Christchurch City Council

English Park Lighting Towers
BU 0623-008 EQ2
BU 0623-010 EQ2

Detailed Engineering Evaluation

Quantitative Assessment Report

Christchurch City Council

English Park Lighting Towers

Quantitative Assessment Report

127 Cranford Street, St. Albans

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Status:	Final

Summary

English Park Lighting Towers
BU 0623-008 EQ2
BU 0623-010 EQ2

Detailed Engineering Evaluation
Quantitative Report - Summary
Final

Background

This is a summary of the quantitative report for the lighting tower structures at 127 Cranford Street, and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on 10 October 2012, available drawings and a Structural Integrity Report for the existing Lighting Towers dated May 2010.

Key Damage Observed

Key damage observed:

- Minor cracking to the grouting beneath the base plate;
- Surface corrosion to the access hatch at the base of the towers, typical for all four towers.

Critical Structural Weaknesses

No critical structural weaknesses have been identified for the English Park Lighting Towers.

Indicative Building Strength

Based on the information available, and from undertaking a quantitative assessment, the lighting towers' original capacity has been assessed to be in excess of 100% NBS as an Importance Level 2 (IL2) structure.

Recommendations

a. The CCC considers whether further geotechnical analysis is required to investigate the site performance and quantify the risk of liquefaction induced deformations.

Contents

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council to undertake a detailed seismic assessment of the Lighting Towers at English Park, located at 127 Cranford Street, Christchurch following the M6.3 Christchurch earthquake on 22 February 2011.

The purpose of the assessment is to determine if the Lighting Tower structures are 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].

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 19 July 2011. This document sets out a methodology for both initial qualitative and detailed quantitative assessments.

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

a. The importance level and occupancy of the building.

b. The placard status and amount of damage.

c. The age and structural type of the building.

d. 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:

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

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

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

d. 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	Improvement of Structural Performance	
					Legal Requirement	NZSEE Recommendation
Low Risk Building	A or B	Low	Above 67	Acceptable (improvement may be desirable)	The Building Act sets no required level of structural improvement (unless change in use) This is for each TA to decide. Improvement is not limited to 34%NBS.	100%NBS desirable. Improvement should achieve at least 67%NBS
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally. Improvement recommended		Not recommended. Acceptable only in exceptional circumstances
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement required under Act)	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 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 at least 67%NBS. A strengthening solution to anything less than 67%NBS would not provide an adequate reduction to the level of risk.

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

3.1.4 Our Ethical Obligation

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

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

4 Background Information

4.1 Lighting Towers Description

The English Park Lighting Towers are located in each of the four corners of English Park, located at 127 Cranford Street, Christchurch. The existing towers were retrofitted onto new 900mm diameter precast pile foundations. The piles have a minimum length of 5m, and based on the construction drawings are inferred to be founded 0.5m below the top of a gravel layer.

The towers are approximately 20 metres high, and are constructed with steel welded hexagonal sections tapering in size from the bottom to top. The distance between the flats across the hexagons vary from 375mm at the base to approximately 175mm at the top.

The tower foundations consist of a 760 x 760 x 75mm steel base plate fixed to the pile cap with grouted M36 threaded rods. There is a steel flat bar assembly welded to the base of the threaded rods providing fixity to the driven concrete pile.

4.2 Survey

4.2.1 Post 22 February 2011 Rapid Assessment

A structural (Level 2) assessment of the English Park Pavilion and Lighting Towers was undertaken on 16 March 2011 by Opus International Consultants.

4.2.2 Further Inspections

Further inspections were undertaken by Opus International Consultants on 10 October 2012.

The above investigations included visual inspection of all structural elements above foundation level. Physical access of the towers was limited to the lowermost 2m, and all measurements above this were based on visual observation.

4.3 Original Documentation

Copies of the following construction drawings were provided by CCC:

- English Park Redevelopment – Lighting Tower Foundation, structural drawings (City Solutions) dated August 2001 and approved for construction.

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

No copies of the calculations have been obtained as part of the documentation set.

5 Structural Damage

The following damage has been noted:

5.1 Foundations

Minor cracking and damage was observed to the grouted layer beneath the tower base plate. This was typical for all four towers.

We note that elongation of the M36 threaded rods was observed with approximately 5mm of separation between the threaded rod nut and steel base plate following the M6.3 Christchurch earthquake on 22 February 2011. The threaded rod nuts have since been retightened by City Care. Refer to Appendix 1 for photographs of the threaded rods before and after retightening.

5.2 Steel Towers

Surface corrosion was observed around the access hatches at the bases of the towers.

6 General Observations

The Lighting Tower structures behaved well and as expected given the date and type of construction. The visual damage observed during our inspections was very minor. No distress was observed at the base of the tower, or around the base plate and bolt fixings.

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 16 May 2012, and the SESOC guidelines "Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes" [5] issued on 18 September 2012.

7.1 Quantitative Assessment Methodology

The assessment assumptions and methodology have been included in Appendix 3 of the report due to the technical nature of the content. A brief summary follows:

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

A modal response spectral analysis and equivalent static analysis were carried out using the spectral values established from NZS 1170.5, with an updated Z factor of 0.3 (B1/VM4). These analyses were used to establish the seismic actions on the towers. Based on the actions determined from the analyses, an assessment of the Lighting Towers capacities was made.

Axial-moment and moment curvature analyses were carried out on the reinforced concrete foundations using the spCOLUMN analysis programme.

The hexagonal hollow tower section shear and flexural capacities were assessed using NZS 3404 [7].

7.2 Limitations and Assumptions in Results

The observed level of damage suffered by the Lighting Towers was deemed low enough to not affect their capacity. Therefore the analysis and assessment was based on them being in an undamaged state. There may have been damage that was unable to be observed that could cause the capacities to be reduced; therefore the current capacities of the Lighting Towers 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.

7.3 Quantitative Assessment

A summary of the structural performance of the building is shown in the following table. Note that the values given represent the worst performing elements in the Lighting Towers, as these effectively define the structure's capacity.

