*Christchurch City Council*

# Tram Barn
# BU 1221-001 EQ2

## Detailed Engineering Evaluation

## Quantitative Assessment Report

*Christchurch City Council*

# Tram Barn

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## Quantitative Assessment Report

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# Contents

# Executive Summary

Tram Barn Building
BU 1221-001 EQ2

Detailed Engineering Evaluation
Quantitative Report - SUMMARY
Final

7 Tramway Lane, Christchurch

## Background

This is a summary of the quantitative report for the building structure at 7 Tramway Lane, central Christchurch, known as the Tram Barn and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 16 May 2012, visual inspection on 23 March 2012 and 24 September 2012, available drawings and qualitative assessment calculations.

## Key Damage Observed

Key damage observed includes:-

- Pounding between the roof parapet at the south-west corner and the building to the west.
- Differential settlement of up to 40mm between the building and the carpark building immediately to the north.
- Settlement of the reinforced masonry wall along the north-western boundary by up to 50mm.
- Differential settlement of the ground floor slab by up to 20mm at the southern end of the building.
- The eastern end of the concrete wall along the southern boundary has diagonal shear cracks up to 0.2mm wide.
- The spandrels along the east face have moderate cracks up to 3 mm wide in the lower corners.
- Cracks between the columns and window frames along the east elevation indicate differential movement.
- Minor cracking visible in precast concrete panels along south and west boundaries.
- Minor horizontal cracking visible in the south-east corner column.

## Critical Structural Weaknesses

The following critical structural weaknesses have been identified:

a. Plan irregularity – the precast concrete shear walls which resist the seismic loads are located on the southern and western elevations and result in a significant eccentricity between the centre of rigidity and the centre of mass of the building.
b. Differential settlement – the performance of the ground at the site has resulted in differential settlement occurring to the north-western boundary wall and also to the northern building superstructure.

## Indicative Building Strength (from quantitative assessment)

Based on the information available, and from undertaking a quantitative assessment, the building's original capacity has been assessed to be in the order of 70% NBS as governed by the reinforced masonry wall along the north-west elevation. The masonry wall has settled around 50mm relative to the piled building and as a result the wall is no longer restrained at the top. As a result of this damage the wall has a current seismic capacity of 18% NBS. The building is therefore classed as an earthquake prone building.

Due to damage sustained to the wall it is recommended that this wall is replaced with a new wall complying with the New Building Standard. This will result in the building having a seismic capacity of 84% NBS as governed by the connection between the floor diaphragm and the precast concrete walls. As an interim measure the wall could be propped to allow occupancy to resume. If the wall is propped the building will have a capacity of 84% NBS.

The northern building has been independently assessed by Holmes Consulting Group, and they have reported that the building has a seismic capacity of around 63% NBS.

The building appears to have performed well throughout the Canterbury Earthquake sequence. The detailing of the structure appears to have largely contributed to that.

## Recommendations

It is recommended that:

a) The masonry wall on the north-west boundary could be propped as an interim measure. Once this wall has been propped full occupancy of the building is expected to resume subject to CERA approval.

b) A remedial solution is investigated for the masonry wall on the north-west boundary. Once the wall is removed and replaced with a new element complying with the New Building Standard the Tram Barn will have a seismic capacity of 84% NBS.

# 1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council (CCC) to undertake a detailed seismic assessment of the Tram Barn building, located at 7 Tramway Lane, Christchurch following the Canterbury Earthquake Sequence since September 2010.

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) [3] [4].

A previous assessment completed in June 2012 for the building, reported that the building had a preliminary seismic capacity of 44%NBS. This report follows the recommendations to carry out a quantitative assessment of the building. Further site specific geotechnical investigations were also carried out as recommended and are included in this report.

The overall extent of the property owned by the Christchurch City Council extends across two separate buildings. Please refer to Section 4.1 of this report for further details.

# 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 definition in the Building Act). CERA have adopted the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 16 May 2012. 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:

c. The importance level and occupancy of the building.

d. The placard status and amount of damage.

e. The age and structural type of the building.

f. Consideration of any critical structural weaknesses.

Christchurch City Council requires any building with a capacity of less than 34% of New Building Standard (including consideration of critical structural weaknesses) to be strengthened to a target of 67% as required under 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).

The Earthquake Prone Building policy for the territorial authority shall apply as outlined in Section 2.3 of this report.

### Section 115 – Change of Use

This section requires that the territorial authority 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 territorial authorities as being 67% of the strength of an equivalent new building or as near as practicable. 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:

a. 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
b. 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
c. 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
d. There is a risk that other property could collapse or otherwise cause injury or death; or
e. 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 (EPB) 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 4 September 2010.

The 2010 amendment includes the following:

f. A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;

g. A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;

h. A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,

i. 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.

Where an application for a change of use of a building is made to Council, the building will be required to be strengthened to 67% of New Building Standard or as near as is reasonably practicable.

## 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:

- increase in the basic seismic design load for the Canterbury earthquake region (Z factor increased to 0.3 equating to an increase of 36 – 47% depending on location within the region);
- 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.*

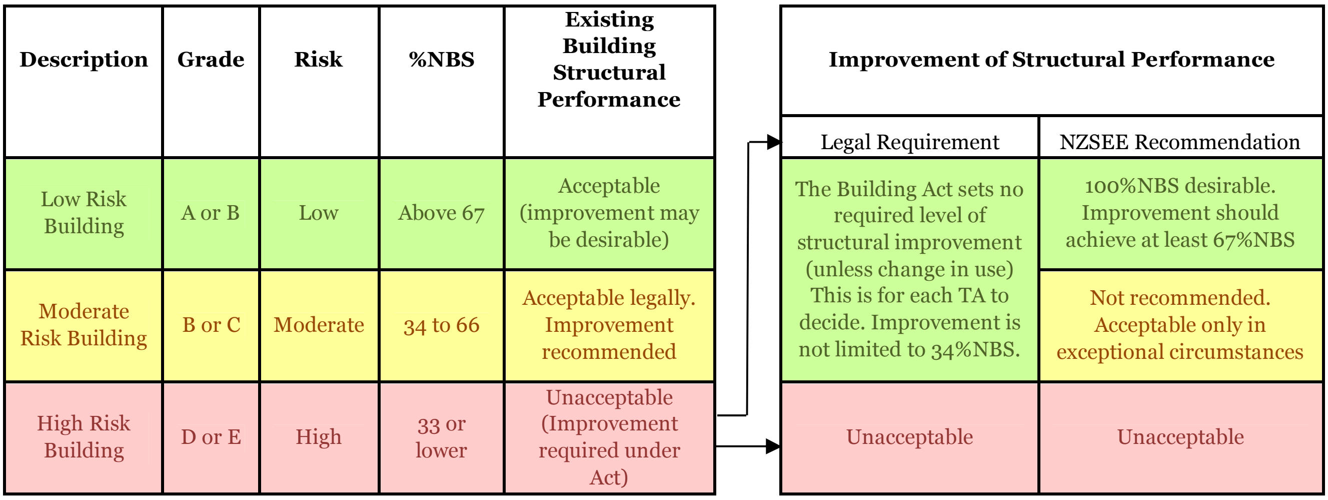
*1.2 Ensuring that responsible steps are taken to minimise the risk of loss of life, injury or suffering which may result from your engineering activities, either directly or indirectly."*

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 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).

Table 1: %NBS compared to relative risk of failure

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

## 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[1] 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 and from the DBH guidance document dated 12 June

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

2012 [6], 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/territorial authority guidelines.

### 3.1.3 Strengthening

Industry guidelines (NZSEE 2006 [2]) strongly recommend that every effort be made to achieve improvement to as near as reasonably practicable to 100%NBS. While in some cases this will not be practicable, NZSEE recommends that 67%NBS be regarded as a minimum target for structural capacity. The legal minimum requirement for structural improvement is 34%NBS, however, each territorial authority has been commissioned to develop guidelines to deal with the range of buildings likely to be encountered.

### 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.

# 4 Background Information

## 4.1 Building Description

The Tram Barn is a single storey reinforced concrete building located at 7 Tram Way Lane in central Christchurch. The ground floor area is used as a tram workshop while the building roof is used for car parking, accessed via the adjacent Millennium Hotel carpark building immediately to the north.

The overall extent of the property owned by the Christchurch City Council extends across two separate buildings as shown in Figure 2 and as described below.

### Southern building

The southern building, constructed in 1994, is 17.17m wide in the east-west direction and 24.4m long in the north-south direction. All of this building is owned by the CCC.

The roof deck is approximately 6.3m above ground floor level and has a 1m high parapet to the western, southern and eastern sides.

On the western side of the building there is an internal mezzanine floor used for offices. The floor is constructed from light weight timber framing and is braced with plasterboard linings.

The building is separated from the northern building by a 40mm wide seismic gap at roof level.

### Northern building

The northern building, constructed in 1996, is approximately 57m long in the east-west direction and 22m wide in the north-south direction. The only section of this building owned by the CCC is a triangular wedge at the eastern end of the building, as shown in Figure 2 below. This triangular area of the building is immediately adjacent to the basement to the west.

Due to the location of the property boundary with respect to the physical location of the two buildings the following scope has been developed for the assessment of the Tram Barn building:

a) Southern building (ground floor and roof) – detailed assessment by Opus and included in this report.

b) Northern building (ground floor) – detailed assessment by Opus and included in this report.

c) Northern building (roof) – this building was independently assessed by Holmes Consulting Group and was found to have a post-earthquake strength of 63%NBS.

The property boundary was surveyed by CCC on 14 August 2012 and was found to be located within 20mm of the centreline of the reinforced masonry external wall along the north-west boundary.

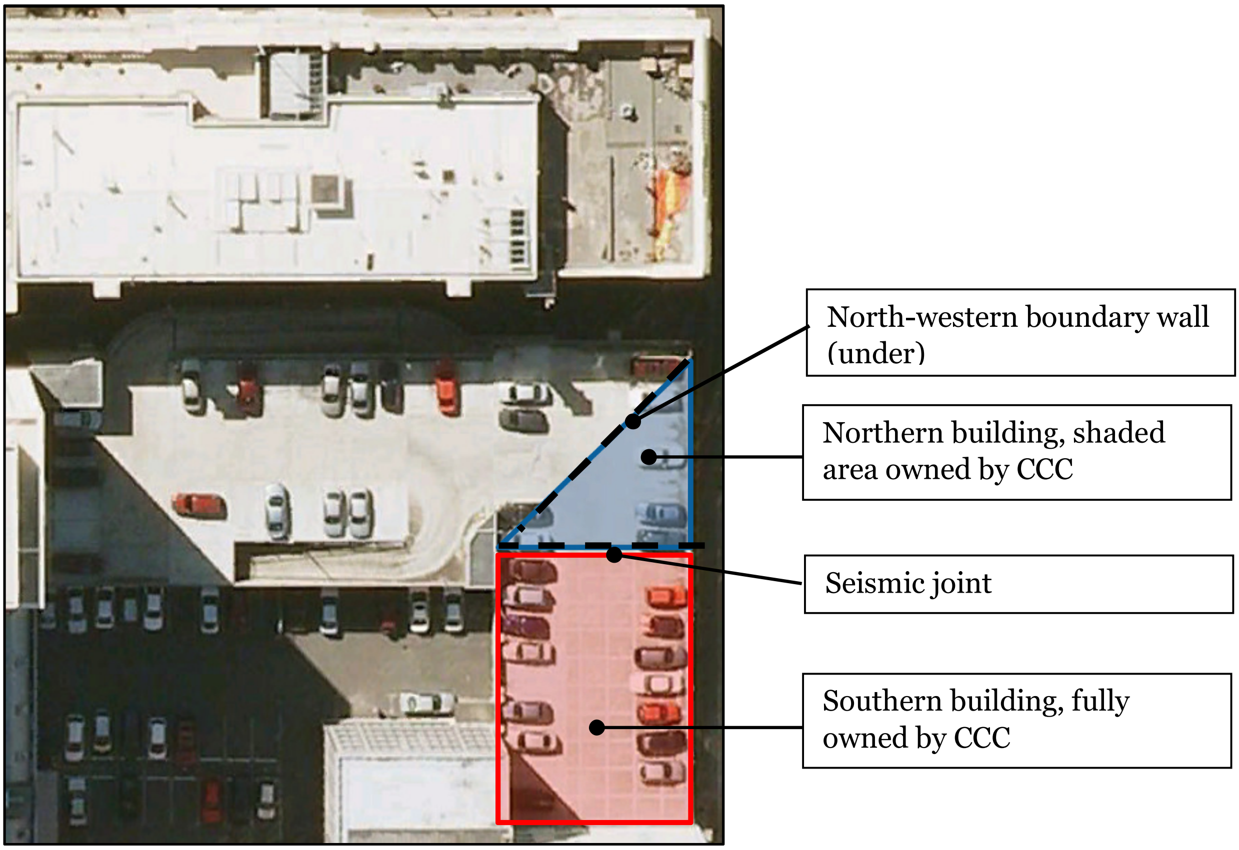


Figure 2 - Site plan

## 4.2 Gravity Load Resisting Systems

Gravity loads at roof level are resisted by 75mm thick precast concrete Unispan flat slab units with a mesh reinforced 75mm thick topping spanning 6.2m north-south between precast concrete beams. The beams are 400mm wide and reduce in overall depth from 750mm on the eastern side of the building to 500mm on the western side of the building. Each beam is supported by two 400x600mm precast concrete columns, with one column located on the eastern elevation and the second column located 4.47m from the western wall. On the eastern side of the building there is a 2m deep by 12m long pit used for servicing the trams. The pit is formed from 200mm thick reinforced concrete walls.

Gravity loads are also resisted by the southern and western boundary walls, which are constructed from 180mm thick precast concrete panels.

The wall along the north-western boundary in the building to the north is constructed from reinforced masonry units and is restrained at roof level with cast in steel plates bolted to the underside of the roof slab. This wall does not form part of the gravity or lateral load resisting system for the building.

The ground floor slab consists of a 100mm thick mesh reinforced concrete slab on grade.

The concrete columns and walls are supported by bored concrete piles. Several piles have been detailed to act as tension piles to resist seismic induced uplift forces.

## 4.3 Lateral Load Resisting Systems

Lateral resistance in the north-south and east-west directions is provided by the precast concrete walls along the southern and western elevations. Lateral loads are transferred to the walls by the mesh reinforced topping on the roof slab. Adjacent precast panels stop 400mm short of each other and horizontal reinforcing bars extend out of each panel into the gap, which was then poured as an insitu concrete stitch joint. The end faces of the panels were detailed to be roughened prior to pouring the insitu stitch. The walls along the southern and western elevations are therefore considered to each act as one single element.

The irregular layout of the precast concrete walls along the southern and western elevations results in the building having a significant torsional response to resisting lateral loads. While the large stiffness of the precast concrete walls results in these elements resisting the majority of the load, the torsional demands imposed by the irregular layout of the structure are resisted by the moment frame along the eastern elevation (formed by the concrete columns and spandrel beams), by the precast concrete wall along the northern elevation and also to a lesser extent by each of the east-west concrete moment resisting frames supporting the roof level structure.

The reinforced masonry wall along the north-western boundary resists self-weight seismic loads through in-plane and out of plane flexure.

## 4.4 CBD Red Zone Cordon

Following the Lyttelton Earthquake of 22 February 2011, the central business district (CBD) suffered major damage to a large proportion of its building stock resulting in a central area of the city being cordoned off and closed to the public, forming what is known as the Red Zone. The Red Zone extent, as of 24 September 2012, is displayed below in Figure 3.

This building is located within the Red Zone.

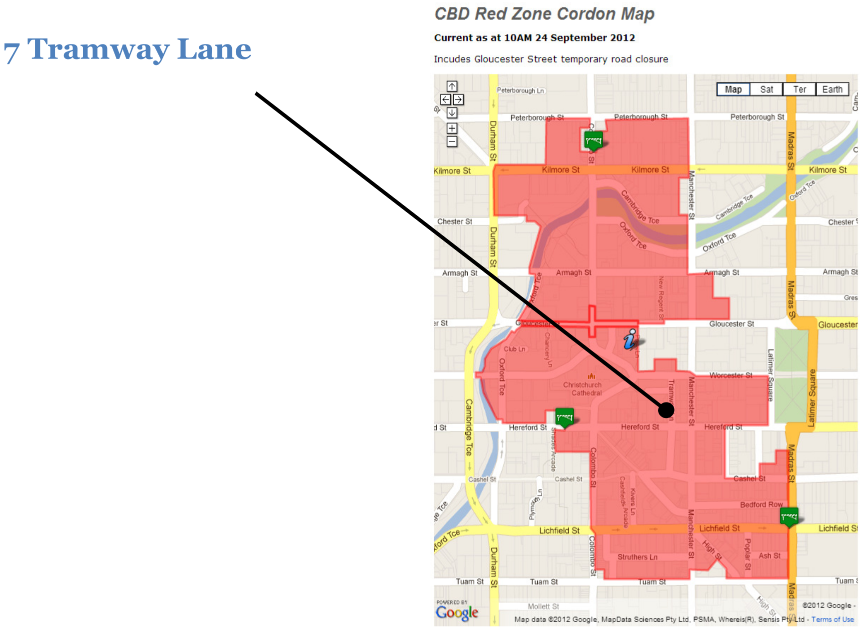


Figure 3: Building Location relative to current Red Zone cordon

# 5 Survey

## 5.1 Post 22 February 2011 Rapid Assessment

A structural (Level 3) Assessment was undertaken on this building on 25 March 2011 by Opus International Consultants, resulting in a Yellow placard being issued due to hazards from neighbouring buildings. Photos of the building can be found in Appendix 1.

## 5.2 Further Inspections

Further inspections were undertaken by Opus International Consultants on 26 March and 24 September 2012. The building remains Yellow placarded on the basis of a CERA placard.

## 5.3 Original Documentation

Copies of the following construction drawings were provided by the CCC:

- "Tramcar Building, Hereford Place, Option 2" structural drawings marked for construction. The drawings were prepared by Holmes Consulting Group in August 1994. These drawings relate to the southern building.

- "The Randolph Carpark, Christchurch" structural drawings marked for construction. The drawings were prepared by Holmes Consulting Group in April 1996. These drawings relate to the northern building but do not detail the reinforced masonry wall along the north-western boundary of the southern building.

The drawings have been used to confirm the structural systems, investigate potential critical structural weaknesses (CSW) and identify details which required particular attention.

# 6 Structural Damage

The following structural damage has been noted:

## 6.1 Roof Level

a. There is a 50mm separation between the Tram Barn building and the six storey building to the west. There is evidence of pounding having occurred between the adjacent building and the roof parapet on the Tram Barn building.
b. The carpark building immediately to the north has settled around 40mm relative to the southern building, and has also moved slightly towards the north. This movement has opened up the seismic joint, which is now leaking some water into the ground floor area below.
c. Some water appears to be leaking through the roof slab and onto the precast beams, however the extent of this prior to the earthquakes is unknown.

## 6.2 Ground Floor

The following damage was noted on the ground floor:

a. The reinforced masonry wall along the north-western boundary has settled by around 50mm and has pulled the restraint fixings out of the roof level slab soffit, resulting in the wall now cantilevering from ground level. This has resulted in damage to the wall. The wall also has a number of diagonal shear cracks through it, indicative of ground movement. This masonry wall is directly behind the retaining wall forming the basement in the northern building.
b. The eastern end of the concrete wall along the southern boundary has diagonal shear cracks up to 0.2mm wide.
c. An area of slab between the tram tracks at the southern end of the building has settled by 20mm.
d. One of the large aluminium window frames on the eastern elevation has fallen out.
e. The office area on the mezzanine floor has minor cracking in the gib plasterboard.
f. The spandrels along the east face have moderate cracks up to 3 mm wide in the lower corners.
g. Cracks between the columns and window frames along the east elevation indicate differential movement.
h. Minor cracking visible in precast concrete panels along south and west boundaries.
i. Minor horizontal cracking visible in the south-east corner column.

# 7 Detailed Seismic Assessment

The detailed seismic assessment has been based on the NZSEE 2006 [2] guidelines for the "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" together with the "Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure" [3] draft document prepared by the Engineering Advisory Group on 19 July 2011, and the SESOC guidelines "Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes" [5] issued on 21 December 2011.

## 7.1 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. During our assessment the following potential CSW's have been identified:

a. Plan irregularity – the irregular layout of the precast concrete shear walls results in a significant eccentricity between the centre of rigidity and the centre of mass of the building. This generates a torsional response in the building during a seismic event. These perimeter concrete shear walls resist the torsional seismic loads and are located on the southern and western elevations.
b. Differential settlement – the performance of the ground at the site has resulted in differential settlement occurring to the north-western boundary wall and also to the northern building superstructure. The settlement of the masonry boundary wall has resulted in it no longer being restrained at the top.

