its foundations to allow it to be shifted/ slid away from existing footprint. The existing concrete footings would be broken up and removed from site, with deep piles installed and the building lowered back upon and tied to the piles. The piles are likely to be constructed from either steel or reinforced concrete and would extend to the Riccarton Gravels at least 37m bgl. The piles would be subject to further design to confirm required embedment depth and dimensions.

7.5 Option 5 - Ground Improvement

Improving the ground beneath the Main Hall could comprise installing stone columns or compaction grout columns to densify the surficial soils and provide resistance to liquefaction induced subsidence. The columns would likely extend to 4-5m bgl to provide bearing into the dense to very dense sand identified in the CPT’s. These ground improvements would be designed to ensure subsidence of less than 100mm occurs in any future ULS seismic event.

For stone column installation, the existing timber floor would be removed, with the building jacked up from its foundations to allow it to be shifted/ slid away from existing footprint. The existing concrete footings would be broken up and removed from site, with the stone columns installed prior to a new concrete ribraft foundation being poured, and the building lowered back upon the new foundations.

Alternatively, grout column installation does not require the building to be lifted up or a new ribraft foundation constructed. Grout columns can be installed through pressurised hoses beneath the entire building, with relatively minimal disruption to the structure.

While the cost of installing stone columns is considerably cheaper than grout columns, the additional cost of lifting the building up and constructing a new ribraft foundation may make this option less economical.

8 Discussion

CPT analysis indicated surficial soils at the site comprise sandy SILT to 0.5m depth, underlain by a relatively thick layer of SAND of varying densities down to the GRAVEL (Riccarton) at 37m below ground level. Due to the high groundwater table expected (1.3m bgl) and the loose granular soils identified in the site investigations, soils at the Wainoni Aranui Community Centre are considered susceptible to liquefaction.

Liquefaction analysis identified two potentially liquefiable layers between depths of 1.3m to 3.0m and 6.0m to 20m for the site. Liquefaction induced settlement in the order of 15mm (SLS) and 130mm (ULS) is predicted in a future earthquake event at the Site.

Due to the relatively flat land and substantial setback distance from the nearest waterway, the site is considered to have a low risk of Lateral Spread.

The founding depths of the Main Hall foundations are 500mm and 350mm below the existing ground level. Variability in the underlying geology at these two founding depths is considered negligible over the entire building footprint and cannot be attributed as the sole cause for the differential settlement observed.

Liquefaction analysis indicates that the site could be considered a “Green/Blue-TC3”, in accordance with the Department of Building and Housing zonation (DBH, 2011). For the extent of reported damage sustained to the hall foundations, the DBH guidance document indicates the observed differential settlement exceeds the tolerance for serviceability and a foundation re-level is required. It is considered that foundation remedial options outlined in the Department of Building and Housing guidance document (DBH, 2012) are considered applicable for the Main Hall.

Potential options to remediate the settlement of the Main Hall Foundations include shallow foundation solutions comprising re-levelling or replacement with a ribraft foundation, replacement or integrating deep piles with the existing foundation and ground improvement, as summarised in table 4. It is acknowledged that any one or a combination of the options could be adopted to remediate the current damage and mitigate the effect of future seismic events. It would be prudent to check the consentability of any proposed remediation option/s prior to developing a concept further.

Option	**Description**	**Design Case**	**Likely Seismic Settlements**		**Consequence**	**Risk**
			Differential	**Total**		
1-Do Minimum	Accept perimeter profile and repair/replace internal piles and flooring to match	SLS	15mm	15mm	Floor level exceeds serviceability limits	moderate
		ULS	60-130mm	130mm	Collapse and Structural Damage	low-moderate
2-Relevel	Re-level and repair entire perimeter footing by: jacking off shallow pads or screwpiles injecting low mobility grout or resin	SLS	15mm	15mm	Floor level exceeds serviceability limits	moderate
		ULS	60-130mm	130mm	Collapse and Structural Damage	low-moderate
3-Ribraft Foundation	Lift or move building from footprint, remove and replace foundation with concrete ribraft.	SLS	15mm	15mm	Floor level exceeds serviceability limits	moderate
		ULS	60-130mm	130mm	Collapse and Structural Damage	low
4-Deep Piled Foundations	Support for Perimeter and internal floor beams by deep piled foundation to 37m depth.	SLS	<10mm	<10mm	Floor level exceeds serviceability limits	low
		ULS	<10mm	<10mm	Collapse and Structural Damage	low
5-Ground Improvement	Stone Columns (with concrete ribraft foundation replacement) or Grout Columns	SLS	<10mm	<10mm	Floor level exceeds serviceability limits	low
		ULS	<50mm	<100mm	Collapse and Structural Damage	low

Table 4: Risk Matrix for Foundation Remedial Options

None of the shallow foundation repair options described above (Options 1, 2, 3) will remediate or reduce the susceptibility of soils at the site to liquefy. Ground improvement and/or a deep piled foundation replacement (Options 4 & 5) would be the only options which could further reduce or mitigate the effects of liquefaction induced settlement at the site. However, due to the relatively limited extent of damage sustained to the Main Hall in the recent earthquakes, these options may not be warranted.

Collapse of the Main Hall is assessed to be unlikely to occur in a ULS earthquake event whether Options 1 to 5 are adopted. The cost of mitigation is considered relatively large verses the cost of re-levelling on the basis the existing foundations are relatively easy to repair after a future earthquake. Accordingly, Options 1 and 2 are recommended for the Main Hall building provided the Client accepts that settlements in the order of 15mm and 130mm are likely in a respective SLS or ULS seismic event, and is aware that the probability of such events occurring is elevated at present and will be for some time.

9 Limitation

This report has been prepared solely for the benefit of Ministry of Education as our client with respect to the brief. The reliance by other parties on the information or opinions contained in the report shall, without our prior review and agreement in writing, be at such parties' sole risk.

10 References

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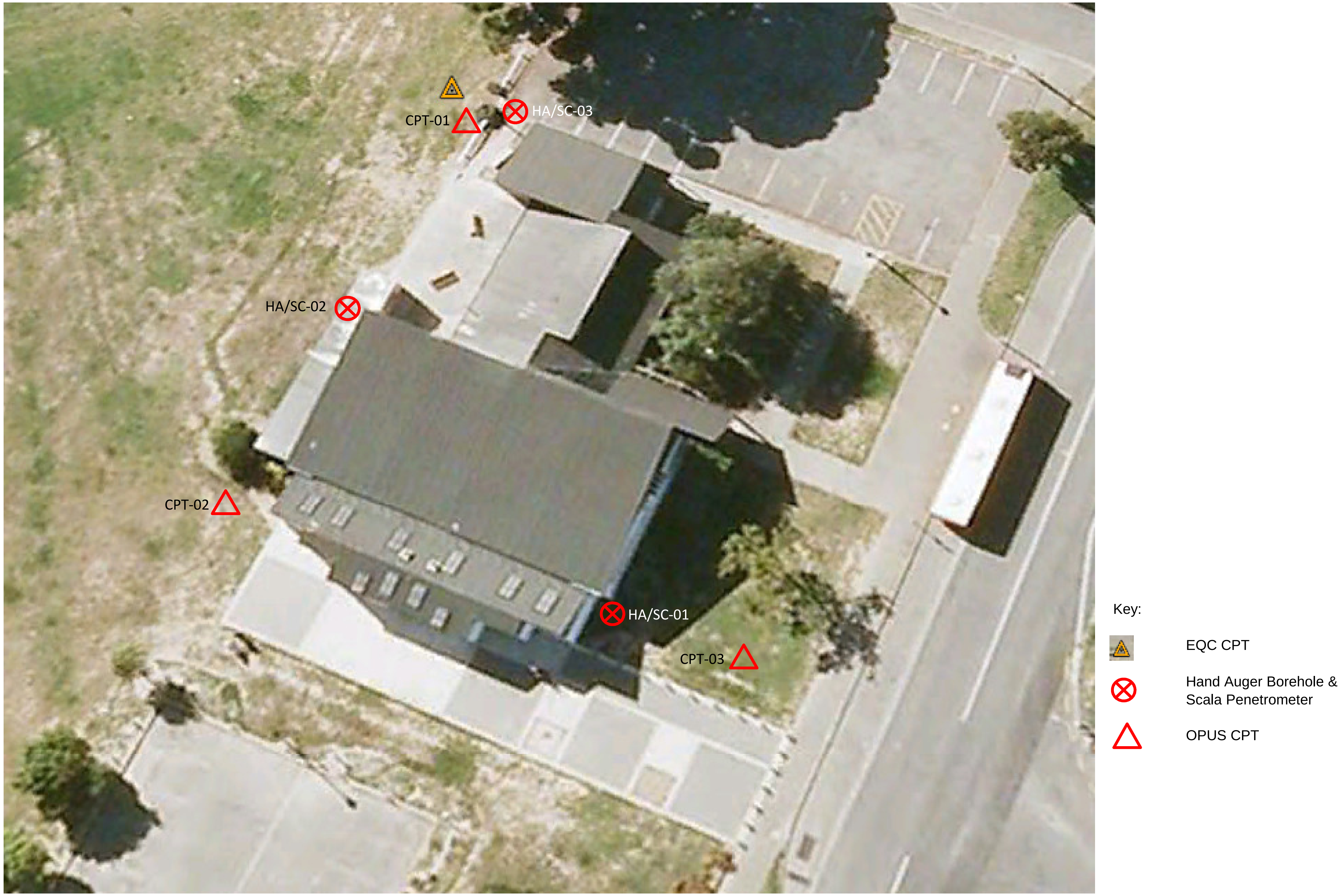
Appendix A:
Test Location Plan



Opus International Consultants Ltd
Christchurch Office
20 Moorhouse Ave
PO Box 1482
Christchurch, New Zealand
Tel: +64 3 363 5400   Fax: +64 3 365 7857

Project: Wainoni Aranui Community Centre
Geotechnical Appraisel

Project No: 6-QUCCC.19 006SC

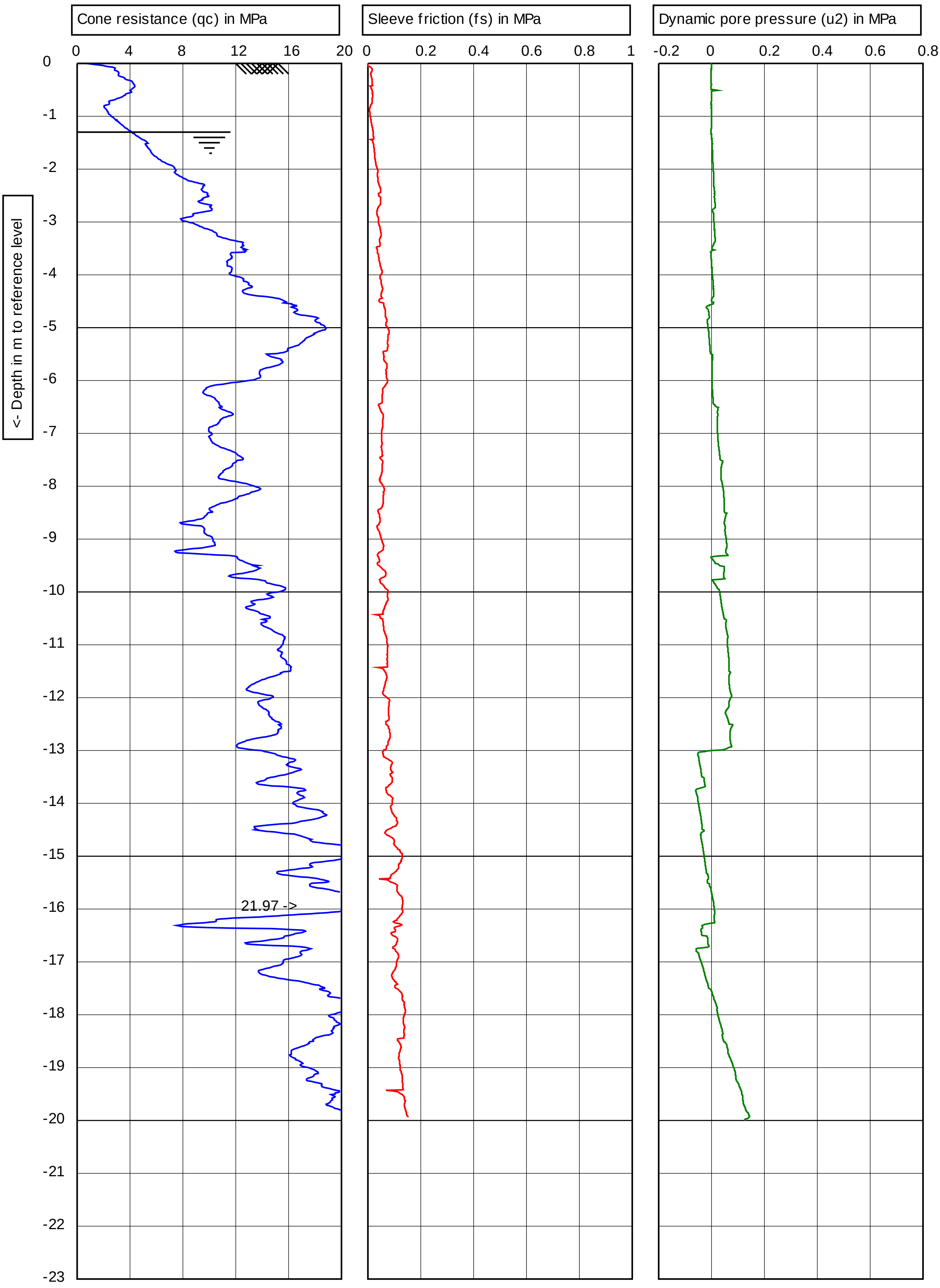
Client: Christchurch City Council

Test Location Plan

Drawn: Mark Broughton
Geotechnical Engineer

Date: 2/02/2012

Appendix B:
Cone Penetrometer Test Results



CPTask V1.20

OPUS
HAMILTON LABORATORIES

u2
150 cm²
10 cm²

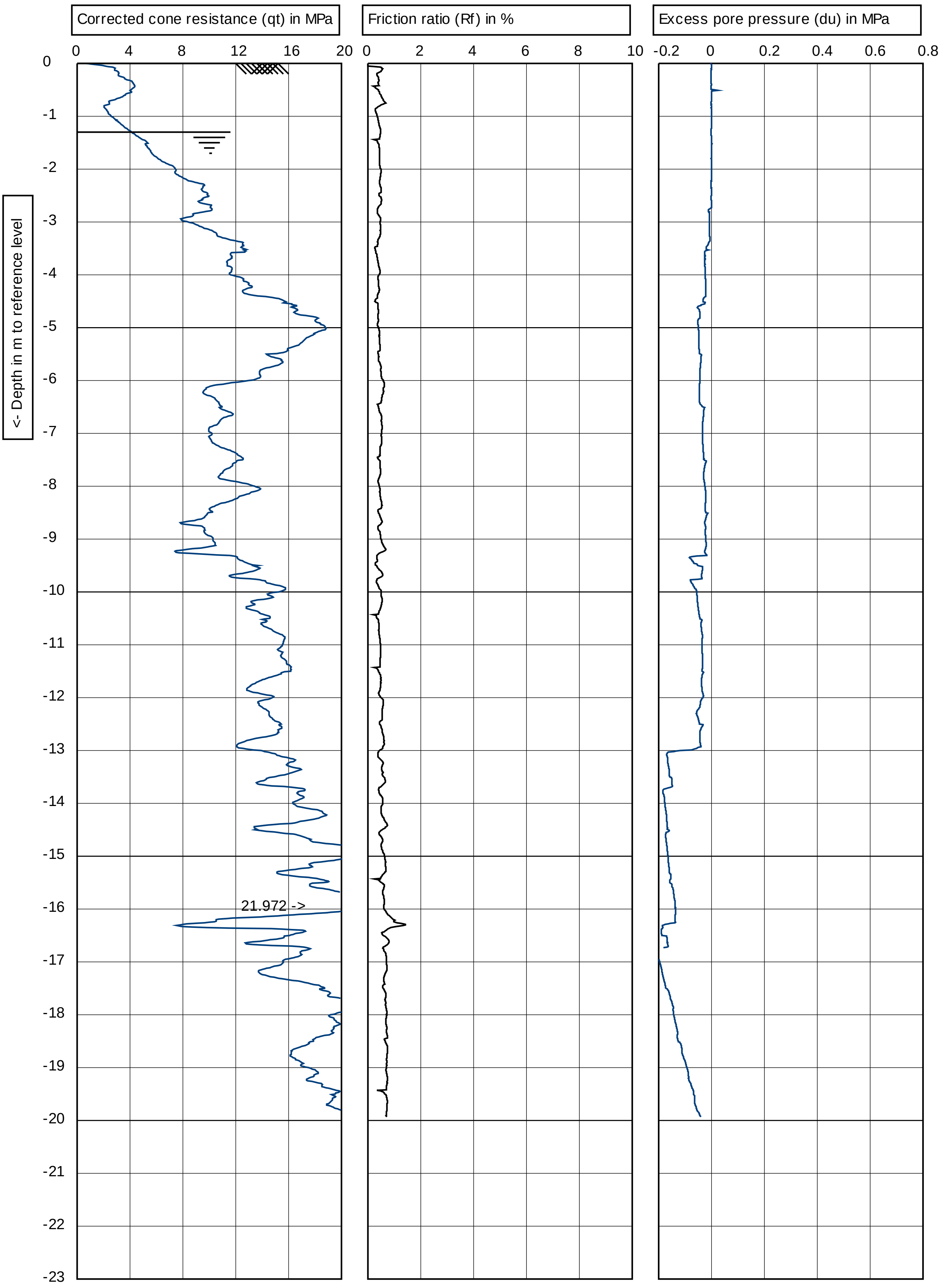
Test according to A.S.T.M standard D-5778-95