Table 2: Summary of Seismic Performance

Structural Element/System	Failure Mode or description of limiting criteria based on displacement capacity of critical element.	% NBS based on calculated capacity
Foundation Pile	Flexure governed failure mode, resulting in buckling failure of the moment resisting foundation column.	>100%
Base Connection – Threaded rod bolted fixings	Flexural failure of the bolted base plate connection, resulting in yielding and elongation of the steel threaded rods.	>100%
Hexagonal Steel Section	Flexure governed failure mode, resulting in compression buckling failure of the hexagonal hollow section.	>100%
Foundations	Axial compression failure governed by the capacity of the reinforced concrete foundation pile.	>100%

8 Summary of Geotechnical Appraisal

A summary of the geotechnical assessment report is attached as Appendix 2. A summary of this report is as follows:

a. There is both liquefaction induced subsidence and lateral spreading hazards at this site due to the geological conditions and the close proximity to St. Albans stream.

b. Based on the relatively good performance of the English Park in the recent seismic events, the towers are unlikely to collapse but may tilt due to ground deformation. Similar performance would be expected in a future ULS earthquake.

c. Further site investigations and corresponding assessment would be required to confirm the future performance of the lighting towers and quantify the risk of liquefaction induced deformations.

9 Conclusions

a. The seismic performance of the lighting towers is governed by the capacity of the steel hexagonal section, which has an expected strength of >100% NBS at IL2. This would classify the lighting towers as not being an earthquake risk.

b. Further site investigations and corresponding assessment would be required to quantify the risk of liquefaction induced deformations during a future ULS earthquake.

10 Recommendations

a. The CCC considers whether further geotechnical analysis is required to investigate the site performance and quantify the risk of liquefaction induced deformations.

11 Limitations

a. This report is based on an inspection of the Lighting Towers 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 6, 16 May 2011.

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

[5] SESOC, Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes, Structural Engineering Society of New Zealand, 18 September 2012.

[6] DBH, 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.

[7] NZS 3404: Part 1:1997, *Steel Structures Standard, The Design of Steel Structures,* Standards New Zealand.

Appendix 1 - Photographs

English Park Lighting Towers		
No.	Item description	Photo
General		
1.	North-west corner	
2.	Side elevation of lighting towers	
3.	Front elevation of lighting towers	

4.	Base plate with threaded bolts fixed to the existing concrete footings	
5.	Steel base plate area of 760 x 760mm	
6.	Hexagonal section with chord length of 220mm	

7.	Steel base plate thickness of 75mm	
8.	Threaded rods of diameter 36mm, and 850mm in length	
9.	Welded hexagonal steel section	

10.	Servicing/access hatch	
11.	Lighting tower cantilevering 20.1m above ground	
12.	Hexagonal section 375mm between the flats	

13.	Existing concrete footing	
14.	Existing concrete footing	
15.	Elongation of the 36mm threaded rods, observed following M6.3 Christchurch earthquake	

16.	Elongation of the 36mm threaded rods, observed following M6.3 Christchurch earthquake	

Appendix 2 - Geotechnical Appraisal

English Park Lighting Towers – Geotechnical Summary

Introduction

This brief memo outlines the soil profile and the ground performance of the soil underlying the four lighting towers at English Park, St Albans, Chirstchurch.

Structural Drawings

Structural drawings of the English Park Lighting Towers foundations have been made available from City Design dated August 2001. The drawings indicate that the towers are constructed on 900mm diameter reinforced cylindrical concrete piles that are founded at a minimum depth of 5.5m below ground level with an embedment of 0.5m into the inferred gravel layer.

Regional Geology

The site is mapped as Holocene aged Springston Formation alluvial sand and silt overbank deposits (1:25,000 Geological Map of the Christchurch Urban Area).

Expected Ground Conditions

A review of the Environmental Canterbury (ECan) wells database showed three wells located within approximately 300 m of the property. The locations of Boreholes and Cone Penetrometer Test's (CPT) undertaken by the Earthquake Commission (EQC) have been reviewed. Five CPT's and one Borehole have been identified approximately 100m south of the park. CPTs have also been made available from the English Park Stadium, along the eastern side of the Park.

Material logs available from the above sources have been used to infer the ground conditions at the site, as shown in Table 1 below.

Table 1: Inferred Ground Conditions

Stratigraphy	Thickness (m)	Depth Encountered (m)
Sandy SILT	2.0 – 5.0m	Surface
SAND	0.8-2.5m	2.0-5.0m
Gravelly SAND to Sandy GRAVEL	1.5-2.2m	4.5-5.8m
SAND	16.4-18.4m	6.0-8.0m
GRAVEL (Riccarton Formation)	-	24.4m

A groundwater depth of approximately 2.0m to 3.0m below ground level has been interpreted from groundwater surface depth maps (Canterbury Geotechnical Database (2012)).

Liquefaction Hazard

A liquefaction hazard study was conducted by the Canterbury Regional Council (ECan) in 2004 to identify areas of Christchurch susceptible to liquefaction during an earthquake. The English Park Lighting Towers are located in an area identified as having 'high liquefaction ground damage potential', for a low groundwater scenario. High ground damage potential indicates that ground subsidence is likely to be greater than 300mm in a future seismic event.

The Earthquake Commission's (EQC) geotechnical consultants have prepared maps showing areas of liquefaction interpreted from high resolution aerial photos for the September 2010 earthquake and the aftershocks of February 2011, June 2011 and December 2011. There has been evidence from these aerial photos of liquefaction ejecta on the site or in the vicinity after all the seismic events mentioned above.

English Park has been zoned as N/A-Urban Non-residential by the Department of Building and Housing (DBH). However, the neighbouring residential properties surrounding English Park have been zoned as Green-TC3 “blue zone”, which is determined to have a moderate to significant risk of land damage due to liquefaction in future significant earthquakes.

Lateral Spreading Hazard

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.

St Albans stream runs along the eastern side of English Park, approximately 20m east of the nearest lighting tower. Following the Canterbury earthquake sequence of 2010-2011, there has been evidence of tension cracking on the embankment fill on the south eastern corner of the English Park Stadium adjacent to the eastern most lighting tower.

Based on the inferred underlying ground conditions, there is a lateral spreading hazard at this site, particularly to the lighting tower closest to the waterway.

Discussion

As a result of the 4th September 2010 to December 2011 Canterbury Earthquakes; liquefaction ejecta, ground cracking and subsidence has occurred throughout English Park.

The relatively deep alluvial formations underlying these towers define this site as Class D – deep or soft site, in accordance with NZS 1170.5:2004.

No site specific deep investigation results have been available for review at the time of reporting.

Observations suggest that the towers are in a vertical state.

Surrounding CPTs have refused at depths ranging from 4.0 to 7.0m below ground level, indicating the presence of the shallow gravel layer. The ground conditions are relatively variable in the vicinity, and therefore the thickness or competency of the underlying bearing layer is unknown.