## 7.2 Seismic Coefficient

The seismic design parameters based on current design requirements from NZS1170.5:2004 and the NZBC clause B1 for this building are:

- Site soil class D, clause 3.1.3 NZS 1170.5:2004
- Site hazard factor, $Z = 0.3$, B1/VM1 clause 2.2.14B
- Return period factor, $R_u = 1.0$ from table 3.5, AS/NZS 1170.0:2002, for an Importance Level 2 structure with a 50 year design life
- Expected maximum ductility factor $\mu = 1.25$ for the precast concrete walls. This ductility factor is based on recommendations in the SESOC December 2011 Practice Note "Design of conventional structural systems following the Canterbury earthquakes".

## 7.3 Quantitative Assessment Methodology

The assessment assumptions and methodology have been included in Appendix 2 of the report due to the technical nature of the content. A brief summary follows:

A 3D model of the building was created in ETABS, which is a finite element structural analysis programme.

An equivalent static analysis was carried out using the spectral values established from NZS1170.5. This analysis was used to establish the actions on the structural elements, and based on the actions determined from the analyses, an assessment of the building capacities was made.

The displacement based assessment procedure outlined in the NZSEE guidelines and moment curvature analyses from XTRACT were used to calculate the displacement capacity of the north-east corner column. These capacities were then compared to the expected displacement demand derived from the acceleration response spectra from NZS 1170.5.

## 7.4 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 may 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:

a. Simplifications made in the analysis, including boundary conditions such as foundation fixity.
b. Assessments of material strengths based on limited drawings, specifications and site inspections
c. The normal variation in material properties which change from batch to batch.

d. Approximations made in the assessment of the capacity of each element, especially when considering the post-yield behaviour.

## 7.5 Quantitative Assessment Results

A summary of the structural performance of the building is shown below in Table 2. 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. This will be considered further when developing any required strengthening options.

Table 2: Summary of Seismic Performance, $\mu = 1.25$

| Structural Element/System | Failure mode and description of limiting criteria based on capacity of critical element. | % NBS based on calculated capacity |
|---|---|---|
| Precast concrete wall panels | Flexural failure, resulting in compression failure of unconfined concrete due to repeated cycles of load and lack of confining reinforcement. Once the wall becomes unstable in the potential plastic hinge zone, the wall loses its ability to take gravity load i.e. its support of the floors above.<br>Shear – loss of ability to resist gravity loads<br>Starter bars connecting diaphragm to perimeter walls | >100%<br>>100%<br>84% |
| Precast concrete spandrel panels on east elevation | Flexural failure, resulting in compression failure of unconfined concrete due to repeated cycles of load and lack of confining reinforcement. Once the spandrel becomes unstable in the potential plastic hinge zone, the eastern wall loses its ability to take resist lateral loads. This will affect the building by making it more torsional.<br>Shear failure | >100%<br>>100% |
| Concrete portal frames | Flexural failure<br>Shear failure<br>Beam-column joint | >100%<br>>100%<br>>100% |
| Concrete column drift in north-east corner | Excessive column drift leading to loss of axial capacity and therefore partial collapse. | >100% |
| Concrete block wall in north-west corner | Out of plane flexural failure when the wall is restrained at the top.<br>Out of plane flexural failure when the wall is cantilevering (current condition due to the settlement of the wall and the resultant failure of the top fixings) | 70%<br>18% |
| Pounding with the building to the west | Pounding with the building to the west. This is unlikely to be an initiator of damage to the structure as the existing damage has only occurred to the roof parapet, which would fail before transferring significant seismic load into the building. | [1] |
| Bored concrete piles | Bearing and uplift | 94% |

| Structural Element/System | Failure mode and description of limiting criteria based on capacity of critical element. | % NBS based on calculated capacity |
|---|---|---|
| Northern elevation precast concrete wall | Wall flexure | >100% |

Notes

1. Guidance from NZSEE 2006 suggests that for buildings of similar height where floor levels are aligned, pounding effects are considered to be insignificant, and will generally result in some local damage, mostly non-structural and nominal structural. Local damage to the floor slabs could adversely affect floor slab performance if it results in damage or loss of post tensioning anchorage. This effect can be addressed during the strengthening phase.

## 7.6 Discussion of Results

The quantitative assessment results show that the primary structure has a seismic capacity of approximately 84%NBS. Building elements with a seismic capacity less than 100% NBS include the connection of the roof diaphragm to the walls, the foundation beam under the north-east wall, and the reinforced masonry wall at ground floor level along the north-west elevation. In an undamaged state the masonry wall has a capacity of 70% NBS, however the settlement of this wall relative to the piled structure has resulted in it no longer being restrained at roof level, thereby causing it to cantilever from ground level. In this condition the wall has a seismic capacity of 18% NBS.

Once the north-west boundary masonry wall is propped the building will have a seismic capacity of around 84% NBS as governed by the connection between the floor diaphragm and the precast concrete walls.

Due to damage sustained to the wall it is recommended that this wall is replaced with a new wall complying with the New Building Standard. This will result in the building having a seismic capacity of 84% NBS.

The short return wall along the northern elevation of the building resists a significant proportion of the demands resulting from the irregular layout of the walls. The critical component of this system is the bored concrete piles that resist the uplift forces resulting from the wall flexural actions. The piles under this wall have a seismic capacity of 94%NBS.

The seismic capacities of the columns along the eastern elevation were calculated to be greater than 100% NBS using force based methods. The displacement capacity of the north-east column (this column was chosen as it is furthest away from the centre of rigidity of the building) was calculated by generating in- and out-of-plane moment-curvature relationships. This was done using the assumed geometrical and material properties based on guidance from NZSEE (2006) [2]. This capacity was compared against the displacement demand obtained from the computer model and was found to be greater than 100% NBS.

# 8 Summary of Geotechnical Appraisal

A geotechnical assessment has been completed as part of this quantitative assessment and is included as Appendix 4 of this report. A brief summary of the report is as follows.

## 8.1 Liquefaction Potential

Liquefaction induced subsidence of up to 80mm has been predicted (CLiq analysis) in a future ULS seismic event at this site. This subsidence is predicted to occur within the top 15m of soils underlying the site.

An alternative analysis was also performed using an inbuilt transition layer reduction routine which removes layers that are progressing from soft material to stiff material or vice versa, these layers will not strictly liquefy therefore may be excluded from the analysis. By ignoring these layers, the analysis predicts a 20% to 26% reduction in settlement to 60mm maximum.

In the liquefaction analysis, non-liquefiable layers have been identified below the groundwater table between 0 to 7m depth (Unit 2 and Unit 3 (where $q_c$>15MPa). These layers comprises of either fine-grained clayey silt or dense sands. The presence of these layers is likely to reduce the potential for differential settlement and ground surface damage at the site.

Records indicate the ground in the vicinity of the Tram Barn site may have undergone peak ground shaking in the order of 0.16-0.64g in the 2010 and 2011 earthquake events.

There is currently a significant risk of a magnitude 6 or greater earthquake event occurring which could induce liquefaction and ground settlement at the site.

## 8.2 Lateral Spreading

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 increasing distance from the stream. The magnitude of future lateral spreads and the area of land that may be affected will depend on the characteristics of the earthquake shaking.

The topography is relatively flat across the site. The nearest waterway to the site is the Avon River, which is located 360m north of the site. Due to this distance to the watercourse and the lack of lateral ground movement recorded during the earthquake events of 2010 and 2011, the land at the Tram Barn building is considered to have a low risk of lateral spreading.

## 8.3 Conclusions

The Tram Barn buildings foundations have performed relatively well following the recent seismic events. Based on the underlying soil profile, it has been inferred that the underlying piles extend to a depth of approximately 10.0m below ground level. A liquefaction assessment predicts this building is likely to experience less than 30mm differential settlement in future ULS seismic events, where differential settlements can be approximated as 50 to 70% of the total settlement.

Uniform settlement has occurred to the northern masonry wall. Observations from the shallow investigations have not identified any damage to the shallow foundations. A number of remedial options may be undertaken based on whether the wall is to remain or be rebuilt. Options may include restraining the wall in its current position, re-levelling the wall, or replacing the wall.

## 8.4 Summary

For the Tram Barn building it is recommended that:

- The existing foundations are accepted on the basis that no evidence of damage was observed or expected given the relatively minor ground damage. In this case the Christchurch City Council needs to accept that there is a potential for up to 30mm of differential settlement that may occur in a future ULS seismic event.

- The settlement observed in the concrete floor slab is accepted and the Christchurch City Council accept the risk of further settlement in a future ULS seismic event. Minor works may need to be undertaken to provide adequate clearance between the concrete and tram.

For the northern masonry wall it is recommended that:

- The selection of the most appropriate foundation option for the wall should consider the susceptibility of soils at the site to liquefy in future seismic events, occupant's safety and the economics of undertaking such remedial solution.

- Specific analysis of the foundations would be required in the detailed design phase. A dependable bearing capacity of 90 kPa is indicated for surficial soils at the site and needs to be addressed in the design of shallow foundations (if adopted), or a shallow foundation ground treatment should be considered.

# 9 Conclusions

a) The seismic capacity of the building in its original condition was calculated to be 70%NBS as governed by the out of plane flexural capacity of the masonry wall on the north-west boundary. All other elements have a seismic capacity greater than 84%NBS as governed by the capacity of the floor diaphragm to precast wall connection.

b) The masonry wall has settled around 50mm relative to the piled structure and as a result the wall is no longer restrained at the top. The wall therefore has a current seismic capacity of 18% NBS.

c) The masonry wall should be propped as an interim measure. Once this wall has been propped full occupancy of the building is expected to resume subject to CERA approval.

d) It is recommended that the masonry wall be replaced due to the damage sustained to it. Once the wall is removed and replaced with a new element complying with the New Building Standard the Tram Barn will have a seismic capacity of 84% NBS.

e) The northern building has been independently assessed by Holmes Consulting Group, and they have reported that the building has a seismic capacity of around 63% NBS.

f) The building appears to have performed well throughout the Canterbury Earthquake sequence. The detailing of the structure appears to have largely contributed to that.

g) Differential settlement has also occurred between the southern building and the carpark building to the north.

h) Pounding has occurred between the Tram Barn building and the building to the west, however the potential for pounding forces to be transferred to the structure is limited by the capacity of the roof level parapet.

# 10 Recommendations

a) The masonry wall on the north-west boundary should be propped.

b) Investigate a remedial solution for the masonry wall on the north-west boundary.

# 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 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 (2011), *Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes*, Structural Engineering Society of New Zealand, 21 December 2011.

[6] DBH (2012), *Guidance for engineers assessing the seismic performance of non-residential and multi-unit residential buildings in greater Christchurch*, Department of Building and Housing, June 2012

# Appendix 1 – Photographs

<table>
<tr><td colspan="3">Tram Barn</td></tr>
<tr><td>**No.**</td><td>**Item description**</td><td>**Photo**</td></tr>
<tr><td colspan="3"><u>General</u></td></tr>
<tr><td>**1.**</td><td>General view of the Tram Barn from the north</td><td></td></tr>
<tr><td>**2.**</td><td>General view of the Tram Barn from the east, looking north</td><td></td></tr>
</table>

| | | |
|---|---|---|
| **3.** | Damage to spandrels (north east column in photo) |  |
| **4.** | Separation of window frames from columns on east boundary |  |

| | | |
|---|---|---|
| **5.** | Ground floor area, looking south |  23/03/2012 14:26 |
| **6.** | Horizontal cracks in panels extending out from panel connection |  |

| | | |
|---|---|---|
| **7.** | Ground floor area, looking north |  23/03/2012 14:27 |
| **8.** | Reinforced masonry wall along the north-western boundary |  23/03/2012 14:32 |
| **9.** | Failed masonry wall restraint to roof slab |  |

| | | |
|---|---|---|
| **10.** | Visible settlement adjacent to the north-western boundary wall |  |
| **11.** | Differential settlement of slab next to service pit |  |
| **12.** | Differential settlement along the length of the seismic joint at roof level |  |

| | | |
|---|---|---|
| **13.** | Roof parapet in the south-western corner of the roof |  23/03/2012 15:01 |
| **14.** | Pounding damage between the roof parapet and the building to the west |  |

# Appendix 2 - Quantitative Assessment

## METHODOLOGY AND ASSUMPTIONS

## A2.1. Reference Documents

- AS/NZS 1170.0:2002, *Structural design actions, Part 0: General principles,* Standards New Zealand.
- AS/NZS 1170.1:2002, *Structural design actions, Part 1: Permanent, imposed and other actions,* Standards New Zealand.
- NZS1170.5:2004, *Structural design actions, Part 5: Earthquake actions – New Zealand,* Standards New Zealand.
- NZS 3101: Part 1:2006, *Concrete Structures Standard, The Design of Concrete Structures,* Standards New Zealand.
- NZS3101: Part 2:2006, *Concrete Structures Standard, Commentary on the Design of Concrete Structures,* Standards New Zealand.
- NZSEE: 2006, *Assessment and improvement of the structural performance of buildings in earthquakes,* New Zealand Society for Earthquake Engineering.
- 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 6, 16 May 2012.

## A2.2. Analysis Parameters

The following parameters are used for the seismic analysis

- Site Soil Category D (deep and soft soil);
- Seismic Hazard Factor Z = 0.3;
- Return Period Factor $R_u$ = 1.0 (Importance Level 2 structure, 50 year design life);
- Ductility Factor $\mu = 1.25$ (Nominally Ductile Structure – in accordance with SESOC (2011), *Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes*, Structural Engineering Society of New Zealand, 21 December 2011).
- Structural Performance Factor $S_p$ =0.9 (NZS3101:Part 1:2006 Clause 2.6.2.2.1)

## A2.3. Material Properties

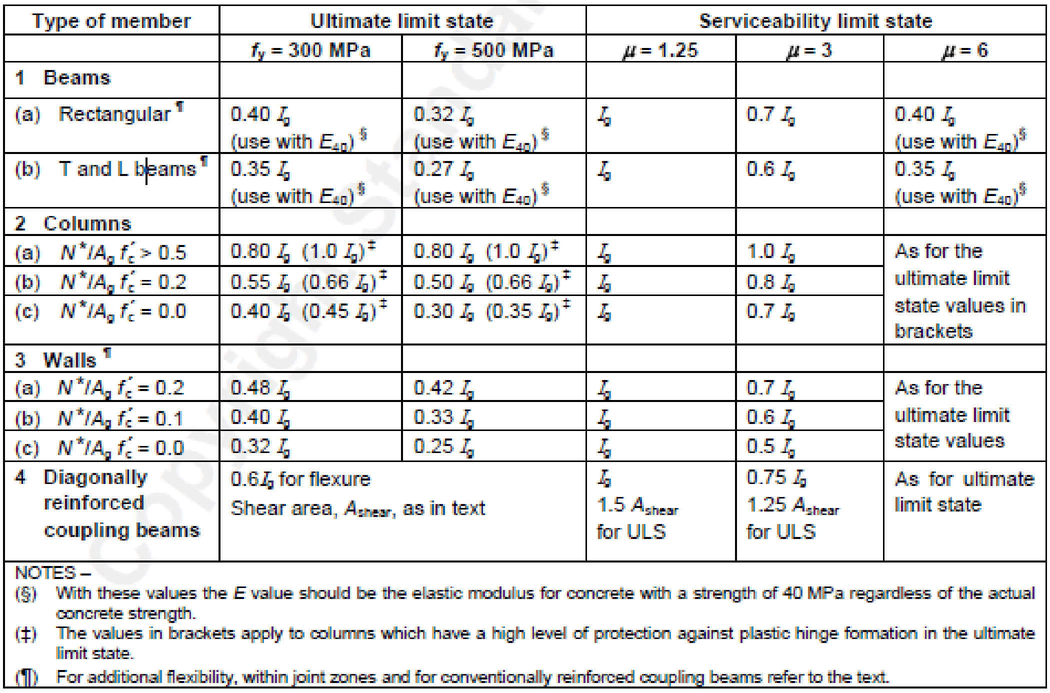
**Table A3: Analysis Material Properties**

| | |
|---|---|
| Retrofitted concrete nominal compressive strength, *f'c* (MPa) (1) | 37.5 |
| High strength reinforcing nominal yield strength, *fy* (MPa) (2) | 464 |
| Mild reinforcing nominal yield strength, *fy* (MPa) (4) | 300 |

Notes:
1. Based on guidance from *NZSEE 2006,* probable concrete compressive strength is based on a value of 1.5 times the nominal compressive strength (Cl. 7.1.1)
2. Based on guidance from *NZSEE 2006,* probable reinforcement yield strength is based on a value of 1.08 times the nominal yield strength (Cl. 7.1.1)
3. Based on guidelines from *Bridge Manual 2004,* probable concrete compressive strength for historical construction.
4. Based on guidelines from *Bridge Manual 2004,* characteristic yield strength of reinforcement for historical construction.

## A2.4. Effective Section Properties

**Table A4: Effective Section Properties from NZS 3101:2006**



| Type of member | Ultimate limit state | | Serviceability limit state | | |
|---|---|---|---|---|---|
| | $f_y$ = 300 MPa | $f_y$ = 500 MPa | $\mu$ = 1.25 | $\mu$ = 3 | $\mu$ = 6 |
| 1 Beams | | | | | |
| (a) Rectangular [¶] | 0.40 $I_g$<br>(use with $E_{40}$) [§] | 0.32 $I_g$<br>(use with $E_{40}$) [§] | $I_g$ | 0.7 $I_g$ | 0.40 $I_g$<br>(use with $E_{40}$)[§] |
| (b) T and L beams [¶] | 0.35 $I_g$<br>(use with $E_{40}$) [§] | 0.27 $I_g$<br>(use with $E_{40}$) [§] | $I_g$ | 0.6 $I_g$ | 0.35 $I_g$<br>(use with $E_{40}$)[§] |
| 2 Columns | | | | | |
| (a) $N^*/A_g f'_c$ > 0.5 | 0.80 $I_g$ (1.0 $I_g$)[‡] | 0.80 $I_g$ (1.0 $I_g$)[‡] | $I_g$ | 1.0 $I_g$ | As for the ultimate limit state values in brackets |
| (b) $N^*/A_g f'_c$ = 0.2 | 0.55 $I_g$ (0.66 $I_g$)[‡] | 0.50 $I_g$ (0.66 $I_g$)[‡] | $I_g$ | 0.8 $I_g$ | |
| (c) $N^*/A_g f'_c$ = 0.0 | 0.40 $I_g$ (0.45 $I_g$)[‡] | 0.30 $I_g$ (0.35 $I_g$)[‡] | $I_g$ | 0.7 $I_g$ | |
| 3 Walls [¶] | | | | | |
| (a) $N^*/A_g f'_c$ = 0.2 | 0.48 $I_g$ | 0.42 $I_g$ | $I_g$ | 0.7 $I_g$ | As for the ultimate limit state values |
| (b) $N^*/A_g f'_c$ = 0.1 | 0.40 $I_g$ | 0.33 $I_g$ | $I_g$ | 0.6 $I_g$ | |
| (c) $N^*/A_g f'_c$ = 0.0 | 0.32 $I_g$ | 0.25 $I_g$ | $I_g$ | 0.5 $I_g$ | |
| 4 Diagonally reinforced coupling beams | 0.6$I_g$ for flexure<br>Shear area, $A_{shear}$, as in text | | $I_g$<br>1.5 $A_{shear}$ for ULS | 0.75 $I_g$<br>1.25 $A_{shear}$ for ULS | As for ultimate limit state |

NOTES –
(§) With these values the *E* value should be the elastic modulus for concrete with a strength of 40 MPa regardless of the actual concrete strength.
(‡) The values in brackets apply to columns which have a high level of protection against plastic hinge formation in the ultimate limit state.
(¶) For additional flexibility, within joint zones and for conventionally reinforced coupling beams refer to the text.

