Tonkin & Taylor

T&T Ref: 52118
11 July 2011

StructureSmith Ltd
PO Box 26502
Epsom
Auckland 1344

Attention: Ashley Smith

Dear Sirs

CTV Building Geotechnical Advice

1 Introduction

1.1 General

This report presents the review of geotechnical conditions at the site of the CTV building, 249 Madras Street, Christchurch, together with the results of an assessment of the dynamic response parameters for the foundations of the building.

This work has been carried out for the Department of Building and Housing (DBH) through the Hyland/Structure Smith Consultant Team, responsible for the investigation of the structural performance of the building during and following the Christchurch earthquake of 22 February 2011.

1.2 Background

The Christchurch area has recently experienced a number of earthquakes and aftershocks, generally starting with the magnitude (M_w) 7.1 Darfield earthquake on 4 September, 2010. Whilst that event caused extensive damage to unreinforced masonry buildings, residential areas and infrastructure, there were no major building collapses and no fatalities.

A magnitude 4.9 aftershock on 26 December 2010 caused some further damage in the Central Business District (CBD) but this was eclipsed by the 22 February 2011 aftershock, now termed the Christchurch earthquake. This magnitude 6.3 event, with very high level ground motions, led to the collapse of two multi-level buildings, one being the CTV building.

The Department of Building and Housing has engaged a joint team of Structure Smith and Hyland Fatigue and Earthquake Engineering to carry out the technical investigation into the performance of the CTV building.

1.3 Objectives and scope

The DBH Project Plan for the Christchurch Earthquake CBD Building Performance Investigation sets out the objectives of the investigation as follows:

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- to determine the facts about the performance of buildings in the Christchurch CBD, establishing the causes of, and contributing factors to the building failures
- to provide a comprehensive analysis of these causes and contributing factors, including, as context, the building standards and construction practices when these buildings were constructed or alterations made to them.

To this end, a detailed scope of work has been set out in the Project Plan, of which the following items may be relevant to the geotechnical and foundation aspects:

i. Review and report on:
   - The original design and construction, including, the foundations and soils investigations
   - The cause(s) of the collapse of the building

ii. Investigation to include consideration of:
   - The design codes, construction methods, and building controls in force at the time the buildings were designed and constructed and changes over time as they applied to these buildings;
   - Knowledge that a competent structural / geotechnical engineer could reasonably be expected to have of the seismic hazard and ground conditions when these buildings were designed;
   - Changes over time to knowledge in these areas; and
   - Any policies or requirements of any agency to upgrade the structural performance of the buildings.

iii. Reports to include:
   - Relevant facts about the design
   - Factors that may have contributed to the collapse
   - Recommendations on changes needed in codes, standards, design and/or construction practices necessary to achieve levels of safety in major earthquakes in New Zealand.

[Note: The above are extracts from the DBH Project Plan and Terms of Reference that are considered relevant to the geotechnical and foundation aspects.]

2 Geotechnical information & site conditions

2.1 Geotechnical report

A geotechnical report was prepared by others in June 1986. The investigations for the site at that time comprised 8 hand-augered boreholes, supplemented by 3 machine-augered boreholes and two deeper boreholes drilled by cable-tool methods.

In judging the quality and usefulness of the report, the following observations are of interest:

(i) The hand-augered boreholes (H/A's) were all between 3 and 4 m depth. Five of these were logged as terminating on gravel but clearly this horizon was inferred by 'feel' as the gravel was not penetrated. The remaining three H/A's were logged as terminating in sand, all in the NE corner of the site.

(ii) The three machine-augered boreholes were put down specifically to prove the gravel but the logs imply no penetration into the gravel.

(iii) The two cable-tool boreholes (BH14 and 15) provide useful deeper information, one with 5.7 m thickness of gravel and the other with just 100 mm of gravel.

(iv) The interpretation of sub-surface conditions by the geotechnical engineer, based on (i), (ii) & (iii) above, was that there is no gravel in the NE corner of the site and elsewhere there is a gravel layer of 5 to 6 m thickness at about 3.5 to 4 m depth. This was supported by their review of adjacent investigations to the south and southeast of the site.

(v) Whilst the interpretation of (iv) above seems reasonable, there are other possible interpretations. For example, the H/A's did not penetrate gravel so may just have terminated on a very thin layer, as logged in the machine borehole BH14.

(vi) The logged profiles in all the 1986 investigation points show very similar conditions over the top 3.5 to 4 metres. It is therefore just the presence or absence of the gravel that makes the difference in foundation conditions and the response characteristics.

(vii) There is some quantitative information on the 1986 logs with regard to the nature of the sands. A few Standard Penetration Tests (SPT's) were performed in the shallow machine-augered holes and some in one of the deep cable-tool holes, summarised as follows:

- B/H 10: 1 SPT only at top of gravel
- B/H 11: 1 SPT in sand (N = 10) and 1 SPT in gravel
- B/H 13: 1 SPT in sand (N=15), and 2 other SPT's above and below sand
- B/H 15: No SPT's in top 6 m; 6 SPT's in 6 m to 12 m depth range, generally N>30
- B/H 15: No SPT's
- The significance of this limited data is discussed later.

(viii) In 1986, one water level was measured in the 13 boreholes set at 2.8 m depth below ground level and within the "silty, fine to medium sand" layer consistently logged at this level in all boreholes.

2.2 Site sub-surface conditions

The top four metres of the soil profile appear very consistent over the whole site, with silt (moist, firm) generally down to 1.5 m depth, overlying silty fine to medium sand (SM or SW). The water level is within this sand.

The geotechnical report of 1986 interpreted site conditions to differ below this level as follows:

- Over the major portion of the site, a thick dense gravel layer of 5 to 6 m thickness is present, overlying a deep layer of dense sand.
- For the remainder of the site, over the NE quadrant, the gravel is not present and is replaced by more sand and silt.

The report pointed out that *"... the transition between the gravel and soft sediments overlying the sand .... is quite abrupt and crosses the north-east corner of the site."*

The limited SPT data indicate that the sands of the top 6 m and above the gravel are "medium dense" and the sands below 6 m and below the gravel are "dense" to "very dense".

2.3 Foundation recommendations

Amongst other items, the 1986 geotechnical report provides criteria for shallow (spread) foundations on the basis of both bearing capacity and settlement. The design information was summarised on a useful chart providing allowable bearing pressures for different footing sizes, shapes and depth, for the two characteristic areas of the site.