Conclusions and Recommendations

Note that this assessment is based on limited site investigation data made available.

There is both liquefaction induced subsidence and lateral spreading hazards at this site due to the ground subsidence and sand boils that were observed in the recent earthquakes.

Based on the relatively good performance of the English Park Lighting Towers in the recent seismic events, the towers are unlikely to collapse but may tilt due to ground deformation. Similar performance would be expected in a future ULS earthquake.

Further site investigations and corresponding assessment would be required to confirm the future performance of the lighting towers and quantify the risk of liquefaction induced deformations.

Appendix A:
Surrounding Site Investigations

Lighting Towers

Boreholes

CPT

Number	BH Refernce
7	M35/1899
8	M35/14863
3	STA-POD08-BH02

Number	CPT Reference
1	CPT-STA-49
2	CPT2 (Geotech Ltd)
3	STA-POD08-CPT03
4	STA-POD13-CPT08
5	STA-POD13-CPT11
6	CPT-HIS-0412

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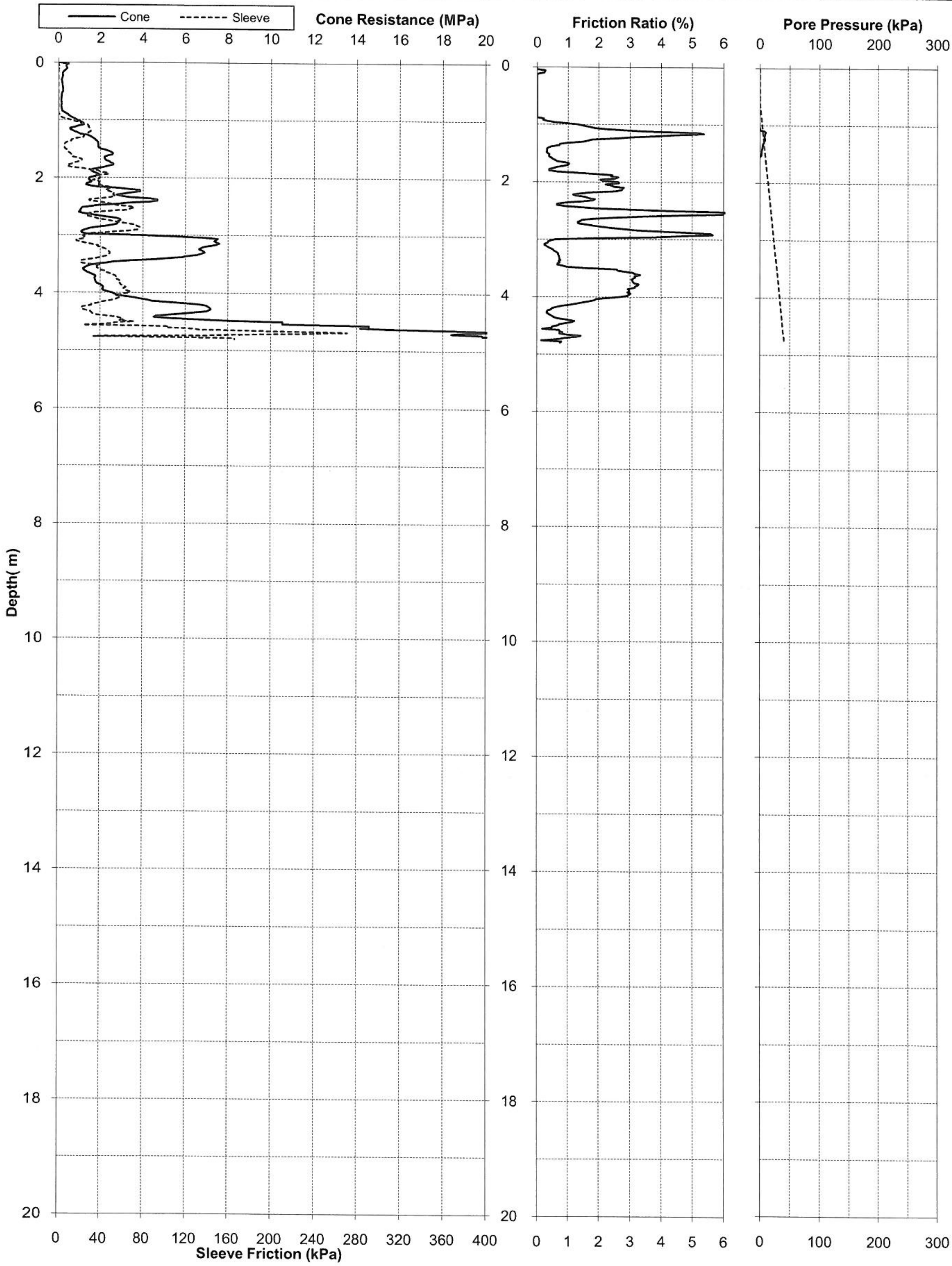
Project: English Park Lighting Towers
Project No.: 6-QUCCC.65
Client: Christchurch City Council