G.L. **0**

W.L.: **-1.3**

Project: **Geotechnical Investigation**
Location: **Wainoni Community Centre**
Position:

Predrill : **0**
Date: **6/10/2011**
Cone no.: **C10CFIIP.C10021**
Project no.: **6-quccc.19**
CPT no.: **CPT01** **1/6**



CPTask V1.20

OPUS
HAMILTON LABORATORIES

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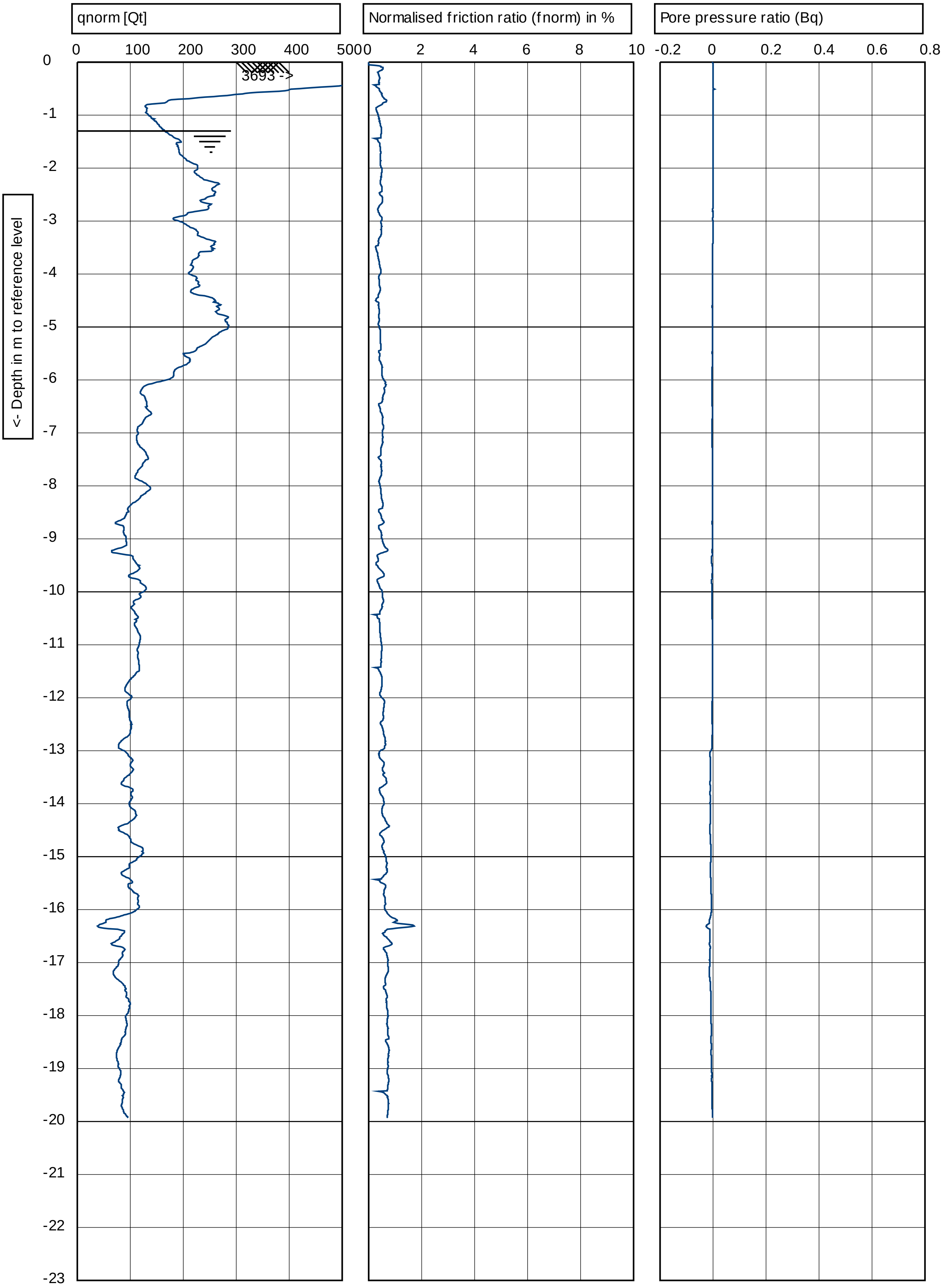
Test according to A.S.T.M standard D-5778-95

G.L. **0**
W.L.: **-1.3**

Project: **Geotechnical Investigation**
Location: **Wainoni Community Centre**
Position:

Predrill :	**0**
Date:	**6/10/2011**
Cone no.:	**C10CFIIP.C10021**
Project no.:	**6-quccc.19**
CPT no.:	**CPT01** **2/6**



OPUS

HAMILTON LABORATORIES

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Test according to A.S.T.M standard D-5778-95

G.L. **0**

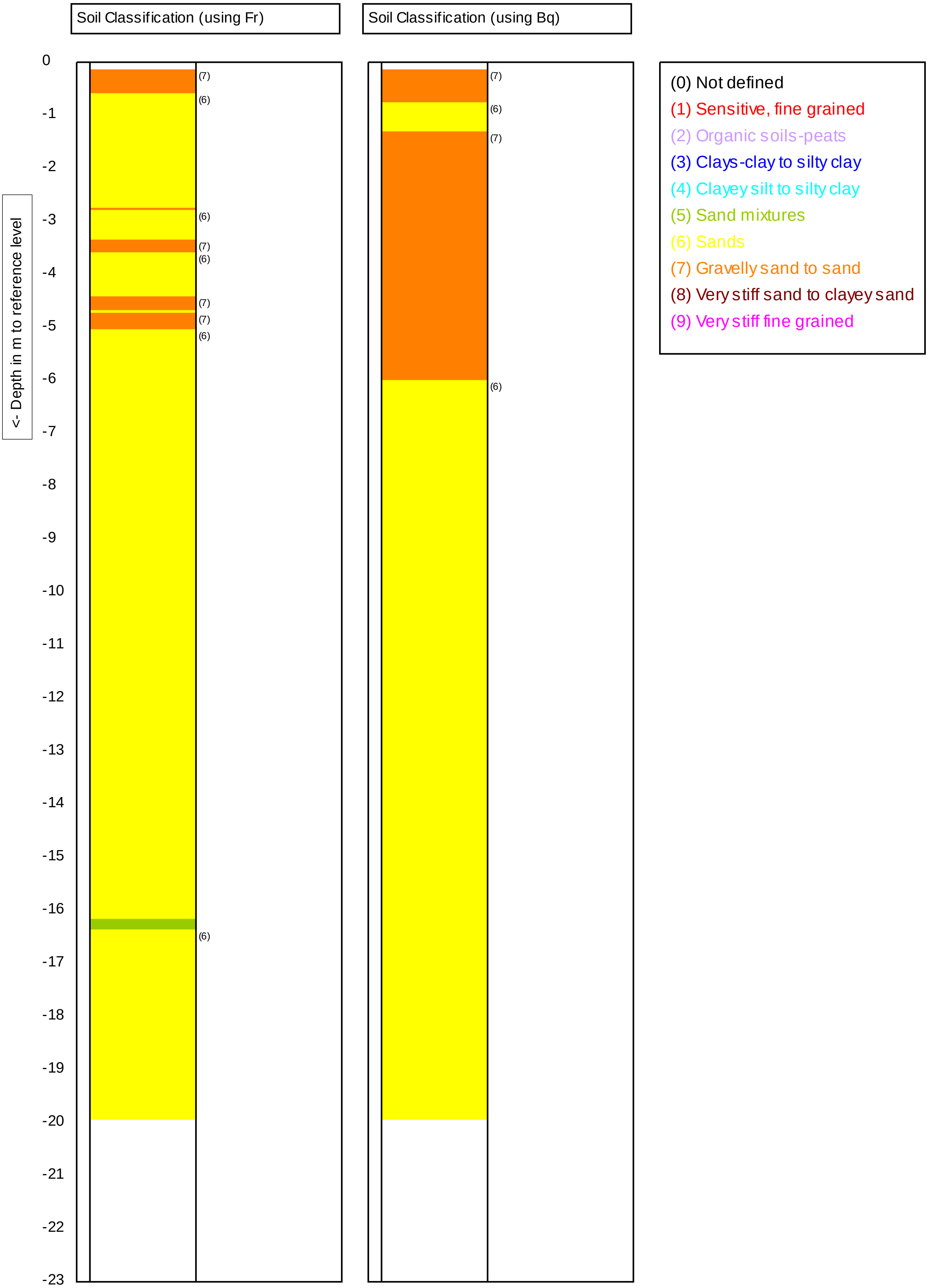
W.L.: **-1.3**

Project: **Geotechnical Investigation**
Location: **Wainoni Community Centre**
Position:

Predrill : **0**
Date: **6/10/2011**
Cone no.: **C10CFIIP.C10021**
Project no.: **6-quccc.19**
CPT no.: **CPT01**

3/6

CPTask V1.20



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HAMILTON LABORATORIES

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Project: **Geotechnical Investigation**

Location: **Wainoni Community Centre**

Position:

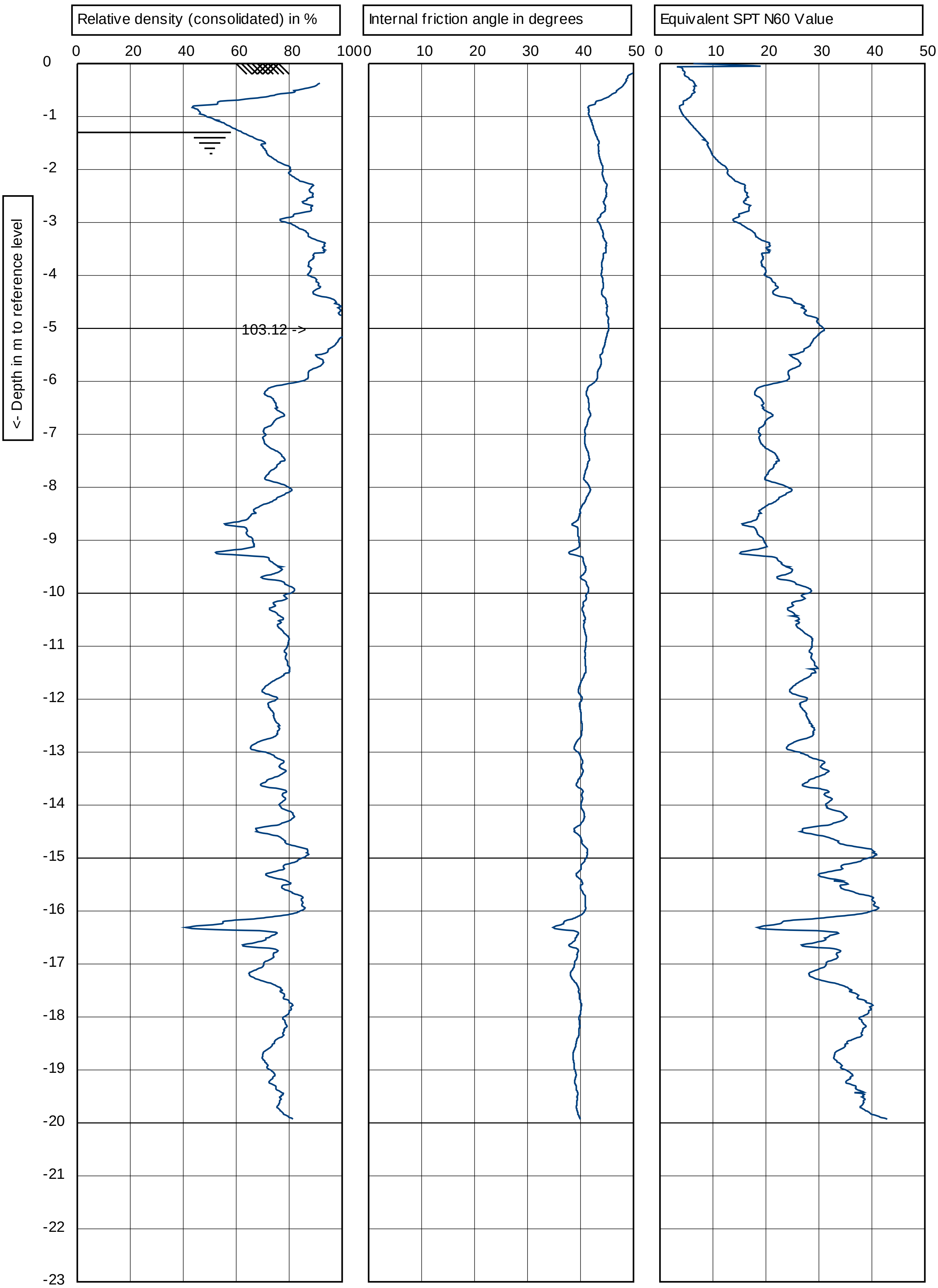
Predrill : **0**

Date: **6/10/2011**

Cone no.: **C10CFIIP.C10021**

Project no.: **6-quccc.19**

CPT no.: **CPT01**



CPTask V1.20

OPUS
HAMILTON LABORATORIES

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	G.L. 0	W.L.: -1.3	Date:	6/10/2011
Project:	**Geotechnical Investigation**		Cone no.:	**C10CFIIP.C10021**
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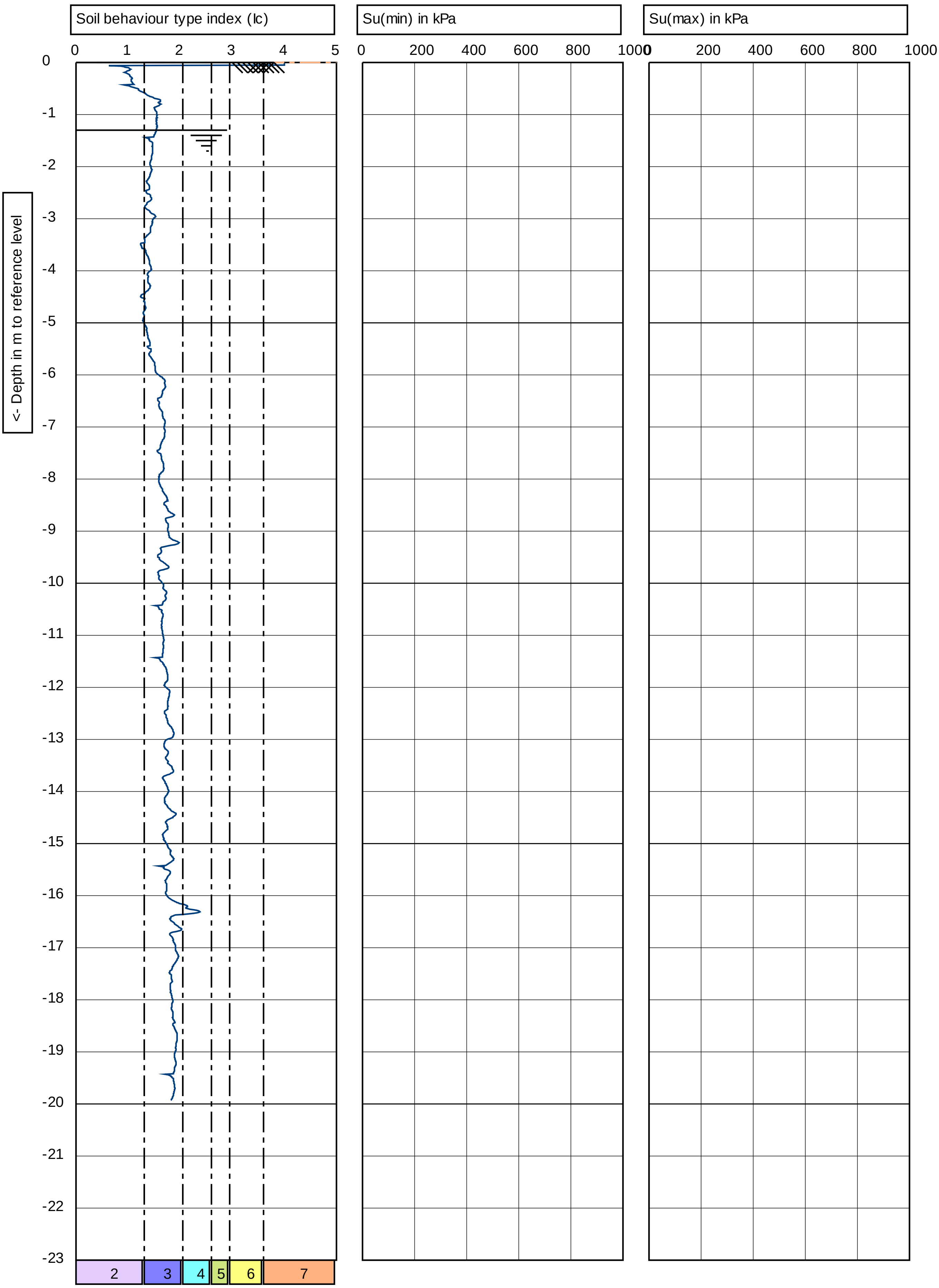
Position:			CPT no.:	**CPT01** 5/6



