Haast Courts-Block I - Garages 1-10 + 5A
BU 0792-009 EQ2

Detailed Engineering Evaluation

Quantitative Report

Version FINAL

43 Haast Street, Linwood

INFRASTRUCTURE | MINING & INDUSTRY | DEFENCE | PROPERTY & BUILDINGS | ENVIRONMENT

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43 Haast Street, Linwood

Christchurch City Council

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Date

20th December 2012

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Quantitative Report Summary

Haast Courts-Block I - Garages 1-10 + 5A

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

43 Haast Street, Linwood

Background

This is a summary of the Quantitative report for the building structure, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on 26 November 2012 and available drawings itemised in 5.2.

Building Description

Haast Courts Block I is located at 43 Haast Street, Linwood, Christchurch. The original buildings were constructed in 1983 along with residential Blocks A, B, C and D and garage Blocks J and K. The 1993 addition of garage 6A connected two previously separate buildings into Block I. The use of the Block I is parking and storage.

The main structural components are reinforced precast concrete panels. The flat roof consists of lightweight corrugated metal sheeting on timber roof joists spanning between the reinforced precast concrete partition walls. The reinforced precast concrete wall panels are supported on reinforced concrete strip footings with the floor being formed by a reinforced concrete slab on grade. The roof construction of the additional garage matches that of the original construction, however the external walls are formed from timber frame with a lightweight external cladding.

Key Damage Observed

There was minor cracking noted to reinforced concrete floor and wall elements, however it is unclear whether this damage is a result of the recent seismic activity.

Building Capacity Assessment

Based on the results of the quantitative assessment the building scored 55% NBS. Therefore the building is Earthquake Risk. This value has been assessed using the structural systems identified and confirmed in the structure during visual inspections. Adequate foundations to resist overturning or nominal diaphragm action of the roof structure would increase this score, however these mechanisms could not be confirmed.

Recommendations

No further action is required to comply with the building code, however given that the building is Earthquake Risk, the development of strengthening concepts to strengthen the building to a minimum of 67% NBS is recommended.

1. Background

GHD has been engaged by Christchurch City Council (CCC) to undertake a detailed engineering evaluation of Toilets Marshland Reserve.

This report is a Quantitative Assessment of the building structure, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011.

2. Compliance

This section contains a brief summary of the requirements of the various statutes and authorities that control activities in relation to buildings in Christchurch at present.

2.1 Canterbury Earthquake Recovery Authority (CERA)

CERA was established on 28 March 2011 to take control of the recovery of Christchurch using powers established by the Canterbury Earthquake Recovery Act enacted on 18 April 2011. This act gives the Chief Executive Officer of CERA wide powers in relation to building safety, demolition and repair. Two relevant sections are:

Section 38 – Works

This section outlines a process in which the chief executive can give notice that a building is to be demolished and if the owner does not carry out the demolition, the chief executive can commission the demolition and recover the costs from the owner or by placing a charge on the owners' land.

Section 51 – Requiring Structural Survey

This section enables the chief executive to require a building owner, insurer or mortgagee carry out a full structural survey before the building is re-occupied.

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

The qualitative assessment is a desk-top and site inspection assessment. It is based on a thorough visual inspection of the building coupled with a review of available documentation such as drawings and specifications. The quantitative assessment involves analytical calculation of the buildings strength and may require non-destructive or destructive material testing, geotechnical testing and intrusive investigation.

It is anticipated that factors determining the extent of evaluation and strengthening level required will include:

- The importance level and occupancy of the building
- The placard status and amount of damage
- The age and structural type of the building
- Consideration of any critical structural weaknesses
- The extent of any earthquake damage

2.2 Building Act

Several sections of the Building Act are relevant when considering structural requirements:

Section 112 – Alterations

This section requires that an existing building complies with the relevant sections of the Building Code to at least the extent that it did prior to any alteration. This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

Section 115 – Change of Use

This section requires that the territorial authority (in this case Christchurch City Council (CCC)) be satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'. Regarding seismic capacity 'as near as reasonably practicable' has previously been interpreted by CCC as achieving a minimum of 67% NBS however where practical achieving 100% NBS is desirable. The New Zealand Society for Earthquake Engineering (NZSEE) recommend a minimum of 67% NBS.