Site Location Plan

Drawn: Opus Geotechnical Engineer
Date: 20-Dec-12

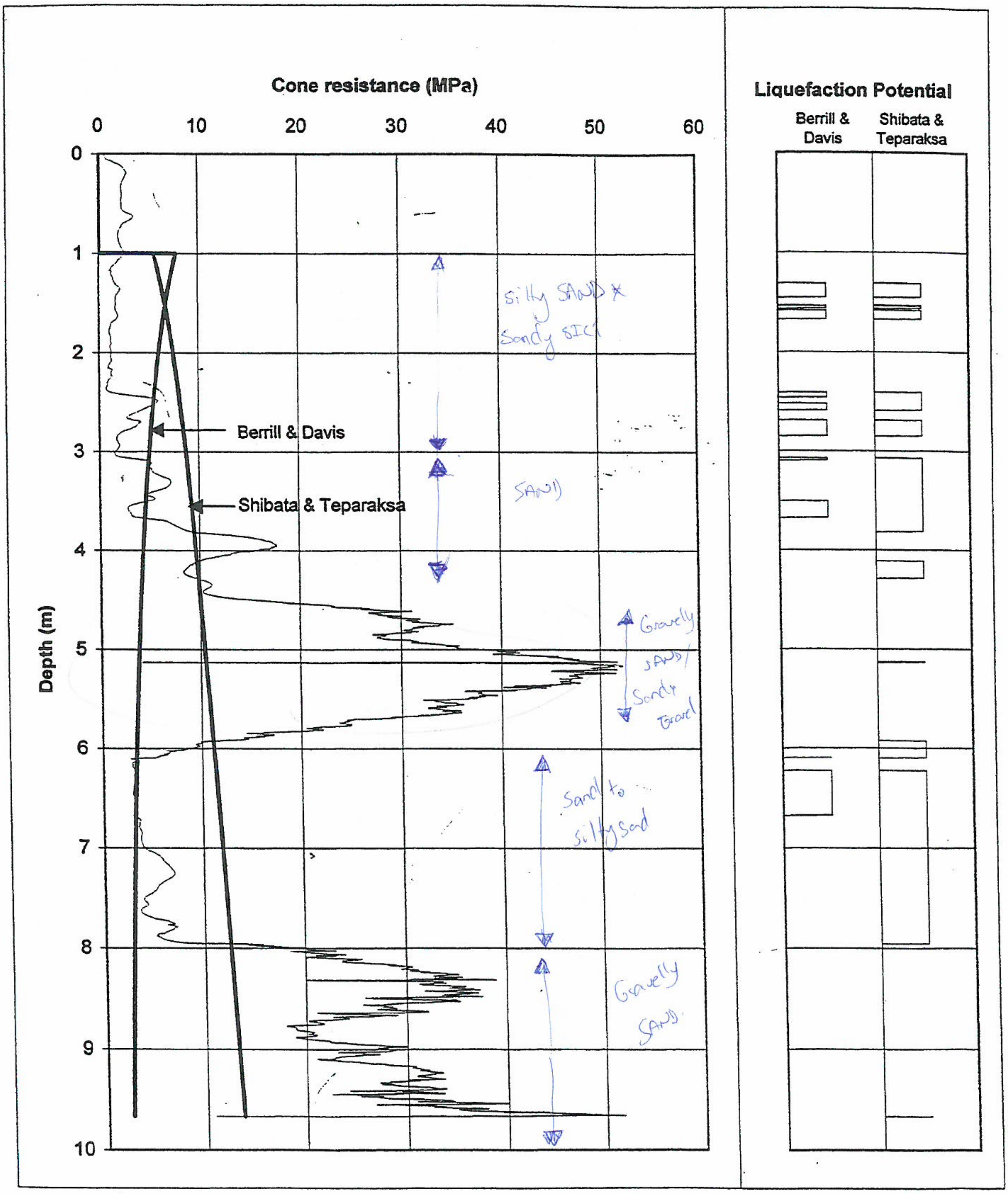
Project:	Christchurch 2011 Earthquake - EQC Ground Investigations			**Page:** 1 of 1	**CPT-STA-49**
Test Date:	24-May-2011	**Location:**	St Albans	**Operator:**	Geotech
Pre-Drill:	1.2m	**Assumed GWL:**	0.6mBGL	**Located By:**	Survey GPS
Position:	2480670.7mE	5744314.6mN	7.605mRL	**Coord. System:**	NZMG & MSL
Other Tests:				**Comments:**	



Liquefaction Potential Analysis

GEOTECH CONSULTING LTD

Project:	English Park Redevelopment	Hole No:	CPT2
Client:	City Design	Job No:	2091



Note:	Left hand graph shows critical values of Qc. Right hand graph shows liquefaction analyses taking into account soil types.