OPUS
HAMILTON LABORATORIES

CPTask V1.20

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Test according to A.S.T.M standard D-5778-95

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W.L.: **-1.3**

Project: **Geotechnical Investigation**
Location: **Wainoni Community Centre**
Position:

Predrill : **0**
Date: **6/10/2011**
Cone no.: **C10CFIIP.C10021**
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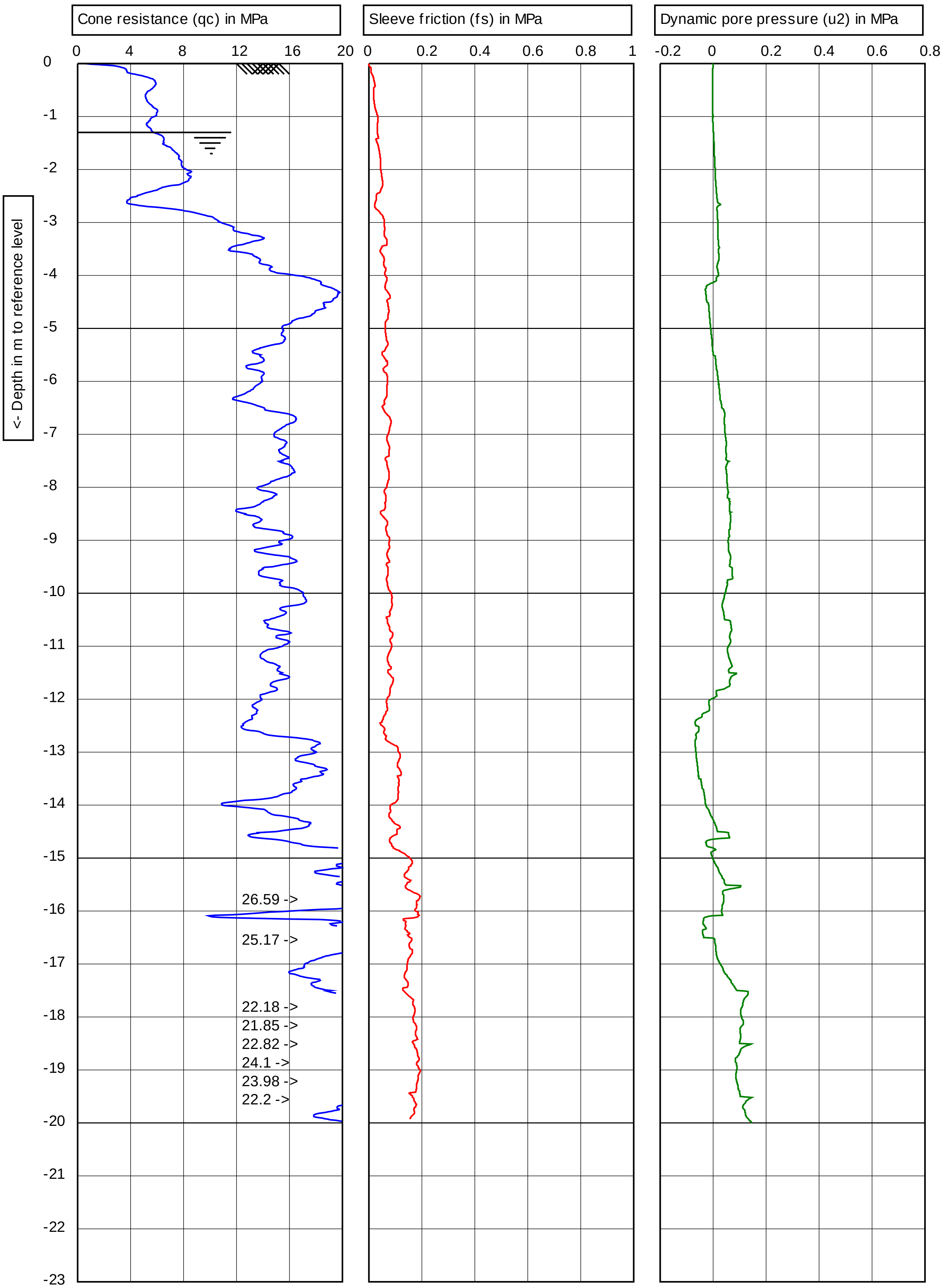
CPT no.: **CPT01** **6/6**



CPTask V1.20

OPUS
HAMILTON LABORATORIES

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10 cm²

Test according to A.S.T.M standard D-5778-95

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Project: **Geotechnical Investigation**
Location: **Wainoni Community Centre**
Position:

Predrill : **0**
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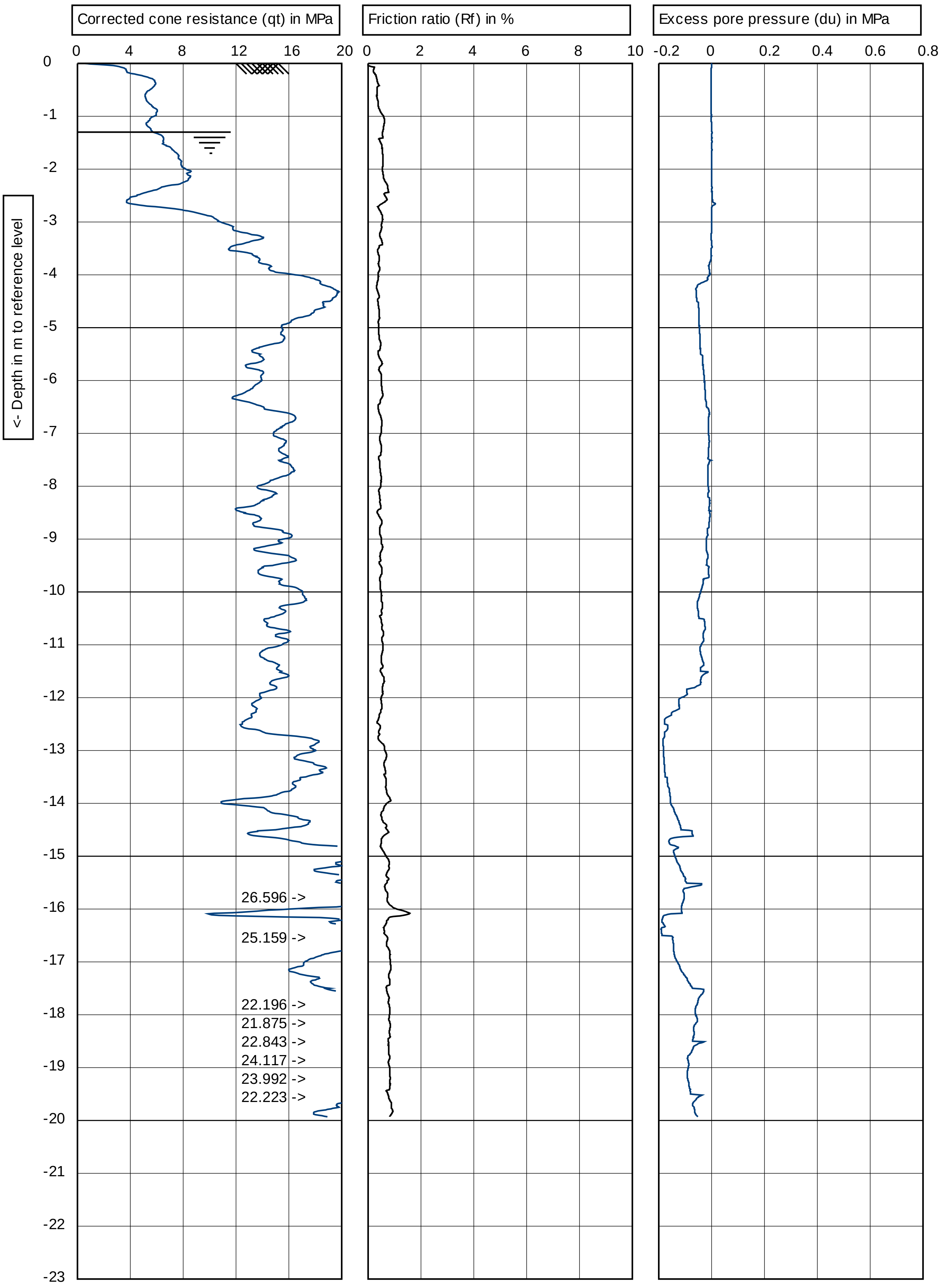
Project no.: **6-QUCCC.19**
CPT no.: **CPT02** **1/6**



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HAMILTON LABORATORIES

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Test according to A.S.T.M standard D-5778-95

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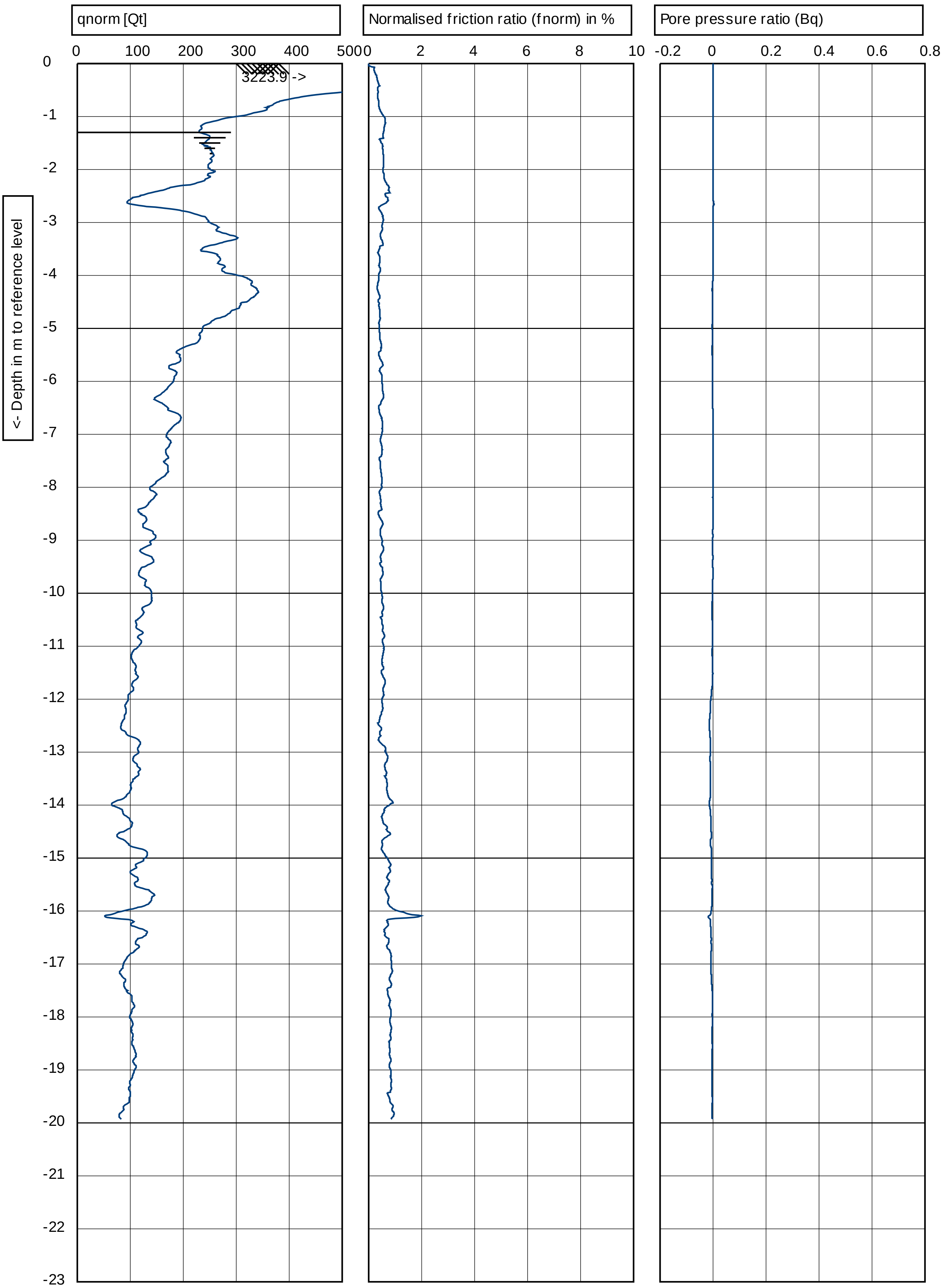
Project: **Geotechnical Investigation**
Location: **Wainoni Community Centre**
Position:

Predrill : **0**
Date: **6/10/2011**
Cone no.: **C10CFIIP.C10021**
Project no.: **6-QUCCC.19**
CPT no.: **CPT02** **2/6**

CPTask V1.20



OPUS

HAMILTON LABORATORIES

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Test according to A.S.T.M standard D-5778-95

G.L. **0**

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Project: **Geotechnical Investigation**

Location: **Wainoni Community Centre**

Position:

Predrill :	**0**
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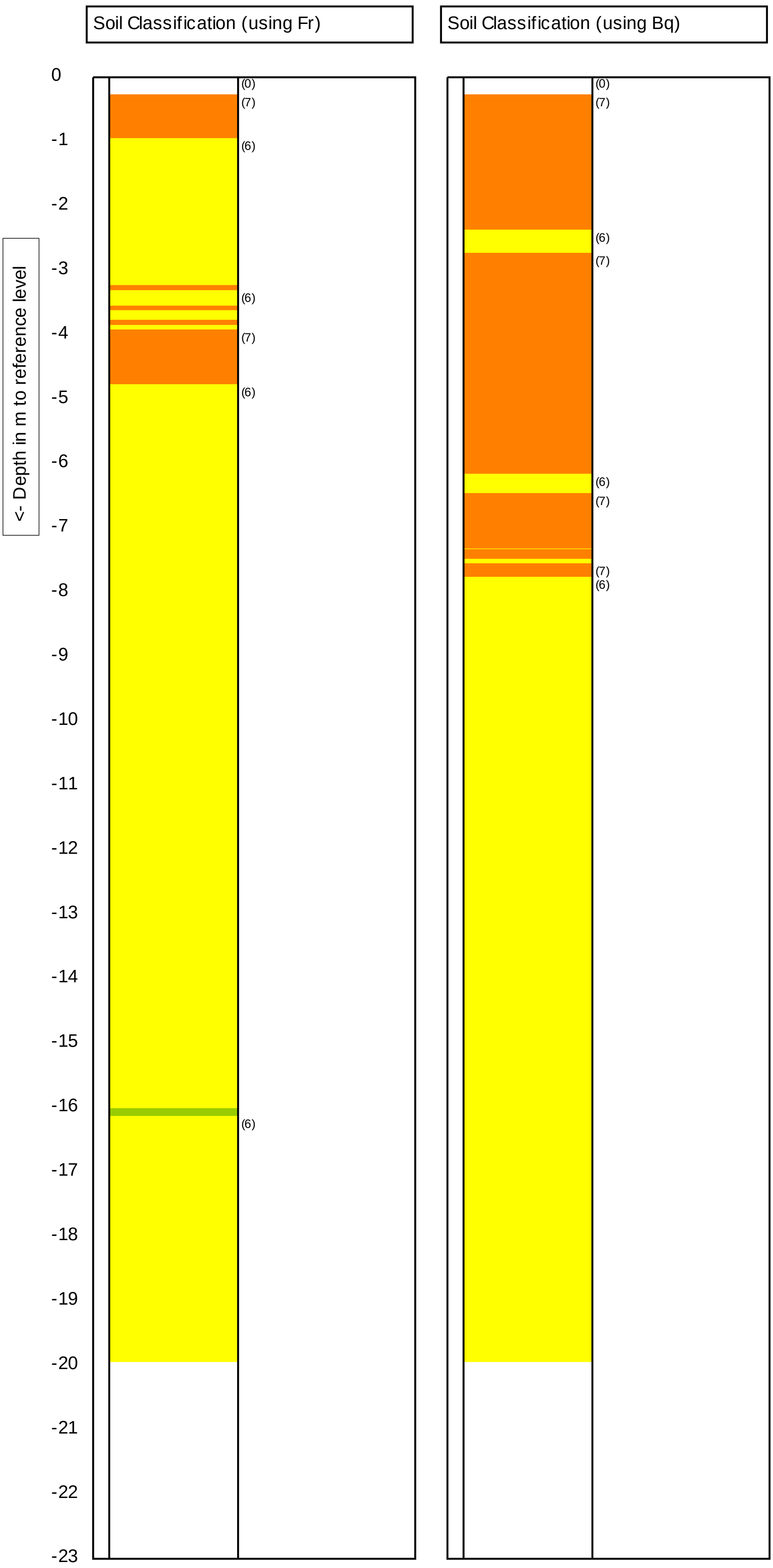
CPT no.:	**CPT02**

CPTask V1.20



(0) Not defined
(1) Sensitive, fine grained
(2) Organic soils-peats
(3) Clays-clay to silty clay
(4) Clayey silt to silty clay
(5) Sand mixtures
(6) Sands
(7) Gravelly sand to sand
(8) Very stiff sand to clayey sand
(9) Very stiff fine grained

CPTask V1.20

OPUS
HAMILTON LABORATORIES

u2
150 cm²
10 cm²

Test according to A.S.T.M standard D-5778-95

G.L. **0**   W.L.: **-1.3**

Project: **Geotechnical Investigation**
Location: **Wainoni Community Centre**
Position:

Predrill :	**0**
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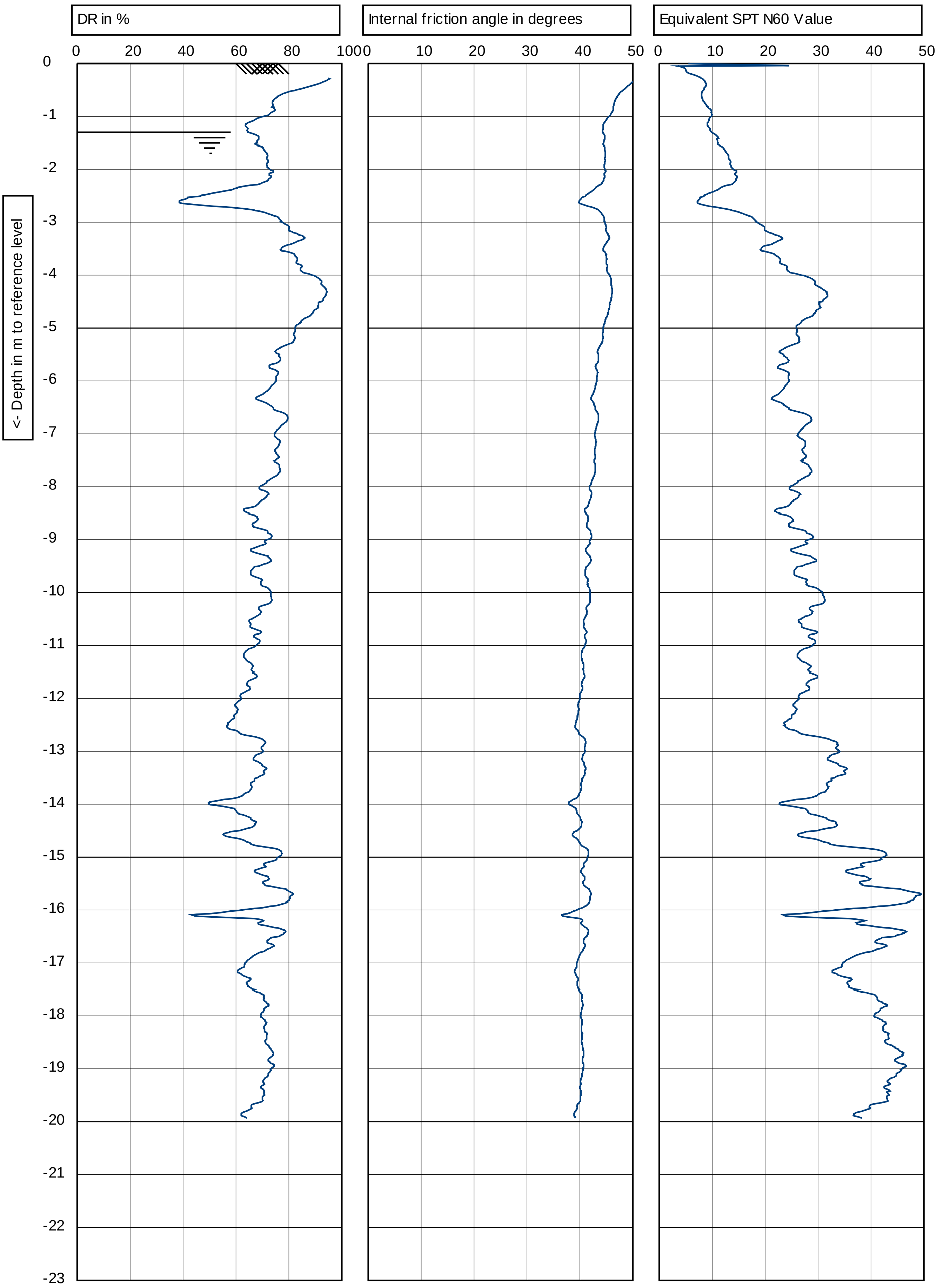
CPT no.:	**CPT02** **4/6**



CPTask V1.20

OPUS
HAMILTON LABORATORIES

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10 cm²

Test according to A.S.T.M standard D-5778-95

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W.L.: **-1.3**

Project: **Geotechnical Investigation**
Location: **Wainoni Community Centre**
Position:

Predrill : **0**
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Cone no.: **C10CFIIP.C10021**
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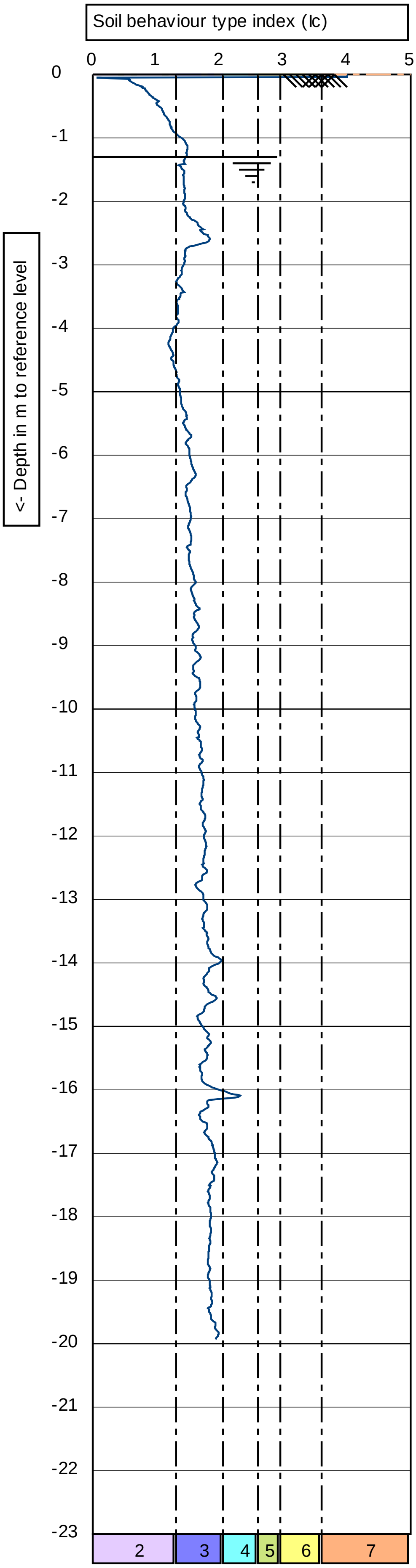
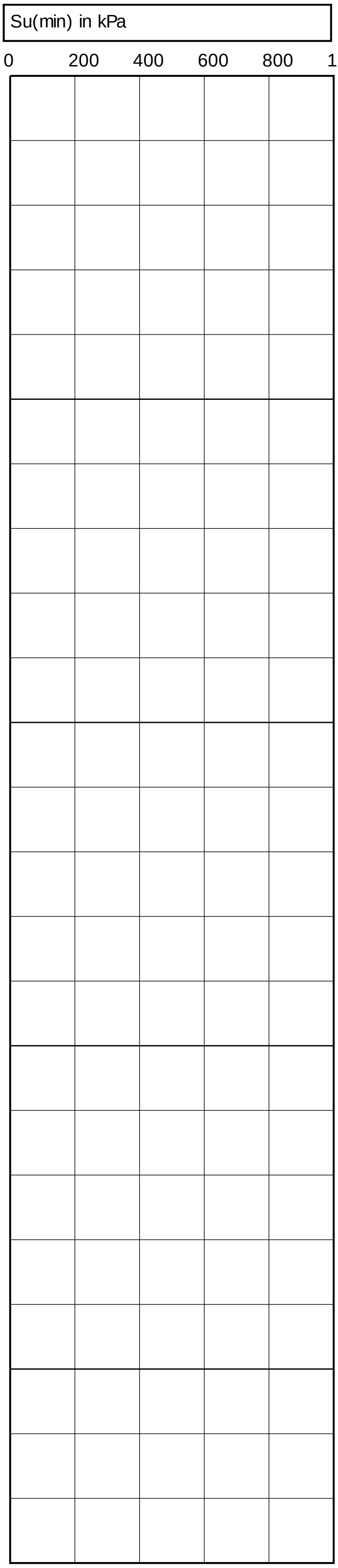
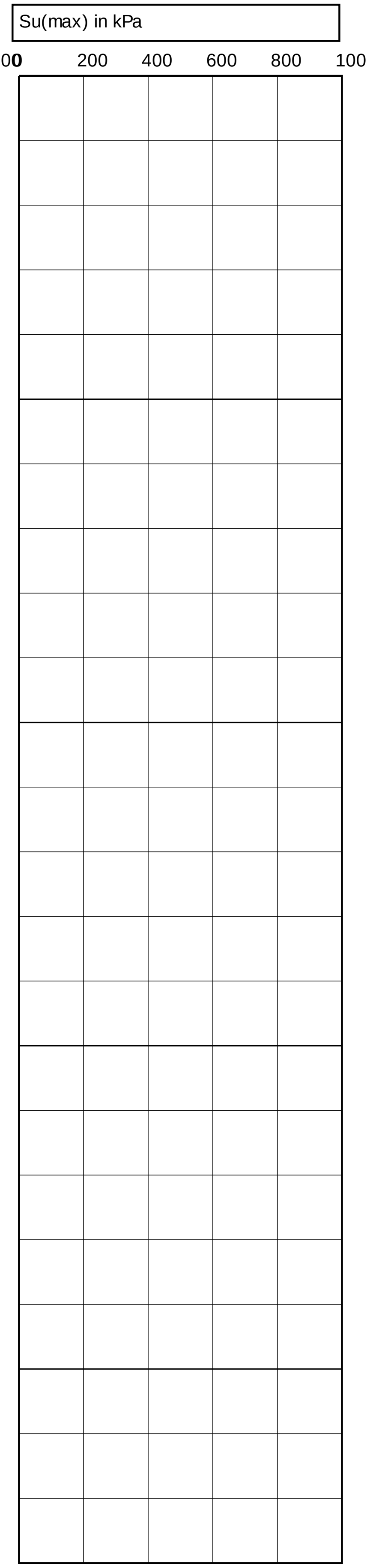
CPT no.: **CPT02** **5/6**







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HAMILTON LABORATORIES

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Test according to A.S.T.M standard D-5778-95

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Project: **Geotechnical Investigation**
Location: **Wainoni Community Centre**
Position:

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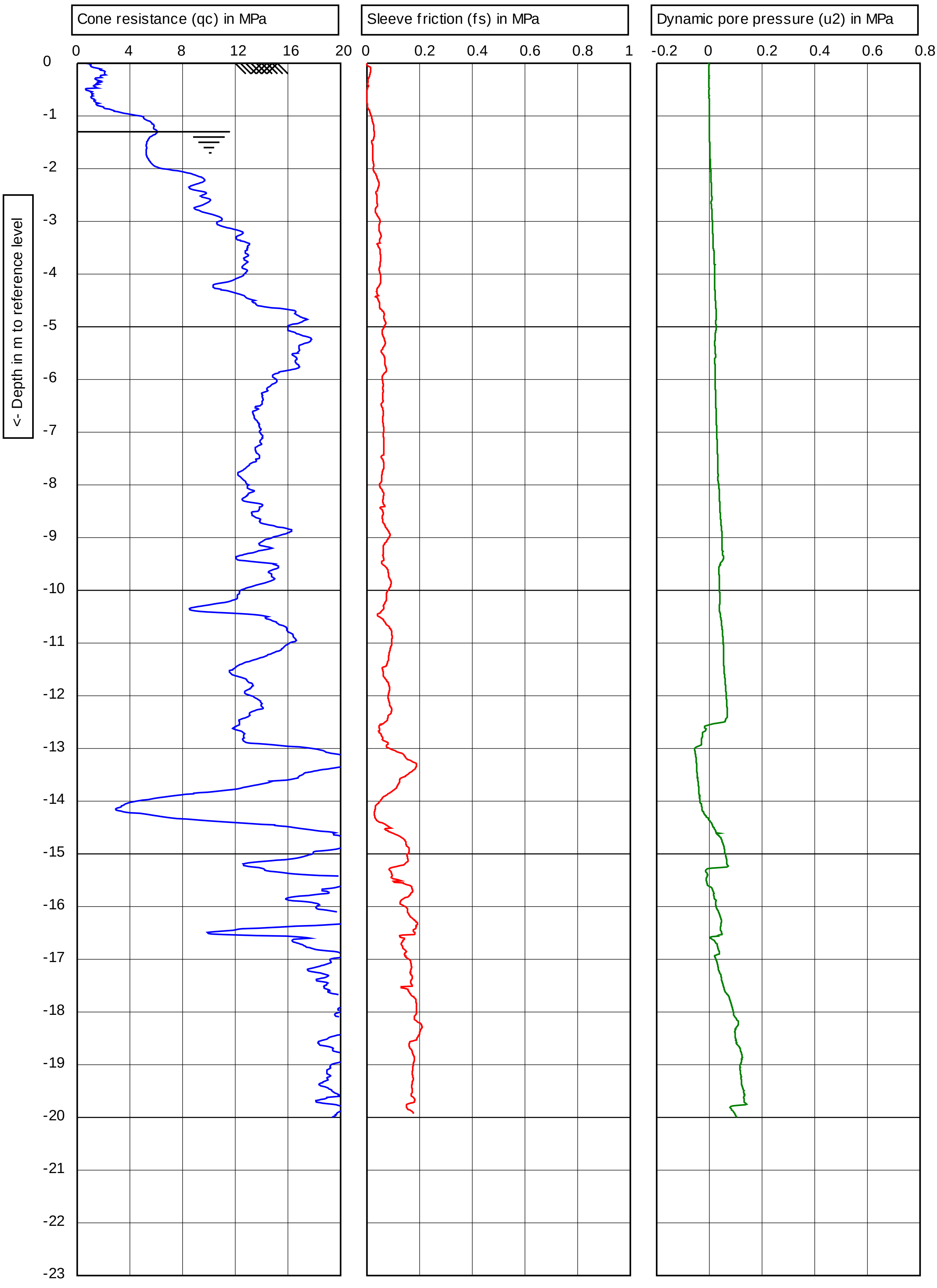
CPT no.: **CPT02**
6/6

CPTask V1.20



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Project: **Geotechnical Investigation**		Cone no.:	**C10CFIIP.C10021**
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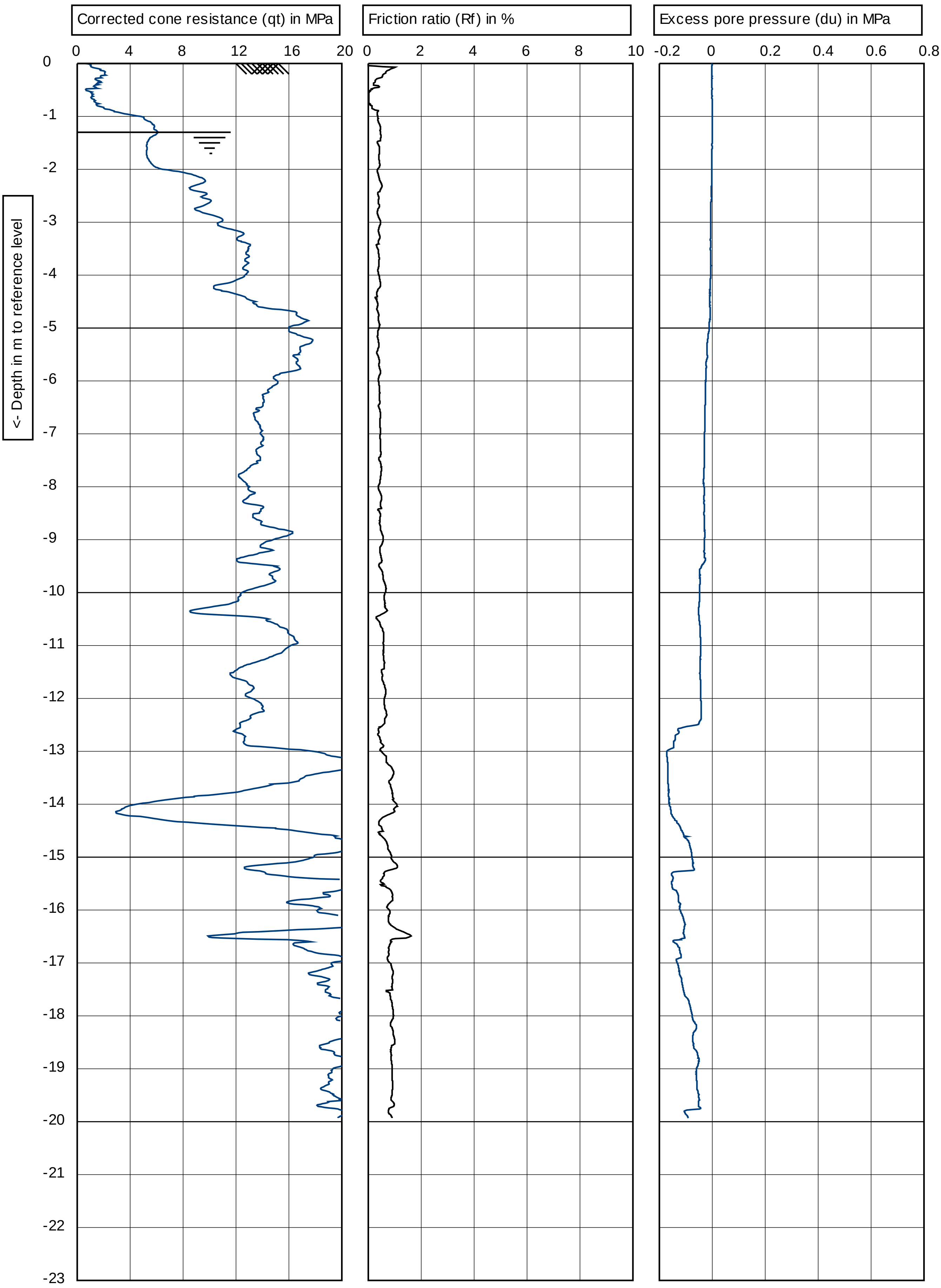
Position:		CPT no.:	**CPT03** 1/6