2.2.1 Section 121 – Dangerous Buildings

The definition of dangerous building in the Act was extended by the Canterbury Earthquake (Building Act) Order 2010, and it now defines a building as dangerous if:

- In the ordinary course of events (excluding the occurrence of an earthquake), the building is likely to cause injury or death or damage to other property; or
- In the event of fire, injury or death to any persons in the building or on other property is likely because of fire hazard or the occupancy of the building; or
- There is a risk that the building could collapse or otherwise cause injury or death as a result of earthquake shaking that is less than a 'moderate earthquake' (refer to Section 122 below); or
- There is a risk that other property could collapse or otherwise cause injury or death; or
- A territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

Section 122 – Earthquake Prone Buildings

This section defines a building as earthquake prone if its ultimate capacity would be exceeded in a 'moderate earthquake' and it would be likely to collapse causing injury or death, or damage to other property. A moderate earthquake is defined by the building regulations as one that would generate ground shaking 33% of the shaking used to design an equivalent new building.

Section 124 – Powers of Territorial Authorities

This section gives the territorial authority the power to require strengthening work within specified timeframes or to close and prevent occupancy to any building defined as dangerous or earthquake prone.

Section 131 – Earthquake Prone Building Policy

This section requires the territorial authority to adopt a specific policy for earthquake prone, dangerous and insanitary buildings.

2.3 Christchurch City Council Policy

Christchurch City Council adopted their Earthquake Prone, Dangerous and Insanitary Building Policy in 2006. This policy was amended immediately following the Darfield Earthquake of the 4th September 2010.

The 2010 amendment includes the following:

- A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
- A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;
- A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
- Repair works for buildings damaged by earthquakes will be required to comply with the above.

The council has stated their willingness to consider retrofit proposals on a case by case basis, considering the economic impact of such a retrofit.

We anticipate that any building with a capacity of less than 33% NBS (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67% NBS of new building standard as recommended by the Policy.

If strengthening works are undertaken, a building consent will be required. A requirement of the consent will require upgrade of the building to comply 'as near as is reasonably practicable' with:

- The accessibility requirements of the Building Code.
- The fire requirements of the Building Code. This is likely to require a fire report to be submitted with the building consent application.

2.4 Building Code

The building code outlines performance standards for buildings and the Building Act requires that all new buildings comply with this code. Compliance Documents published by The Department of Building and Housing can be used to demonstrate compliance with the Building Code.

After the February Earthquake, on 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

- Hazard Factor increased from 0.22 to 0.3 (36% increase in the basic seismic design load)
- Serviceability Return Period Factor increased from 0.25 to 0.33 (80% increase in the serviceability design loads when combined with the Hazard Factor increase)

The increase in the above factors has resulted in a reduction in the level of compliance of an existing building relative to a new building despite the capacity of the existing building not changing.

3. Earthquake Resistance Standards

For this assessment, the building's earthquake resistance is compared with the current New Zealand Building Code requirements for a new building constructed on the site. This is expressed as a percentage of new building standard (%NBS). The new building standard load requirements have been determined in accordance with the current earthquake loading standard (NZS 1170.5:2004 Structural design actions - Earthquake actions - New Zealand).

The likely capacity of this building has been derived in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) guidelines 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes' (AISPBE), 2006. These guidelines provide an Initial Evaluation Procedure that assesses a buildings capacity based on a comparison of loading codes from when the building was designed and currently. It is a quick high-level procedure that can be used when undertaking a Qualitative analysis of a building. The guidelines also provide guidance on calculating a modified Ultimate Limit State capacity of the building which is much more accurate and can be used when undertaking a Quantitative analysis.

The New Zealand Society for Earthquake Engineering has proposed a way for classifying earthquake risk for existing buildings in terms of %NBS and this is shown in Figure 3.1 below.

Description	Grade	Risk	%NBS	Existing Building Structural Performance	Improvement of Structural Performance	
					Legal Requirement	NZSEE Recommendation
Low Risk Building	A or B	Low	Above 67	Acceptable (improvement may be desirable)	The Building Act sets no required level of structural improvement (unless change in use) This is for each TA to decide. Improvement is not limited to 34%NBS.	100%NBS desirable. Improvement should achieve at least 67%NBS
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally. Improvement recommended		Not recommended. Acceptable only in exceptional circumstances
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement	Unacceptable	Unacceptable

Figure 3.1NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE

Figure 3.2 compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% risk of exceedance in 50 years (i.e. 0.2% in the next year). It is noted that the current seismic risk in Christchurch results in a 6% risk of exceedance in the next year.