We have checked this chart and confirm that it is appropriate, helpful and reasonably conservative for static design of footings at this particular site.

The chart can be used to derive spring stiffness under the spread foundations for static long term loadings.

2.4 Sufficiency of investigations and geotechnical information

The scope and methodology of the investigations appear to have been typical for Christchurch sites of the 80's and 90's period. It is understood that the geotechnical engineer had extensive experience in the area and had considerable knowledge of site conditions which would allow a relatively economic approach. For example, because of the knowledge of conditions in the surrounding areas, the use of hand-augered boreholes was common practice. The characterisation of sub-surface conditions below hand-auger depth was based on just two deeper boreholes.

The interpretation of site conditions on the basis of the investigation and regional knowledge appears reasonable and indeed "most likely" but still uncertainties remain.

There is no mention of "liquefaction" in the report but there is a statement in the 1986 report:

> *"The near surface fine sediments have only moderate resistance to seismic loading."*

The report twice includes recommendations for additional investigation. It is not known if such recommendations were implemented.

With the benefit of hindsight, and with the benefit of a heightened awareness of liquefaction that has gradually developed over the last two decades, it would now seem prudent to carry out more investigation and more testing to give quantitative measures of density, strength and stiffness. If the investigation were to be carried out now, it would likely include much deeper boreholes with more SPT's and probably a number of Cone Penetration Tests (CPT's).

3 Foundation response parameters

3.1 Factors influencing foundation response

In order to carry out a dynamic analysis of the building, the structural analysis team requires representation of the soil-foundation interaction as "subgrade reaction" stiffnesses. In effect these are springs per unit area with units of kN/m stiffness per metre squared or kN/m^3. It is often convenient to think of the stiffness in terms of kN/mm per m^2, which is equivalent to MN/m^3.

In deriving these spring stiffnesses for the dynamic analyses, the following factors need to be taken into account:

a) Soils are non-linear so that at the very small dynamic strains, the dynamic stiffness is very much higher than the more usual static stiffness.

b) In addition to (a) above, the short-term dynamic stiffness is greater than the long-term static stiffness due to water in the pores for which the pore pressures are only able to dissipate partially during rapid cyclic loading.

c) Soil stiffness is also load dependent. Under higher confining stresses imposed by the building, the incremental stiffness is significantly higher than the first-load stiffness.

d) Unit spring stiffness is dependent on footing size. The larger the footing the greater the depth of influence and hence more compression per unit load.

e) Unit spring stiffness is also dependent on footing shape. Square footings distribute load in two lateral dimensions whereas strip footings distribute in one direction. Strip footings, therefore, tend to have greater depth of influence and hence lower stiffness.

f) Spring stiffness is depth dependent. Deeper footings in relation to lateral dimension have higher stiffness.

In summary, the subgrade reaction values for the dynamic analysis should be expected to be much greater than for static dead-load analysis, and to vary with footing shape, size and depth.

3.2 Method of determining response parameters

To address the factors above, a method specifically developed for dynamic situations has been employed that allows for different depths and shapes of footing, and is based on the dynamic shear modulus (G). The method is given by Barkan (1948)[1].

The dynamic shear modulus was not measured for the site but a range can be estimated, based on the few SPT values and published examples. The shear modulus is related to the shear wave velocity and density of the soils.

3.3 Foundation arrangement

The foundation details of the building are given on the original design drawings dated August 1986.

Because of the different shapes, sizes and depths, and because of need to provide details for the two characteristic soil profiles, the various foundation elements have been assigned Type Numbers, as shown on Figure 1.

3.4 Dynamic subgrade reaction stiffness values

The results of the computations using the Barkan formulae are given in spreadsheets in Appendix A. These spreadsheets are presented, representing probable lower bound soil stiffness parameters, most likely parameters and probable upper bound parameters.

The parameters (shear wave velocity, density and Poisson's ratio) are considered to be average values for the soil profile to depth of three times the footing width below the bearing surface. The major part of the site with the gravel present is termed the "stiff area", and the NE quadrant area without gravel is termed the "soft area".

3.5 Static subgrade reaction stiffness values

The static values may be deduced separately from published data (e.g. Bowles, 1988[2]):

- Loose sand 5 – 16 MN/m^3
- Medium dense sand 19 – 80 MN/m^3
- Dense sand and gravel 64 – 128 MN/m^3

Taking account of the layering under the footings and assuming 3B depth of influence (where B = footing width), Footing Type 1 would have a range of subgrade reaction stiffness of 51 to 116 MN/m^3 and Type 1b (soft area) of 10 to 80 MN/m^3. Comparing these with the dynamic values, it would seem reasonable to take static values as equal to half the lower bound dynamic values.

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1 Barkan, D.D. (1948): Dynamics of bases and foundations, McGraw Hill.

2 Bowles, J.E. (1988): Foundation Analysis & Design, McGraw Hill, p. 409.

4 Discussion

4.1 Practice in 1980's

The computer models may have become more sophisticated and more powerful over the past few decades but the methods of determining the input subgrade reaction stiffness have changed little. What seems to have changed is the awareness that normal published values of stiffness significantly underestimate the values required for dynamic conditions.

There is no mention of subgrade reaction stiffness in the 1986 geotechnical report. However, there is a chart showing working pressures to limit settlements to 25 mm, from which subgrade reaction stiffnesses for different foundation types and sizes can be computed. If this chart had been used for this purpose, it would have given stiffness values at the extreme "soft" end of the range, even for static conditions. This is reasonable and conservative for the normal static settlement assessment but is not necessarily conservative for the dynamic analysis.

4.2 Liquefaction

We understand that one area of localised liquefaction was reported on the adjacent empty site to the west. Otherwise there have been no reports of obvious liquefaction in the immediate vicinity of the CTV Building.

Notwithstanding this, the sands noted in all the boreholes in the depth range 1.5 to 3.5 m (silty fine-to-medium sand) are typical of those soils that liquefied in the February 22nd event. The ground water level is within this layer and the foundations were established just above. For the NE Quadrant (BH14: fine to medium sand), the possibly liquefiable material extends to 6 m depth.

There are few quantitative measurements of density but the limited SPT results (N = 10 to 15) indicate that liquefaction would have been likely in the top 6 m at the accelerations experienced. In borehole BH14, there were Standard Penetration Tests in deeper sands below 8 m depth but these all indicated a dense state (N>40), not susceptible to liquefaction.