CPTask V1.20



CPTask V1.20

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HAMILTON LABORATORIES

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W.L.: **-1.3**

Project: **Geotechnical Investigation**
Location: **Wainoni Community Centre**
Position:

Predrill :	**0**
Date:	**6/10/2011**
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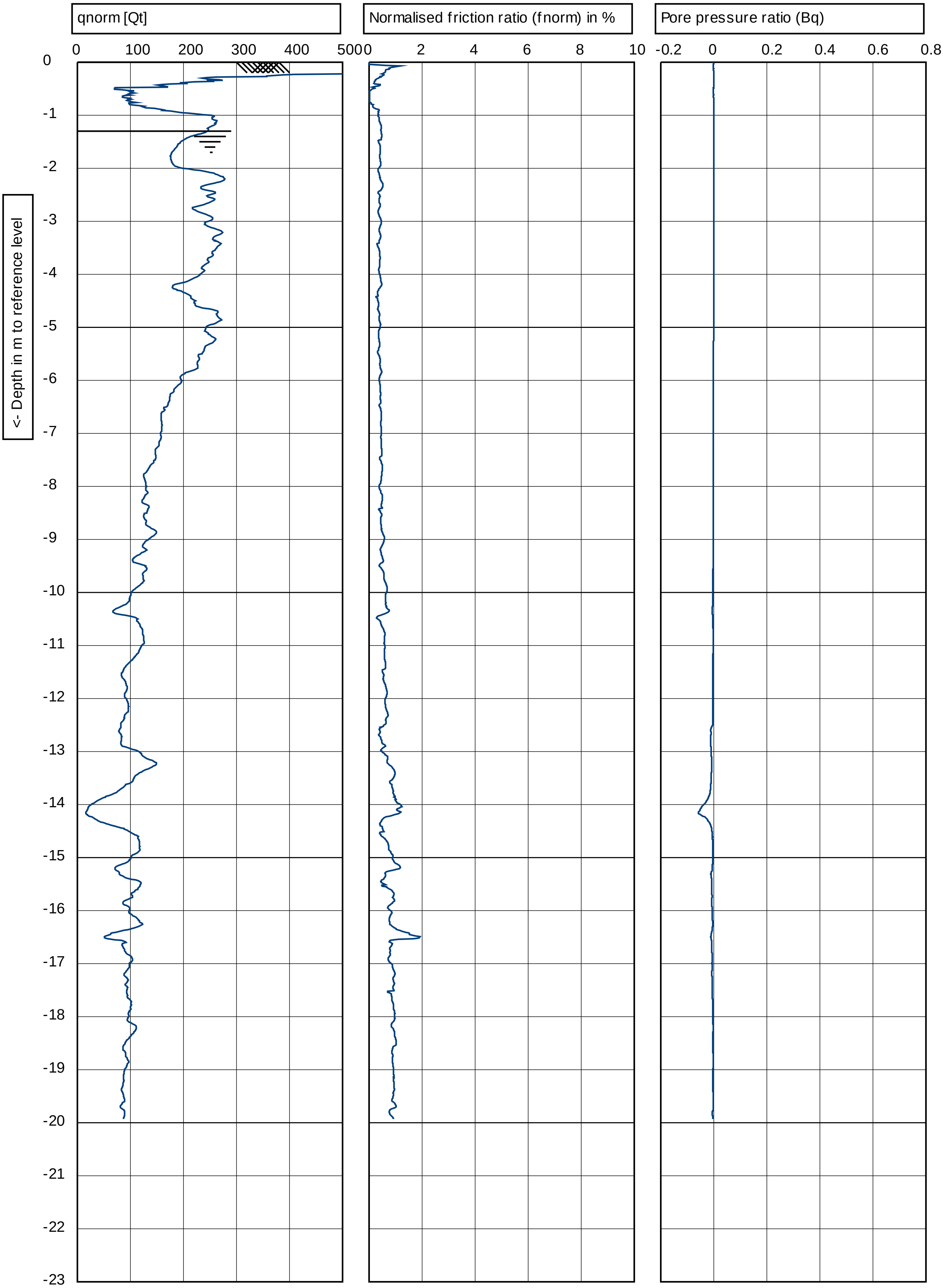
CPT no.:	**CPT03** **2/6**



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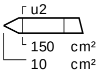
HAMILTON LABORATORIES



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Project: **Geotechnical Investigation**

Location: **Wainoni Community Centre**

Position:

Predrill : **0**

Date: **6/10/2011**

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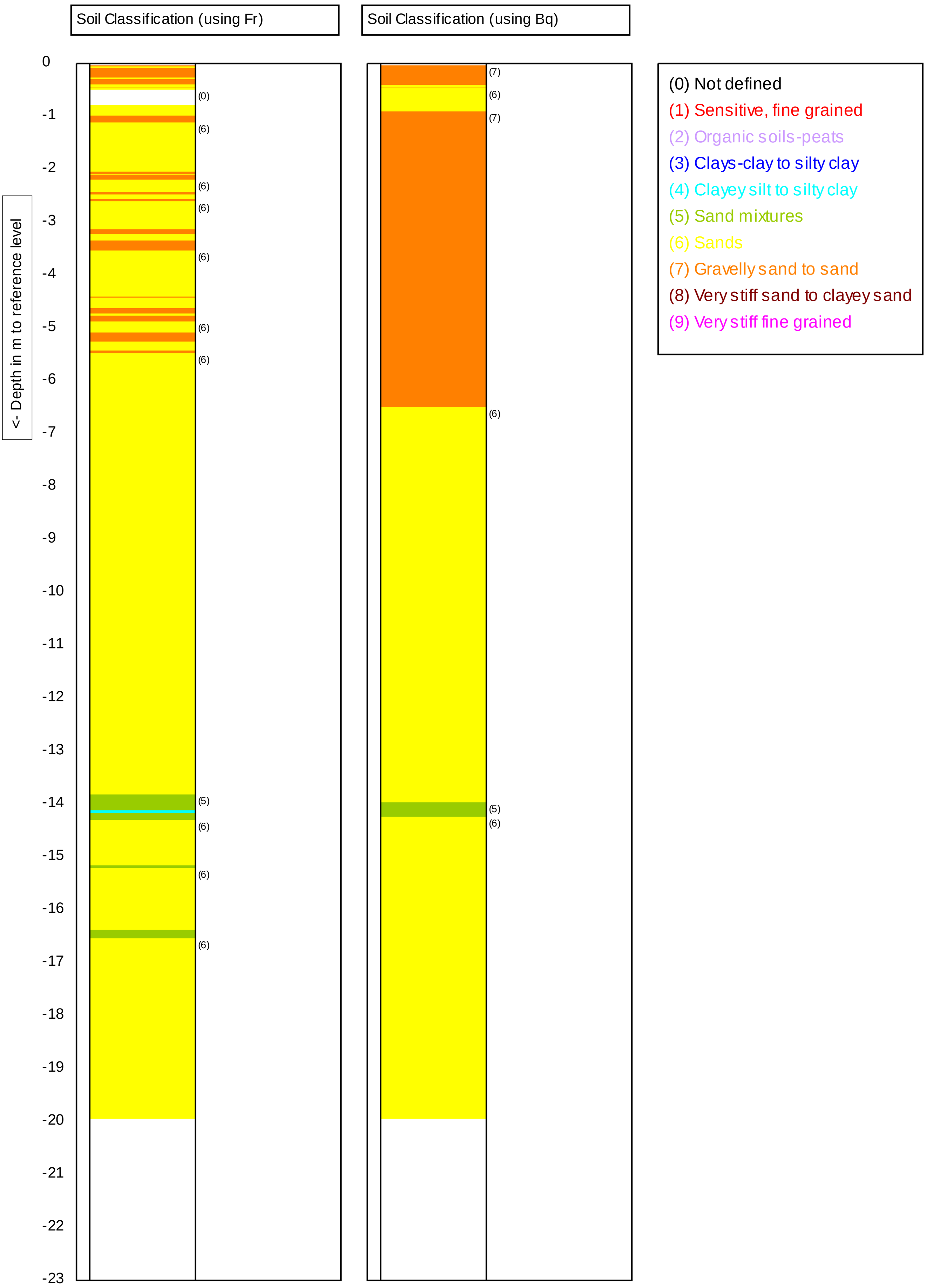
Project no.: **6-QUCCC.19**

CPT no.: **CPT03** **3/6**

CPTask V1.20



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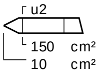
HAMILTON LABORATORIES



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G.L. **0**

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Predrill : **0**

Date: **6/10/2011**

Project: **Geotechnical Investigation**

Location: **Wainoni Community Centre**

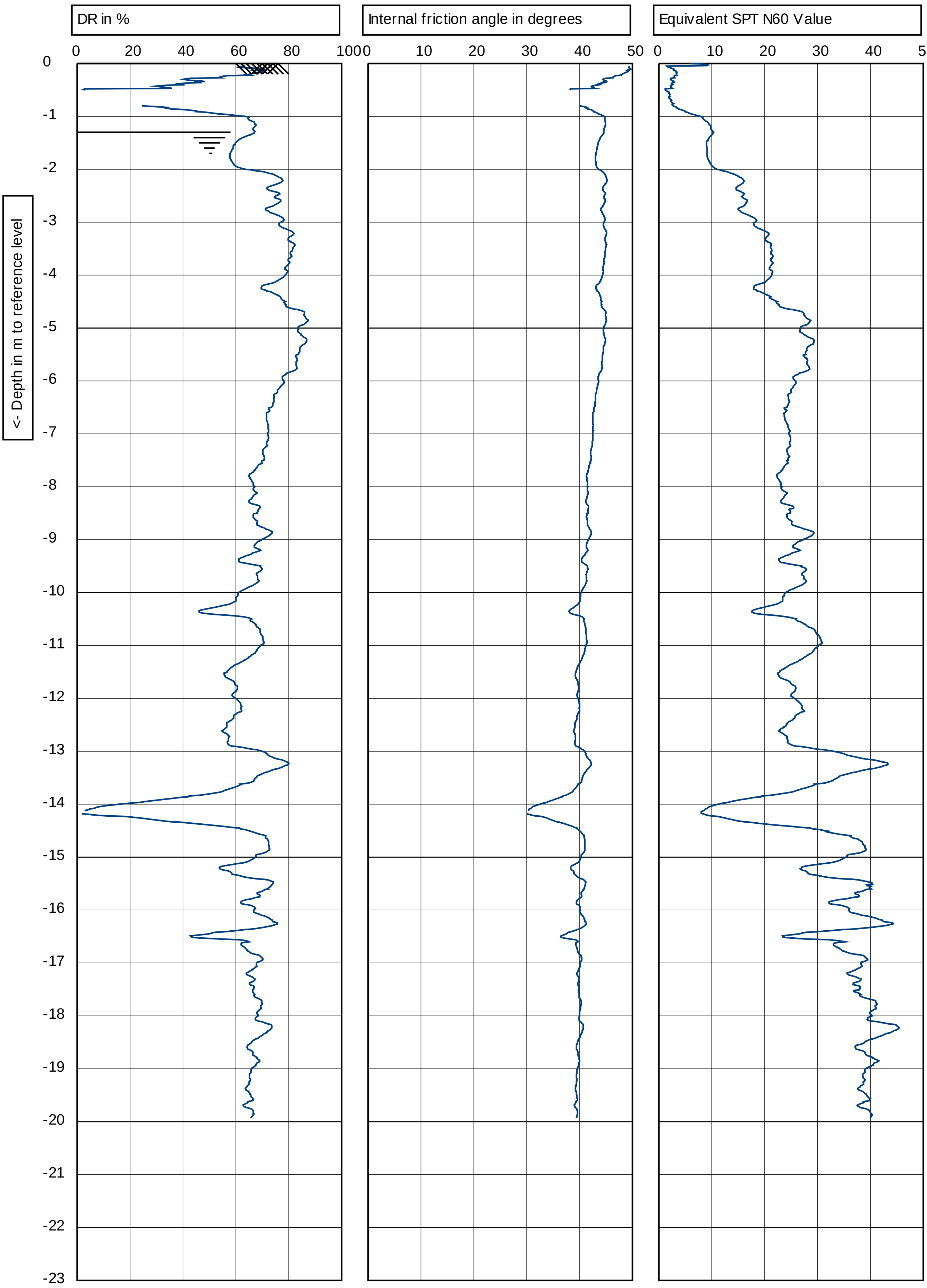
Position:

Cone no.: **C10CFIIP.C10021**

Project no.: **6-QUCCC.19**

CPT no.: **CPT03**

CPTask V1.20



OPUS
HAMILTON LABORATORIES

u2
150 cm²
10 cm²

Test according to A.S.T.M standard D-5778-95		Predrill :	**0**
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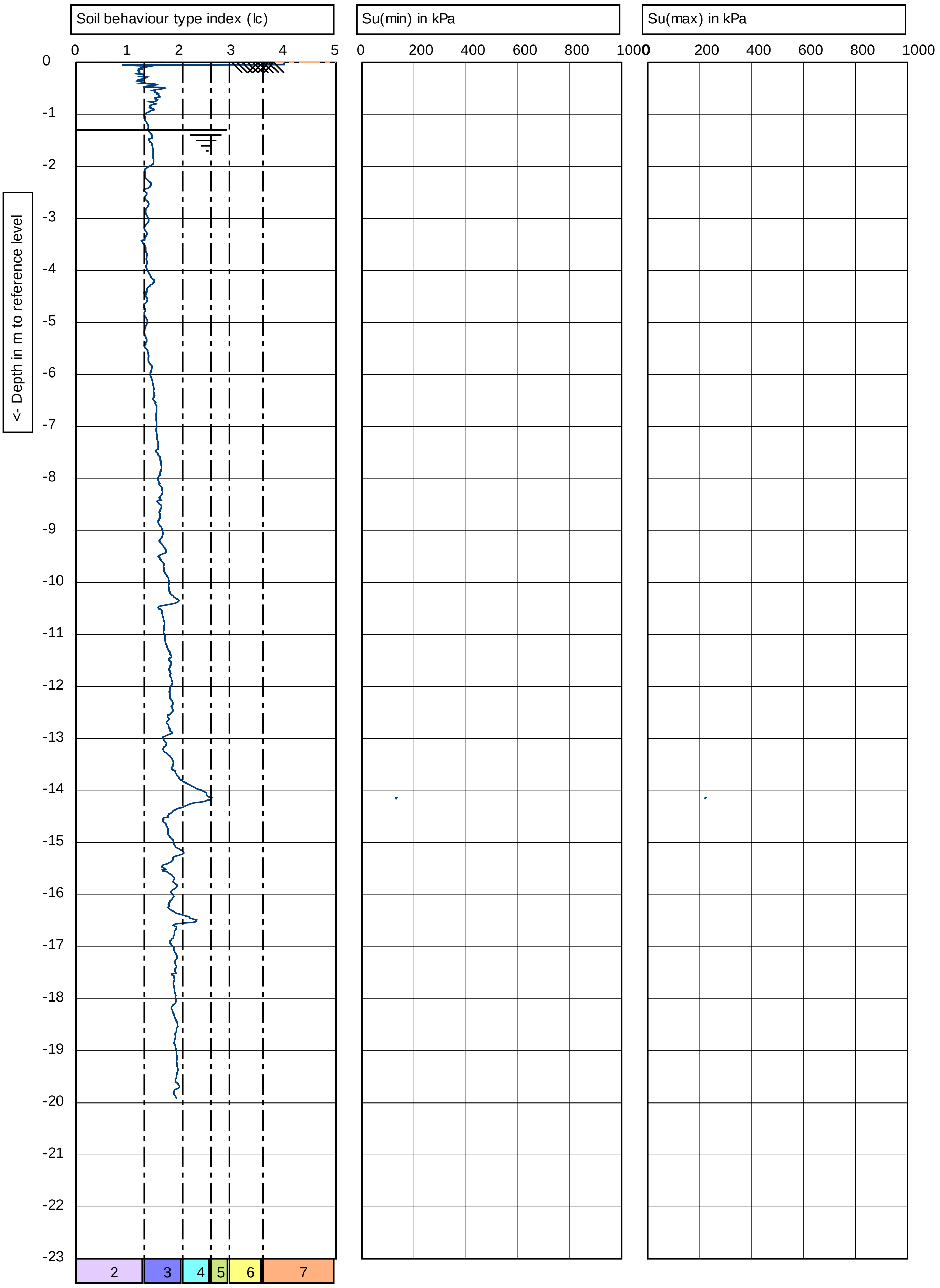
Project: **Geotechnical Investigation**		Cone no.:	**C10CFIIP.C10021**
Location: **Wainoni Community Centre**		Project no.:	**6-QUCCC.19**
Position:		CPT no.:	**CPT03** **5/6**



HAMILTON LABORATORIES

Test according to A.S.T.M standard D-5778-95		Predrill :	**0**	
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Project: **Geotechnical Investigation**		Cone no.:	**C10CFIIP.C10021**	
Location: **Wainoni Community Centre**		Project no.:	**6-QUCCC.19**	
Position:		CPT no.:	**CPT03**	**6/6**