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

Figure 3.2 %NBS compared to relative risk of failure

4. Building Description

4.1 General

Haast Courts Block I is located at 43 Haast Street, Linwood, Christchurch. The original buildings were constructed in 1983 along with residential Blocks A, B, C and D and garage Blocks J and K. The 1993 addition of garage 6A connected two previously separate buildings into Block I. The use of the building is parking and storage.

The main structural components are reinforced precast concrete panels. The flat roof consists of lightweight corrugated metal sheeting on timber roof joists spanning between the reinforced precast concrete partition walls. The reinforced precast concrete wall panels are supported on reinforced concrete strip footings with the floor being formed by a reinforced concrete slab on grade. The roof construction of the additional garage matches that of the original construction, however the external walls are formed from timber frame walls with a lightweight external cladding.

The building is approximately 6.3m wide by 35m long, and stands 2.3m high with plan area of approximately 220m^2. The nearest building to Hasst Courts Block I is Haast Courts Block H, approximately 2m to the north. The nearest waterway to the flat site is the Avon River, approximately 250m to the north.

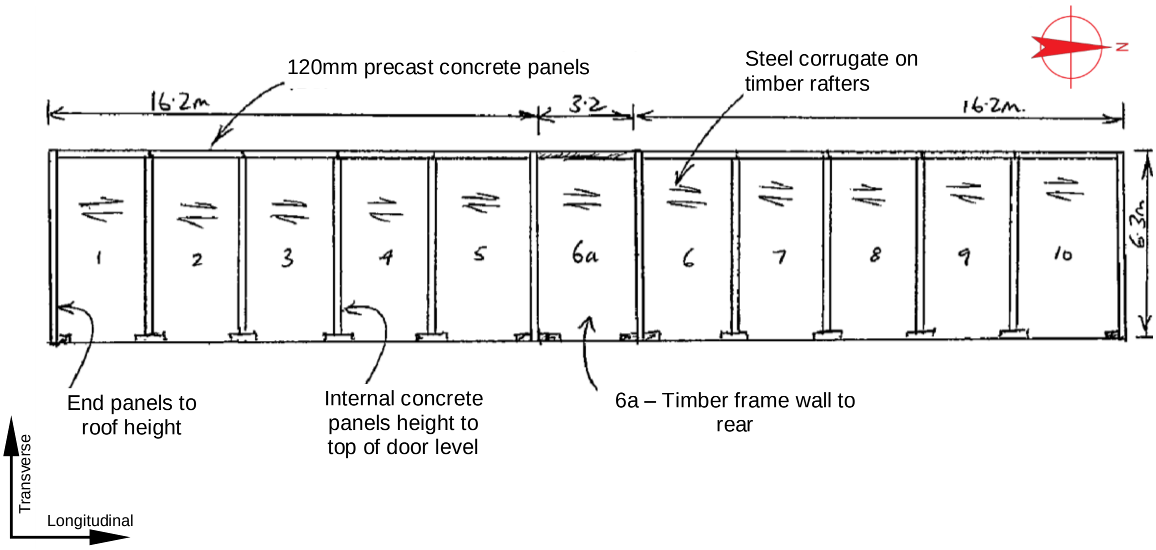


Figure 4.1 Plan of Original Structure

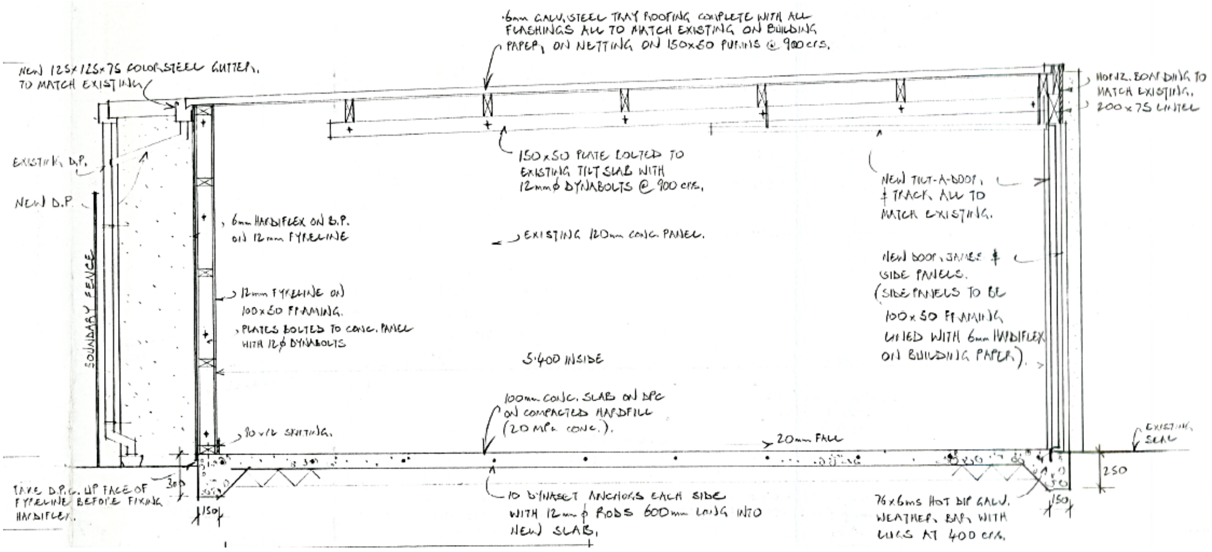


Figure 4.2 Section of Additional Garage

4.2 Gravity Load Resisting System

Gravity roof loads are transferred through the corrugate metal sheeting to the timber roof joists which span between the partition walls. These reinforced precast concrete partition wall panels support the gravity roof load and transfer them down to the reinforced concrete strip footing where they are distributed into the ground beneath. The gravity floor loads are transferred directly to the ground beneath by the reinforced concrete floor slab on grade.

4.3 Lateral Load Resisting System

Lateral loads are resisted primarily by the panel action of the reinforced concrete precast panels.

In the transverse direction, the lateral roof loads span horizontally between the in-plane partition walls, which transfer the lateral roof loads to the foundations by panel action. The foundations then transfer the lateral loads into the ground beneath. The rear walls, which are subject to out-of-plane loading, spans between the ground and cleats near the panel corners at roof level. The cleats form connections to the two adjoining orthogonal partition walls, which transfer the lateral loads to these in-plane partition walls. Again the lateral load in the in-plane partition walls are resisted by panel action.