CPT LOG

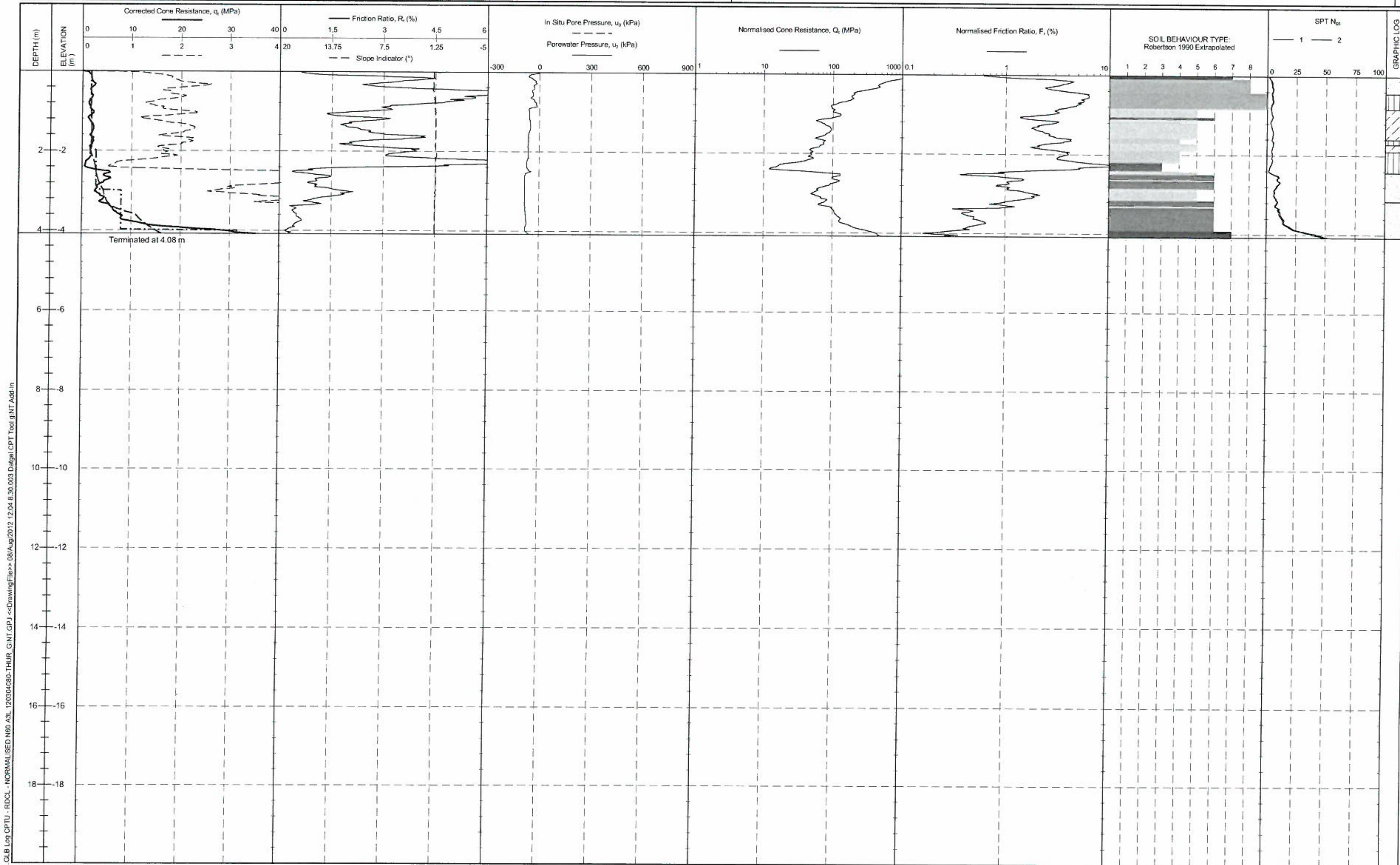
PointID **STA-POD13-CPT08**

CLIENT	: EQC	AREA	: 120304080
ENGINEER	: Reuben George	EASTING	: 2480698.0 m
PROJECT	: TC3 Investigation	NORTHING	: 5744144.0 m
LOCATION	: Christchurch	COORD. SYS.	:
PROJECT No.	: 120304080	ELEVATION	: 0.00 m

METHOD: Robertson 1990

- 1 - Sensitive, fine grained
- 2 - Organic soil - peats
- 3 - Clays - clay to silty clay
- 4 - Silt mixtures - clayey silt to silty clay
- 5 - Sand mixtures - silty sand to sandy silt
- 6 - Sands - clean sand to silty sand
- 7 - Gravelly sand to sand
- 8 - Very stiff sand to clayey sand
- 9 - Very stiff fine grained

SHEET : 1 OF 1
STATUS :
DATE : 07/08/2012



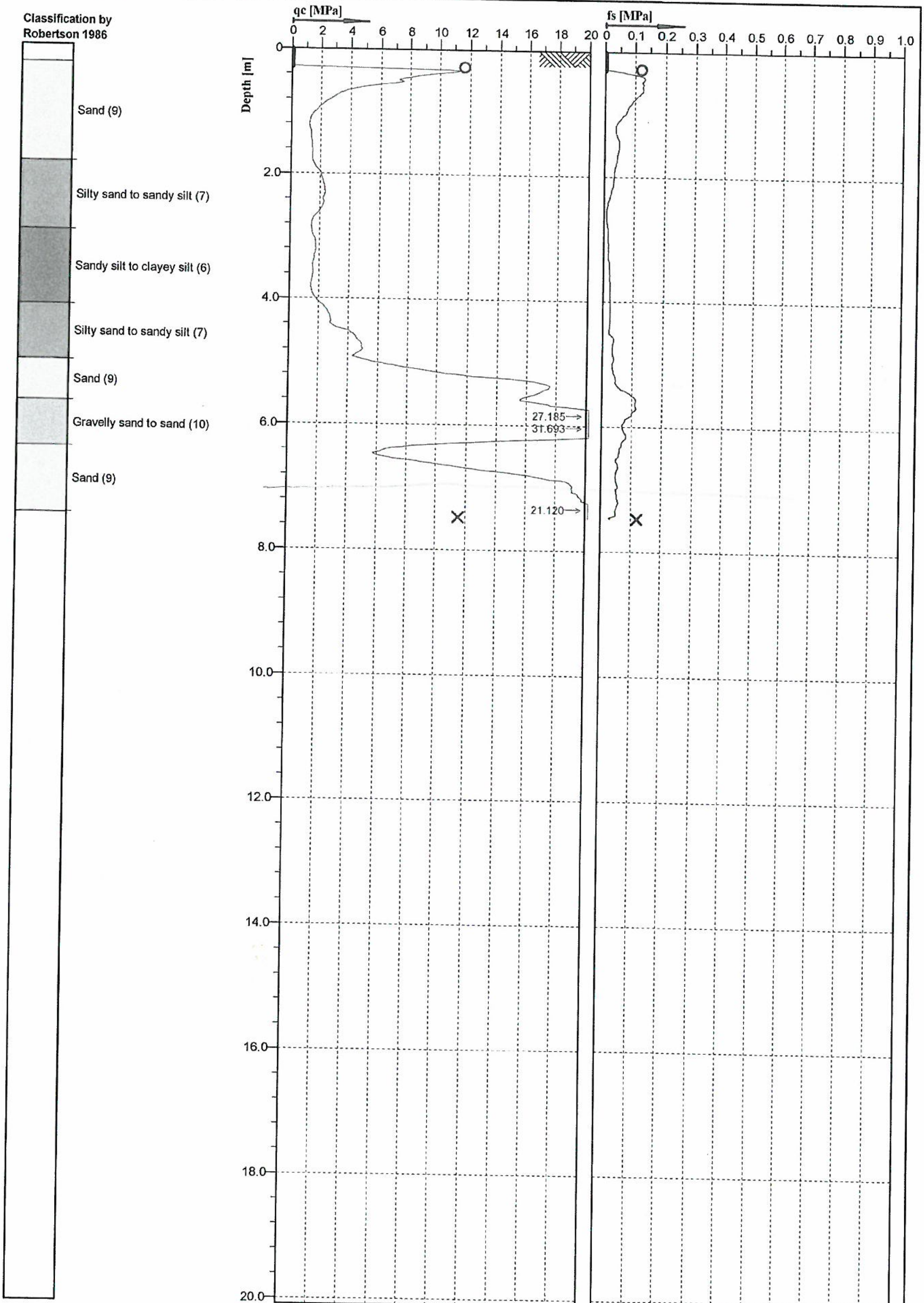
GRAPHIC LOG (depth, m)	MATERIAL DESCRIPTION
0.44	Very stiff sand to clayey sand
0.84	Very stiff fine grained
1.60	Sand mixtures - silty sand to sandy silt
1.72	Silt mixtures - clayey silt to silty clay
1.90	Sand mixtures - silty sand to sandy silt
2.44	Silt mixtures - clayey silt to silty clay; Clays - clay to silty clay
3.16	Sands - clean sand to silty sand; Sand mixtures - silty sand to sandy silt
4.08	Sands - clean sand to silty sand; Gravelly sand to sand

RIG	:	CHECKED BY	: RG
CONE TYPE	: C+F+W2+S	CHECKED DATE	: 08/08/2012
CONE ID	: 4045	APPROVED BY	:
OPERATOR	: TM	APPROVED DATE	:

— — q_t Moving Average Over 1 m
— · — q_t Stepped Average Over 1 m
- - - q_c Strata Average
Dissipation Test

REMARK

Next to BH.