CPTask V1.20

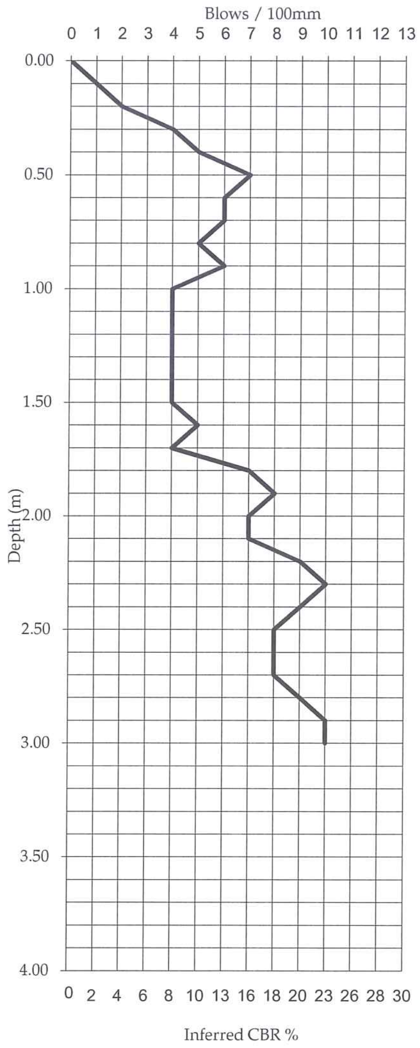
Appendix C:
Shallow Soils Investigation

AUGER / SCALA PENETROMETER TEST REPORT

Project :	**Wainoni Community Centre Geotechnical Assessment**
Location :	**31 Hampshire Street**
Client :	**Christchurch City Council**
Contractor :	**Opus Christchurch Laboratory**
Test number :	**Scala / Auger 1**
Shear vane number :	**N/A**
Shear vane correction :	**N/A**
Water level (m):	**1.7**

Project No :	**6QUCCC.19/045LC**
Lab Ref No :	**6224**
Client Ref No :	

Scala Penetrometer



Test Results

Depth (m)	Shear Strength (kPa)	Soil Description
0 - 0.1		Dark brown organic topsoil, damp, rootlets (Topsoil & grass)
0.1 - 1.2		Light brown SAND, loosely packed, moist
1.2 - 1.7		Grey fine - medium SAND, dense, wet
1.70 EOB		Grey SAND, saturated (auger hole collapse)

Test Methods

Determination of Penetration Resistance of a Soil, NZS 4402 : 1988, Test 6.5.2	Field Descriptions of Soils and Rocks by
Shear Strength using a Hand Held Shear Vane: NZ Geotechnical Soc Inc 8/2001	NZ Geotechnical Society Dec 2005
Inferred CBR values taken from Austroads Pavement Design Manual 2004	Inferred CBR values are not IANZ accredited

Date tested : 29 September 2011
Date reported : 17 October 2011

This report may only be reproduced in full

IANZ Approved Signatory

Designation : *Senior Civil Engineering Technician*
Date : 17 October 2011

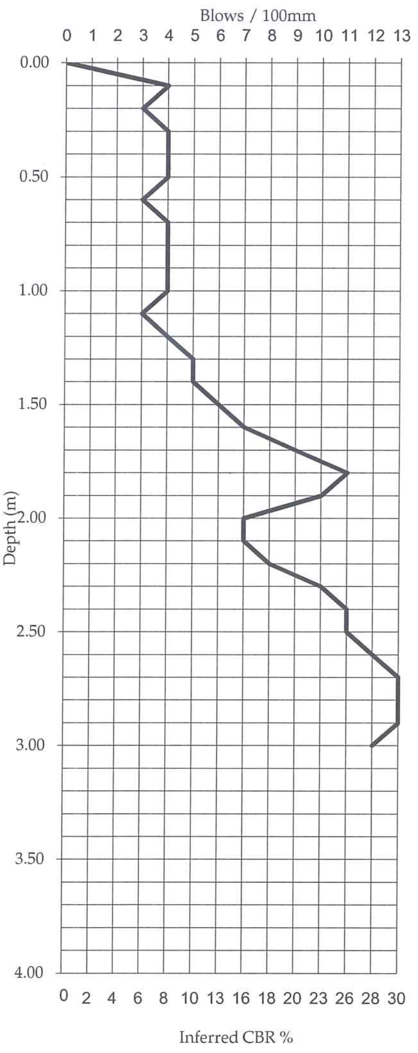
All tests reported herein have been performed in accordance with the laboratory's scope of accreditation

AUGER / SCALA PENETROMETER TEST REPORT

Project : **Wainoni Community Centre Geotechnical Assessment**
Location : **31 Hampshire Street**
Client : **Christchurch City Council**
Contractor : **Opus Christchurch Laboratory**
Test number : **Scala / Auger 2**
Shear vane number : **N/A**
Shear vane correction : **N/A**
Water level (m): **1.7**

Project No : **6QUCCC.19/045LC**
Lab Ref No : **6224**
Client Ref No :

Scala Penetrometer



Test Results

Depth (m)	Shear Strength (kPa)	Soil Description
0 - 0.1		Dark brown Sandy SILT, trace fine gravel, dry, firm low plasticity
0.1 - 0.9		Light brown SAND, trace dark brown silt, loose, dry
0.9 - 1.1		Light brown SAND, trace dark brown silt,loose, moist
1.1 - 1.7		Grey SAND, moist
1.70 EOB		Grey fine - medium SAND, saturated (auger hole collapse)

Test Methods

Determination of Penetration Resistance of a Soil, NZS 4402 : 1988, Test 6.5.2	Field Descriptions of Soils and Rocks by
Shear Strength using a Hand Held Shear Vane: NZ Geotechnical Soc Inc 8/2001	NZ Geotechnical Society Dec 2005
Inferred CBR values taken from Austroads Pavement Design Manual 2004	Inferred CBR values are not IANZ accredited

Date tested : 29 September 2011
Date reported : 5 October 2011

This report may only be reproduced in full

IANZ Approved Signatory

Designation : *Senior Civil Engineering Technician*
Date : 5 October 2011

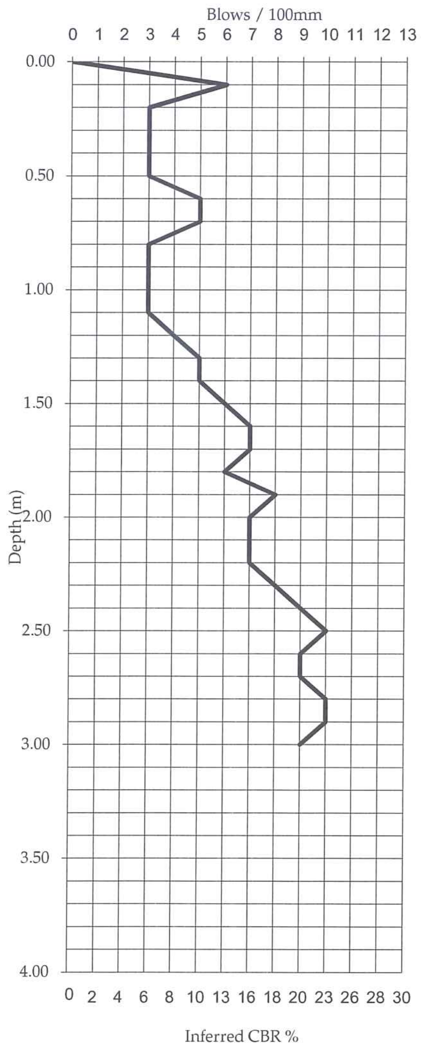
All tests reported herein have been performed in accordance with the laboratory's scope of accreditation

AUGER / SCALA PENETROMETER TEST REPORT

Project : **Wainoni Community Centre Geotechnical Assessment**
Location : **31 Hampshire Street**
Client : **Christchurch City Council**
Contractor : **Opus Christchurch Laboratory**
Test number : **Scala / Auger 3**
Shear vane number : **N/A**
Shear vane correction : **N/A**
Water level (m): **1.7**

Project No : 6QUCCC.19/045LC
Lab Ref No : 6224
Client Ref No :

Scala Penetrometer



Test Results

Depth (m)	Shear Strength (kPa)	Soil Description
0 - 0.2		Dark brown Sandy SILT, trace fine gravel, dry firm, low plasticity
0.2 - 0.5		Dark brown Sandy SILT, moist, loosely packed
0.5 - 1.0		Grey SAND moist, some light brown dry silt dense
1.0 - 1.3		Grey SAND moist, some light brown silt trace fine gravel, dense
1.3 - 1.7		Grey fine - medium SAND, wet
1.70 EOB		Grey fine - medium SAND, saturated (auger hole collapse)

Test Methods

Determination of Penetration Resistance of a Soil, NZS 4402 : 1988, Test 6.5.2	Field Descriptions of Soils and Rocks by
Shear Strength using a Hand Held Shear Vane: NZ Geotechnical Soc Inc 8/2001	NZ Geotechnical Society Dec 2005
Inferred CBR values taken from Austroads Pavement Design Manual 2004	Inferred CBR values are not IANZ accredited

Date tested : 29 September 2011
Date reported : 5 October 2011

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IANZ Approved Signatory RJones

Designation : *Senior Civil Engineering Technician*
Date : 5 October 2011

new zealand accredited laboratory

All tests reported herein have been performed in accordance with the laboratory's scope of accreditation

TEST PIT INVESTIGATION
TEST REPORT

Project : **Wainoni Community Centre Geotechnical Assessment**
Location : **31 Hampshire Street**
Client : **Christchurch City Council**
Contractor : **Opus Christchurch Laboratory**
Shear vane number : **N/A**
Shear vane correction : **N/A**
Water level (m) : **1.7**
Reduced level (m) : **N/A**

Project No : **6QUCCC.19/045LC**
Lab Ref No : **6224**
Client Ref No : **Test Pit 1**

Test Results

Depth (m)	Shear Strength (kPa)	Sample Details	Material Description
0 - 0.1	N/A	N/A	Light brown SAND, loosely packed, dry
0.1 - 0.4	N/A	N/A	Dark brown SAND trace silt, loosely packed, low plasticity
0.4 - 0.6 EOB	N/A	N/A	Light brown SAND, loosely packed, moist (building foundation finished at 400mm)

Test Methods	Notes
Shear Strength using a Hand Held Shear Vane: NZ Geotechnical Soc Inc 8/2001	IANZ accreditation does not apply to material descriptions
Field Description of Soils and Rocks in Engineering Use, NZ Geomechanics Society	

Date tested : 29 September 2011
Date reported : 17 October 2011

This report may only be reproduced in full

Approved

Designation : *Senior Civil Engineering Technician*
Date : 17 October 2011

TEST PIT INVESTIGATION
TEST REPORT

Project : **Wainoni Community Centre Geotechnical Assessment**
Location : **31 Hampshire Street**
Client : **Christchurch City Council**
Contractor : **Opus Christchurch Laboratory**
Shear vane number : **N/A**
Shear vane correction : **N/A**
Water level (m) : **1.7**
Reduced level (m) : **N/A**

Project No : **6QUCCC.19/045LC**
Lab Ref No : **6224**
Client Ref No : **Test Pit 2**

Test Results

Depth (m)	Shear Strength (kPa)	Sample Details	Material Description
0 - 0.2	N/A	N/A	Dark brown Sandy SILT, trace fine gravel, dry, firm low plasticity
0.2 - 0.6 EOB	N/A	N/A	Light brown SAND, trace dark brown silt, loose, dry
			

Test Methods	Notes
Shear Strength using a Hand Held Shear Vane: NZ Geotechnical Soc Inc 8/2001 Field Description of Soils and Rocks in Engineering Use, NZ Geomechanics Society	IANZ accreditation does not apply to material descriptions

Date tested : 29 September 2011
Date reported : 17 October 2011

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Designation : *Senior Civil Engineering Technician*
Date : 17 October 2011

TEST PIT INVESTIGATION
TEST REPORT

Project :	**Wainoni Community Centre Geotechnical Assessment**
Location :	**31 Hampshire Street**
Client :	**Christchurch City Council**
Contractor :	**Opus Christchurch Laboratory**
Shear vane number :	**N/A**
Shear vane correction :	**N/A**
Water level (m) :	**1.7**
Reduced level (m) :	**N/A**

Project No :	**6QUCCC.19/045LC**
Lab Ref No :	**6224**
Client Ref No :	**Test Pit 3**

Test Results

Depth (m)	Shear Strength (kPa)	Sample Details	Material Description
0 - 0.2	N/A	N/A	Dark brown Sandy SILT, trace fine gravel, dry firm, low plasticity
0.2 - 0.5			Dark brown Sandy SILT, moist, loosely packed
0.5 - 0.6 EOB			Grey SAND moist, some light brown dry silt dense

Test Methods	Notes
Shear Strength using a Hand Held Shear Vane: NZ Geotechnical Soc Inc 8/2001	IANZ accreditation does not apply to material descriptions
Field Description of Soils and Rocks in Engineering Use, NZ Geomechanics Society	

Date tested : 29 September 2011
Date reported : 17 October 2011

This report may only be reproduced in full

Approved

Designation : *Senior Civil Engineering Technician*
Date : 17 October 2011

Appendix D:
Site Location Plan

Key:

⚠	EQC CPT
⊕	Ecan Borehole

SOURCE: 1) canterburyrecovery.projectorbit.com (Accessed on 02/05/12)
2) http://arcims.ecan.govt.nz/ecanmapping/ (Accessed on 02/05/12)

OPUS

Opus International Consultants Ltd
Christchurch Office
20 Moorhouse Ave
PO Box 1482
Christchurch, New Zealand
Tel: +64 3 363 5400 Fax: +64 3 365 7857

Project: Wainoni Aranui Community Centre
Geotechnical Appraisel
Project No: 6-QUCCC.19 006SC
Client: Christchurch City Council

Site Location Plan

Drawn: Mark Broughton
Geotechnical Engineer
Date: 2/02/2012

Appendix E:
Geotechnical Inspection Notes

APPENDIX B

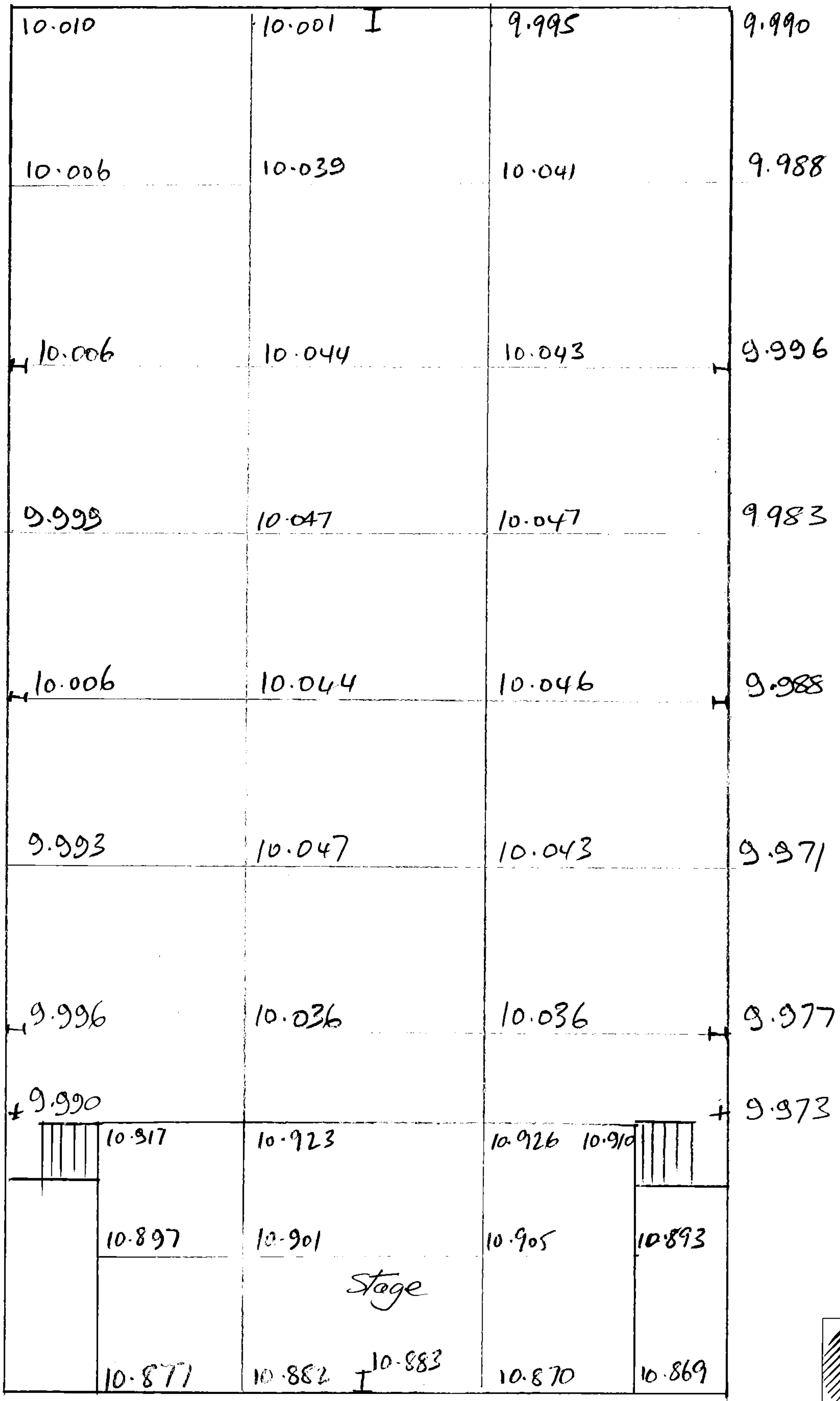
Figure 1 – Geotechnical Inspection Observations

Appendix F:
Levels Survey

Calculation Sheet

Project/Task/File No:	Sheet No 1 of 2
Project/Description: Wainoni Aranui Community Centre	Office: CH
	Computed: AJB 28/6/2011
Main Hall Floor Levels	Checked: / /

Note: Nominal datum of 10.000 taken on floor inside main entrance lobby against wall to North Side.