The partition wall panels are connected by cleats to the rear wall line and to narrow piers along the garage entrance line. The narrow dimensions of the piers render them ineffective as lateral restraints in the longitudinal direction, thus the partition wall panels are only effectively restrained against lateral out-of-plane loading by the single cleat near roof level at the rear wall line and at the ground level. Lateral out-of-plane loading on the partition require it to span in an inefficient manner from the rear wall along its length. The lateral loads transferred to the rear wall by the cleat connection are resisted by the panel action of the reinforced precast concrete wall.

Other lateral load resisting systems in the longitudinal direction may provide restraint to the partition walls, however their dependability is unclear. The partition walls may cantilever from the foundation level provided there is enough base fixity to restrain the lateral loads. There may also be a nominal diaphragm action of the corrugate metal sheeting to transfer lateral partition wall loads through the roof structure to the rear wall. As the lateral load resisting element of the rear wall is offset from the seismic demand of the overall structure, there will be a resultant torsion in the structure.

The lateral loads in the additional garage timber frame will be restrained by the stiffer adjacent reinforced precast concrete panels similar to above.

5. Assessment

5.1 Site Inspection

An inspection of the building was undertaken on the 26th of November 2012. Both the interior and exterior of the building were inspected. The main structural components of the building were all able to be viewed.

The inspection consisted of observing the building to determine the structural systems and likely behaviours of the building during earthquake. The site was assessed for damage, including observing the ground condition, checking for damage areas where damage would be expected for the structure type observed and noting general damage observed throughout the building in both structural and non-structural elements.

A Hilti PS 200 Ferroscan was used to confirm the position, depth and diameter of the reinforcement in the precast concrete panels. This scanning equipment using electro-magnetic fields allowed for the determination of the capacity of the various walls in the building. In the case of conflicting results, the most conservative bar diameter was chosen for the capacity calculations.