In summary, a thin layer, between water level at 2.5 – 3 m depth and gravel at 3.5 to 4 m depth, may have liquefied during and following the February earthquake. At the NE quadrant, this may have extended deeper. The limited thickness of the layer and the confining effect of the larger footings would mean complete bearing future would be unlikely, but "yield" with resulting settlement and differential settlement could have occurred.

No evidence was found from the levelling survey, foundation inspection or pits dug adjacent to the footings that indicated the lean of the core tower was due to the September earthquake and aftershocks.

5 Strong ground motion records

Assessment of the ground motions at the CTV site may be deduced from the four strong-motion recordings surrounding the CBD. A fifth station to the east may also be relevant for comparison with the others, as this would be off the gravel areas and is at about the same focal distance.

The five stations of interest are shown on Figure 2 and are listed below:

- Botanical Gardens: CGBS
- Cathedral College: CCCC
- Christchurch Hospital: CHHC
- Rest Home Colombo Street: REHS

- Page Road Pumping Station: PRPC

The last two of these (REHS and PRPC) show significantly higher amplification than the others, both with respect to Peak Ground Accelerations (PGA) and spectral accelerations.

A borehole (BH 103) drilled for the Department of Building and Housing (DBH) at the REHS site logs significant thickness of "very soft organic silt" and "very soft peat". Other borehole records near REHS (ECAN bores 2140 and 2142) suggest the presence of peat or clayey soils of about 6m thickness. These support the general knowledge of the area that would predict "silts, clayey silt and peat" north of Peterborough Street.

The PRPC station is located in a known liquefaction zone, with a nearby borehole (ECAN – M35/5124) logging sand to 27m depth, overlying sands and gravels.

The other three stations (CGBS, CCCC, CHHC) are all expected to have generally similar profiles of variable inter-bedded silts, silty and gravelly sands, overlying dense sands.

The response characteristics are generally governed by the soil profile to rock which would be about 300 m depth, and so thin layers near surface are not likely to make much difference. The exception, of course, could be the presence of deep liquefiable soils and/or the presence of peat. For this reason we consider the REHS and PPPC records should be disregarded and the CTV site response should be assumed as similar to the average of the other three stations.

6 Conclusions and recommendations

i. The geotechnical investigation carried out in 1986 was typical of the time and appropriate for the expected development. The report contained recommendations for further investigation. A modern investigation would now likely involve more deeper boreholes with more sampling and SPT's. Cone Penetration Tests would offer the opportunity of mapping the "transition" between gravel/no-gravel areas and also quantitative data for liquefaction analysis. Shear wave measurements would enable assessment of dynamic response parameters for dynamic analyses.

ii. Liquefaction was not mentioned in the 1986 geotechnical report though the potential for liquefaction in Christchurch was well known at the time. Some of the soils at depth could have been subject to liquefaction or strength loss.

iii. The type of foundations employed for the CTV building were typical for the size of the building and the Christchurch CBD. Provided liquefaction was not an issue, the shallow spread footings would seem appropriate and design recommendations were conservative for static conditions.

iv. Design codes, construction methods and building controls are likely to change in relation to liquefaction. As a minimum, we recommend that any future geotechnical report provide a statement on the potential for liquefaction.

v. Notwithstanding the issue of liquefaction, consideration of other earthquake effects should become part of foundation designs in future. This should include the provision of dynamic response parameters, based on site-specific measurements described in (i) above.

7 Applicability

This report has been prepared for the benefit of the Department of Building and Housing and the Hyland/StructureSmith Consultant Team with respect to the particular brief given to us and it may not be relied upon in other contexts or used for any other purpose without our prior review and agreement.

This opinion is not intended to be advice that is covered by the Financial Advisers Act 2010.

Tonkin & Taylor Ltd

Environmental and Engineering Consultants

Report prepared by:

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Tim Sinclair

Technical Director

Copy to: Clark Hyland

12-Jul-11
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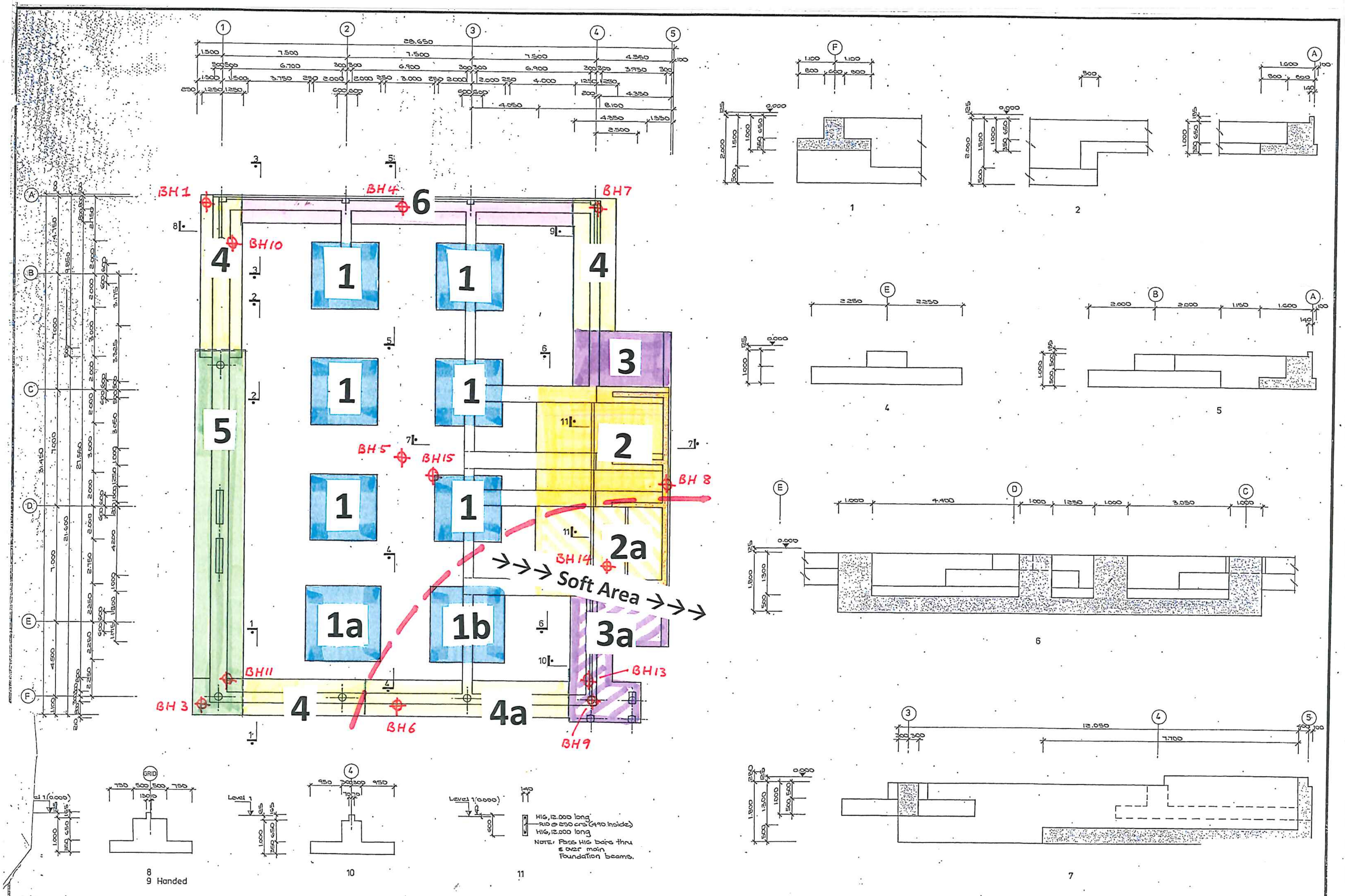