10.010	10.001	9.995		9.990
10.006	10.039	10.041		9.988
10.006	10.044	10.043		9.996
9.999	10.047	10.047		9.983
10.006	10.044	10.046		9.988
9.993	10.047	10.043		9.971
9.996	10.036	10.036		9.977
9.990				9.973
10.917	10.923	10.926	10.910	
10.897	10.901	10.905	10.893	
10.877	10.882 10.883	10.870	10.869	

Calculation Sheet

Project/Task/File No:

Project/Description: Wainoni Avonui Community Centre

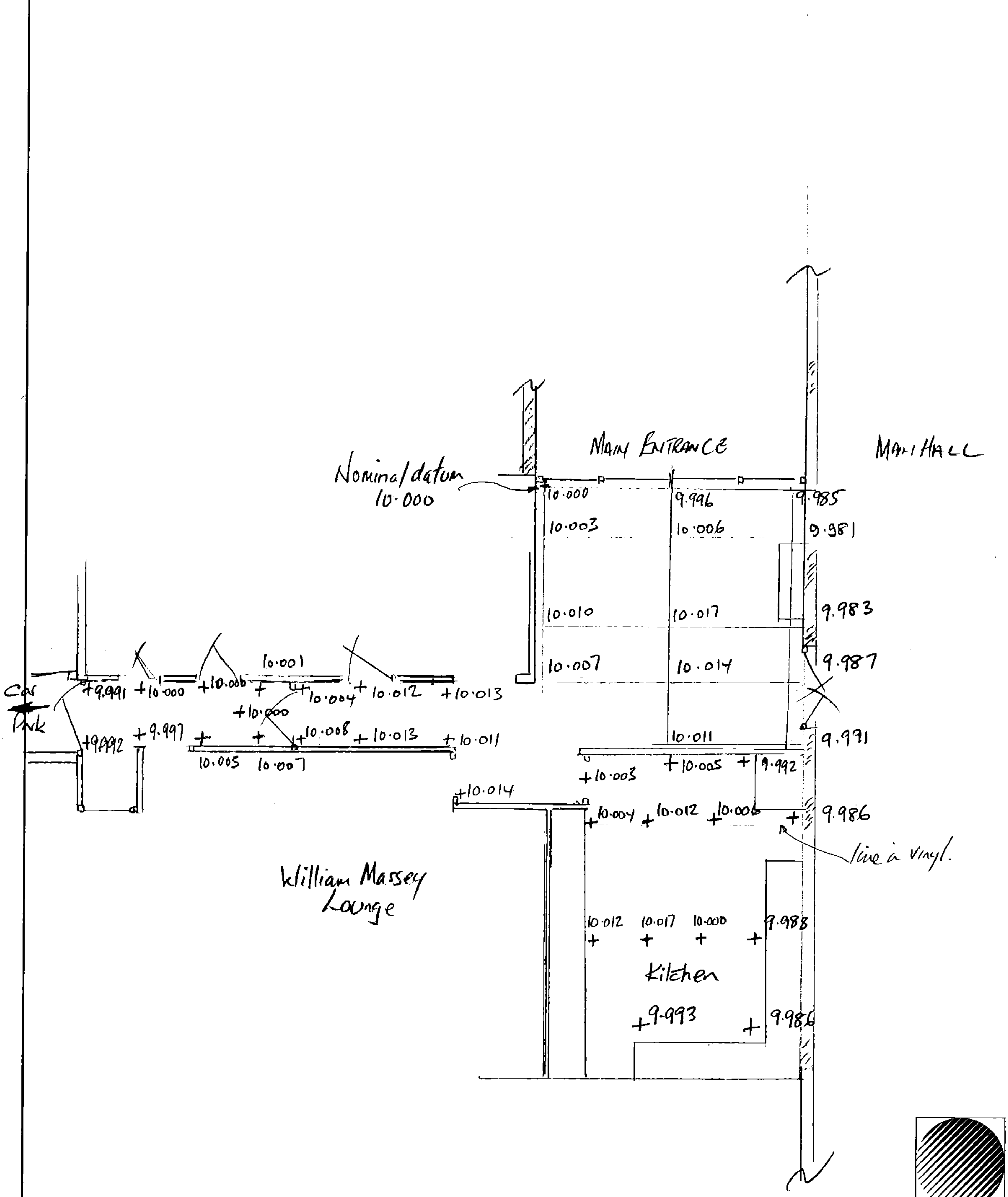
Sheet No 2 of 2

Office: CH

Computed: AB 20 / 6 / 2011

Checked: / /

Levels in Main Lobby Hallway and Kitchen



CSF 400 (7/2000)

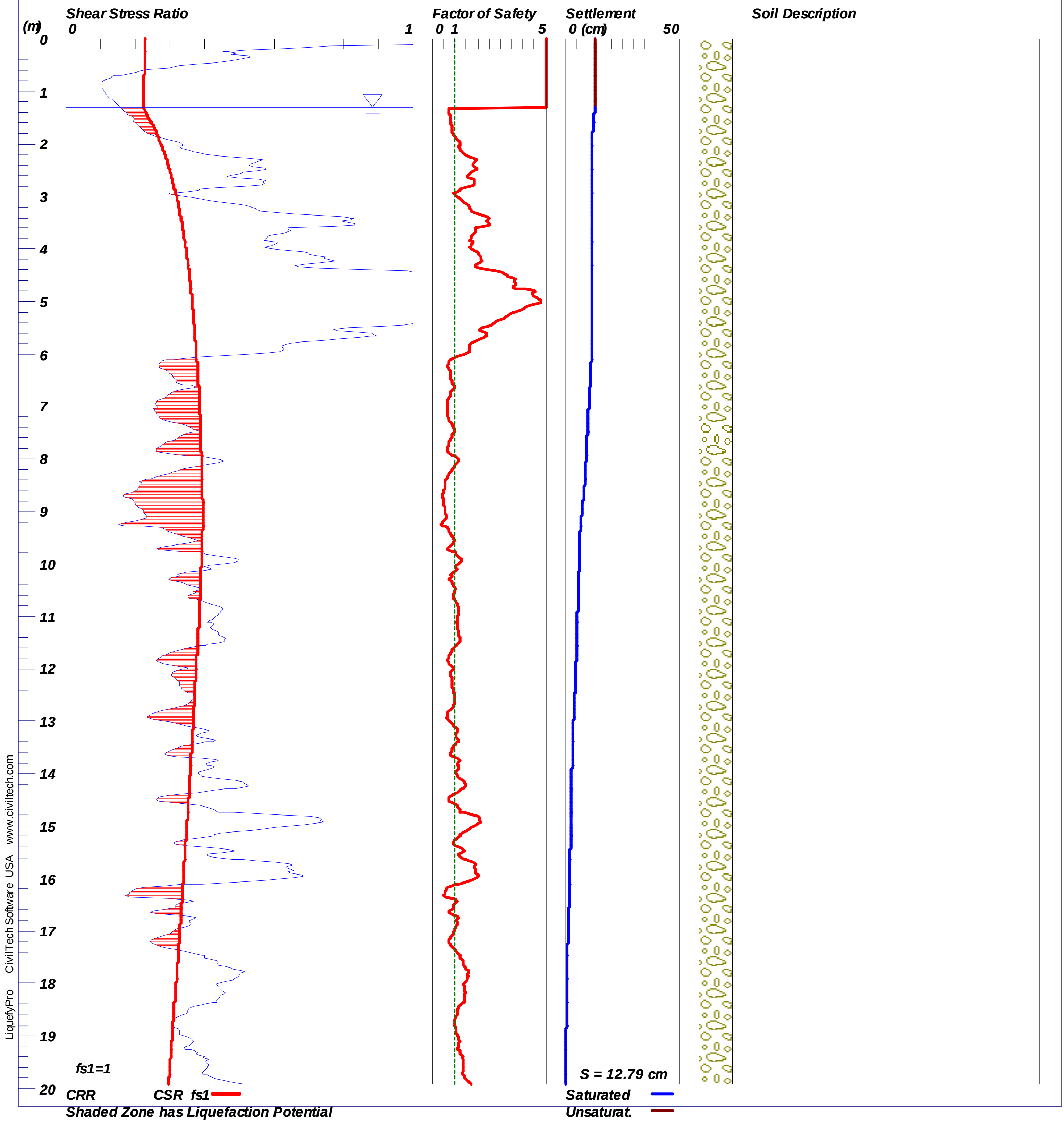
Appendix G:
Liquefaction Analysis

LIQUEFACTION ANALYSIS

Wainoni Aranui Community Centre

Hole No.=CPT 001   Water Depth=1.3 m   Surface Elev.=0

Magnitude=7.5
Acceleration=0.35g



CivilTech Corporation **ULS** **Plate A-1**

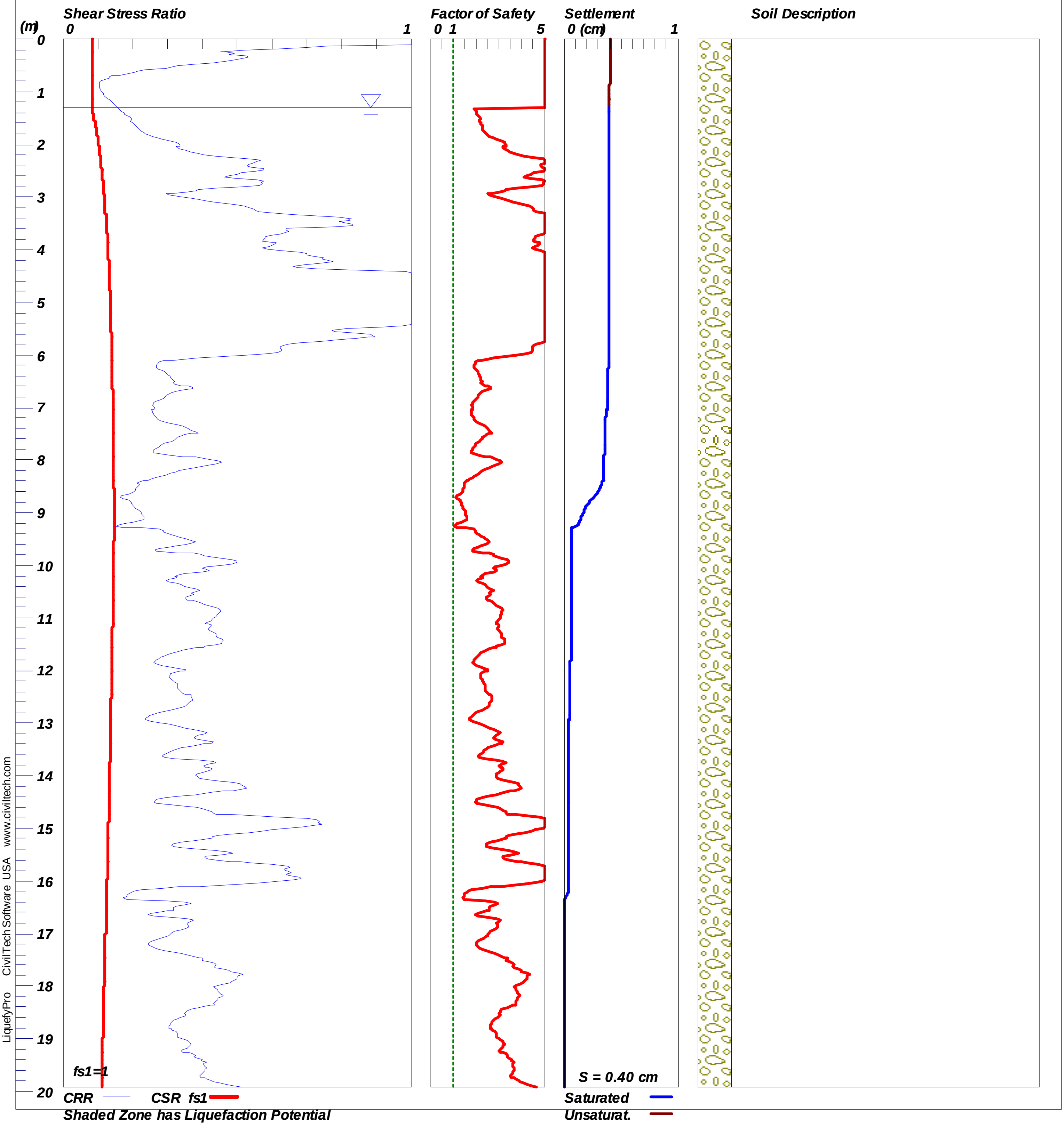
LIQUEFACTION ANALYSIS

Wainoni Aranui Community Centre

Hole No.=CPT 001 Water Depth=1.3 m Surface Elev.=0

Magnitude=7.5

Acceleration=0.13g



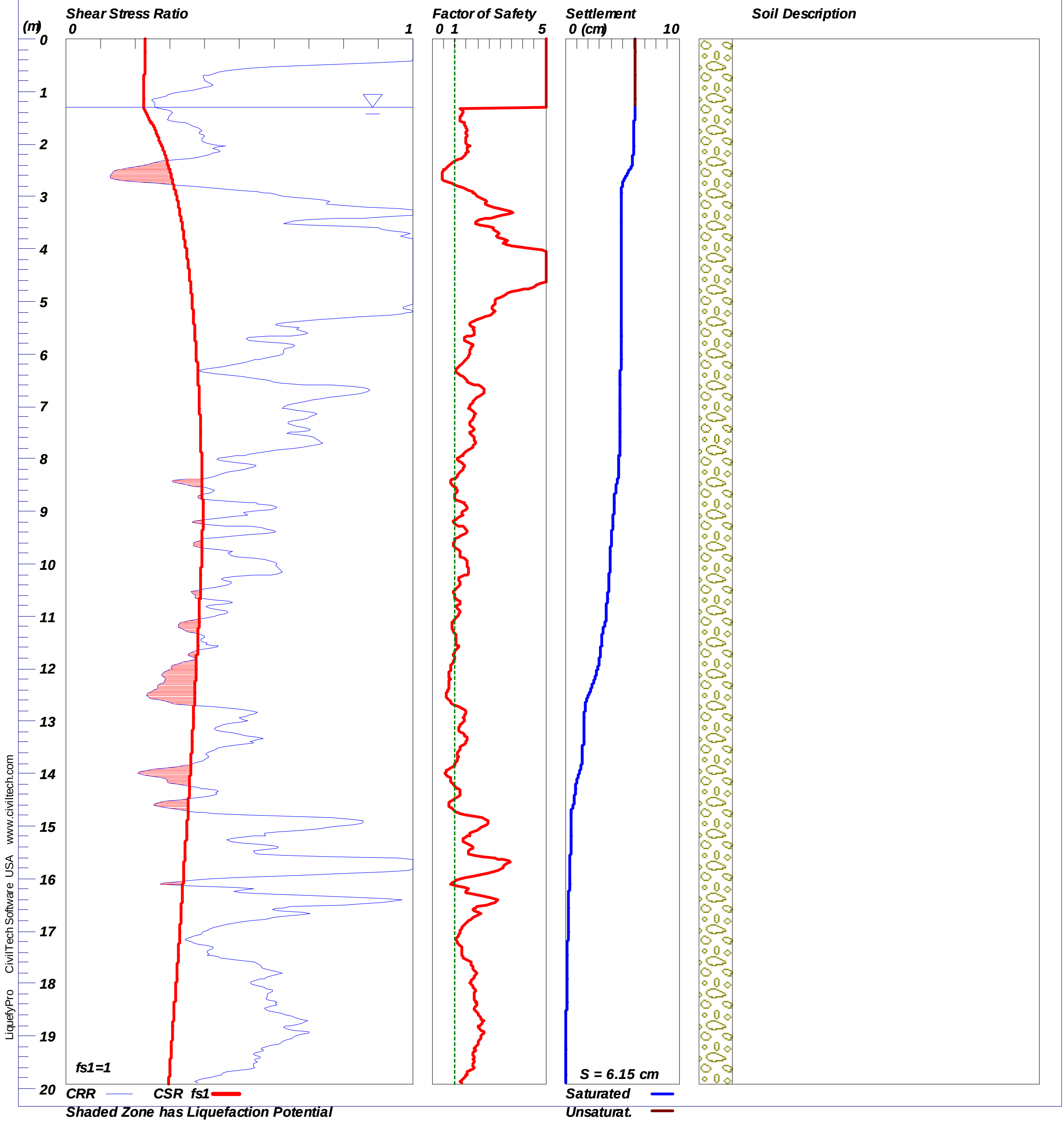
CivilTech Corporation **SLS** **Plate A-1**