## A2.5. Assessment Methodology

Equivalent Static Analysis

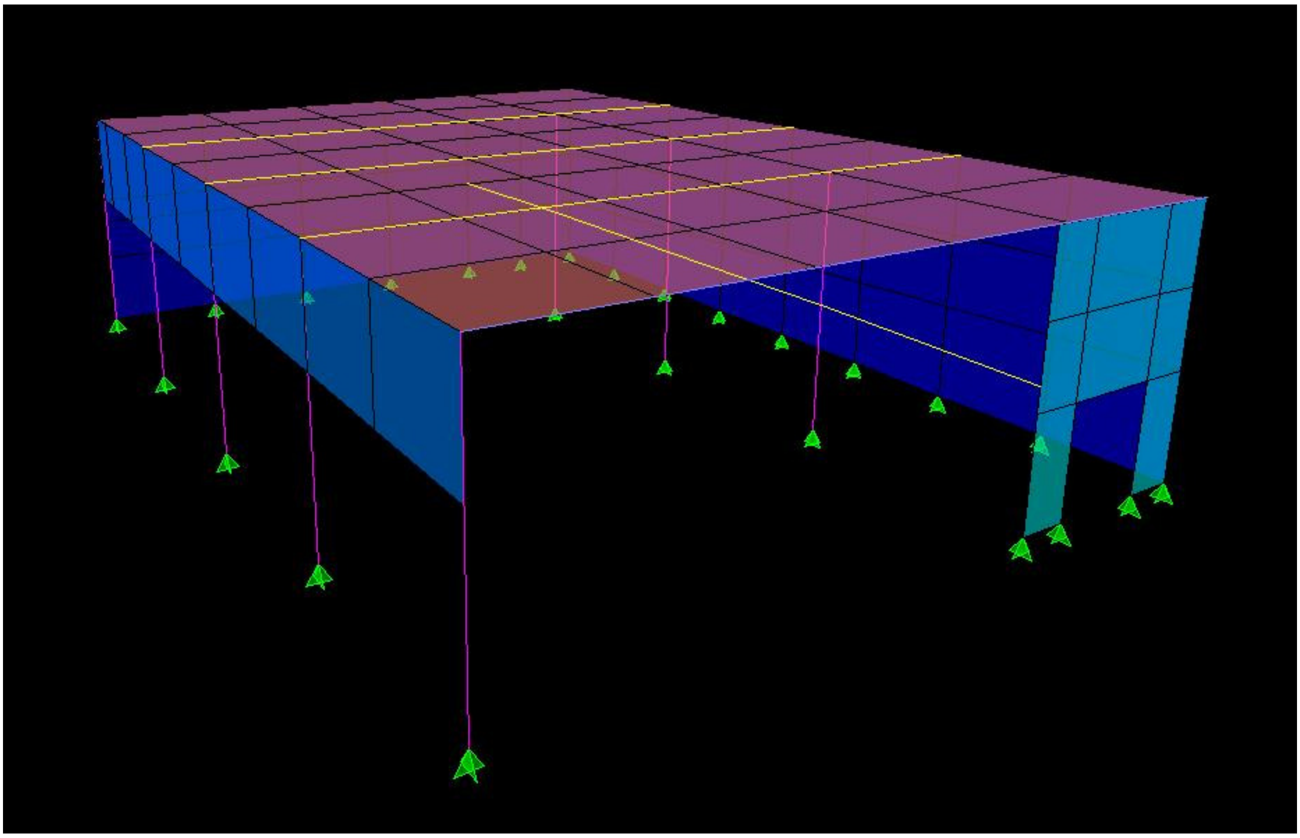


Figure A.1 – ETABS model

The building was assessed at Importance Level 2 (IL2).

The building modes of free vibration outputted from ETABS are:

$T_1$ = 0.18 seconds (translational mode);

$T_2$ = 0.09 seconds (translational mode);

$T_3$ = 0.05 seconds (torsional mode).

The building was analysed as being nominally ductile ($\mu$ = 1.25). The design actions were applied separately in each perpendicular direction, with 100% for the first axis plus 30% on the second axis, and then 30% on the first axis and 100% in the second axis, as required by NZS1170.5:2004 for nominally ductile and brittle structures (Clause 5.3.1.2). These actions were also shifted by applying an accidental eccentricity of ±0.1 times the plan dimension of the structure at right angles to the direction of loading (Clause 5.3.2)

Element force demands were extracted from the equivalent static analysis and compared to calculated capacities based on material properties assumed in Table A1. The results of these capacity to demand ratio checks are summarised in further detail in the report and presented as %NBS.

The displacement capacity of the north-east column was determined by generating an out-of-plane moment-curvature relationship using Xtract. The model was prepared by providing the known geometric properties and assumed material properties based on guidance from NZSEE (2006), *Assessment and improvement of the structural performance of buildings in earthquakes,* New Zealand Society for Earthquake Engineering. The method for calculating the drift capacity of the column was then determined using the NZSEE (2006), *Assessment and improvement of the structural performance of buildings in earthquakes,* New Zealand Society for Earthquake Engineering document, and 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.

The maximum displacement experienced by the north-east column in the ETABS model was compared against the capacity to determine the approximate %NBS.

# Appendix 3  - DEEP Spreadsheet

# Detailed Engineering Evaluation Summary Data

V1.11

## Location

Building Name: Tram Barn
Unit No: Street
Building Address: 7 Tram Way Lane
Legal Description:
Degrees Min Sec
GPS south:
GPS east:
Building Unique Identifier (CCC): BU 1221-001 EQ2

Reviewer: Alistair Boyce
CPEng No: 209860
Company: Opus International Consultants
Company project number: 6QUCCC.64
Company phone number: 03 363 5400
Date of submission: 1/02/2013
Inspection Date: 23-Mar-12
Revision: Final
Is there a full report with this summary? yes

## Site

Site slope: flat
Soil type:
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): 10.00

## Building

No. of storeys above ground: 1 single storey = 1
Ground floor split? no
Storeys below ground
Foundation type: bored cast-insitu concrete piles
Building height (m): 6.00
Floor footprint area (approx): 520
Age of Building (years): 18

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: 2004-

Strengthening present? no

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

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

### Gravity Structure

Gravity System: frame system
Roof: concrete
Floors: precast concrete with topping
Beams: precast concrete
Columns: precast concrete
Walls: load bearing concrete

slab thickness (mm) 75
unit type and depth (mm), topping Unispan, 75mm topping
overall depth (mm) 750
typical dimensions (mm x mm) 600x400
#N/A

### Lateral load resisting structure

Lateral system along: concrete shear wall
Ductility assumed, μ: 1.25
Period along: 0.18 ##### enter height above at H31
Total deflection (ULS) (mm):
maximum interstorey deflection (ULS) (mm):

**Note: Define along and across in detailed report!**

enter wall data in "IEP period calcs" worksheet for period calculation
estimate or calculation? calculated
estimate or calculation?
estimate or calculation?

Lateral system across: concrete shear wall
Ductility assumed, μ: 1.25
Period across: 0.18 ##### enter height above at H31
Total deflection (ULS) (mm):
maximum interstorey deflection (ULS) (mm):

enter wall data in "IEP period calcs" worksheet for period calculation
estimate or calculation? calculated
estimate or calculation?
estimate or calculation?

### Separations:

north (mm):
east (mm):
south (mm): 180
west (mm): 50

leave blank if not relevant

### Non-structural elements

Stairs: timber
Wall cladding: precast panels
Roof Cladding:
Glazing: aluminium frames
Ceilings:
Services(list):

describe supports
thickness and fixing type

## Available documentation

Architectural none
Structural full
Mechanical none
Electrical none
Geotech report

original designer name/date
original designer name/date Holmes Consulting Group, 1994
original designer name/date
original designer name/date
original designer name/date

## Damage

Site:
(refer DEE Table 4-2)

Site performance:
Settlement: 25-100m
Differential settlement: 0-1:350
Liquefaction:
Lateral Spread: none apparent
Differential lateral spread: none apparent
Ground cracks:
Damage to area:

Describe damage:
notes (if applicable): Settlement of NW masonry wall
notes (if applicable): Minor settlement at southern wall
notes (if applicable):
notes (if applicable):
notes (if applicable):
notes (if applicable):
notes (if applicable):

Building:

Current Placard Status: yellow

Along Damage ratio: 0%
Describe (summary):

Describe how damage ratio arrived at:

Across Damage ratio: 74%
Describe (summary):

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

Diaphragms Damage?: no
Describe: Potentially some minor opening up of the roof slab, result

CSWs: Damage?: no
Describe:

Pounding: Damage?: yes
Describe: Damage to roof parapet on western side of roof

Non-structural: Damage?:
Describe:

## Recommendations

Level of repair/strengthening required: minor structural
Building Consent required: yes
Interim occupancy recommendations: partial occupancy

Describe: Note: the NW masonry wall has a capacity of 18% as it i
Describe:
Describe:

Along Assessed %NBS before e'quakes: 100% ##### %NBS from IEP below
Assessed %NBS after e'quakes: 100%

If IEP not used, please detail assessment methodology: Quantitative seismic assessment

Across Assessed %NBS before e'quakes: 70% ##### %NBS from IEP below
Assessed %NBS after e'quakes: 18%

# Appendix 4 - Geotechnical Appraisal

*Tram Barn Building*

# Geotechnical Assessment

## 7 Tramway Lane

## Christchurch CBD

OPUS

*Tram Barn Building*

# Geotechnical Assessment

## 7 Tramway Lane

## Christchurch CBD

Prepared By
Brayden Barnett
Graduate Geotechnical Engineer

Reviewed By
Greg Saul
Principal Geotechnical Engineer

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Date: 25 October 2012
Reference: 6-QUCCC.64 035SC
Status: Draft 1

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# Executive Summary

Opus International Consultants Ltd (Opus) has been commissioned by Christchurch City Council (CCC) to complete a geotechnical investigation and assessment for the Tram Barn building at 7 Tramway Lane, Christchurch. It is intended that this assessment will provide an evaluation of the liquefaction and lateral spreading potential; indicate subsurface soil properties, founding dimensions of the northern masonry wall and an indication of likely founding depths of the underlying piles.

A site specific investigation programme was undertaken which included:

- Two Test Pits and Scala Penetrometer Tests were completed on 27 July 2012 by Taggart Earthmoving.
- Two Boreholes to depths ranging between 30.3m and 27.3m bgl (BH1 and BH2 respectively).
- Two CPT piezocones proceeded to depths of 13.3m and 16.9m bgl (CPTu001 to CPTu02 respectively).

Results from the deep site investigations infer that the underlying soils at the Tram Barn site consist of clayey SILT to silty SAND overlying a clayey SILT layer encountered at approximately 2.5m depth. Beneath the clayey SILT are layers of sandy SILT, clayey SILT and a thick layer of SAND down to the Riccarton Gravel layer at approximately 25m depth. The Northern Masonry Wall appears to be founded on sandy GRAVEL (Fill material).

Liquefaction induced (free-field) subsidence of up to 80mm has been predicted (CLiq analysis) in a future ULS seismic event at this site. This subsidence is predicted to occur within the top 15m of soils underlying the site.

The deep site investigations indicated a dense sand layer from a depth of approximately 9m below ground level. As the pile depths were unable to be made available, we would expect that the piles underlying the Tram Barn building would d be installed beyond this depth to approximately 10m. At this founding depth we would expect settlement in the range of 10 to 15mm, which the structure may have experienced. Differential subsidence is expected to be approximately 50% to 70% of the total subsidence.

The topography is relatively flat across the site. The nearest waterway to the site is the Avon River, which is located 360m north of the site. Due to this distance to the watercourse and the lack of lateral ground movement recorded during the earthquake events of 2010 and 2011, the land at the Tram Barn building is considered to have a low risk of lateral spreading.

The Tram Barn buildings foundations have performed relatively well following the recent seismic events. Based on the underlying soil profile, it has been inferred that the underlying piles extend to a depth of approximately 10.0m below ground level. A liquefaction assessment predicts this building is likely to experience less than 30mm differential settlement in future ULS seismic events.

Uniform settlement has occurred to the Northern Masonry Wall. Observations from the shallow investigations have not identified any damage to the shallow foundations. A number of remedial options may be undertaken based on whether the wall is to remain or be rebuilt.

# Contents

**List of Tables**

**Appendices**

# 1 Introduction

Opus International Consultants Ltd (Opus) has been commissioned by Christchurch City Council (CCC) to complete a geotechnical investigation and assessment for the Tram Barn building at 7 Tramway Lane, Christchurch. It is intended that this assessment will provide an evaluation of the:

- Ground conditions and ground water conditions beneath the Tram Barn Building.
- Founding dimensions and corresponding static bearing capacity of soils underneath the northern concrete masonry wall.
- Nature of liquefaction at the site and assess the potential for future liquefaction and consequential ground damage due to settlement and lateral spreading.
- Possible founding depth of existing concrete piles and the liquefaction potential of soil the underlying soil.

This Geotechnical Assessment report follows on from the Tram Barn Building Geotechnical Desktop Appraisal report issued by Opus dated 23 May 2012.

# 2 Site Description

The Tram Barn Building is located at 7 Tramway Lane, in the Christchurch CBD. The building is bounded to the east by Tramway Lane which is perpendicular to Hereford Street and Worcester Street.

The Tram Barn building is bounded by Tramway Lane and the 8 storey Design and Arts College of New Zealand building to the east. The building is bounded to the south by the 8 storey 161 Hereford Suites building. A paved carpark building is located to the west.

The building is located at NZ Grid Map Grid position 2480821 mE and 5741705 mN.

# 3 Reported Ground Damage

Opus observations of ground damage have been recorded during various site inspections and are outlined (including photographs) in the Tram Barn Building Geotechnical Desk study previously issued.

The northern masonry wall has settled by up to approximately 50mm. Evidence of the settlement is illustrated by distortion of the steel lateral restraints connecting the wall to the roof. All of these restraints have been either pulled out of this wall or the bolts have been pulled out of the roof. The settlement of this wall is also consistent with the 30mm of settlement around the south end of the wall relative to the railway tracks. The cladding between the concrete portal frame and masonry wall shows that the block wall has settled relative to the portal frame. The concrete block wall is suspected to be founded on shallow foundations which are consistent with the settlement observed.

Settlement and ground heave of the footpath along Tramway Lane and the access way directly north of the building, is inferred to have resulted from liquefaction subsidence of the underlying soils.

The two sets of railway tracks closest to the western side of the building and the corresponding concrete slab appear to have settled by up to 40mm.

# 4 As-built Records

Extracts from the Structural Drawings prepared by Holmes Consulting Group illustrating the foundation details have been available for review. We understand that Option 2 for the Tram Barn building involving a system of reinforced concrete piles capable of resisting tension and compression loads was adopted for construction. The piles are 450mm in diameter and are of an unknown depth. Capping of the piles generally consist of 0.6m to 1.4m deep concrete continuous ground beams connecting adjacent piles.

Strip footings (approximately 300mm deep) support the 180mm thick precast concrete wall panels. A 100mm thick reinforced concrete floor slab is connected to the wall panels via reinforcing steel. The foundation type of the concrete block wall is unknown. A 1.5m deep and 12m long tram servicing pit located along the eastern side of the buildings footprint is founded on a 250mm thick concrete slab.

Structural Drawing extracts obtained from Holmes Consulting dated July 1999 indicate that the Northern masonry wall is founded on a shallow strip footing 600mm deep and 350mm wide.

Communications with long serving Tram staff infer that the “franki” piles supporting the Tram Barn building are likely to be founded at a depth of approximately 9 to 10m below ground level (bgl).

# 5 Seismic Considerations

## 5.1 Seismic Category

The relatively deep alluvial formations underlying the Tram Barn building defines this site as Class D – deep or soft soil site, in terms of the seismic design requirements of NZS 1170.5:2004.

## 5.2 Importance Level

An importance level (IL) of 2 has been considered appropriate in the liquefaction assessment of this site, in accordance with NZS 1170.5:2004.

## 5.3 Recorded Peak Ground Accelerations

The site has been subjected to strong seismic shaking with a number of recent earthquakes, especially the 22 February 2011 earthquake which produced very strong ground shaking in Christchurch, CBD. The nearest seismic strong motion recording station to the site is at the Christchurch Botanic Gardens (CBGS) situated approximately 1.4km west of the site. Table 2 indicates the peak ground accelerations recorded at the CBGS site for the various significant recent earthquakes (Cousins, 2012).

**Table 1: Peak Ground Acceleration Values for CBGS Strong Motion Recorder**

| Earthquake Magnitude and Date | Peak Ground Acceleration |
|---|---|
| M7.1, 4 September 2010 | 0.18g |
| M6.3, 22 February 2011 | 0.64g |
| M5.6, 13 June 2011 | 0.21g |
| M6.0, 13 June 2011 | 0.18g |
| M6.0, 23 December 2011 | 0.16g |

## 5.4 Design Peak Ground Accelerations

Interim guidance published on 27th April 2012 (DBH, 2012) recommends for the assessment of liquefaction for soil class D sites; the Geotechnical Engineer should apply a peak horizontal ground acceleration of 0.13g for a 1 in 25 year event (SLS) and 0.35g for a 1 in 500 year event (ULS).

**Table 2: Preliminary Design Peak Ground Accelerations**

| Importance Level =2⁽¹⁾ | SLS⁽²⁾ | ULS⁽³⁾ |
|---|---|---|
| Annual Probability of Exceedance | 1/25yr | 1/500yr |
| Peak Ground Acceleration (PGA) | 0.13g | 0.35g |

*Notes:*

1) *The proposed buildings on the property are designated in terms of AS/NZS 1170 as Importance Level 2.*
2) *SLS-Serviceability Limit State*
3) *ULS-Ultimate Limit State*

## 5.5 Likelihood of Future Damaging Seismic Events

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 13% probability of another Magnitude 6 or greater earthquake occurring in the year between 9 September 2012 and 9 September 2013 in the Canterbury region. This seismic event may cause liquefaction induced land damage at the site similar to that experienced, dependent on the location of the earthquakes epicentre. This confirms that there is currently a significant risk of liquefaction and further ground 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 Geotechnical Investigation Scope

## 6.1 Shallow Geotechnical Investigations

Two Test Pits and Scala Penetrometer Tests were completed on 27 July 2012 by Taggart Earthmoving. The test pits were undertaken along the northern masonry wall to confirm the foundation type and dimensions whilst assessing the underlying bearing capacity of the soils. The results have been included in Appendix E.

## 6.2 Deep Geotechnical Investigations

Deep site specific investigations were initiated by Opus on behalf of the Christchurch City Council and undertaken by McMillan Drilling Services Ltd.

The investigations included:

- Two boreholes to depths ranging between 30.3m and 27.3m bgl (BH1 and BH2 respectively). Split spoon SPTs were performed at 1.5m intervals until the Riccarton gravel layer was reached. Solid nose SPTs were carried out in this gravel layer. Both boreholes were conducted using a Sonic drilling rig which extended the borehole into the underlying Riccarton Gravel formation.

- Two CPT Piezocones (CPTu001 to CPTu02) proceeded to depths of 13.3m and 16.9m bgl respectively.

The locations of the Boreholes and CPTs were surveyed using a handheld GPS and are shown on the Site Investigation Plan, located in Appendix B.

# 7 Soil Profile

The following soil profile has been interpreted from the geotechnical investigations completed at the site.

**Table 3: Interpreted Soil Profile and Liquefaction Potential**

| Unit | Stratigraphy | Thickness (m) | Depth Encountered from (m) bgl | Liquefiable SLS | Liquefiable ULS |
|---|---|---|---|---|---|
| 1 | Fill Material | 0.5 – 1.5m | Surface | No | No |
| 2 | SILT to Clayey SILT<br>$q_c$ = 1 - 9 MPa<br>'N' = 2 - 5 | 5.8 -7.0m | 0.5-1.5m | No | No |
| 3 | SAND<br>$q_c$ = 4 - 35 MPa<br>'N' = 2 - 34 | 12.4-13.6m | 7.7-8.5m | No | Yes (when $q_c$ < 15MPa) |
| 4 | SILT<br>'N' = 0 - 10 | 3.5-3.8m | 21.2-21.3m | No | No |
| 5 | Sandy GRAVEL (Riccarton Formation)<br>'N' = 60+ | - | 24.7-25.7m | No | No |

*Notes:*

*N values quoted are raw values and have not been corrected for the SPT hammer energy efficiency (estimated to be 85%).*

*$q_c$ values have been obtained from the CPT tests.*

Groundwater was encountered at a depth of approximately 2.0 to 2.5m below ground level.

# 8 Liquefaction

A liquefaction assessment has been completed using CLiq software (Version 1.7, 2012) adopting the Robertson Method with settlements calculated using Zhang et al (2002). Cone Penetrometer Tests (CPT's) form the basis for prediction of liquefaction potential, with a Magnitude 7.5 earthquake considered, and a groundwater depth of 2.0m.

Both the serviceability limit state (SLS) and ultimate limit state (ULS) seismic loadings have been assessed (with PGA's as specified in Table 2), with liquefaction induced settlement estimates given over the complete soil depth of the test (refer Appendix F).

**Table 4: Estimated Liquefaction Induced Settlements for SLS and ULS Seismic Events**

| Event | Mag/PGA | | CPT-01 | CPT-02 |
|---|---|---|---|---|
| SLS | M7.5/0.13g | Total# | 20mm | Negligible |
| | | Excluding Transition Layers^ | Negligible | Negligible |
| | | Below 10m Depth* | Negligible | Negligible |
| ULS | M7.5/0.35g | Total# | 80mm | 75mm |
| | | Excluding Transition Layers^ | 60mm<br>[-20%] | 55mm<br>[-26%] |
| | | Below 10m Depth* | 15mm | 10mm |

*Note:*

*# Settlement calculation over the total CPT depth.*

*^ Thin layers that are not truly representative as they are in 'transition' from either soft to stiff soils or visa-versa.*

** Settlement below the pile (assumed) bearing depth.*

*Negligible = subsidence < 10mm.*

For comparison, a liquefaction assessment was completed for soils at the site based on the recorded earthquake magnitudes and PGA's, as indicated in Table 1. The results are shown below in Table 5.