Figure 1: Foundation Types

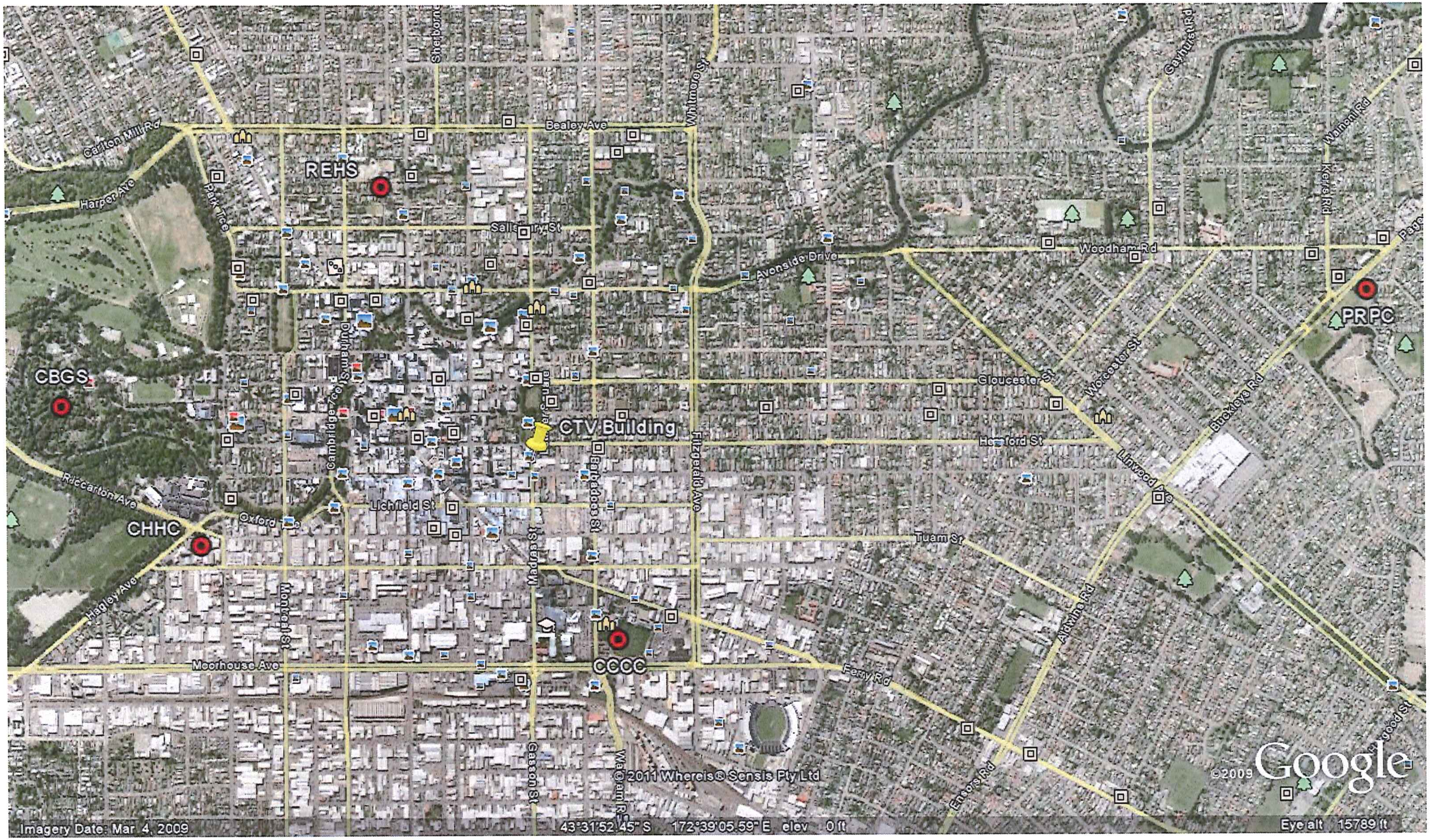

Figure 2: Locations of Strong Motion Recording Stations

Appendix A: Dynamic response parameters

CTV BUILDING FOUNDATION SPRING STIFFNESS

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Most likely values

Shear wave velocity:

Stiff area	300 m/s
Soft area	200 m/s

Shear modulus:

Stiff area	162.00 MPa
Soft area	70.00 MPa

Density:

Stiff area	1800 kg/m^3
Soft area	1750 kg/m^4

Poissons Ratio:

Stiff area	0.3
Soft area	0.4

Footing type	B (m)	L (m)	D (m)	L/B	D/B	Depth factor	Barcan: β_z	Spring (MN/m^3)	Comment
1	4	4	1	1.00	0.25	1	2.12	122.71	
1a	4.5	4.5	1	1.00	0.22	1.2	2.12	130.89	
1b	4.5	4.5	1	1.00	0.22	1.2	2.12	65.98	Soft area
2	7.3	7.7	1.8	1.05	0.25	1.3	2.13	85.40	
2a	5.4	7.7	1.8	1.43	0.33	1.35	2.18	53.14	Soft area
3	3.3	5.8	1	1.76	0.30	1.33	2.22	117.22	
3a	3.3	5.8	1	1.76	0.30	1.33	2.22	78.59	Soft area
4	2.5	9.8	1	3.92	0.40	1.4	2.44	159.69	
4a	2.5	12.5	1	5.00	0.40	1.4	2.53	73.94	Soft area
5	3	21.6	1	7.20	0.33	1.35	2.69	104.35	
6	1.7	20	1	10.00	0.59	1.5	2.87	185.42	

CTV BUILDING FOUNDATION SPRING STIFFNESS

31/05/2011 16:03

<u>Lower Bound(Soft)</u>

Shear wave velocity:

Stiff area	280 m/s
Soft area	150 m/s

Shear modulus:

Stiff area	133.28 MPa
Soft area	37.13 MPa

Density:

Stiff area	1700 kg/m^3
Soft area	1650 kg/m^4

Poissons Ratio:

Stiff area	0.25
Soft area	0.35

Footing type	B (m)	L (m)	D (m)	L/B	D/B	Depth factor	Barcan: β_z	Spring (MN/m^3)	Comment
1	4	4	1	1.00	0.25	1	2.12	94.22	
1a	4.5	4.5	1	1.00	0.22	1.2	2.12	100.51	
1b	4.5	4.5	1	1.00	0.22	1.2	2.12	32.30	Soft area
2	7.3	7.7	1.8	1.05	0.25	1.3	2.13	65.57	
2a	5.4	7.7	1.8	1.43	0.33	1.35	2.18	26.01	Soft area
3	3.3	5.8	1	1.76	0.30	1.33	2.22	90.01	
3a	3.3	5.8	1	1.76	0.30	1.33	2.22	38.48	Soft area
4	2.5	9.8	1	3.92	0.40	1.4	2.44	122.62	
4a	2.5	12.5	1	5.00	0.40	1.4	2.53	36.20	Soft area
5	3	21.6	1	7.20	0.33	1.35	2.69	80.13	
6	1.7	20	1	10.00	0.59	1.5	2.87	142.38	

CTV BUILDING FOUNDATION SPRING STIFFNESS

31/05/2011 16:03

Upper Bound (stiff)

Shear wave velocity:

Stiff area	320 m/s	
Soft area	250 m/s	0.78125

Shear modulus:

Stiff area	204.80 MPa
Soft area	115.63 MPa

Density:

Stiff area	2000 kg/m^3
Soft area	1850 kg/m^4

Poissons Ratio:

Stiff area	0.35
Soft area	0.45

Footing type	B (m)	L (m)	D (m)	L/B	D/B	Depth factor	Barcan: β_z	Spring (MN/m^3)	Comment
1	4	4	1	1.00	0.25	1	2.12	167.06	
1a	4.5	4.5	1	1.00	0.22	1.2	2.12	178.20	
1b	4.5	4.5	1	1.00	0.22	1.2	2.12	118.90	Soft area
2	7.3	7.7	1.8	1.05	0.25	1.3	2.13	116.26	
2a	5.4	7.7	1.8	1.43	0.33	1.35	2.18	95.75	Soft area
3	3.3	5.8	1	1.76	0.30	1.33	2.22	159.59	
3a	3.3	5.8	1	1.76	0.30	1.33	2.22	141.62	Soft area
4	2.5	9.8	1	3.92	0.40	1.4	2.44	217.41	
4a	2.5	12.5	1	5.00	0.40	1.4	2.53	133.23	Soft area
5	3	21.6	1	7.20	0.33	1.35	2.69	142.07	
6	1.7	20	1	10.00	0.59	1.5	2.87	252.45	

Tonkin & Taylor

T&T Ref: 52118
23 January 2012

Structure Smith Ltd
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Epsom
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Attention: Ashley Smith

Dear Sirs

CTV Building Geotechnical Advice - Addendum

This addendum presents additional information in relation to the geotechnical advice given in our letter report Ref 52118 of 11th July 2011. Section 5.0 of that report considered the ground conditions of five strong motion recording stations surrounding the CBD. We understand that records from two other recording stations have subsequently become available. The two stations of interest, namely at the Westpac Building (Station 503) and the Police Station (Station 501), are closer to the CTV Building site.

To determine whether the locations of these additional recording stations are similar to the CTV site, we have examined the available subsurface information and comment as follows:

1. Westpac Building (503)

i. The locations for the site investigation points surrounding the Westpac Building are shown on Figure A1 attached. (Note: Station 503E is marked on this plan but not in exact location due to co-ordinate discrepancy.)

ii. One ECAN borehole log is available at this location (M35 – 7403), attached for information as Figure A2.

iii. Tonkin & Taylor have carried out investigations for the Triangle Centre (T&T Ref No. 52157) immediately north west of the Westpac Building. The logs of boreholes give detailed subsurface information to about 27 m depth. Whilst not being able to include the logs of boreholes and cone penetration tests (CPT), the information enables us to have confidence in interpretation of the subsurface profile.

iv. The GeoNet DELTA site gives descriptive information, including assumed layer thicknesses, which agree with the above information.

v. Conclusion: The site conditions here are very similar to the conditions for the major portion of the CTV Building site, over which the gravel layer is present.

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2. Police Station (501)

i. The locations of the site investigation points surrounding the Police Station are shown on Figure A3 attached. The recording station CCPS (501) is also marked.

ii. Several ECAN borehole logs are available for the immediate vicinity of the Police Station. Log for M35-2148 and M35-8097 are attached for information as Figures A4 and A5 (1 & 2)

iii. Recent CBD investigations give detailed near-surface information (ref CPT CBD-58 attached as Figure A6- 1 & 2).

iv. The GeoNet DELTA site gives information that is reasonably consistent with the other sources.

v. Conclusion: The site conditions at the Police Station are similar to those at CTV Building site, though slightly more favourable/stiffer due to greater depth of near-surface gravel.

In general, for the purpose of seismic site class characterisation, I consider both of these recording stations have sufficiently similar conditions to the CTV site to warrant use of time histories for dynamic analysis.