LIQUEFACTION ANALYSIS

Wainoni Aranui Community Centre

Hole No.=CPT 002   Water Depth=1.3 m   Surface Elev.=7   *Magnitude=7.5*

Acceleration=0.35g



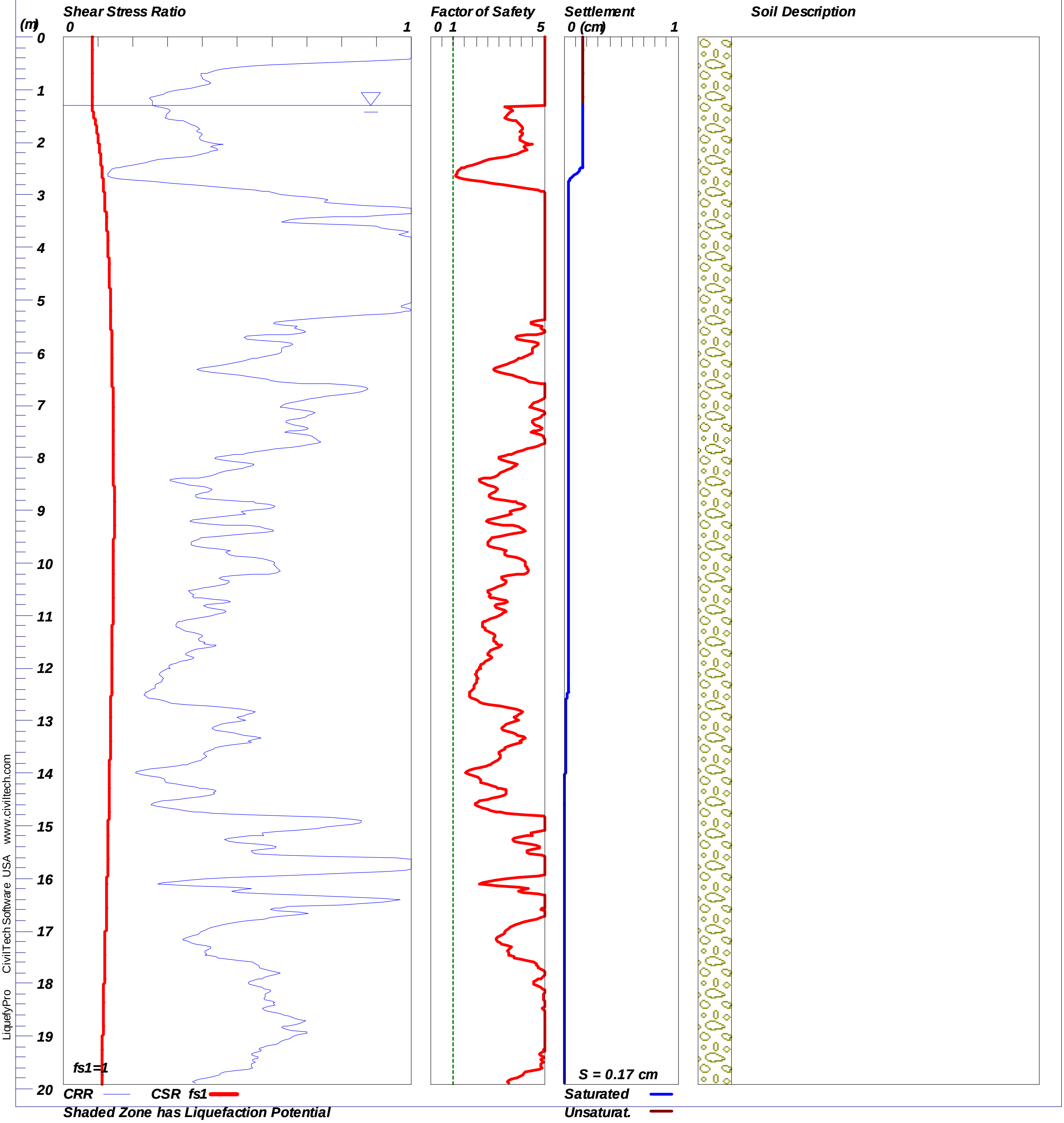
CivilTech Corporation   **ULS**   **Plate A-1**

LIQUEFACTION ANALYSIS

Wainoni Aranui Community Centre

Hole No.=CPT 002   Water Depth=1.3 m   Surface Elev.=7

Magnitude=7.5
Acceleration=0.13g

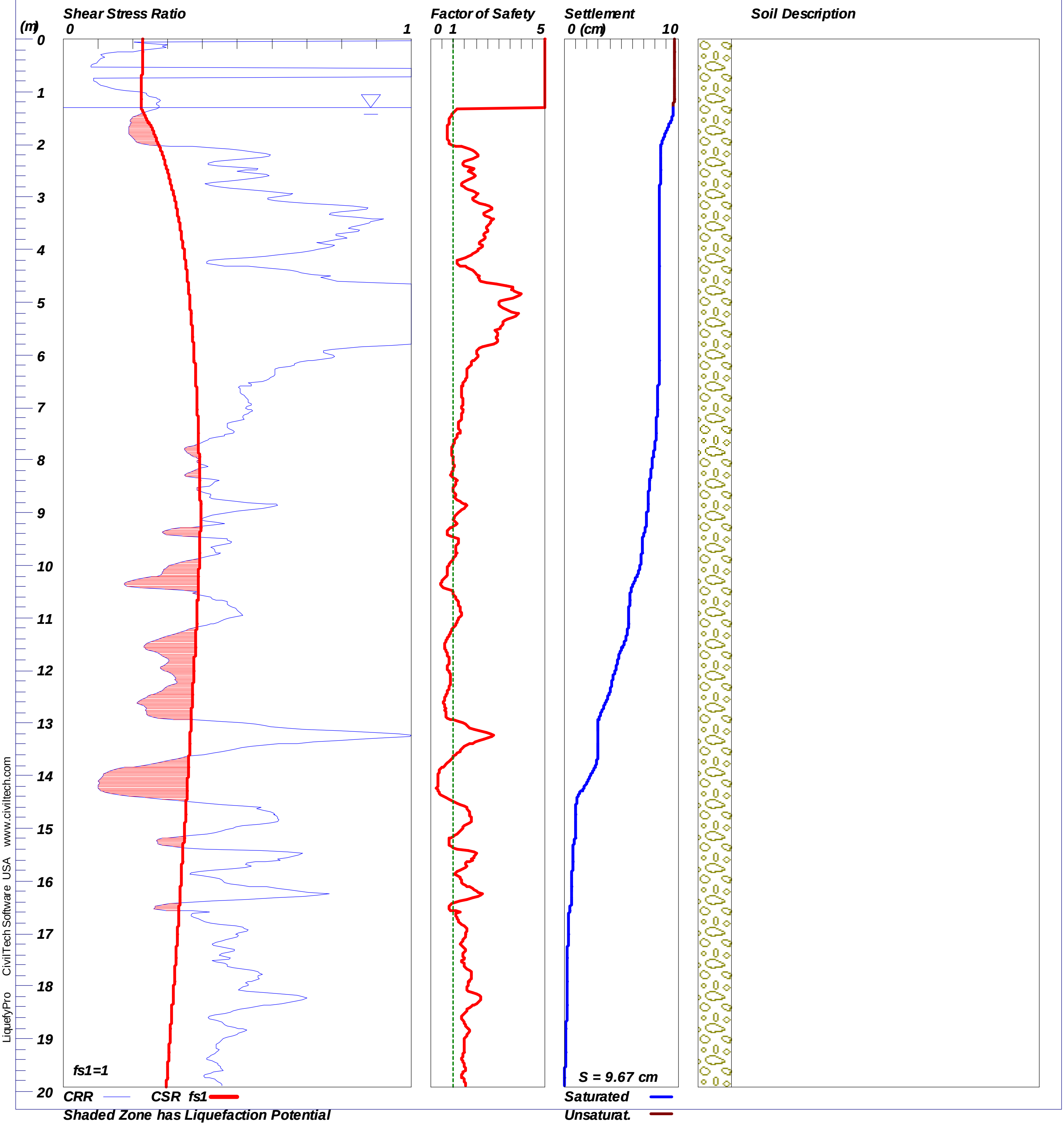


CivilTech Corporation   **SLS**   **Plate A-1**

LIQUEFACTION ANALYSIS

Wainoni Aranui Community Centre

Hole No.=CPT 003   Water Depth=1.3 m   Surface Elev.=0   *Magnitude=7.5*
Acceleration=0.35g



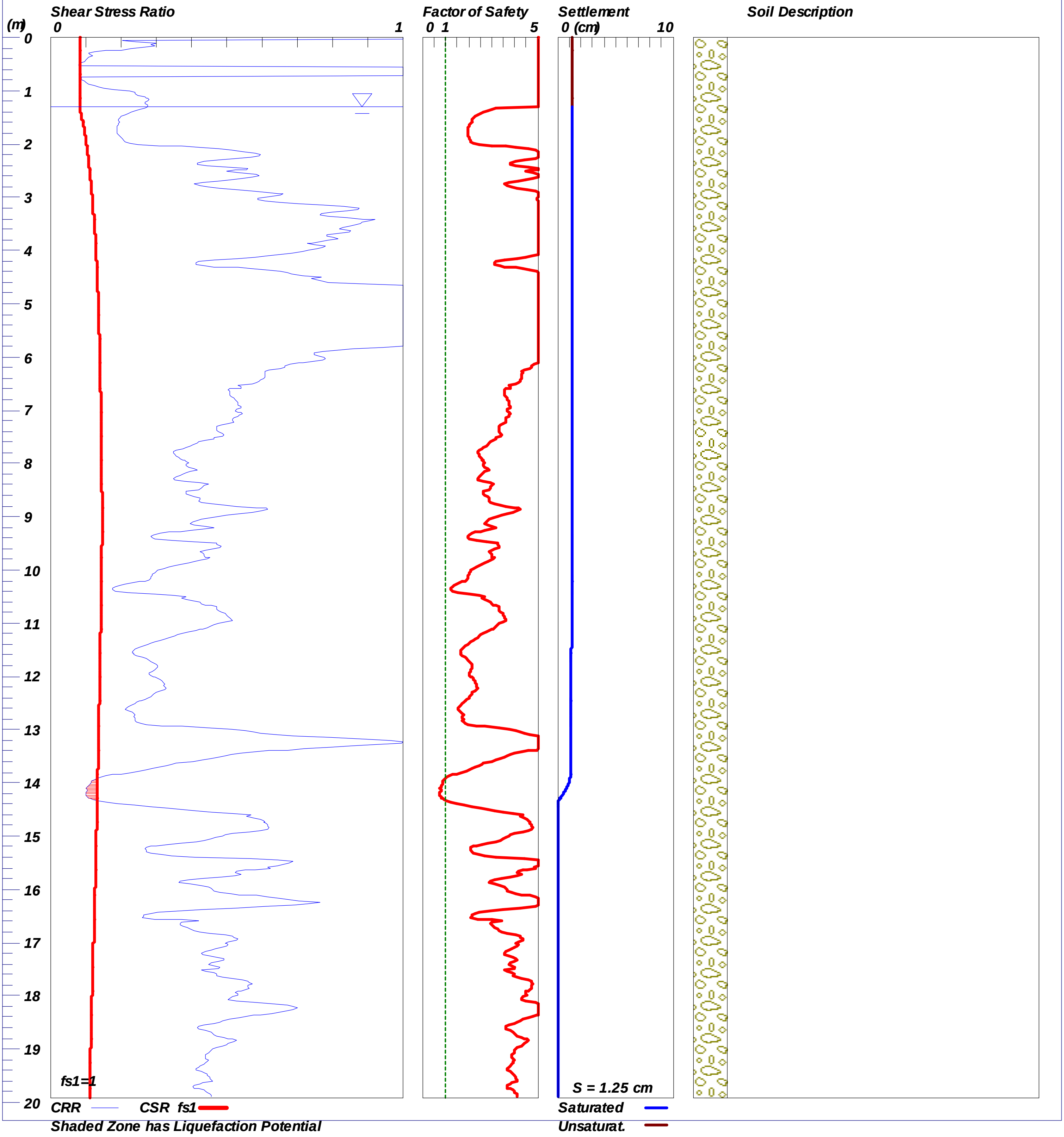
CivilTech Corporation   **ULS**   **Plate A-1**

LIQUEFACTION ANALYSIS

Wainoni Aranui Community Centre

Hole No.=CPT 003   Water Depth=1.3 m   Surface Elev.=0

Magnitude=7.5
Acceleration=0.13g



CivilTech Corporation **SLS** **Plate A-1**

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Appendix C – CERA DEEP Data Sheet

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name: Community Centre (Community rooms)
Unit No: Street
Building Address: Aranui, Christchurch 31 Hampshire Street
Legal Description:
Degrees Min Sec
GPS south:
GPS east:
Building Unique Identifier (CCC):

Reviewer: Alistair Boyce
CPEng No: 209860
Company: Opus Consultants Ltd.
Company project number: 6-QUCC.19
Company phone number: 03 363 5400
Date of submission: 23-May-12
Inspection Date:
Revision: Final
Is there a full report with this summary? yes

Site

Site slope: flat
Soil type: sandy silt
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):
Soil Profile (if available):
If Ground improvement on site, describe:
Approx site elevation (m):

Building

No. of storeys above ground: 1 single storey = 1
Ground floor split? no
Storeys below ground 0
Foundation type: strip footings
Building height (m): 6.00
Floor footprint area (approx):
Age of Building (years): 36

Ground floor elevation (Absolute) (m):
Ground floor elevation above ground (m):
if Foundation type is other, describe:
height from ground to level of uppermost seismic mass (for IEP only) (m): 6
Date of design: 1976-1992

Strengthening present? no
If so, when (year)?
And what load level (%g)?
Brief strengthening description:

Use (ground floor): other (specify)
Use (upper floors): other (specify)
Use notes (if required): Comunity Rooms
Importance level (to NZS1170.5): IL2

Gravity Structure

Gravity System: load bearing walls
Roof: timber framed
Floors: timber
Beams: timber
Columns: load bearing walls
Walls: fully filled concrete masonry

rafter type, purlin type and cladding
joist depth and spacing (mm)
type
typical dimensions (mm x mm)
#N/A

Lateral load resisting structure

Lateral system along: fully filled CMU
Ductility assumed, μ: 1.25
Period along: 0.40 ##### enter height above at H31
Total deflection (ULS) (mm):
maximum interstorey deflection (ULS) (mm):

Note: Define along and across in detailed report!

note total length of wall at ground (m):
wall thickness (m):
estimate or calculation? estimated
estimate or calculation?
estimate or calculation?

Lateral system across: fully filled CMU
Ductility assumed, μ: 1.25
Period across: 0.40 ##### enter height above at H31
Total deflection (ULS) (mm):
maximum interstorey deflection (ULS) (mm):

note total length of wall at ground (m):
wall thickness (m):
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:
Roof Cladding:
Glazing:
Ceilings:
Services(list):

Available documentation

Architectural
Structural
Mechanical
Electrical
Geotech report

original designer name/date
original designer name/date
original designer name/date
original designer name/date
original designer name/date

Damage

Site:
(refer DEE Table 4-2)

Site performance:
Describe damage:

Settlement:
Differential settlement:
Liquefaction:
Lateral Spread:
Differential lateral spread:
Ground cracks:
Damage to area:

notes (if applicable):
notes (if applicable):
notes (if applicable):
notes (if applicable):
notes (if applicable):
notes (if applicable):
notes (if applicable):

Building:

Current Placard Status: green

Along Damage ratio:
Describe (summary):
Describe how damage ratio arrived at:

Across Damage ratio: #DIV/0!
Describe (summary):

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

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

Recommendations

Level of repair/strengthening required: none Describe:
Building Consent required: yes Describe:
Interim occupancy recommendations: partial occupancy Describe:

Along Assessed %NBS before: ##### %NBS from IEP below
Assessed %NBS after: 5%

Across Assessed %NBS before: ##### %NBS from IEP below
Assessed %NBS after: 5%

IEP

Period of design of building (from above): 1976-1992
h_n from above: 6m

Seismic Zone, if designed between 1965 and 1992:
not required for this age of building
not required for this age of building

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

Note:1 for buildings designed prior to 1976 as public buildings, to code at time, use 1.25
Note 2: for RC buildings designed between 1976-1984, use 1.2
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)

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

2.2 Near Fault Scaling Factor

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

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

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:
Z_{1992}, from NZS4203:1992
Hazard scaling factor, **Factor B:** #DIV/0!

2.4 Return Period Scaling Factor

Building Importance level (from above):
Return Period Scaling factor from Table 3.1, **Factor C**:

2.5 Ductility Scaling Factor

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

2.6 Structural Performance Scaling Factor:

Sp:		
Structural Performance Scaling Factor **Factor E**:	#DIV/0!	#DIV/0!

2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom}$ x A x B x C x D x E $\%NBS_b$: **#DIV/0!** **#DIV/0!**

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

3.1. Plan Irregularity, factor A: 1

3.2. Vertical irregularity, Factor B: 1

3.3. Short columns, Factor C: 1

3.4. Pounding potential
Pounding effect D1, from Table to right
Height Difference effect D2, from Table to right

Therefore, Factor D: 0

3.5. Site Characteristics

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

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

3.6. Other factors, Factor F

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

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)
List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

3.7. Overall Performance Achievement ratio (PAR) **0.00** **0.00**

4.3 PAR x (%NBS)b: PAR x Baselline %NBS: **#DIV/0!** **#DIV/0!**

4.4 Percentage New Building Standard (%NBS), (before) **#DIV/0!**

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