**Table 5: Estimated Liquefaction Induced Settlements for Recorded Seismic Events**

| Event | Mag/PGA/$PGA_{7.5}$ | CPT-01 | CPT-02 |
|---|---|---|---|
| 4 September 2010 | M7.1/0.26g/0.23g | 40mm | 30mm |
| 22 February 2011 | M6.3/0.64g/0.49g | 65mm | 60mm |
| 13 June 2011 | M5.6/0.16g/0.1g | Negligible | Negligible |
| 13 June 2011 | M6.0/0.22g/0.15g | Negligible | Negligible |
| 23 December 2011 | M6.0/0.21g/0.14g | Negligible | Negligible |
| *Negligible = subsidence <10mm.* | | | |

# 9 Lateral Spreading

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 increasing distance from the stream. The magnitude of future lateral spreads and the area of land that may be affected will depend on the characteristics of the earthquake shaking.

The topography is relatively flat across the site. The nearest waterway to the site is the Avon River, which is located 360m north of the site. Due to this distance to the watercourse and the lack of lateral ground movement recorded during the earthquake events of 2010 and 2011, the land at the Tram Barn building is considered to have a low risk of lateral spreading.

# 10 Northern Masonry Wall

The Northern Masonry Wall extends from the north west corner of the Tram Barn on a diagonal trajectory towards the north east of the buildings footprint. This wall has settled uniformly by approximately 50mm, which has also affected the ability to use the large northern roller door.

Two test pits were undertaken along the wall to an approximate depth of 0.8m below ground level (bgl). The foundations were measured and are shown on the drawing presented in Appendix G. Visual observations of the foundations at these locations confirmed that these were shallow strip footings and presented no evidence of cracking or spalling.

The surficial soil profile beneath the wall, with dependable bearing capacities correlated to Scala Penetrometer tests (Stockwell, 1977) and other presumptive geotechnical properties (Look, 2007) have been summarised in Table 7 below. The bearing capacities stated are indicative only, and do not take into account load eccentricity or loss of soil shear strength with liquefaction. It is recommended that detailed analysis of individual footings be carried out in design. Founding conditions should be confirmed at the time of construction.

**Table 6: Inferred Design Properties for Surficial Soils under Northern Masonry wall.**

| Stratigraphy | *Dependable Bearing Capacity (kPa) | Thickness (m) | Soil Friction Angle Ø (degrees)^ | Cohesion C or undrained shear strength Su (kPa) | Depth Encountered from (m) bgl |
|---|---|---|---|---|---|
| Sandy GRAVEL | 90 - 140 | 0-1.6m | 35 | 0 | Surface |
| Sandy GRAVEL | 190 - 285 | - | 35 | 0 | 1.6m |

**The Dependable Bearing Capacity obtained from Stockwell (1977) correlations (incorporating Φ=0.5 as per B1/VM4) of the surficial soils. To be confirmed when the actual foundation dimensions are selected and should be reassessed using Terzaghi (1943) Bearing Capacity Equations in accordance with B1/VM4.*

*^ Obtained from Table 5.5 (Look, B. Handbook of Geotechnical Investigation and Design Tables (2007))*

The test pits were only performed on the southern side of the wall due to the basement of the Heritage Carpark located on the adjacent side. A fill depth underlying the masonry wall of up to 8m has been inferred. This depth has been evaluated based on the fact that the carpark is founded at depth of approximately 5.0m below ground level. At this depth, there is a soft silt layer which may be unsuitable for founding on; therefore it has likely resulted in the silt being replaced with compacted fill to the sand layer at approximately 7.5m below ground level.

# 11 Discussion

## 11.1 Soil Profile

Results from the Deep Soil Investigations infer that the underlying soils at the Tram Barn site consist of clayey SILT to silty SAND overlying a clayey SILT layer encountered at approximately 2.5m depth. Beneath the clayey SILT are layers of sandy SILT, clayey SILT and a thick layer of SAND down to the Riccarton Gravel layer at approximately 25m depth.

Results from the shallow soil investigations alongside the northern masonry wall infer that the underlying surficial soils beneath the northern masonry wall consist of sandy Gravel (fill material) down to an inferred depth of 8m.

## 11.2 Liquefaction Potential

Liquefaction induced (free-field) subsidence of up to 80mm has been predicted (CLiq analysis) in a future ULS seismic event at this site. This subsidence is predicted to occur within the top 15m of soils underlying the site.

An alternative analysis was also performed using an inbuilt transition layer reduction routine which removes layers that are progressing from soft material to stiff material or vice versa, these layers will not strictly liquefy therefore maybe excluded from the analysis. By ignoring these layers, the analysis predicts a 20% to 26% reduction in settlement to 60mm maximum.

In the liquefaction analysis, non-liquefiable layers have been identified below the groundwater table between 0 to 7m depth (Unit 2 and Unit 3 (where $q_c$>15MPa)). These layers comprises of either fine-grained clayey silt or dense sands. The presence of these layers is likely to reduce the potential for differential settlement and ground surface damage at the site.

Records indicate the ground in the vicinity of the Tram Barn site may have undergone peak ground shaking in the order of 0.16-0.64g in the 2012 and 2011 earthquake events.

There is currently a significant risk of a magnitude 6 or greater earthquake event occurring which could induce liquefaction and ground settlement at the site.

## 11.3 Tram Barn Building

As outlined earlier in this report, extracts from the Structural Drawings prepared by Holmes Consulting Group illustrating the foundation details have been available for review. We understand that Option 2 for the Tram Barn building involving a system of reinforced concrete piles capable of resisting tension and compression loads were adapted for construction. The piles are approximately 450mm in diameter and are of an unknown depth. Capping of the piles generally consist of 0.6m to 1.4m deep continuous concrete ground beams between adjacent piles.

The deep site investigations indicated a dense sand layer from a depth of approximately 9m below ground level. As the pile depths were unable to be made available, we would expect that the piles underlying the Tram Barn building would d be installed beyond this depth to approximately 10m. At this founding depth we would expect settlement in the range of 10 to 15mm, which the structure may have experienced. Differential subsidence is expected to be approximately 50% to 70% of the total subsidence.

Deep piled foundations are beneficial as they distribute the load from the structure into competent underlying soils, reducing the effect of potentially liquefiable soil layers on the foundations.

Provided that the pile-column connection has maintained integrity and the piles have adequate capacity, the foundation system should be accepted. This conclusion is supported by the good performance of the building and relatively favourable soil conditions underlying the site.

### Concrete Floor Slab

Up to 40mm of differential settlement has occurred to the concrete floor slab that encompasses the two western tram tracks. The concrete floor slab located in the tram housing area is not supported by ground beams, nor are sufficiently tied to the adjacent columns or surrounding ground beams. The concrete slab that has settled appears to be separate from the remainder of the concrete floor slab, and has settled along the joint.

Communications with Tram Barn staff have suggested that this has limited the ability to use these tram rails due to low points of the tram scraping the concrete.

There are several plausible reasons as to why this has occurred. These include: temporary loss of bearing capacity with the tram weight being a contributing factor, consolidation of the underlying fill material with a lack of influence from piles, or liquefaction induced settlement of the underlying material. Remedial options which could be adopted to for this settlement include: removing the miss-aligned concrete, replacing the section of concrete slab, or installing a pile system.

## 11.4 Northern Masonry Wall

The existing footings appear to have settled by approximately 50mm following the Christchurch earthquake sequence of 2010 and 2011. Visual observations of the foundations of the two trial pit locations did not identify any evidence of structural damage to the concrete shallow footings. There are a number of options which could be adopted for the remediation of this wall which include: restraining the wall in its current position, re-level the wall or replace the wall.

# 12 Conclusions

The Tram Barn buildings foundations have performed relatively well during the recent seismic events. Based on the underlying soil profile, it has been inferred that the underlying piles extend to a depth of approximately 10.0m below ground level. A liquefaction assessment predicts this building is likely to experience less than 30mm of differential settlement in future ULS seismic events, where differential settlements has been approximated as 50 to 70% of the total subsidence.

Uniform settlement has occurred to the Northern Masonry Wall. Observations from the shallow investigations have not identified any damage to the shallow foundations. A number of remedial options may be undertaken based on whether the wall is to remain or be rebuilt. Options may include: restraining the wall in its current position, re-levelling or replacing the wall.

# 13 Recommendations

For the Tram Barn Building it is recommended that:

- The existing foundations are accepted on the basis that no evidence of damage was observed or expected given the relatively minor ground damage. In this case the Christchurch City Council needs to accept that there is a potential for up to 30mm of differential settlement to occur in a future ULS seismic event.

- The existing differential settlement observed in the concrete floor slab is less than normally accepted tolerance and should be accepted providing the Christchurch City Council also accepts the risk of further settlement in a future ULS seismic event. Minor works may need to be undertaken to provide adequate clearance between the concrete and and tram.

For the Northern Masonry Wall it is recommended that:

- The selection of the most appropriate foundation option for the wall should consider the susceptibility of soils at the site to liquefy in future seismic events, occupant's safety and the economics of undertaking such remedial solution.

- Specific analysis of the foundations would be required in the detailed design phase. A dependable bearing capacity of 90 kPa is indicated for surficial soils at the site and needs to be addressed in the design of shallow foundations (if adopted), or a shallow foundation ground treatment should be considered.

# 14 Limitation

This report has been prepared solely for the benefit of the Christchurch City Council as our client with respect to the particular brief given to us. Data or opinions in this assessment report may not be used in other contexts, by any other party or for any other purpose.

It is recognised that the passage of time affects the information and assessment provided in this document. Opus's opinions are based upon information that existed at the time of the production of this assessment report. It is understood that the Services provided allowed Opus to form no more than an opinion on the actual conditions of the site at the time the site investigations were undertaken and cannot be used to assess the effect of any subsequent changes in the quality of the site, or its surroundings or any laws or regulations.

# 15 References

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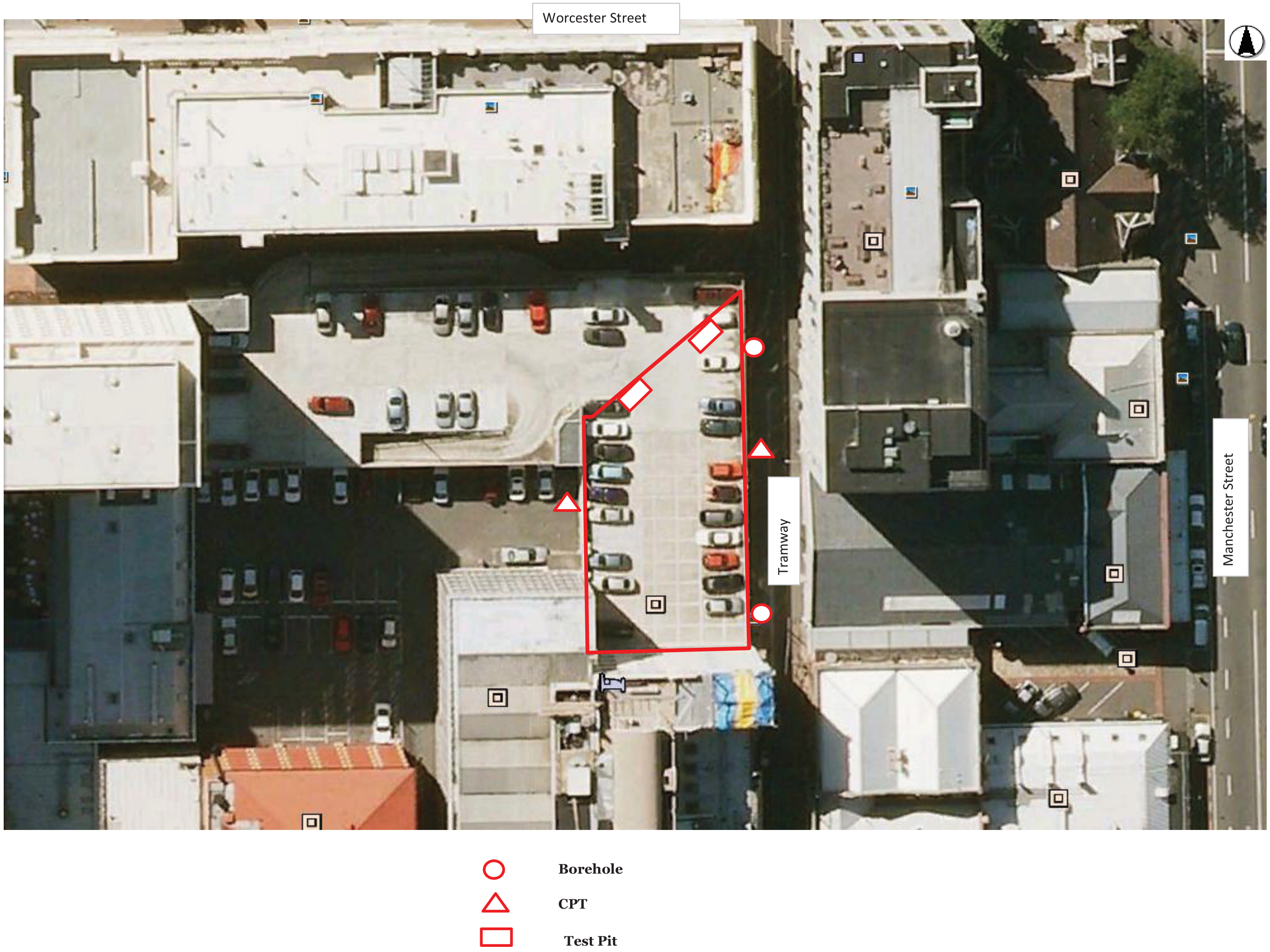
# Appendix A:

## Site Location Plan



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| | |
|---|---|
| **Project:** | Tram Barn Buildng, 7 Tramway Lane |
| **Project No.:** | 6-QUCCC.64/035SC |
| **Client:** | Christchurch City Council |

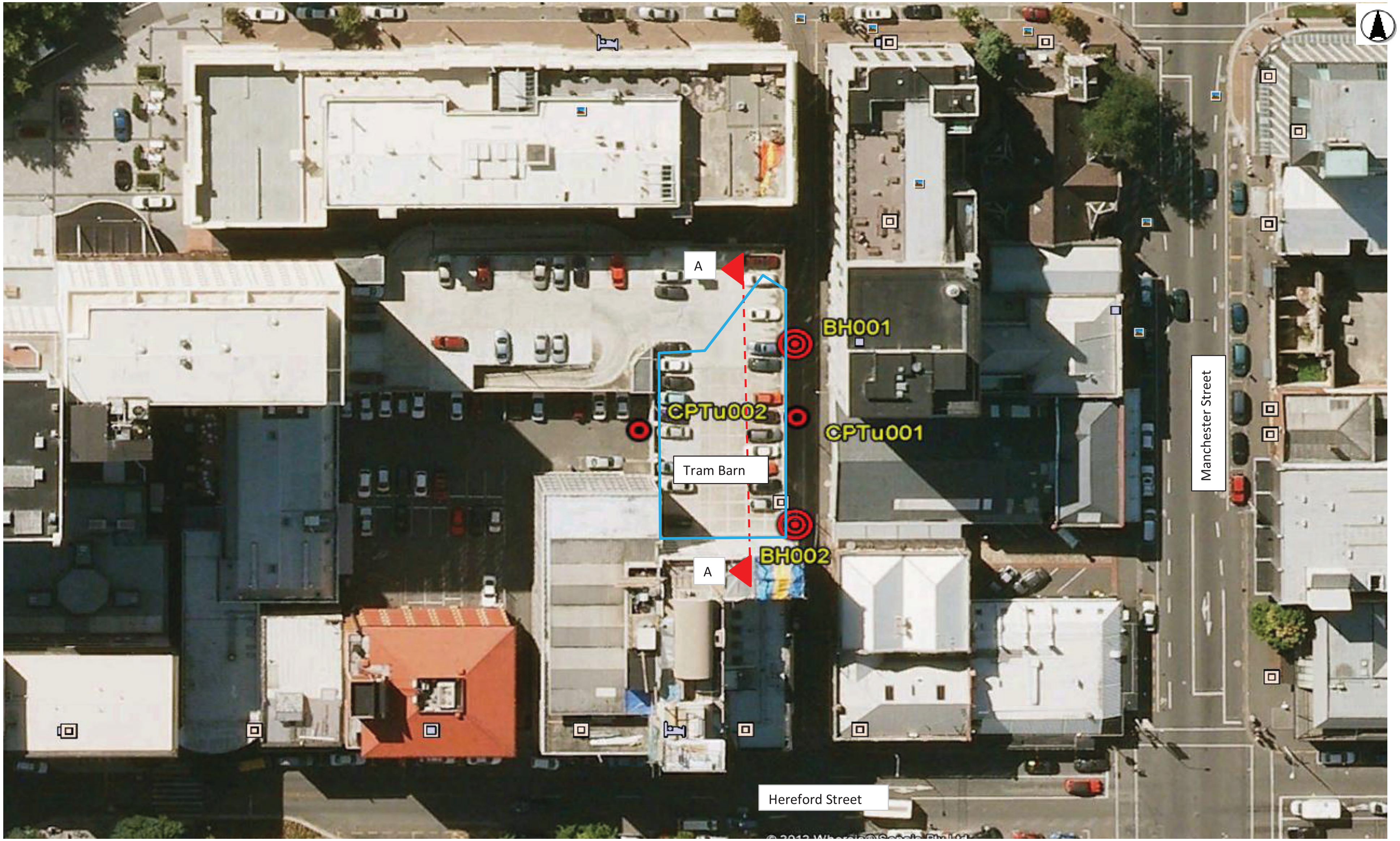
# Site Location Plan

**Drawn:** Opus Geotechnical Engineer

**Date:** 2-Oct-12

# Appendix B:

## Site Investigation Cross-Sections



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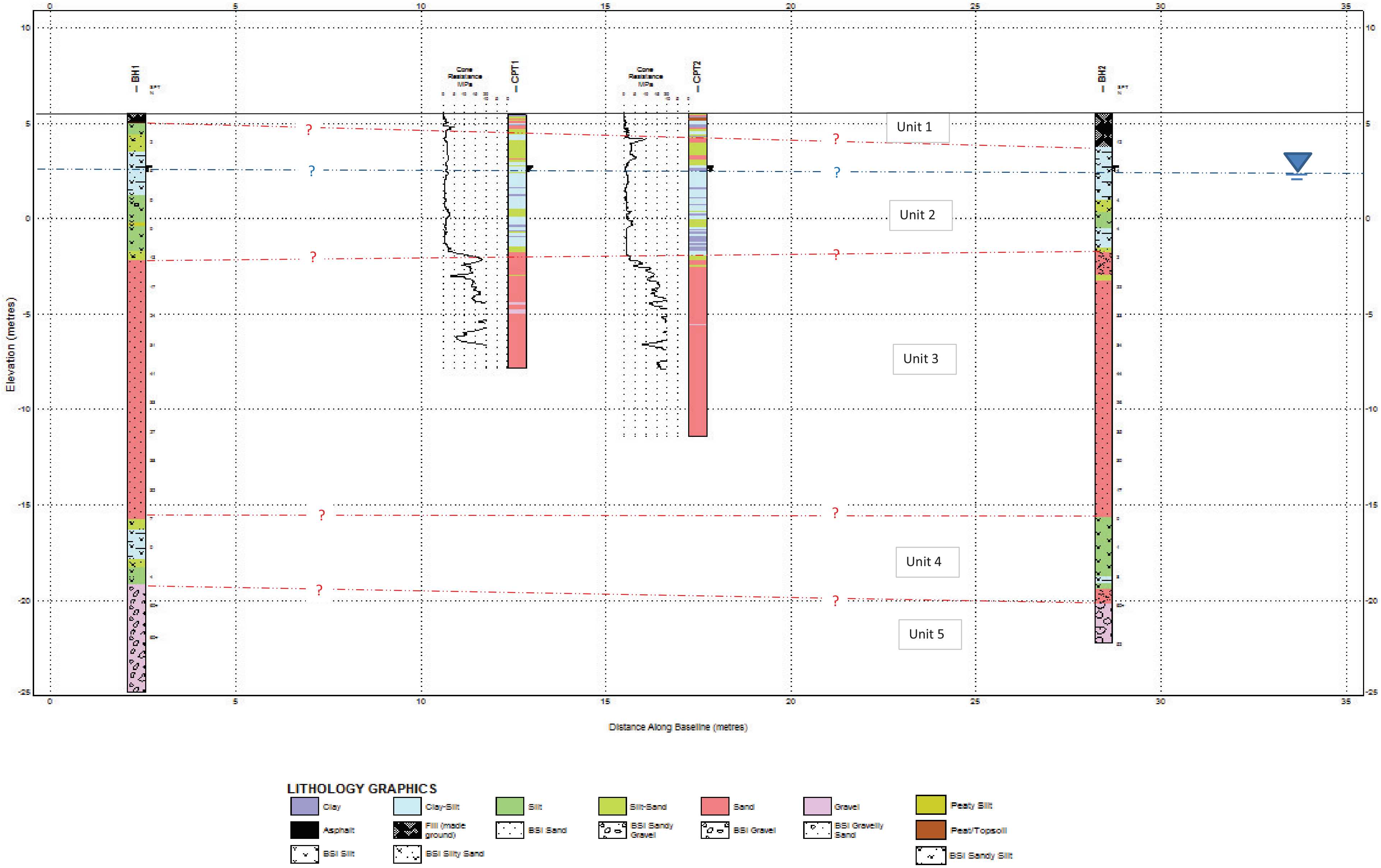
**Project No.:** 6-QUCCC.64/035SC
**Client:** Christchurch City Council