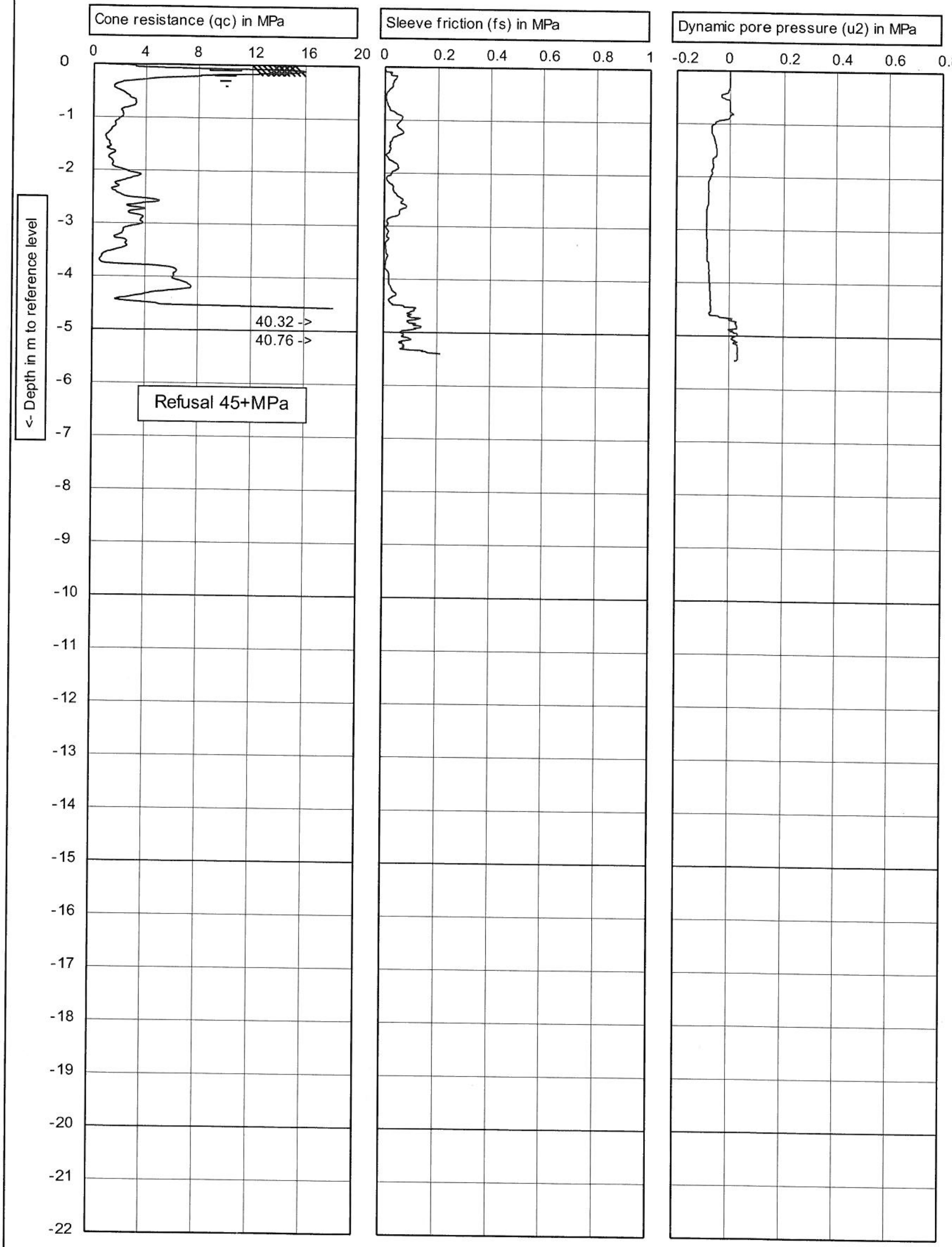


PRO-DRILL
SPECIALIST DRILLING ENGINEERS

U2

Cone No: 4486
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: STA-POD08-CPT03	Position: X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no: 1
Project ID:	Client: EQC	Date: 30/07/2012	Scale: 1 : 82
Project: 93DOVERST		Page: 1/1	Fig:
		File: STA-POD08-CPT03.010	