Tonkin & Taylor Ltd
T J E Sinclair
Technical Director

23-Jan-12
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Westpac Building
CPT-CBD-59
CPT-CBD-68
CPT-CBD-69
M35_7389
M35_16134
503E
M35_16132
M35_2475
M35_16103
M35_16131
M35_5823
M35_6007
M35_9899
M35_1921
M35_16130
M35_2263
M35_16114
M35_16113
M35_2261
M35_2264
M35_1915
M35_7403
M35_9986
M35_9987
M35_7382
M35_7383
M35_7404
M35_1917
M35_2280
M35_2199
M35_4163
M35_16112
M35_11958
M35_11959
M35_2168
High Street Arcade
National Mutual Arcade
Tattersalls Ln
Cashel St
High St
Kivers Ln
Cashfields Arcade
Lichfield St
GeoNet
Canterbury Earthquake Recovery Authority
EQC
EARTHQUAKE COMMISSION
Tonkin & Taylor
124 m
© 2011 Whereis® Sensis Pty Ltd
©2009

Figure A1

Borelog for well M35/7403

Gridref: M35:8078-4149 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 6.7 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -59.4m Drill Date : 27/07/1891

Environment Canterbury Regional Council

Water Level: Artesian

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

Depth(m)	Full Drillers Description	Formation Code
-3.00m	Soft Blue sand	sp?
-7.90m	Hard Blue sand & gravel	sp?
- 23.5m	Soft Blue sand	ch
- 27.4m	Hard Grey sand	ch
- 30.5m	Hard gravel	ri
- 38.4m	Hard Brown sand (Gravel??)	ri
- 39.9m	Soft Yellow sand	br
- 44.5m	Hard Brown sand	br
- 52.5m	Soft Grey sand	br
- 57.6m	Soft Blue sand & peat	br
- 59.4m	Hard Grey sand & gravel	li-1

Figure A2

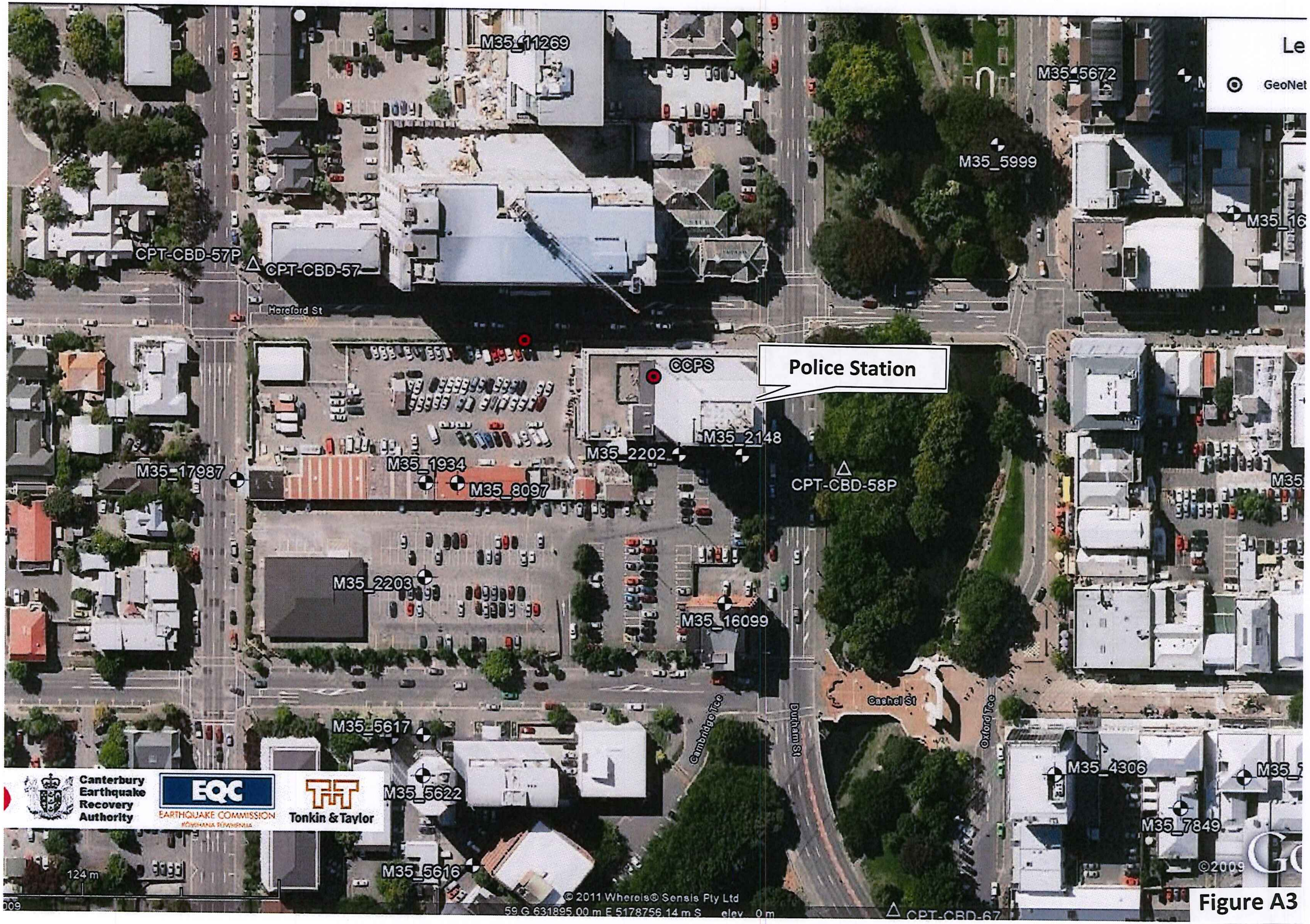

Le
GeoNet
M35_11269
M35_5672
M35_5999
CPT-CBD-57P
CPT-CBD-57
Hereford St
CCPS
Police Station
M35_2148
M35_2202
CPT-CBD-58P
M35_17987
M35_1934
M35_8097
M35_2203
M35_16099
M35_5617
Cambridge Tce
Durham St
Cashel St
Oxford Tce
M35_4306
M35_7849
M35_5622
M35_5616
Canterbury Earthquake Recovery Authority
EQC
EARTHQUAKE COMMISSION
Tonkin & Taylor
124 m
© 2011 Whereis® Sensis Pty Ltd
59 G 631895.00 m E 5178756.14 m S elev 0 m
©2009