# Cross-Section Plan

**Drawn:** Opus Geotechnical Engineer

**Date:** 2-Oct-12



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**Project No.:** 6-QUCCC.64/035SC
**Client:** Christchurch City Council

## Section A-A

**Drawn:** Opus Geotechnical Engineer

**Date:** 2-Oct-12

# Appendix C:

Borehole (BH) Logs

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# BOREHOLE LOG

| | | | | | |
|---|---|---|---|---|---|
| Hole No. | **BH1** | Sheet | 1 of 4 | | |
| Project | Tram Barn Building | Co-ord. | 2480831 E 5741716 N | R.L. | Approx. 5.5 m |
| Location | North East corner of Tram Barn building | Ref. Grid | NZMG | Datum | MSL |
| Hole Length | 30.3 m | | | | |

| GEOLOGY/UNIT | MAIN DESCRIPTION | R.L. (m) | DEPTH (m) | GRAPHIC LOG | TESTS: SPT 'N' VALUE | TESTS: SPT BLOW COUNTS OR SHEAR VALUE | ROCK STRENGTH | ROCK WEATHERING | DEFECT SPACING | DIP degrees 0 90 | DETAILED DESCRIPTION | CORE: RQD (%) | CORE: TOTAL CORE RECOVERY (%) | CORE: SAMPLE TYPE | DRILLING: DRILLING METHOD | DRILLING: DRILLING FLUID LOSS | DRILLING: CASING | BASE OF HOLE & WATER LEVEL | PIEZOMETER DETAILS | OTHER INSTRUMENTATION |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Fill | Asphalt | | | | | | | | | | | | | | | | | | | |
| | Sandy fine to coarse GRAVEL; brownish grey. Very loose; well graded. | | | | | | | | | | Gravel is rounded to sub-rounded, maximum size is 40 mm Ø. Medium to coarse grained sand. | | | | | | | | | |
| | SILT with some sand; grey mottled brown. "Soft"; low plasticity. | | 1 | | | | | | | | Historic demoltion material. Sand is fine grained. | | 80 | Sonic | | | | | | |
| Springston Formation | Silty fine SAND; brownish grey. Very loose to loose; poorly graded. | 4 | | | 2 | 0/0//0/0/1/1 | | | | | | | 91 | SPT | | | | | | |
| | Clayey SILT with some sand; grey mottled brown. "Firm"; high plasticity. | | 2 | | | | | | | | Sand is fine grained. | | 100 | Sonic | | | | | | |
| | Clayey SILT; grey. "Soft"; high plasticity. | | | | | | | | | | | | | | | | | | | |
| | Fine to medium SAND with some silt; grey. Loose; poorly graded. | | 3 | | 5 | 0/1//1/1/1/2 | | | | | | | 80 | SPT | | | | | | |
| | Clayey SILT; grey. "Soft", highly plastic. | 2 | | | | | | | | | | | | | | | | | | |
| | Becomes SILT; grey mottled brown, with minor organic inclusions. | | 4 | | | | | | | | | | 74 | Sonic | | | | | | |
| | Organic SILT with some clay; dark brown. "Soft", moderate plasticity, fibrous inclusions. | | | | | | | | | | | | | | | | | | | |
| | Clayey SILT with some organic fibrous inclusions; grey mottled dark brown. "Soft"; high plasticity. | | 5 | | 0 | 0/0//0/0/0/0 | | | | | | | 80 | SPT | | | | | | |
| | | 0 | | | | | | | | | | | 87 | Sonic | | | | | | |
| | SILT with minor sand and trace clay; grey. "Firm"; low plasticity. | | | | | | | | | | Sand is fine grained. | | | | | | | | | |
| | Dark brown fibrous lense at 5.70 m. | | | | | | | | | | | | | | | | | | | |
| | SILT minor clay; grey to light brown. "Soft"; high plasticity, with trace organic inclusions. | | 6 | | 0 | 0/0//0/0/0/0 | | | | | | | 71 | SPT | | | | | | |
| | | | 7 | | | | | | | | | | 100 | Sonic | | | | | | |
| Christchurch Formation | Silty fine to medium SAND; grey. Medium dense; poorly graded. | -2 | | | | | | | | | | | | | | | | | | |
| | Fine to medium SAND with trace silt; grey. Medium dense; poorly graded. | | | | 13 | 1/2//3/4/3/3 | | | | | | | 51 | SPT | | | | | | |
| | | | 8 | | | | | | | | | | 100 | Sonic | | | | | | |
| | At 8.50 to 8.53 m: Silt lense. | | 9 | | 17 | 1/2//4/4/4/5 | | | | | | | 60 | SPT | | | | | | |
| | | -4 | | | | | | | | | | | 100 | Sonic | | | | | | |

## NOTES

SPT: Safety Auto Trip Hammer #368 used.
SC = Solid Cone

| | | | |
|---|---|---|---|
| Started | 27/08/2012 | Finished | 29/08/2012 |
| Driller | D. Keown | Drilling Co. | McMillan |
| Inclination/ Azimuth | -90° | Drilling Rig | Geoprobe 8140LS (DT45) |
| Logged | B. Barnett | Checked | F. Neeson |
| Client | Christchurch City Council | Job No. | 6-QUCCC.64 |

**BH1**

LOGGED IN ACCORDANCE WITH NZ GEOTECHNICAL SOCIETY (2005) GUIDELINES

SEE ATTACHED KEY SHEET FOR EXPLANATION OF SYMBOLS

Scale 1:33.33

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# BOREHOLE LOG

| | | | |
|---|---|---|---|
| HOLE NO. | **BH1** | SHEET | **2** of **4** |
| PROJECT | **Tram Barn Building** | CO-ORD. | **2480831 E 5741716 N** |
| R.L. | **Approx. 5.5 m** | LOCATION | **North East corner of Tram Barn building** |
| REF. GRID | **NZMG** | DATUM | **MSL** |
| HOLE LENGTH | **30.3 m** | | |

| GEOLOGY/UNIT | MAIN DESCRIPTION | R.L. (m) | DEPTH (m) | GRAPHIC LOG | SPT 'N' VALUE | SPT BLOW COUNTS OR SHEAR VALUE | ROCK STRENGTH | ROCK WEATHERING | DEFECT SPACING | DIP degrees 0 90 | DETAILED DESCRIPTION | RQD (%) | TOTAL CORE RECOVERY (%) | SAMPLE TYPE | DRILLING METHOD | DRILLING FLUID LOSS | CASING | BASE OF HOLE & WATER LEVEL | PIEZOMETER DETAILS | OTHER INSTRUMENTATION |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Christchurch Formation | Fine to medium SAND with trace silt; grey. Medium dense; poorly graded. | | | | | | | | | | | | 100 | Sonic | | | | | | |
| | Fine to medium SAND; grey. Medium dense; poorly graded. | | | | 24 | 2/2//4/5/7/8 | | | | | | | 60 | SPT | | | | | | |
| | At 11.50 m less silt.<br>At 11.80: SAND becomes medium dense to dense. | -6 | 11<br>12 | | | | | | | | | | 100 | Sonic | | | | | | |
| | | | | | 31 | 2/4//7/6/8/10 | | | | | | | 71 | SPT | | | | | | |
| | | -8 | 13 | | | | | | | | | | 100 | Sonic | | | | | | |
| | Becomes fine SAND at 13.50 m. | | 14 | | 41 | 1/2//6/10/12/13 | | | | | | | 91 | SPT | | | | | | |
| | | | 15 | | | | | | | | | | 100 | Sonic | | | | | | |
| | | -10 | | | 33 | 5/6//7/8/9/9 | | | | | | | 80 | SPT | | | | | | |
| | | | 16 | | | | | | | | | | 100 | Sonic | | | | | | |
| | At 16.60 m: Becomes fine to medium SAND. Medium dense. | | 17 | | 27 | 2/4//6/7/7/7 | | | | | | | 71 | SPT | | | | | | |
| | | -12 | 18 | | | | | | | | | | 100 | Sonic | | | | | | |
| | At 18.10 m: Becomes dense. | | | | 38 | 2/3//6/9/11/12 | | | | | | | 80 | SPT | | | | | | |
| | At 18.60 m: Fine to medium SAND with minor silt; grey. Dense; poorly graded. | -14 | 19 | | | | | | | | | | 100 | Sonic | | | | | | |
| | Fine to medium SAND; grey. Medium dense, poorly graded and fragmented shell inclusions. | | | | 32 | 2/4//7/7/9/9 | | | | | Shells fine to medium sized, matrix supported. | | 60 | SPT | | | | | | |

## NOTES

SPT: Safety Auto Trip Hammer #368 used.
SC = Solid Cone

| | | | |
|---|---|---|---|
| STARTED | 27/08/2012 | FINISHED | 29/08/2012 |
| DRILLER | D. Keown | DRILLING CO. | McMillan |
| INCLINATION/ AZIMUTH | -90° | DRILLING RIG | Geoprobe 8140LS (DT45) |
| LOGGED | B. Barnett | CHECKED | F. Neeson |
| CLIENT | Christchurch City Council | JOB NO. | 6-QUCCC.64 |

**BH1**

*LOGGED IN ACCORDANCE WITH NZ GEOTECHNICAL SOCIETY (2005) GUIDELINES*

*SEE ATTACHED KEY SHEET FOR EXPLANATION OF SYMBOLS*

Christchurch Office

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Tel: +64 3 363 5400
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www.opus.co.nz

# BOREHOLE LOG

| | | | | |
|---|---|---|---|---|
| PROJECT<br>Tram Barn Building | CO-ORD.<br>2480831 E 5741716 N | R.L.<br>Approx. 5.5 m | HOLE NO.<br>**BH1** | SHEET<br>3 of 4 |
| LOCATION<br>North East corner of Tram Barn building | REF. GRID<br>NZMG | DATUM<br>MSL | HOLE LENGTH<br>30.3 m | |

| GEOLOGY/UNIT | MAIN DESCRIPTION | R.L. (m) | DEPTH (m) | GRAPHIC LOG | TESTS: SPT 'N' VALUE | TESTS: SPT BLOW COUNTS OR SHEAR VALUE | ROCK STRENGTH | ROCK WEATHERING | DEFECT SPACING | DIP degrees 0 90 | DETAILED DESCRIPTION | CORE: RQD (%) | CORE: TOTAL CORE RECOVERY (%) | CORE: SAMPLE TYPE | DRILLING: DRILLING METHOD | DRILLING: DRILLING FLUID LOSS | DRILLING: CASING | BASE OF HOLE & WATER LEVEL | PIEZOMETER DETAILS | OTHER INSTRUMENTATION |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Christchurch Formation | Fine to medium SAND with trace silt; grey. Medium dense; poorly graded.<br>Fine SAND with minor silt; grey. Medium dense to dense; poorly graded. | | | | | 2/4//7/7/9/9 | | | | | | | | SPT | | | | | | |
| | At 20.7 m to 20.73 m: Lense of fibrous organic material.<br>At 21.0 to 21.05 m: Lense of shell fragements. | | 21 | | | | | | | | Shells fine to medium sized, matrix supported. | | 100 | Sonic | | | | | | |
| | Fine sandy SILT with trace clay; grey. "Firm", moderate plasticity and minor shell fragments. | -16 | | | 7 | 4/4//3/2/1/1 | | | | | Shells fine to medium sized, matrix supported. | | 100 | SPT | | | | | | |
| | Clayey SILT; grey. "Firm", highly plastic and fibrous organic inclusions. | | 22 | | | | | | | | | | 100 | Sonic | | | | | | |
| | SILT with trace clay; brownish grey. "Soft", high plasticity. | | 23 | | 0 | 1/0//0/0/0/0 | | | | | | | 71 | SPT | | | | | | |
| | Becomes SILT with some clay; trace sand. "Firm" to "stiff"<br>Silty fine to medium SAND with trace clay; grey. Loose; poorly graded.<br>SILT with minor clay and trace sand; grey. "Firm"; high plasticity. | -18 | 24 | | | | | | | | Sand is fine to medium grained | | 100 | Sonic | | | | | | |
| | Organic SILT; brown. "Firm"; low plasticity.<br>SILT with minor sand; grey. "Firm"; low plasticity. | | | | 4 | 1/0//0/0/2/2 | | | | | Sand is fine to medium grained. | | 91 | SPT | | | | | | |
| Riccarton Gravels | Fine to coarse GRAVEL with some sand; brownish grey. Very dense; poorly graded. | -20 | 25 | | | | | | | | Gravel is rounded to sub-rounded, maximum size is 60 mm Ø. | | 100 | Sonic | | | | | | |
| | Sandy fine to coarse GRAVEL; light brown. Very dense; well graded. | | 26 | | 60+ | 10/18//22/26/21 for 225 mm | | | | | Gravel is rounded to sub-rounded, maximum size is 30 mm Ø. Sand is medium to coarse grained. | | SC | SPT | | | | | | |
| | Fine to coarse GRAVEL with some sand; light brown. Very dense; poorly graded. | | 27 | | | | | | | | Gravel is rounded to sub-rounded, maximum size is 50 mm Ø. | | 74 | Sonic | | | | | | |
| | Sandy fine to coarse GRAVEL; light brown. Very dense; well graded. | -22 | | | 60+ | 15/24//22 for 135 mm | | | | | Gravel is rounded to sub-rounded, maximum size is 30 mm Ø. | | SC | SPT | | | | | | |
| | | | 28 | | | | | | | | | | 100 | Sonic | | | | | | |
| | | | 29 | | | | | | | | | | SC | SPT | | | | | | |
| | | -24 | | | | | | | | | | | 50 | Sonic | | | | | | |

## NOTES

SPT: Safety Auto Trip Hammer #368 used.
SC = Solid Cone

| | |
|---|---|
| STARTED<br>27/08/2012 | FINISHED<br>29/08/2012 |
| DRILLER<br>D. Keown | DRILLING CO.<br>McMillan |
| INCLINATION/ AZIMUTH<br>-90° | DRILLING RIG<br>Geoprobe 8140LS (DT45) |
| LOGGED<br>B. Barnett | CHECKED<br>F. Neeson |
| CLIENT<br>Christchurch City Council | JOB NO.<br>6-QUCCC.64 |

**BH1**

LOGGED IN ACCORDANCE WITH NZ GEOTECHNICAL SOCIETY (2005) GUIDELINES

SEE ATTACHED KEY SHEET FOR EXPLANATION OF SYMBOLS

BOREHOLE_LOG_A3 W_W/O PHOTO PAGE BH_CPTS.GPJ OPUS CHCH JUL12.GDT 25/10/12

Scale 1:33.33

Christchurch Office

PO Box 1482
Christchurch, NZ

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www.opus.co.nz

# BOREHOLE LOG

| | | | | |
|---|---|---|---|---|
| HOLE NO. | | | | **BH1** |
| PROJECT<br>**Tram Barn Building** | CO-ORD.<br>**2480831 E** | **5741716 N** | R.L.<br>**Approx. 5.5 m** | SHEET<br>**4** of **4** |
| LOCATION<br>**North East corner of Tram Barn building** | REF. GRID<br>**NZMG** | | DATUM<br>**MSL** | HOLE LENGTH<br>**30.3 m** |

| GEOLOGY/UNIT | MAIN DESCRIPTION | R.L. (m) | DEPTH (m) | GRAPHIC LOG | TESTS<br>SPT 'N' VALUE | TESTS<br>SPT BLOW COUNTS OR SHEAR VALUE | ROCK STRENGTH | ROCK WEATHERING | DEFECT SPACING | DIP<br>degrees 0 90 | DETAILED DESCRIPTION | CORE<br>RQD (%) | CORE<br>TOTAL CORE RECOVERY (%) | CORE<br>SAMPLE TYPE | DRILLING<br>DRILLING METHOD | DRILLING<br>DRILLING FLUID LOSS | DRILLING<br>CASING | BASE OF HOLE & WATER LEVEL | PIEZOMETER DETAILS | OTHER INSTRUMENTATION |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Sandy fine to coarse GRAVEL; light brown. Very dense; well graded. | | | | | | | | | | Gravel is rounded to sub-rounded, maximum size is 30 mm Ø. | | 50 | Sonic | | | | | | |
| | End of hole at 30.4 m: Target Depth Reached (approx. 4 m into the Riccarton Gravels) | | | | | | | | | | | | SC | SPT | | | | | | |
| | | | 31 | | | | | | | | | | | | | | | | | |
| | | -26 | | | | | | | | | | | | | | | | | | |
| | | | 32 | | | | | | | | | | | | | | | | | |
| | | | 33 | | | | | | | | | | | | | | | | | |
| | | -28 | | | | | | | | | | | | | | | | | | |
| | | | 34 | | | | | | | | | | | | | | | | | |
| | | | 35 | | | | | | | | | | | | | | | | | |
| | | -30 | | | | | | | | | | | | | | | | | | |
| | | | 36 | | | | | | | | | | | | | | | | | |
| | | | 37 | | | | | | | | | | | | | | | | | |
| | | -32 | | | | | | | | | | | | | | | | | | |
| | | | 38 | | | | | | | | | | | | | | | | | |
| | | | 39 | | | | | | | | | | | | | | | | | |
| | | -34 | | | | | | | | | | | | | | | | | | |

## NOTES

SPT: Safety Auto Trip Hammer #368 used.
SC = Solid Cone

| | |
|---|---|
| STARTED<br>27/08/2012 | FINISHED<br>29/08/2012 |
| DRILLER<br>D. Keown | DRILLING CO.<br>McMillan |
| INCLINATION/ AZIMUTH<br>-90° | DRILLING RIG<br>Geoprobe 8140LS (DT45) |
| LOGGED<br>B. Barnett | CHECKED<br>F. Neeson |
| CLIENT<br>Christchurch City Council | JOB NO.<br>6-QUCCC.64 |

**BH1**

LOGGED IN ACCORDANCE WITH NZ GEOTECHNICAL SOCIETY (2005) GUIDELINES

SEE ATTACHED KEY SHEET FOR EXPLANATION OF SYMBOLS

Scale 1:33.33

Christchurch Office

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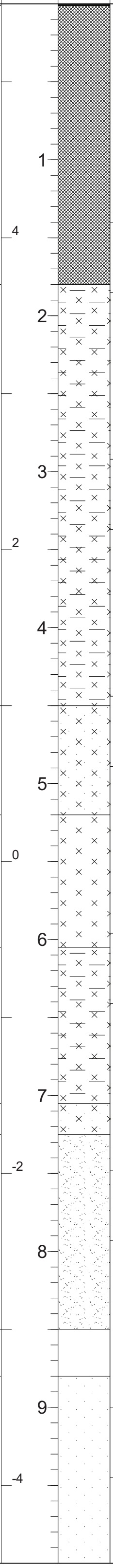
www.opus.co.nz

# BOREHOLE LOG

| | |
|---|---|
| HOLE NO. | **BH2** |
| PROJECT | Tram Barn Building |
| CO-ORD. | 2480831 E 5741690 N |
| R.L. | Approx. 5.5 m |
| SHEET | 1 of 3 |
| LOCATION | South East corner of Tram Barn building |
| REF. GRID | NZMG |
| DATUM | MSL |
| HOLE LENGTH | 27.74 m |

| GEOLOGY/UNIT | MAIN DESCRIPTION | DETAILED DESCRIPTION |
|---|---|---|
| Fill | Asphalt | |
| | Sandy fine to coarse GRAVEL with some silt; brownish grey. Medium dense, well graded. | Gravel is sub-rounded to rounded with maximum gravel size = 40 Ø. Fine to medium grained sand. |
| Springston Formation | Clayey SILT; light grey, mottled brown. "Soft", high plasticity. | |
| | Becomes highly plastic. | |
| | Becomes blueish grey, low plasticity. | |
| | Sandy SILT; blueish grey. "Soft" to "firm", low plasticity. | Very fine to fine grained sand. |
| | SILT with trace sand; blueish grey. "Firm", non plastic and contains fibrous organic material. | Sand is fine to medium grained. |
| | Some fibrous wood inclusions, rootlets and finer fibrous organic material.<br>Becomes dark greyish brown SILT with some clay. "Soft" to "firm", moderately plastic.<br>Trace fibrous organic material from 5.96 m | Peat lense at 5.80 to 5.82 m. |
| | Clayey SILT; blueish grey, mottled brown. "Very soft", highly plasticity.<br>Becomes "soft", higly plastic and contains some fibrous organic material. | |
| | Trace sand and fibrous organic material from 6.90 m. "Soft to firm", low plasticity. | Sand is fine grained. |
| | Sandy SILT; blueish grey. "Firm", non plastic. | Sand is very fine to fine grained |
| Christchurch Formation | Fine to medium SAND with some silt; blueish grey. Very loose.<br>Decrease in silt 7.48 m. | |
| | Fine to medium silty SAND; blueish grey. Very loose to medium dense. | |
| | Fine to medium SAND with trace silt; blueish grey. Medium dense. | |



| SPT 'N' VALUE | SPT BLOW COUNTS OR SHEAR VALUE | ROCK STRENGTH | ROCK WEATHERING | DEFECT SPACING | DIP (degrees 0 90) | RQD (%) | TOTAL CORE RECOVERY (%) | SAMPLE TYPE | DRILLING METHOD | DRILLING FLUID LOSS | CASING | BASE OF HOLE & WATER LEVEL | PIEZOMETER DETAILS | OTHER INSTRUMENTATION |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | 50 | Sonic | | | | | | |
| 12 | 3/4//5/1/3/3 | | | | | | 51 | SPT | | | | | | |
| | | | | | | | 90 | Sonic | | | | | | |
| 4 | 0/1//1/1/1/1 | | | | | | 64 | SPT | | | | | | |
| | | | | | | | 90 | Sonic | | | | | | |
| 4 | 0/1//1/1/1/1 | | | | | | 91 | SPT | | | | | | |
| | | | | | | | 100 | Sonic | | | | | | |
| 4 | 0/0//1/1/1/1 | | | | | | 100 | SPT | | | | | | |
| | | | | | | | 100 | Sonic | | | | | | |
| 2 | 2/2//0/1/0/1 | | | | | | 51 | SPT | | | | | | |
| | | | | | | | 90 | Sonic | | | | | | |
| 22 | 3/5//5/5/5/7 | | | | | | 40 | SPT | | | | | | |
| | | | | | | | 93 | Sonic | | | | | | |