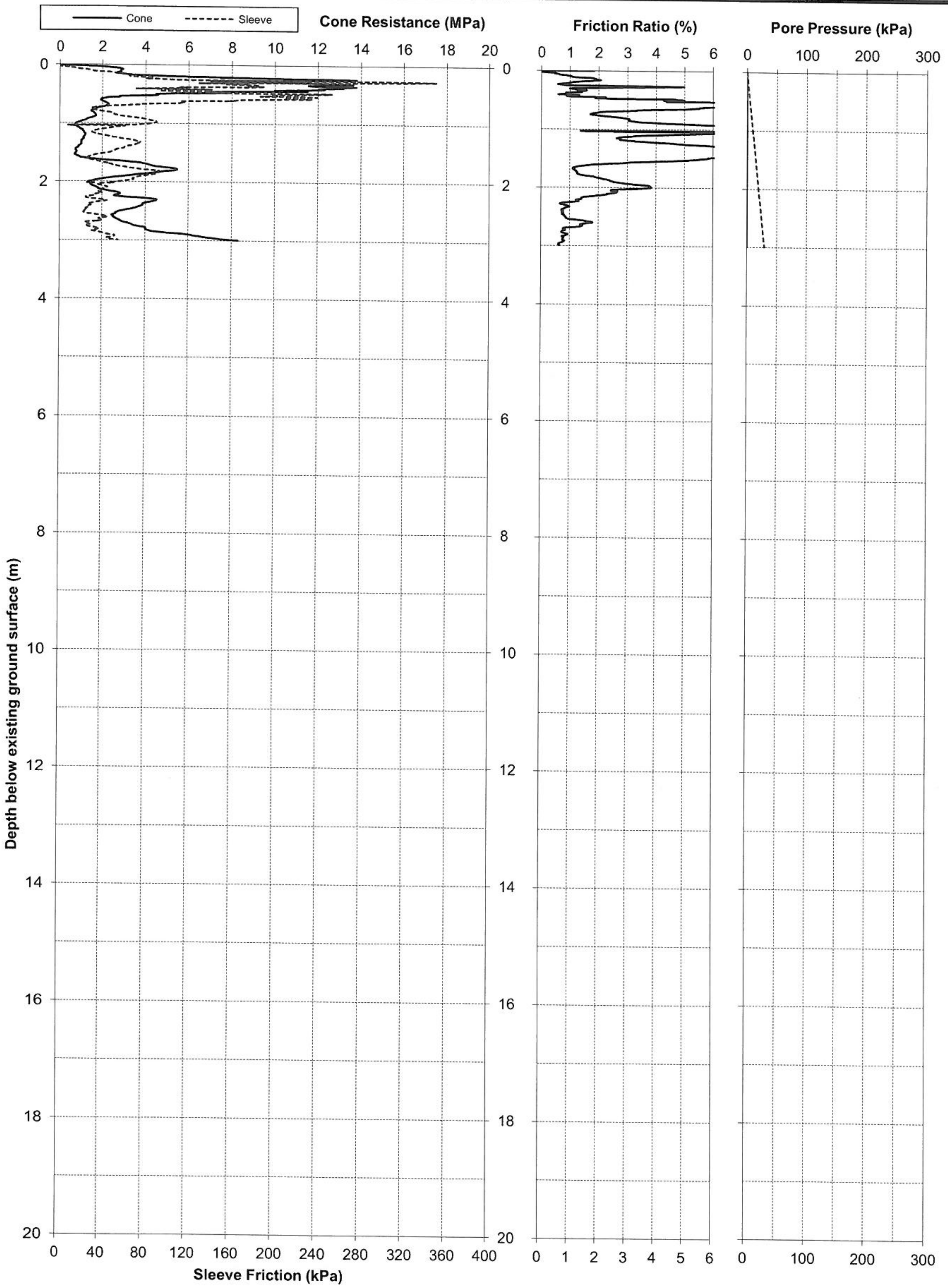
OPUS
HAMILTON LABORATORY

u2
150 cm²
10 cm²

Test according to A.S.T.M standard D-5778-07		Predrill :	0
G.L. **0**	W.L.: **0**	Date:	**6/09/2012**
Project:	**Geotechnical Investigation**	Cone no.:	**C10CFIIP.C10204**
Location:	**GPS: E1570644 N5182457**	Project no.:	**2-68292.12_028**
Position:		CPT no.:	**STA-POD13-CPTI1** **1/6**

CPTask V1.20

Project:	Historical Geotechnical CPT Investigations		**Page:** 1 of 1		**CPT-HIS-0412**
Test Date:	6-Nov-2000	**Suburb:**	Saint Albans		Locations based on supplied address
Pre-Drill:	0 m		**Located By:**	Google Earth	
Position:	5744059 mE	2480564 mN 7.4 mRL	**Coord. System:**	NZMG	
Address:	64/ 68 Trafalgar St		**Datum Reference:**	Lyttelton Vertical Datum (MSL 1937)	



Plotted: 6/11/2012 6:12 p.m.

TONKIN & TAYLOR LTD

BOREHOLE LOG

BOREHOLE No: BH-02
Hole Location: STA-POD08-BH02
(93 Dover Street)
SHEET 1 OF 2

PROJECT: CHCH TC3 GEOTECHNICAL INVESTIGATIONS   LOCATION: ST ALBANS   JOB No: 52003.000

CO-ORDINATES 5744115.35 mN
2480680.72 mE
R.L. 7.49 m
DATUM NZMG, MSL (CCC 20/01/12 Datum -9.043m)

DRILL TYPE: Roto-Sonic
DRILL METHOD: PQDT/Auto SPT
DRILL FLUID: LP2000

HOLE STARTED: 28/8/12
HOLE FINISHED: 28/8/12
DRILLED BY: Pro-Drill
LOGGED BY: GLDS-KJ   CHECKED: BMcD

GEOLOGICAL | ENGINEERING DESCRIPTION

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION. | FLUID LOSS | WATER | CORE RECOVERY (%) | METHOD | CASING | TESTS | SAMPLES | R.L. (m) | DEPTH (m) | GRAPHIC LOG | CLASSIFICATION SYMBOL | MOISTURE CONDITION / WEATHERING | STRENGTH/DENSITY CLASSIFICATION | SHEAR STRENGTH (kPa) 10 25 50 100 200 | COMPRESSIVE STRENGTH (MPa) 1 5 20 50 100 250 | DEFECT SPACING (mm) 50 250 1000 2000

SOIL DESCRIPTION: Soil type, minor components, plasticity or particle size, colour.
ROCK DESCRIPTION: Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.

Geological unit	Core recovery (%)	Method	Tests	Samples	R.L. (m)	Depth (m)	Class. symbol	Moisture	Strength/Density	Soil description
FILL						0	GW	D		Fine to medium GRAVEL with some sand, grey, dry, well graded. Sand is fine to coarse.
YALDHURST MEMBER OF THE SPRINGSTON FORMATION (ALLUVIAL)	100	HAND AUGER			7		ML	M		SILT with some sand, brown, moist, low plasticity. Sand is fine to medium.
			*FC1.5 *ATP1.5	B	6	1.5		W		1.5m- wet. 1.5m- no SPT test.
	100	PQDT			5	2				Sandy SILT, grey, wet, non plastic. Sand is fine to medium.
	100	SPT	*FC3.0 *PL3.0 0/1//0/1/0/0 N=1	B	4	3			VS	3.0m- very soft.
	100	PQDT				4				
	100	SPT	1/1//1/1/1/1 N=4		3	4.5			F	4.5m- firm.
	100	PQDT			2	5	P			PEAT with some sand and some silt, dark brown, firm, wet. Sand is fine to medium.
							SP		L	Fine to medium SAND with some silt, grey, loose, wet, poorly graded.
	100	SPT	3/4//2/1/0/1 N=4			6	GW			Sandy fine to coarse GRAVEL, grey, rounded, loose, wet, well graded. Sand is fine to coarse.
					1		SW			Gravelly fine to coarse SAND, grey, loose, wet, well graded.
	100	PQDT				7	GW			Sandy fine to coarse GRAVEL, grey, rounded, loose, wet, well graded. Sand is fine to coarse.
	100	SPT	5/6//7/4/1/2 N=14		0	7.5			MD	7.5m- medium dense.
CHRISTCHURCH FORMATION (MARINE/ ESTUARINE)	100	PQDT			-1	8	SM			Silty fine to medium SAND, grey, medium dense, wet, poorly graded.
							SP			Fine to medium SAND with minor silt, grey, medium dense, wet, poorly graded.
	100	SPT	*FC9.0 2/2//3/4/4/5 N=16	B	-2	9				
	100	PQDT				10				

Appendix 3 - Methodology and Assumptions

A3.1. Reference Documents

- AS/NZS 1170.0:2002, *Structural design actions, Parts 0: General principles,* Standards New Zealand.
- AS/NZS 1170.1:2002, *Structural design actions, Part 1: Permanent, imposed and other actions,* Standards New Zealand.
- NZS 1170.5:2004, *Structures design actions, Part 5: Earthquake actions – New Zealand,* Standards New Zealand.
- NZS 3404:Part 1:1997, *Steel Structures Standard, The design of Steel 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 5, 19 July 2011.