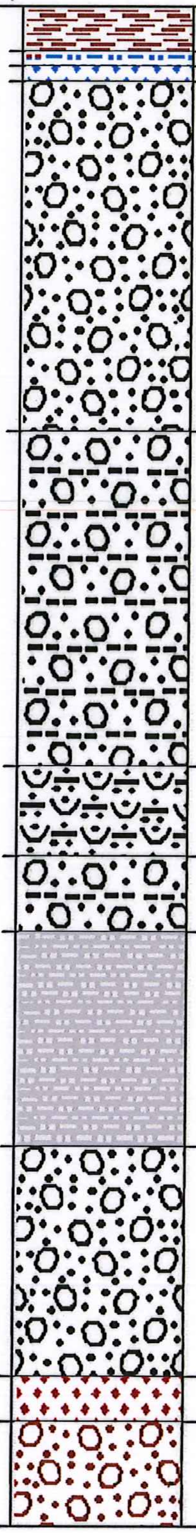
Figure A3

Borelog for well M35/2148

Gridref: M35:8032-4160 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 4.7 +MSD
Driller : Ministry of Works
Drill Method : Cable Tool
Drill Depth : -30.4m Drill Date : 17/08/1967

Environment Canterbury Regional Council

Scale(m) | Water Level 0.0CalcMin | Depth(m)



Depth(m)	Full Drillers Description	Formation Code
-0.89m	Fill, Brown mottled clay	fi
-1.20m	Blue silty sand	sp?
-1.50m	Blue coarse sand	sp?
-8.50m	Grey and Blue sandy gravel	sp?
- 15.2m	Grey silty fine sand with traces of wood & gravel	sp?
- 17.0m	Grey silty medium fine sand with shells	ch
- 18.5m	Grey silty medium fine sand with some gravel	ch
- 22.8m	Grey silty fine sand	ch
- 27.4m	Brown and Grey sandy gravel	ri
- 28.3m	Brown fine sand	ri
- 30.4m	Brown sandy gravel with sandy gravel lens	ri

Scale: -5, -10, -15, -20, -25, -30

Figure A4

Borelog for well M35/8097 page 1 of 2

Environment Canterbury Regional Council

Gridref: M35:8023-4159 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 5.9 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -62.2m Drill Date : 11/10/1892

Water Level: Artesian

Depth(m)	Full Drillers Description	Formation Code
-1.80m	Soft Yellow sand	sp
- 10.7m	Hard Blue gravel	sp
- 15.2m	Soft Blue sand	ch
- 16.8m	Soft Blue clay	ch
- 19.8m	Soft Blue sand	ch
- 23.2m	Soft Blue clay	ch
- 31.1m	Hard Grey gravel and got 1st strata water to rise 1.07m and throw 0.23 l/s (3gpm) at surface	ri

Scale(m): -5, -10, -15, -20, -25, -30

Borelog for well M35/8097 page 2 of 2

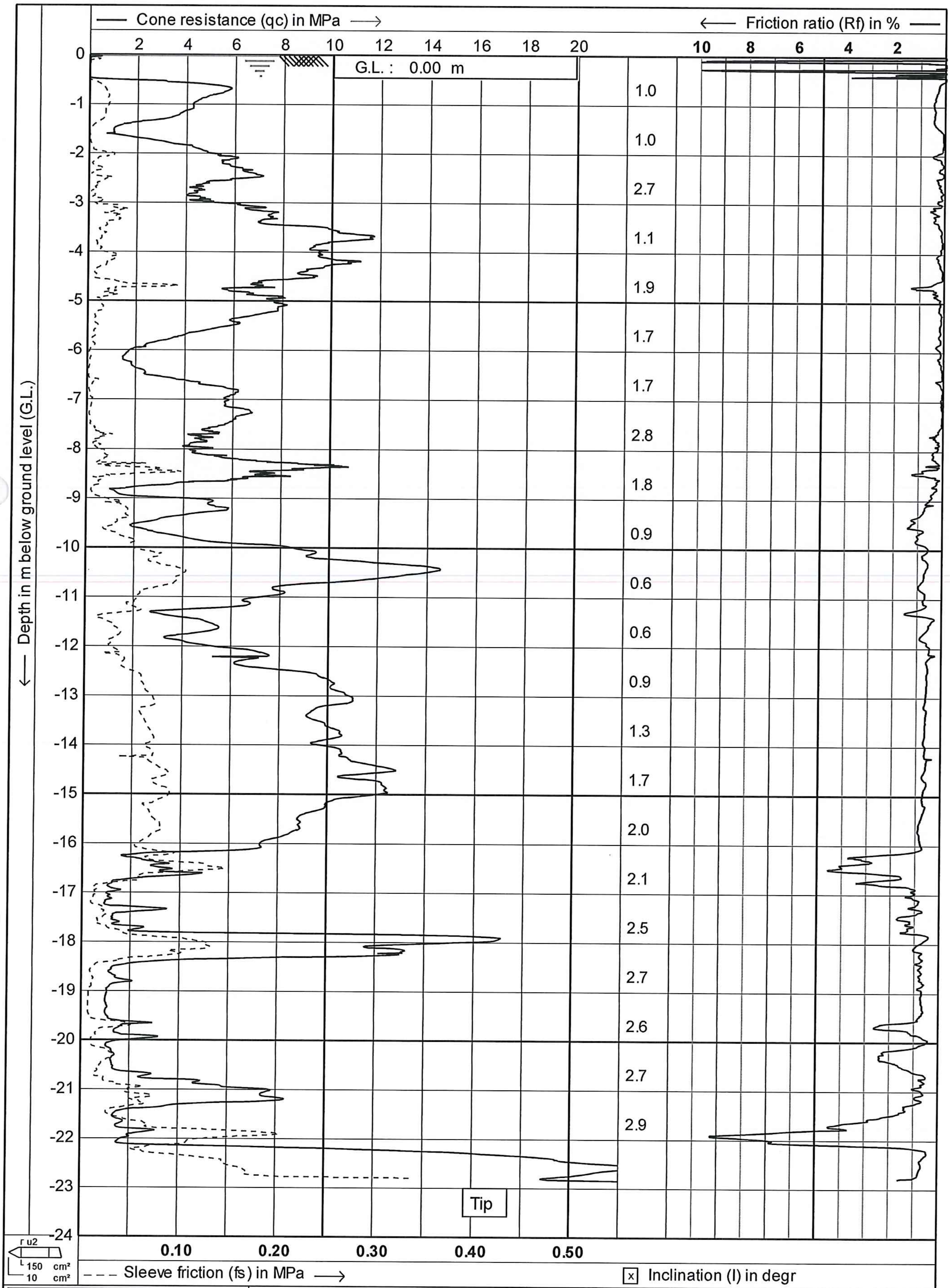
Gridref: M35:8023-4159 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 5.9 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -62.2m Drill Date : 11/10/1892