## NOTES

SPT: Safety Auto Trip Hammer #368 used.
SC = Solid Cone

| | |
|---|---|
| STARTED | 29/08/2012 |
| FINISHED | 31/08/2012 |
| DRILLER | D. Keown |
| DRILLING CO. | McMillans |
| INCLINATION/ AZIMUTH | -90° |
| DRILLING RIG | Geoprobe 8140LS (DT45) |
| LOGGED | J. Claridge |
| CHECKED | F. Neeson |
| CLIENT | Christchurch City Council |
| JOB NO. | 6-QUCCC.64 |

**BH2**

*LOGGED IN ACCORDANCE WITH NZ GEOTECHNICAL SOCIETY (2005) GUIDELINES*

*SEE ATTACHED KEY SHEET FOR EXPLANATION OF SYMBOLS*

Scale 1:33.33

**OPUS**

Christchurch Office

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Christchurch, NZ

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Fax: +64 3 365 7858
www.opus.co.nz

# BOREHOLE LOG

| | | | | |
|---|---|---|---|---|
| PROJECT: Tram Barn Building | CO-ORD.: 2480831 E 5741690 N | R.L.: Approx. 5.5 m | HOLE NO.: BH2 | SHEET: 2 of 3 |
| LOCATION: South East corner of Tram Barn building | REF. GRID: NZMG | DATUM: MSL | HOLE LENGTH: 27.74 m | |

| GEOLOGY/UNIT | MAIN DESCRIPTION | R.L. (m) | DEPTH (m) | GRAPHIC LOG | TESTS: SPT 'N' VALUE | TESTS: SPT BLOW COUNTS OR SHEAR VALUE | ROCK STRENGTH | ROCK WEATHERING | DEFECT SPACING | DIP degrees 0 90 | DETAILED DESCRIPTION | CORE: RQD (%) | CORE: TOTAL CORE RECOVERY (%) | CORE: SAMPLE TYPE | DRILLING: DRILLING METHOD | DRILLING: DRILLING FLUID LOSS | DRILLING: CASING | BASE OF HOLE & WATER LEVEL | PIEZOMETER DETAILS | OTHER INSTRUMENTATION |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Christchurch Formation | Fine to medium SAND with trace silt; blueish grey. Medium dense.<br>At 10.10 m: SAND becomes dark blueish grey. Dense. | | | | | | | | | | | | 93 | Sonic | | | | | | |
| | At 10.50 m: Becomes SAND with some silt. Firm to stiff, medium-dense to dense.<br>At 10.80 m: SAND becomes medium to coarse grained. | | | | 33 | 5/7//9/8/7/9 | | | | | | | 51 | SPT | | | | | | |
| | At 11.90 m: Becomes firm, dense, fine to medium grained. | -6 | 11<br>12 | | | | | | | | | | 100 | Sonic | | | | | | |
| | | | | | 34 | 6/8//8/8/9/9 | | | | | | | 51 | SPT | | | | | | |
| | Trace silt from 12.75 m. | -8 | 13 | | | | | | | | | | 100 | Sonic | | | | | | |
| | At 13.63 to 13.65 m: Lense of shell fragments. | | 14 | | 44 | 6/7//9/11/11/13 | | | | | Shells fine to coarse gravel sized, matrix supported. | | 51 | SPT | | | | | | |
| | SAND becomes fine grained from 14.50 m. | | 15 | | | | | | | | | | 90 | Sonic | | | | | | |
| | At 15.15 m: SAND becomes medium grained, and medium dense. | -10 | | | 26 | 2/3//4/6/7/9 | | | | | | | 51 | SPT | | | | | | |
| | At 15.70 m: Becomes fine grained SAND with some silt; blueish grey. Medium dense.<br>SAND becomes medium grained.<br>Becomes dense at 16.5 m. | | 16 | | | | | | | | | | 85 | Sonic | | | | | | |
| | Trace silt from 16.7 m. | | 17 | | 35 | 4/4//5/8/10/12 | | | | | | | 60 | SPT | | | | | | |
| | At 17.40 m: Becomes medium dense to dense. | -12 | 18 | | | | | | | | | | 86 | Sonic | | | | | | |
| | | | | | 30 | 2/3//5/6/9/10 | | | | | | | 51 | SPT | | | | | | |
| | At 18.80 - 20.05 m: Lense of shell fragments.<br>SAND becomes medium to coarse grained from 19.20 m. | -14 | 19 | | | | | | | | Shells fine to coarse gravel sized, matrix supported. | | 100 | Sonic | | | | | | |
| | At 19.68 m: Fine to medium SAND with trace silt. | | | | 17 | 1/1//2/3/5/7 | | | | | | | 51 | SPT | | | | | | |

## NOTES

SPT: Safety Auto Trip Hammer #368 used.
SC = Solid Cone

| | |
|---|---|
| STARTED: 29/08/2012 | FINISHED: 31/08/2012 |
| DRILLER: D. Keown | DRILLING CO.: McMillans |
| INCLINATION/ AZIMUTH: -90° | DRILLING RIG: Geoprobe 8140LS (DT45) |
| LOGGED: J. Claridge | CHECKED: F. Neeson |
| CLIENT: Christchurch City Council | JOB NO.: 6-QUCCC.64 |

BH2

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www.opus.co.nz

# BOREHOLE LOG

| | | | | |
|---|---|---|---|---|
| HOLE NO. | **BH2** | | | |
| PROJECT | **Tram Barn Building** | CO-ORD. | **2480831 E** **5741690 N** | R.L. **Approx. 5.5 m** |
| SHEET | **3** of **3** | | | |
| LOCATION | **South East corner of Tram Barn building** | REF. GRID | **NZMG** | DATUM **MSL** |
| HOLE LENGTH | **27.74 m** | | | |

| GEOLOGY/UNIT | MAIN DESCRIPTION | R.L. (m) | DEPTH (m) | TESTS: SPT 'N' VALUE | TESTS: SPT BLOW COUNTS OR SHEAR VALUE | ROCK STRENGTH | ROCK WEATHERING | DEFECT SPACING | DIP degrees 0 90 | DETAILED DESCRIPTION | CORE: RQD (%) | CORE: TOTAL CORE RECOVERY (%) | CORE: SAMPLE TYPE | DRILLING: DRILLING METHOD | DRILLING: DRILLING FLUID LOSS | DRILLING: CASING | BASE OF HOLE & WATER LEVEL | PIEZOMETER DETAILS | OTHER INSTRUMENTATION |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Christchurch Formation | Fine to medium SAND with trace silt; blueish grey. Medium dense. | | | | 1/1//2/3/5/7 | | | | | | | | SPT | | | | | | |
| | SAND becomes medium dense to loose.<br>Becomes fine SAND with some silt. Very loose. | | 21 | | | | | | | | | 100 | Sonic | | | | | | |
| | SILT with some sand; blueish grey. "Very soft" to "soft", low plasticity.<br>At 21.40 m: Becomes "soft". Dilatant. | -16 | | 0 | 0/0//0/0/0/0 | | | | | Sand is very fine to fine grained. | | 55 | SPT | | | | | | |
| | SILT with trace of Clay and "firm" at 22.01 m<br>SILT with some clay; dark grey. "Very soft", low plasticity and some fibrous organic material. | | 22 | | | | | | | | | 100 | Sonic | | | | | | |
| | At 22.68 m: Becomes "very soft" to "soft", highly plastic. | | 23 | 1 | 0/0//0/0/0/1 | | | | | | | 100 | SPT | | | | | | |
| | At 23.40 m: Trace rootlets. Becomes soft to firm, low plasticity.<br>At 23.75 m: Becomes SILT with trace sand; blueish grey. "Soft", low plasticity<br>At 24.00 m: No sand. BecomesSILT; grey, mottled brown. "Very soft", low plasticity, dilatant. | -18 | 24 | | | | | | | Sand is fine grained. | | 100 | Sonic | | | | | | |
| | Organic clayey SILT; blackish brown. "Stiff", low plasticity. | | | 8 | 0/1//2/2/2/2 | | | | | Organic odour, stains fingers when rubbed. | | 80 | SPT | | | | | | |
| | SILT with trace sand; grey, mottled brown. "Soft" to "firm", low plasticity, dilatant.<br>Fine SAND with some silt; blueish grey. Dense. | -20 | 25 | | | | | | | Very fine to fine sand. | | 100 | Sonic | | | | | | |
| Riccarton Gravels | Silty fine to coarse GRAVEL with some sand; brown. Medium dense, well graded.<br>At 26.04 m: Trace sand. | | 26 | 60+ | 13/20//15 | | | | | Gravel is sub-angular to sub-rounded, maximum size is 40 mm Ø. Sand is fine to medium grained.<br>At 26.04 m: Maximum gravel size is 25 mm Ø. | | SC | SPT | | | | | | |
| | | -22 | 27 | | | | | | | | | 100 | Sonic | | | | | | |
| | End of Hole at 27.74 m Target depth reached | | 28 | 52 | 3//10/16/13/13 | | | | | | | SC | SPT | | | | | | |
| | | -24 | 29 | | | | | | | | | | | | | | | | |

## NOTES

SPT: Safety Auto Trip Hammer #368 used.
SC = Solid Cone

| | | | |
|---|---|---|---|
| STARTED | 29/08/2012 | FINISHED | 31/08/2012 |
| DRILLER | D. Keown | DRILLING CO. | McMillans |
| INCLINATION/ AZIMUTH | -90° | DRILLING RIG | Geoprobe 8140LS (DT45) |
| LOGGED | J. Claridge | CHECKED | F. Neeson |
| CLIENT | Christchurch City Council | JOB NO. | 6-QUCCC.64 |

**BH2**

LOGGED IN ACCORDANCE WITH NZ GEOTECHNICAL SOCIETY (2005) GUIDELINES

SEE ATTACHED KEY SHEET FOR EXPLANATION OF SYMBOLS

# Appendix D:

## Cone Penetrometer Test (CPT) Results

## CPT ANALYSIS NOTES

### Soil Type

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

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

### Liquefaction Screening

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

🟥 high susceptibility
🟧 medium susceptibility
🟩 low susceptibility

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

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

Low susceptibility is all other cases.

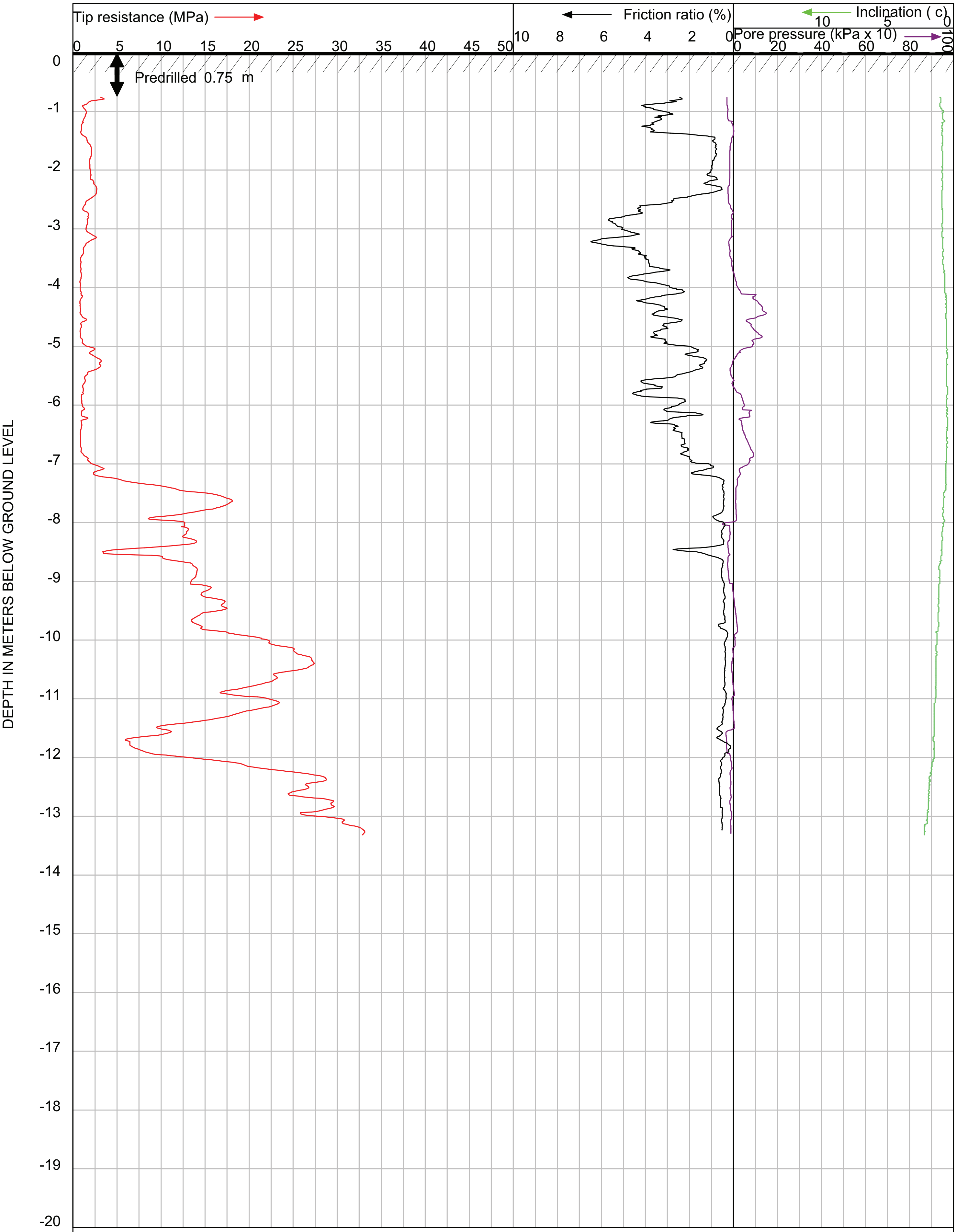
### Relative Density ($D_R$)

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

### Undrained Shear Strength ($S_U$)

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

McMILLAN
DRILLING SERVICES



| | | |
|---|---|---|
| CLIENT | : | OPUS International Consultants Ltd |
| LOCATION | : | Tramway Lane, Christchurch |
| DATE | : | 10-8-2012 |
| OPERATOR | : | H. Pardoe |
| REMARK 1 | : | CPTu001 |
| REMARK 2 | : | Effective Refusal |

JOB # : 10915

TEST # : 1

CONE TYPE/SERIAL #: I-CFXYP20-10/ 111007

**McMILLAN**
DRILLING SERVICES
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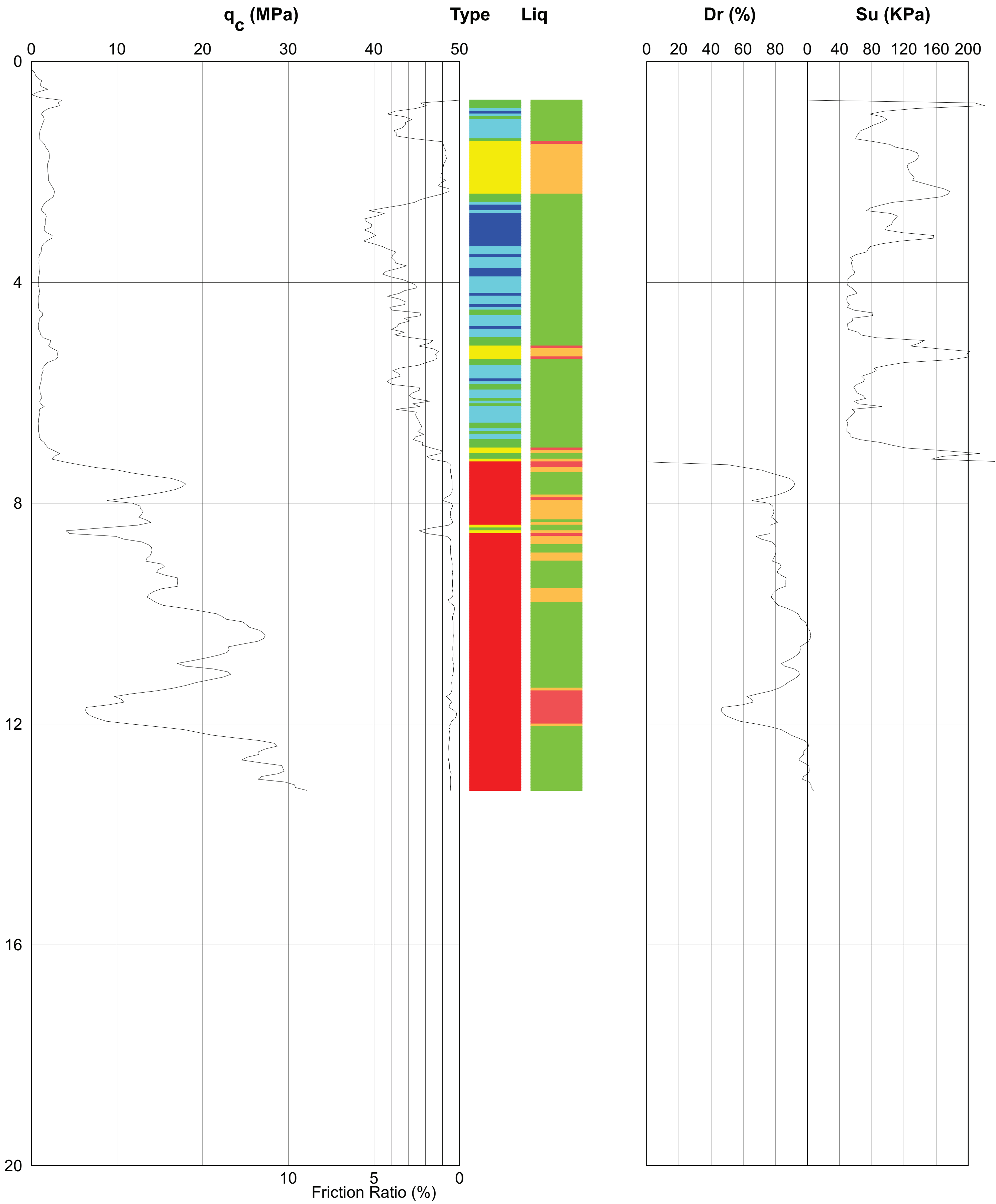
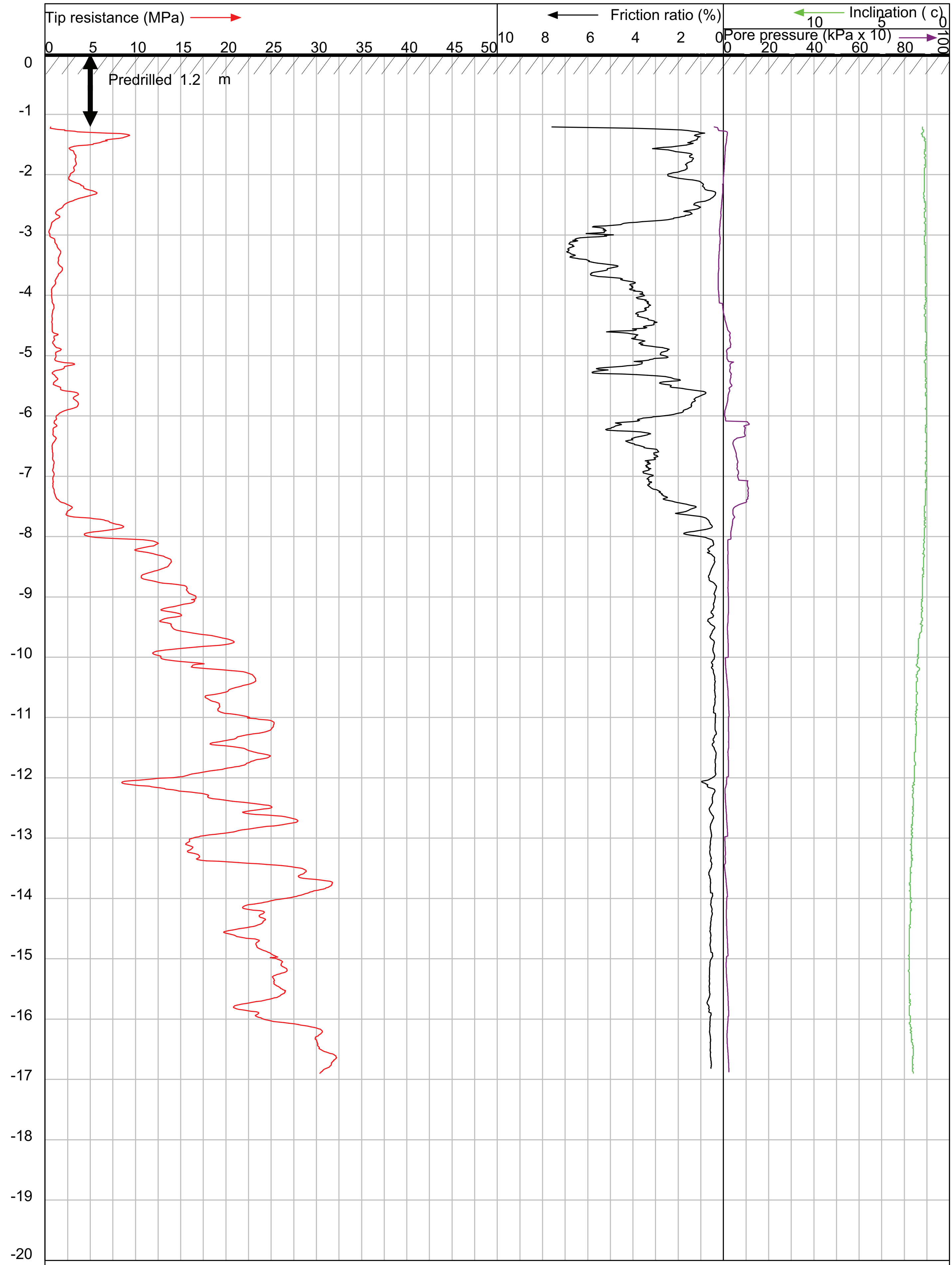
www.drilling.co.nz

# PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



| | | | |
|---|---|---|---|
| Job No: | 10915 | Date: | 10-8-2012 |
| CPT No: | CPTu001 | Operator: | S.Cardona |
| Project: | OPUS International Consultants Ltd | Remark: | Effective Refusal |
| Location: | Tramway Lane, Christchurch | | |

McMILLAN
DRILLING SERVICES

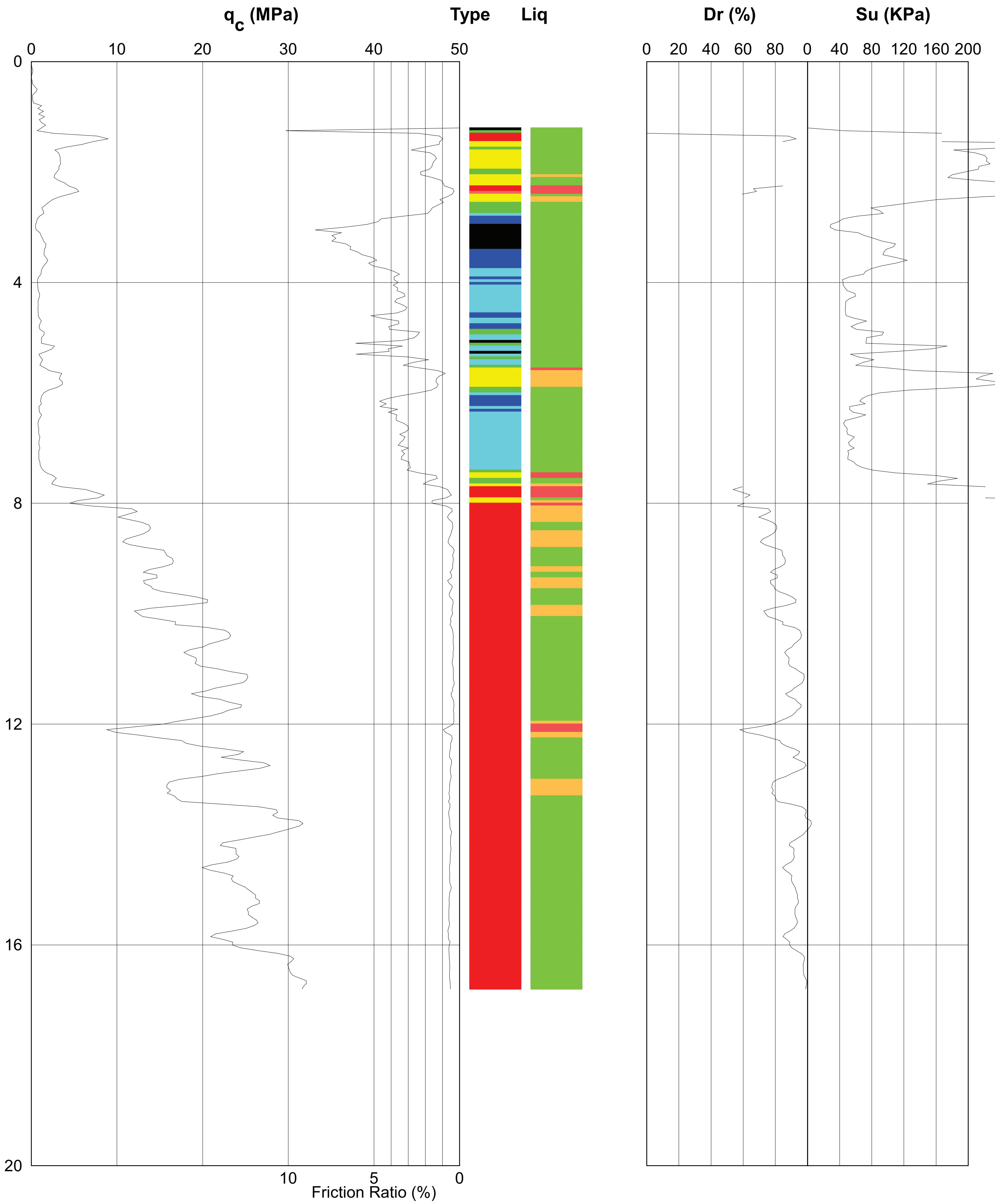


CLIENT : OPUS International Consultants Ltd
LOCATION : Tramway Lane, Christchurch
DATE : 10-8-2012
OPERATOR : H. Pardoe
REMARK 1 : CPTu002
REMARK 2 : Effective Refusal

JOB # : 10915
TEST # : 2
CONE TYPE/SERIAL #: I-CFXYP20-10/ 111007

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

# PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



| | | | |
|---|---|---|---|
| Job No: | 10915 | Date: | 10-8-2012 |
| CPT No: | CPTu002 | Operator: | S.Cardona |
| Project: | OPUS International Consultants Ltd | Remark: | Effective Refusal |
| Location: | Tramway Lane, Christchurch | | |

McMILLAN DRILLING SERVICES

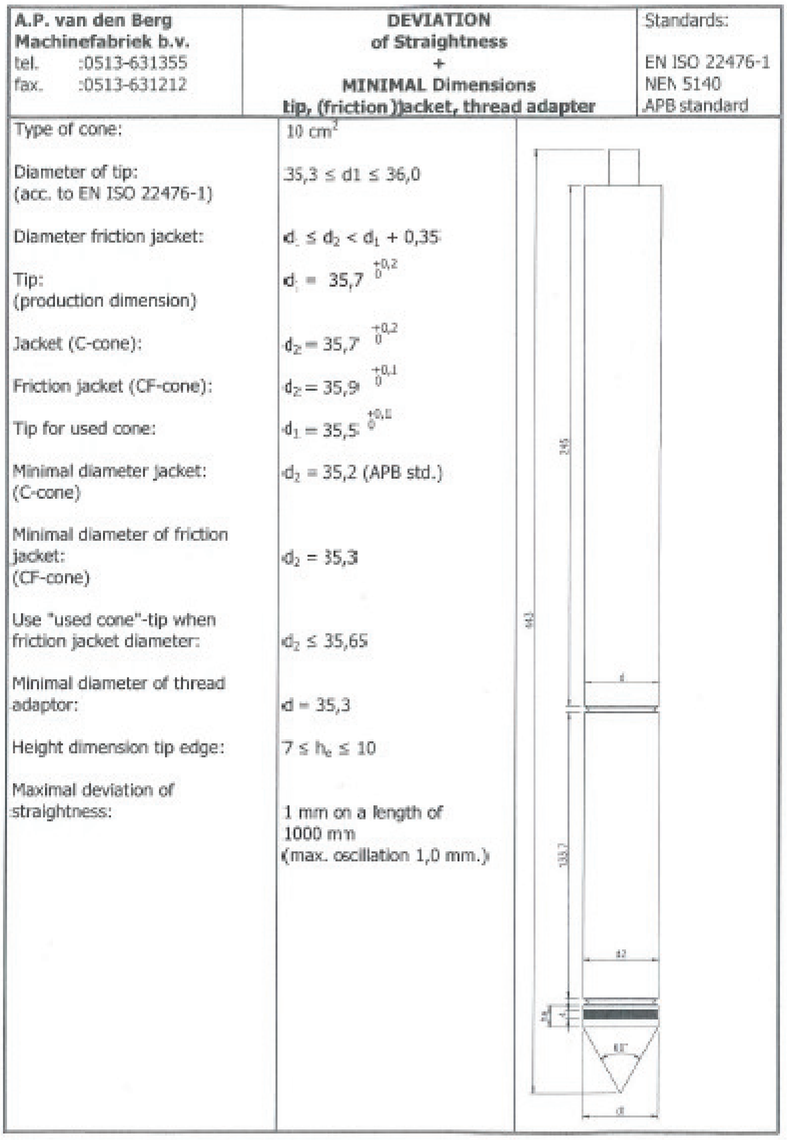
# CPT CALIBRATION AND TECHNICAL NOTES

These notes describe the technical specifications and associated calibration references pertaining to the following cone types:

- ELCI-10CFXY measuring cone resistance, sleeve friction and inclination (standard cone);
- ELCI-CFXYP20-10 measuring cone resistance, sleeve friction, inclination and pore pressure (piezo cone).

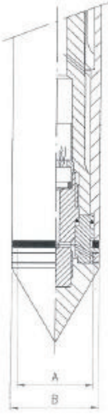
## Dimensions

Dimensional specifications for both cone types are detailed below. All tolerances are routinely checked prior to testing and measurements taken are manually recorded on CPT field sheets. All field sheets are kept on file and available on request.



| **A.P. van den Berg**<br>**Machinefabriek b.v.**<br>tel. :0513-631355<br>fax. :0513-631212 | **DEVIATION**<br>**of Straightness**<br>**+**<br>**MINIMAL Dimensions**<br>**tip, (friction )jacket, thread adapter** | Standards:<br>EN ISO 22476-1<br>NEN 5140<br>APB standard |
|---|---|---|
| Type of cone: | 10 $cm^2$ | |
| Diameter of tip:<br>(acc. to EN ISO 22476-1) | $35{,}3 \le d1 \le 36{,}0$ | |
| Diameter friction jacket: | $d_1 \le d_2 < d_1 + 0{,}35$ | |
| Tip:<br>(production dimension) | $d_1 = 35{,}7\ ^{+0,2}_{0}$ | |
| Jacket (C-cone): | $d_2 = 35{,}7\ ^{+0,2}_{0}$ | |
| Friction jacket (CF-cone): | $d_2 = 35{,}9\ ^{+0,1}_{0}$ | |
| Tip for used cone: | $d_1 = 35{,}5\ ^{+0,1}_{0}$ | |
| Minimal diameter jacket:<br>(C-cone) | $d_2 = 35{,}2$ (APB std.) | |
| Minimal diameter of friction jacket:<br>(CF-cone) | $d_2 = 35{,}3$ | |
| Use "used cone"-tip when friction jacket diameter: | $d_2 \le 35{,}65$ | |
| Minimal diameter of thread adaptor: | $d = 35{,}3$ | |
| Height dimension tip edge: | $7 \le h_e \le 10$ | |
| Maximal deviation of straightness: | 1 mm on a length of 1000 mm<br>(max. oscillation 1,0 mm.) | |

### Cone surface ratio



$A=0{,}25 \times 3{,}14 \times 30{,}9 \times 30{,}9=750\ MM^2$

$B=0{,}25 \times 3{,}14 \times 35{,}7 \times 35{,}7=1000\ MM^2$

$\alpha = A/B \qquad \beta = 1 - A/B$

$\alpha = 750/1000 = 0{,}75$

$\beta = 1 - 0{,}75 = 0{,}25$

## CPT CALIBRATION AND TECHNICAL NOTES (cont.)

### Calibration

Each cone has a unique identification number that is electronically recorded and reported for each CPT test. The identification number enables the operator to compare ‘zero-load offsets’ to manufacturer calibrated zero-load offsets.

The recommended maximum zero-load offset for each sensor is determined as ± 10% of the maximum measuring range although the more conservative trigger point adopted by McMillan Drilling Services is ± 10% of the nominal range.

In addition to maximum zero-load offsets, McMillan Drilling Services also limits the difference in zero load offset before and after the test as ± 1% of the maximum measuring range. See table below:

| | Tip (MPa) | Friction (MPa) | Pore Pressure (MPa) |
|---|---|---|---|
| **Maximum Measuring Range:** | 150 | 1.50 | 3.00 |
| **Nominal Measuring Range:** | 100 | 1.00 | 2.00 |
| **Max. ‘zero-load offset’:** | 10 | 0.10 | 0.20 |
| **Max ‘before and after test’:** | 1.5 | 0.015 | 0.03 |

**Note:** The zero offsets are electronically recorded and reported for each test in the same units as that of each sensor.

# TEST CERTIFICATE

## Icone (all versions)

| | |
|---|---|
| **Supplier:** | A.P. v.d. Berg Machinefabriek, Heerenveen The Netherlands |
| **Production-order:** | 54627 |
| **Client:** | Mc Millan |
| **Cone-type:** | ELCI - CFXY P20-10 |
| **Cone-number:** | 111007 |

| **To test / To check item** | **Required value** | **Checked value** |
|---|---|---|
| Isolation-resistance | >0.5 G-Ohm | 1 Gohm |
| Straightness | S=<0,2 mm | ✓ mm |
| Zero-Value Tip | [✓ Good] | -4,93 MPa |
| Zero-Value Local Friction | [✓ Good] | -0,069 MPa |
| Zero-Value Pore Pressure | [✓ Good] | -209 kPa |
| Zero-Value Inclination X<br>Zero-Value Inclination Y | [✓ -2°< X <+2°<br>-2° < Y <+2°] | -0,0 °<br>0,1 ° |
| Measurements Tip resistance OK? | [✓ Yes] | 0-50 MPa |
| Influence of Tip on Local Friction? (Tip: 100 kN; Mantle free?) | [✓ No influence] | |
| Measurements Local Friction OK? | [✓ Yes] | 0-0.75 MPa |
| Measurements Pore Pressure OK? | [✓ Yes] | 0-2000 kPa |
| Measurements Inclination OK? | [✓ Yes] | ± 1,8° |
| Cone recognition on disconnecting and connecting Icone again? | [✓ Yes] | |
| Software version 1.7 installed? Check at opening screen | [✓ Yes] | |
| Thresholds for rapid exit set to maximum | [✓ Yes] | |

Remarks:

| | | | | | |
|---|---|---|---|---|---|
| Calibrated by: | J.E. Ten Hage | Date: | 12.10.'11 | Sign.: | |
| Final check: | C.J. Ouwejan | Date: | 12.10.'11 | Sign.: | |

# Appendix E:

## Test Pit (TP) Logs

Christchurch Office

PO Box 1482
Christchurch, NZ

Tel: +64 3 363 5400
Fax: +64 3 365 7858
www.opus.co.nz

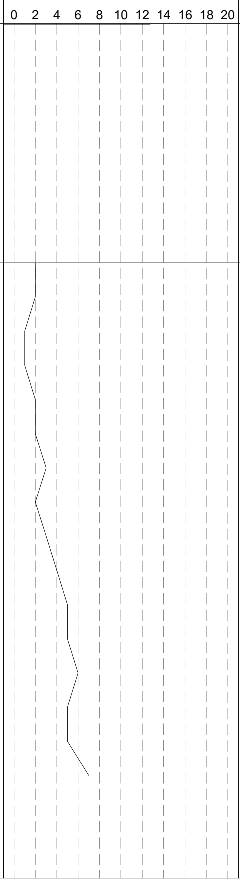
# LOG OF TRIAL PIT

| HOLE NO. | TP1 |
|---|---|
| PROJECT | Tram Barn Building |
| CO-ORD. | 2480818 E 5741718 N |
| R.L. | Approx. 5.5 m |
| SHEET | 1 of 1 |
| LOCATION | 7 Tramway Lane |
| REF. GRID | NZMG |
| DATUM | MSL |
| TOTAL DEPTH | 0.7 m |

DRAFT

| GEOLOGY/UNIT | DESCRIPTION | R.L. (m) | DEPTH (m) | GRAPHIC LOG | MOISTURE CONDITION | SCALA PENETROMETER Blows per 100 mm | SHEAR STRENGTH kPa | OTHER TESTS | SAMPLES |
|---|---|---|---|---|---|---|---|---|---|
| | Asphalt | | | | | | | | |
| | Sandy Fine to coarse GRAVEL with minor silt; brownish grey. "Loose"; well graded. Gravel: max size = 45mm.<br>Rounded to sub-rounded.<br>Sand: medium to coarse grained.<br>Fill Material. | | | | | | | | |
| | Target Depth Reached | 4 | 1<br>2 | | | | | | |

SOIL TESTS



## SKETCH OF EXPOSURE

## NOTES

No oserved damage to the foundations.

| | |
|---|---|
| LOGGED | B.Barnett |
| DATE EXCAVATED | 27/07/2012 |
| OPERATOR | N/A |
| EXCAVATOR | 6 Tonne |
| CLIENT | Chrstchurch City Council |
| JOB NO. | 6-QUCCC.64 |
| | TP1 |

Guideline for the field classification of soil and rock for engineering purposes: NZ Geotechnical Society (2005)
Determination of penetration resistance of a soil, NZS 4402 : 1988, Test 6.5.2
Shear strength using a hand held shear vane: NZ Geotechnical Society (8/2001)

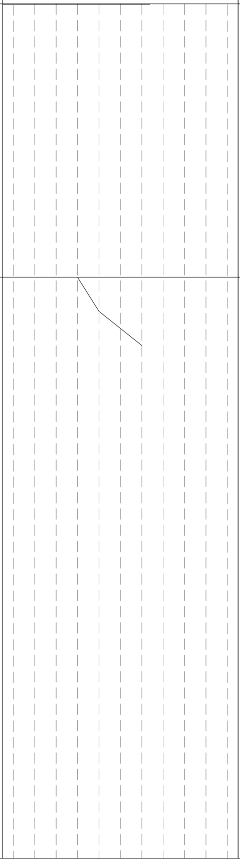
Christchurch Office
PO Box 1482
Christchurch, NZ
Tel: +64 3 363 5400
Fax: +64 3 365 7858
www.opus.co.nz

# LOG OF TRIAL PIT

| | | | | |
|---|---|---|---|---|
| HOLE NO. | **TP2** | | | |
| PROJECT | Tram Barn Building | CO-ORD. 2480825 E 5741725 N | R.L. Approx. 5.5 m | SHEET 1 of 1 |
| LOCATION | 7 Tramway Lane | REF. GRID NZMG | DATUM MSL | TOTAL DEPTH 0.8 m |

DRAFT

| GEOLOGY/UNIT | DESCRIPTION | R.L. (m) | DEPTH (m) | GRAPHIC LOG | MOISTURE CONDITION | SOIL TESTS: SCALA PENETROMETER Blows per 100 mm (0 2 4 6 8 10 12 14 16 18 20) | SOIL TESTS: SHEAR STRENGTH kPa | SOIL TESTS: OTHER TESTS | SAMPLES |
|---|---|---|---|---|---|---|---|---|---|
| | Asphalt | | | | | | | | |
| | Sandy fine to coarse GRAVEL with minor silt; brownish grey. "Loose"; well graded. Gravel: max size = 45mm.<br>Rounded to sub-rounded.<br>Sand: medium to coarse grained.<br>Fill Material. | | | | | | | | |
| | Target Depth Reached | 4 | 1<br>2 | | | | | | |