A3.2. 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 Factory μ = 2.00 (Limited Ductility Structure – in accordance with requirements outlined in NZS3404:1997);
- Structural Performance Factor S_p = 0.7.

A3.3. Material Properties

Table A1: Analysis Material Properties

Mild steel and reinforcing normal yield strength, *fy* (MPa)	250
Probable steel yield strength, *fy* (MPa)	270

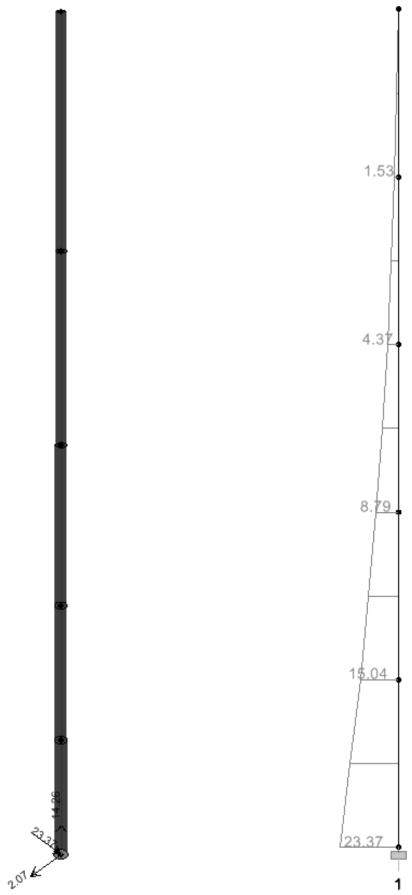
Notes:

i. Based on guidance from *NZSEE 2006*, probable reinforcement yield strength is based on a value of 1.08 times the nominal yield strength (Ci. 7.1.1)

ii. Based on guidance from *Bridge Manual 2004,* characteristic yield strength of reinforcement for historical constructions.

A3.4. Assessment Methodology

Equivalent Static Analysis



The lighting towers were analysed as having limited ductility (μ = 2.00). The seismic design actions were applied at five nodes along the height of the tower in accordance with NZS1170.5:2004 (Section 6.2).

Element force demands were extracted for the equivalent static analysis and compared to calculated capacities based on the material properties assumed in Table A2.3. The results of these demand to capacity checks are summarised in further detail in the report and presented as %NBS.

The flexural capacity of the hexagonal hollow section of the towers was calculated using NZS3404:Part 1:1997, and evaluated against a circular hollow section with similar geometric properties for reference.

Appendix 4 – CERA DEE Spreadsheet

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	English Park Lighting Towers	Reviewer:	Al Boyce
	Unit / No: / Street	CPEng No:	209860
Building Address:	127 Cranford Street	Company:	Opus International Consultants Ltd
Legal Description:		Company project number:	6-QUCCC.65
		Company phone number:	6433635400
	Degrees / Min / Sec		
GPS south:	43 / 30 / 29.21	Date of submission:	5/03/2013
GPS east:	172 / 38 / 11.72	Inspection Date:	10/10/2012
		Revision:	Final
Building Unique Identifier (CCC):	BU 0623-008 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	sandy silt	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	17.50
Ground floor split?	no		Ground floor elevation above ground (m):	
Storeys below ground	0			
Foundation type:	bored cast-insitu concrete piles		if Foundation type is other, describe:	
Building height (m):	20.10		height from ground to level of uppermost seismic mass (for IEP only) (m):	20.1
Floor footprint area (approx):	1			
Age of Building (years):			Date of design:	1992-2004
Strengthening present?	yes		If so, when (year)?	2001
			And what load level (%g)?	
Use (ground floor):	other (specify)		Brief strengthening description:	New foundations constructed under existing towers
Use (upper floors):				
Use notes (if required):	Lighting Towers			
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:			
Roof:			
Floors:			
Beams:			
Columns:	structural steel	typical dimensions (mm x mm)	
Walls:			

Lateral load resisting structure

Lateral system along:	welded and bolted steel moment frame	**Note: Define along and across in detailed report!**		Cantilevered steel tower
Ductility assumed, μ:	2.00		note typical bay length (m)	
Period along:	1.32	1.33 from parameters in sheet	estimate or calculation?	calculated
Total deflection (ULS) (mm):			estimate or calculation?	calculated
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	calculated
Lateral system across:	welded and bolted steel moment frame			Cantilevered steel tower
Ductility assumed, μ:	2.00		note typical bay length (m)	
Period across:	1.32	0.00	estimate or calculation?	calculated
Total deflection (ULS) (mm):			estimate or calculation?	calculated
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	calculated

Separations:

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

Non-structural elements

Stairs:		
Wall cladding:		
Roof Cladding:		
Glazing:		
Ceilings:		
Services(list):		

Available documentation

Architectural	none	original designer name/date	
Structural	partial	original designer name/date	City Design 21/08/01
Mechanical	none	original designer name/date	
Electrical	none	original designer name/date	
Geotech report	none	original designer name/date	

Damage

Site:
(refer DEE Table 4-2)

Site performance:	Poor	Describe damage:	Settlement & Liquefaction observed
Settlement:	0-25mm	notes (if applicable):	
Differential settlement:	0-1:350	notes (if applicable):	
Liquefaction:	2-5 m³/100m²	notes (if applicable):	
Lateral Spread:	50-250mm	notes (if applicable):	
Differential lateral spread:	0-1:400	notes (if applicable):	
Ground cracks:	0-20mm/20m	notes (if applicable):	
Damage to area:	moderate to substantial (1 in 5)	notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

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

Recommendations

	Level of repair/strengthening required:	none	Describe:	
	Building Consent required:	no	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	
Along	Assessed %NBS before e'quakes:	100% ##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Quantitative Assessment
	Assessed %NBS after e'quakes:	100%		
Across	Assessed %NBS before e'quakes:	100% ##### %NBS from IEP below		
	Assessed %NBS after e'quakes:	100%		

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