Environment Canterbury Regional Council

Water Level: Artesian

Depth(m)	Full Drillers Description	Formation Code
- 36.6m	Hard Grey gravel and got 1st strata water to rise 1.07m and throw 0.23 l/s (3gpm) at surface	ri
- 37.2m	Soft Blue sand	br
- 38.1m	Peat	br
- 39.3m	Soft Blue clay	br
- 41.4m	Hard Grey gravel 2nd strata water rise 1.37m, throws 0.46 l/s (6 gpm) at surface	br
- 50.0m	Soft Yellow sand	br
- 51.2m	Soft Blue sand	br
- 51.8m	Soft peat	br
- 54.3m	Soft Blue clay	br
- 57.3m	Hard Grey gravel 3rd strata water rise 2.4m, throws 1.14 l/s (15gpm) 0.6m high	li-1
- 57.9m	Soft peat	li-2
- 59.7m	Soft Blue clay	li-2
- 62.2m	Very hard Grey gravel and got good flow to rise 3.66m and throw 2.28 l/s (30 gpm) 0.6m high and 1.82 l/s (24 gpm) 1.68m above surface	li-2

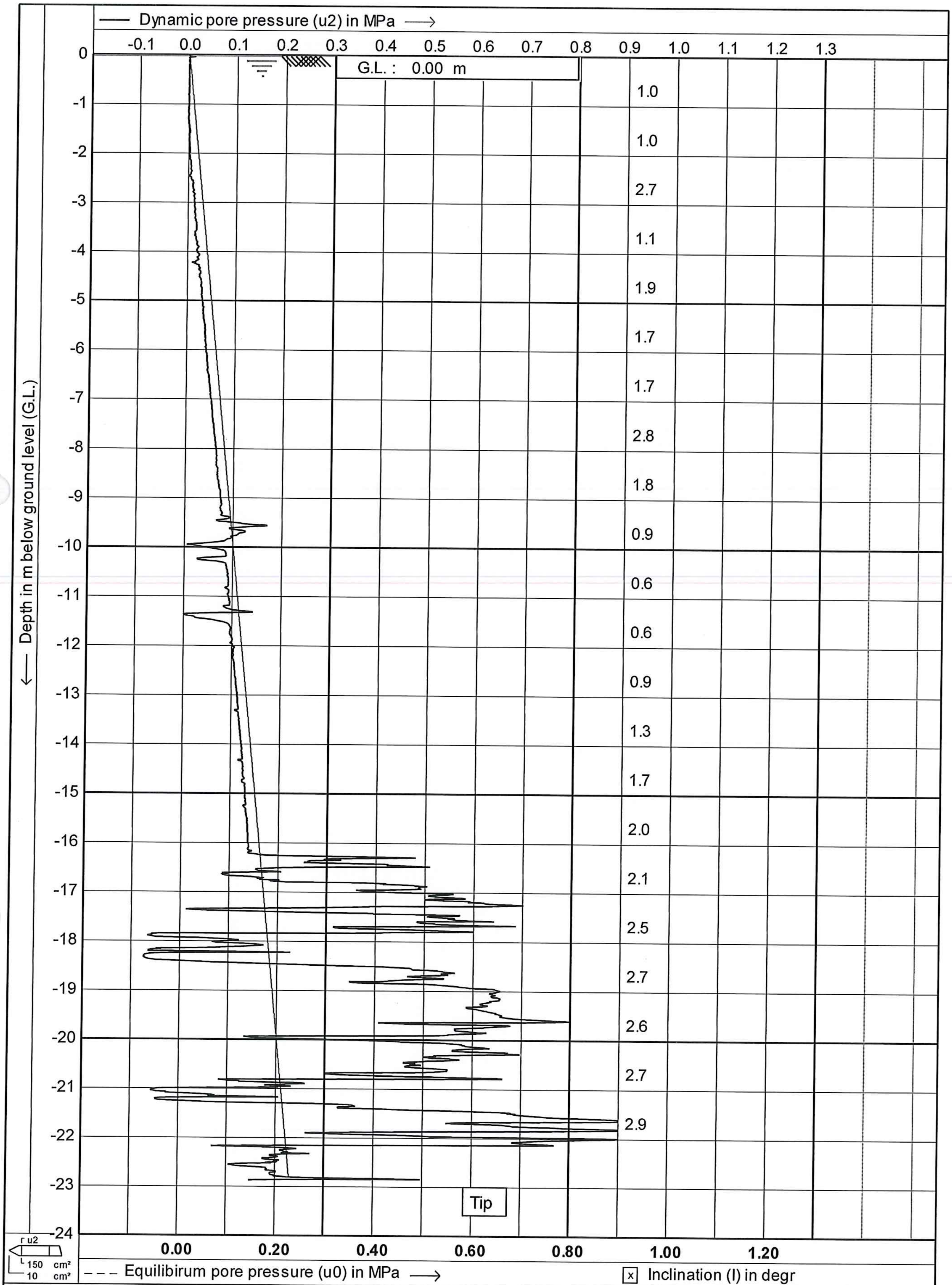
Scale(m): -35, -40, -45, -50, -55, -60



Perry Drilling GEOPROBE Ground Drilling and Environmental Specialists	Test according A.S.T.M. Standard D 5778-07	Date : **29-11-2011**
	Project : **Site Investigations**	Cone no. : **C10CFIIP.F56**
		Project no. : **01TT26**
	Location: **CBD - Christchurch City**	CPT no. : **CBD-58a** **1/14**

CPTask V1.31

Figure A6-1



Perry DRILLING GEO PROBE Ground Drilling and Environmental Specialists	Test according A.S.T.M. Standard D 5778-07	Date : 29-11-2011
	Project : **Site Investigations**	Cone no. : C10CFIIP.F56
		Project no. : **01TT26**
	Location: **CBD - Christchurch City**	CPT no. : **CBD-58a** 2/14

CPTask V1.31

Figure A6-2