## SKETCH OF EXPOSURE

## NOTES

No oserved damage to the foundations.

| | |
|---|---|
| LOGGED: B.Barnett | DATE EXCAVATED: 27/07/2012 |
| OPERATOR: N/A | EXCAVATOR: 6 Tonne |
| CLIENT: Chrstchurch City Council | JOB NO.: 6-QUCCC.64 TP2 |

Guideline for the field classification of soil and rock for engineering purposes: NZ Geotechnical Society (2005)
Determination of penetration resistance of a soil, NZS 4402 : 1988, Test 6.5.2
Shear strength using a hand held shear vane: NZ Geotechnical Society (8/2001)

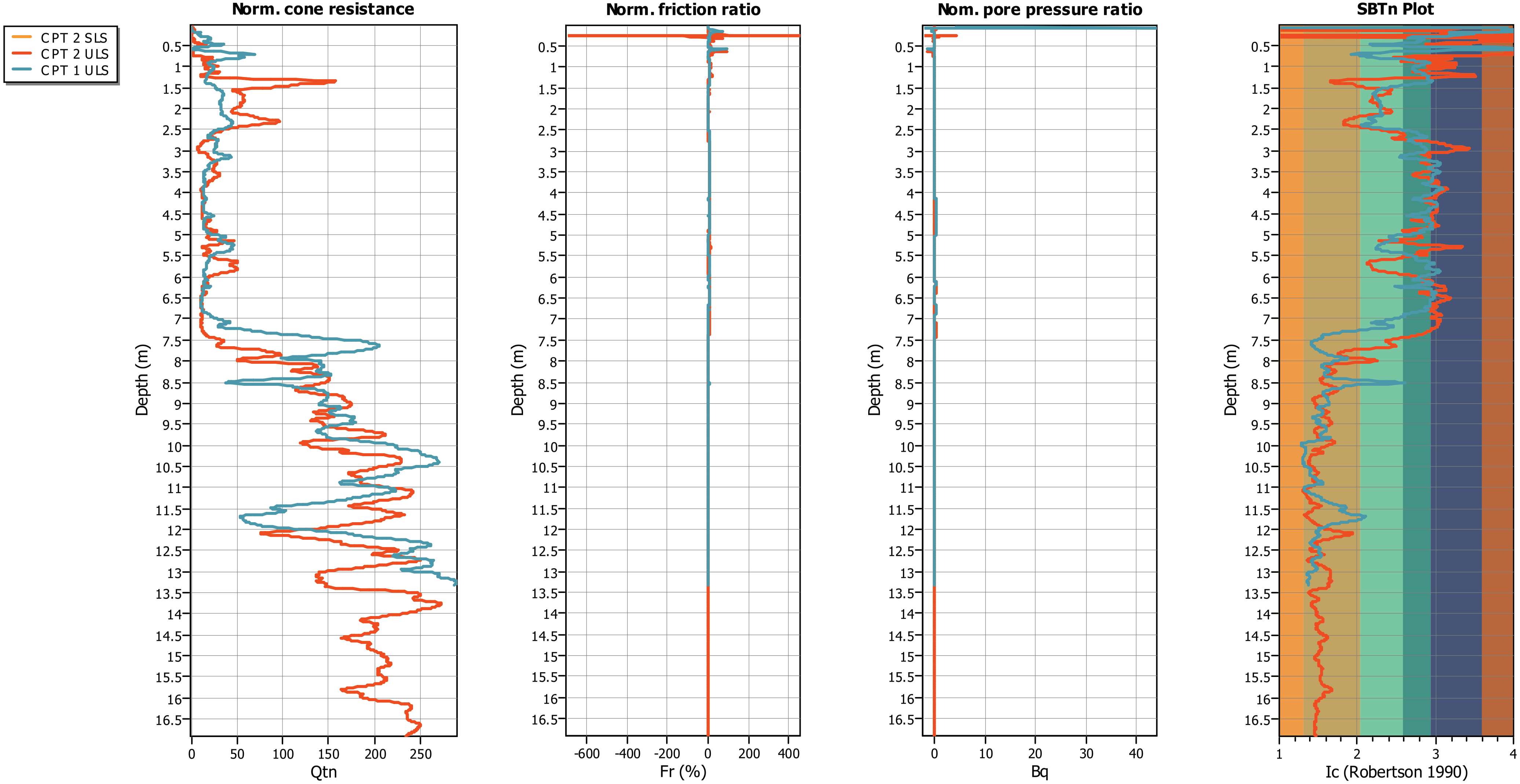
# Appendix F:

## CLiq (v1.7) Liquefaction Analysis

**Opus International Consultants Ltd**
20 Moorhouse Ave
PO Box 1482
Christchurch, New Zealand

**Project: Tram Barn Building**

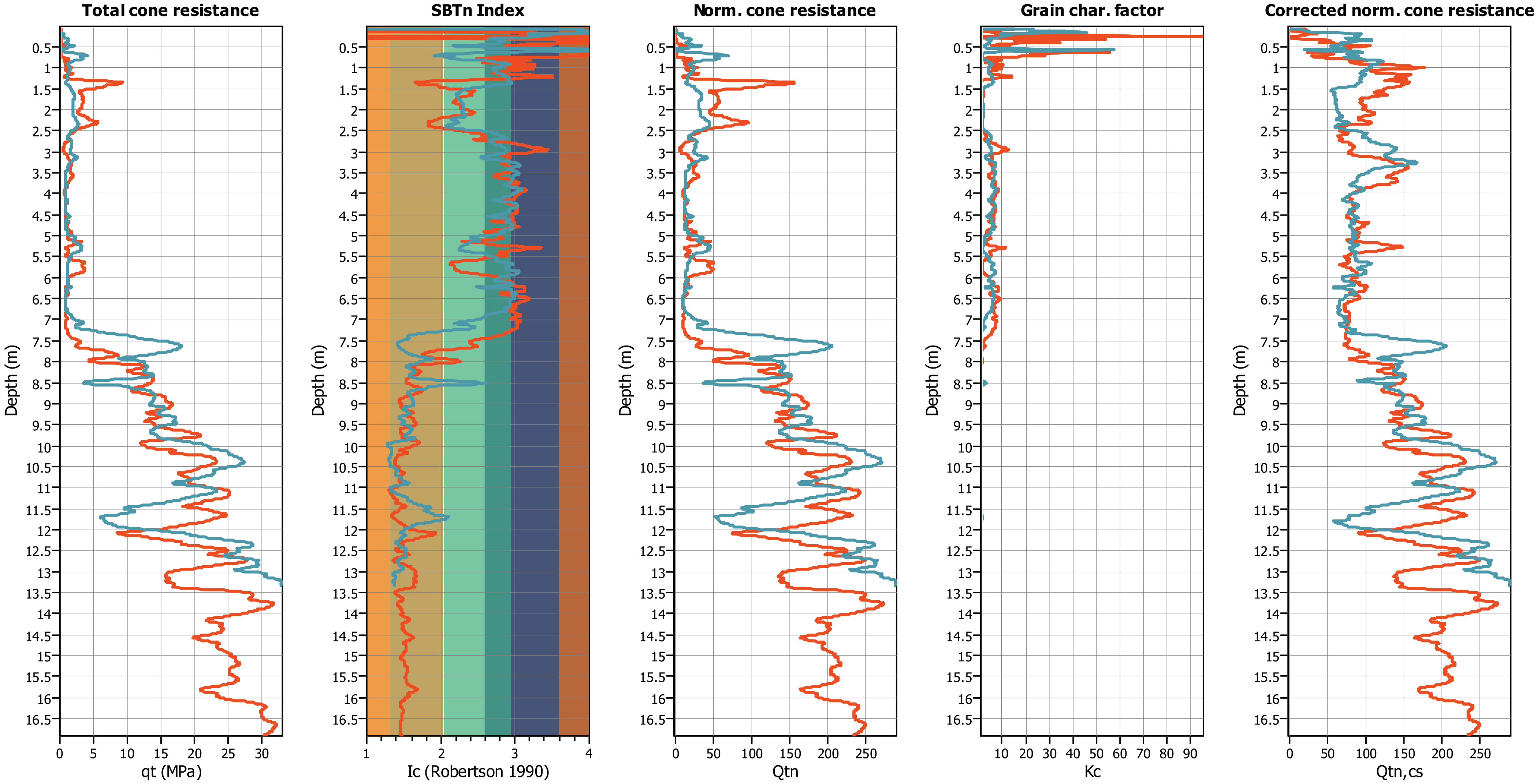
## Overlay Normalized Plots



**Opus International Consultants Ltd**
20 Moorhouse Ave
PO Box 1482
Christchurch, New Zealand

**Project: Tram Barn Building**

## Overlay Intermediate Results


Total cone resistance

SBTn Index

Norm. cone resistance

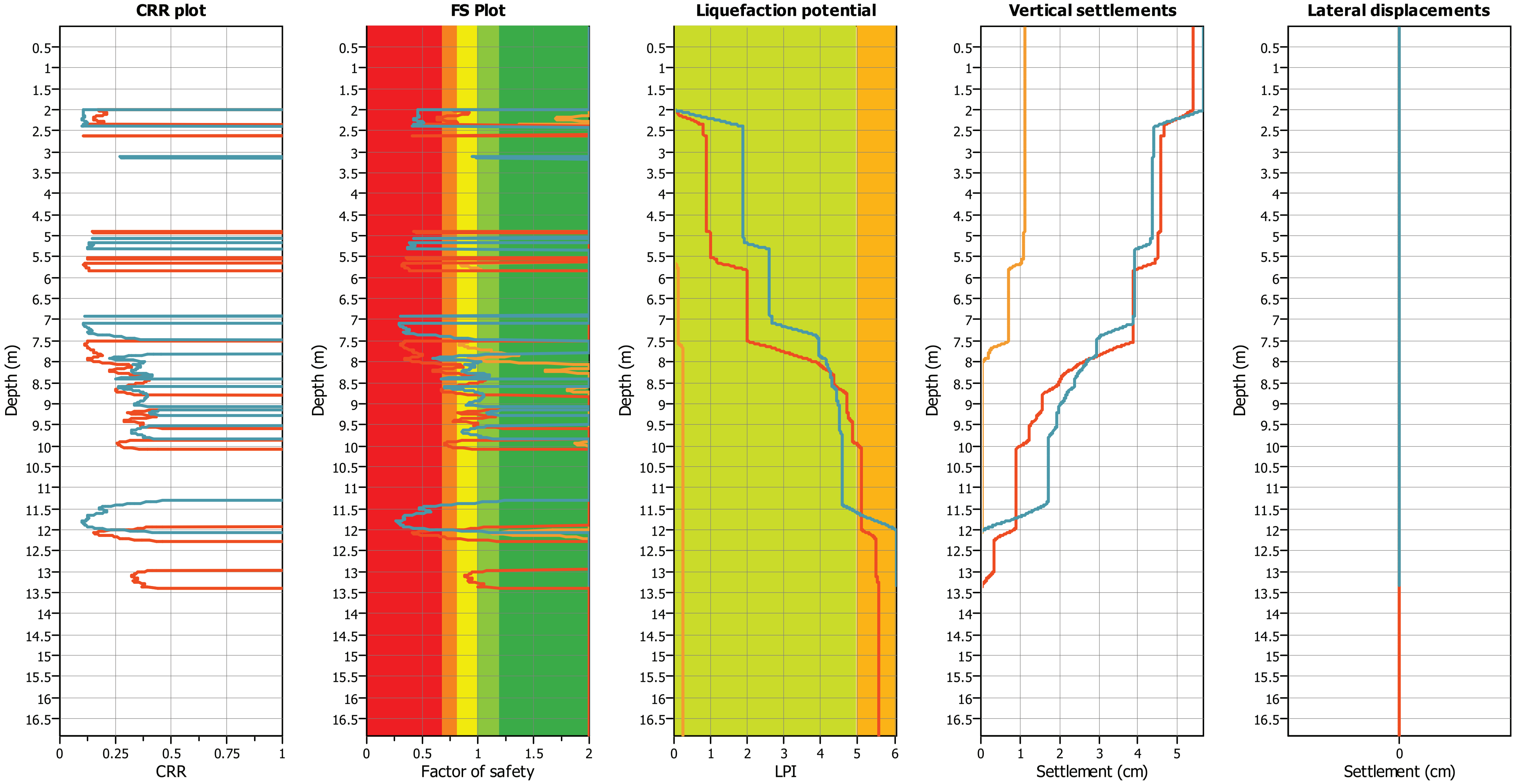
Grain char. factor

Corrected norm. cone resistance

**Opus International Consultants Ltd**
20 Moorhouse Ave
PO Box 1482
Christchurch, New Zealand

**Project: Tram Barn Building**

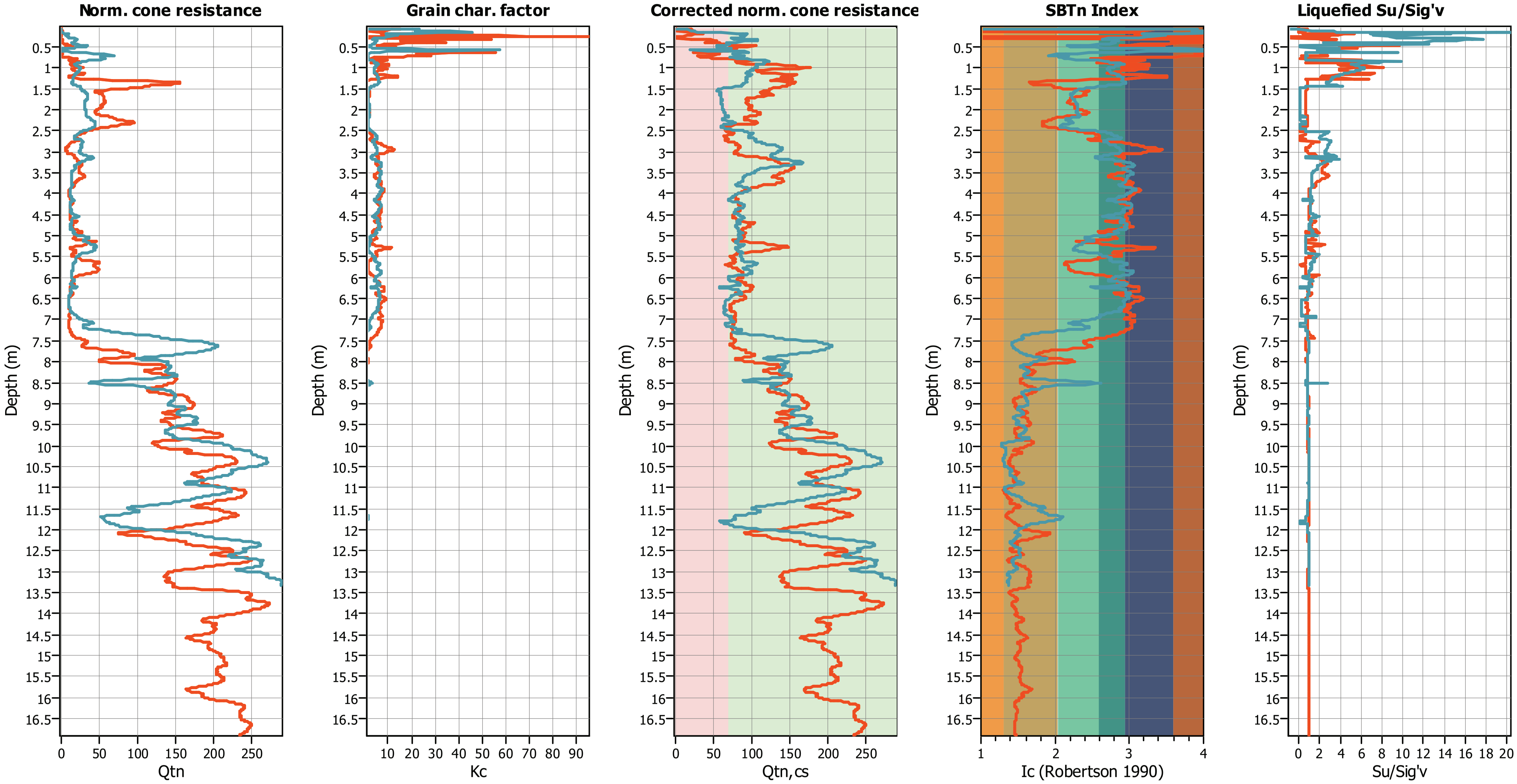
# Overlay Cyclic Liquefaction Plots



**Opus International Consultants Ltd**
20 Moorhouse Ave
PO Box 1482
Christchurch, New Zealand

**Project: Tram Barn Building**

# Overlay Strength Loss Plots



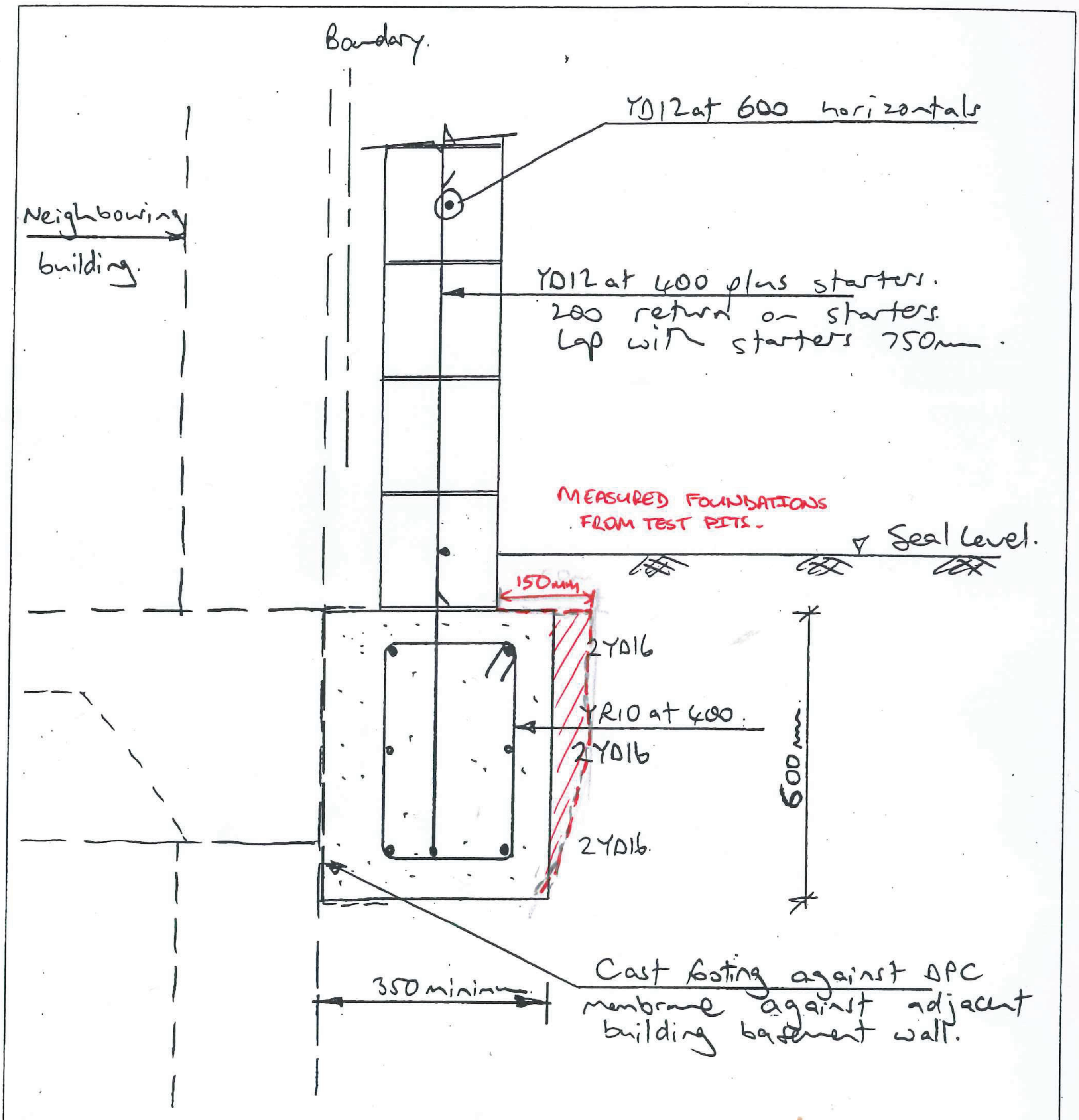
# Appendix G:

Structural Drawing Extract

HolmesConsultingGroup

Project Name: Tramcar Garage Extensions.
Project No: 25871.01
Calcs By: CBLewis
Date: 15/7/99.
Sketch No: 2.
Page No: —
Revision: —

~~CALCS~~/SKETCHES



Note: Remove any soft material under foundation and replace with thoroughly compacted hardfill. Refer also specification.

Blockwall Foundation Detail. (Scale 1:10)

**Opus International Consultants Ltd**
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PO Box 1482, Christchurch Mail Centre,
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w: www.opus.co.nz