5.2 Available Drawings

The construction drawings of the additional structure were made available.

All drawings are attached as Appendix B.

6. Damage Assessment

6.1 Surrounding Buildings

Hornby Courts Block I is located in a residential complex with 8 other residential blocks and 2 blocks of garages. Reportedly some of the older masonry residential units had suffered damage with the collapse of a portion of the gable end of Block G being the most significant.

6.2 Residual Displacements and General Observations

There were no residual displacements identified in the structure. Minor cracking was identified in the reinforced concrete floor slab and precast concrete wall panels, however it is unclear whether this damage is earthquake related. The cracking will not affect structural performance of the elements.

6.3 Ground Damage

No ground damage was observed during our inspection of the site.

7. Structural Analysis

7.1 Seismic Parameters

Earthquake loads shall be calculated using New Zealand Code.

◗ Site Classification	D
◗ Seismic Zone factor (Z)	
(Table 3.3, NZS 1170.5:2004 and NZBC Clause B1 Structure)	0.30 (Christchurch)
◗ Annual Probability of Exceedance	
(Table 3.3, NZS 1170.0:2002)	1/100 (ULS) Importance Level 1
◗ Return Period Factor (Ru)	
(Table 3.5, NZS 1170.5:2004)	0.5 (ULS)
◗ Ductility Factor (μ)	1.25
◗ Ductility Scaling Factor (k_μ)	1.14
◗ Performance Factor (Sp), based on NZS 3.1.0.1	0.925
◗ Gravitational Constant (g)	9.81 m/s^2

An increased Z factor of 0.3 for Christchurch has been used in line with recommendations from the Department of Building and Housing recommendations resulting in a reduced % NBS score.

7.2 Equivalent Static Method

Equivalent Static forces were calculated in accordance with NZS 1170.5:2004. A ductility factor of 1.25 has been assumed given the age and partially filled construction used. The structure is expected to have nominally ductile behavior given the lightly reinforced partially filled concrete masonry construction.

The elastic site hazard spectrum for horizontal loading:

$C(T_1)=C_h\cdot Z\cdot R\cdot N(T,D)$

$C_h=3.0$ – Value from 3.1 table for the period (T=0.4s)

Z=0.3 – Hazard factor determined from the table 3.3 (NZS 1170.5:2004)

R=0.5– Return period factor determined from the table 3.5 (NZS 1170.5:2004)

N (T,D) = 1.0 – Near fault factor- clause 3.1.6. (NZS 1170.5:2004)

$C(T_1)= 3.0\cdot 0.3\cdot 0.5\cdot 1.0 = 0.45$

The horizontal design action coefficient:

$$C_d(T_1)=\frac{C(T_1)\cdot S_p}{k_\mu}=\frac{0.45\cdot 0.925}{1.14}=0.365$$

7.3 Dependable Capacity

7.3.1 In-Plane Shear Capacity of Reinforced Concrete Wall

The in-plane shear capacity of the reinforced concrete wall is the combination of the shear capacity of the concrete and the shear capacity of the transverse reinforcement as detailed in NZS 3101:2006.

$$V^* = \emptyset V_c + \emptyset V_s$$

Concrete shear capacity is derived from

$$V_c = v_c A_{cv}$$

$$A_{cv} = td$$

Where

v_c = concrete shear strength calculated in accordance with Clause 11.3.10.3.5

t = wall thickness

d = wall length as detailed in Clause 11.3.10.3.3

Reinforcement shear capacity is derived from

$$V_s = A_v f_{yt} \frac{d}{s_2}$$

Where

Av = shear reinforcement area

fyt = strength of reinforcement

d = wall length as detailed in Clause 11.3.10.3.3

S2 = centres of reinforcement

7.3.2 Reinforced Concrete Moment Capacity

The following method was used to calculate the moment capacity of the reinforced concrete panels.

$$\emptyset M_n = \emptyset \left(\frac{t}{2} - \frac{a}{2}\right) f_{yt} A_s$$

$$a = {A_s f_{yt}}\big/{\emptyset A_c f'_m}$$

Where

t = wall thickness

A_s = area steel

A_c = area of concrete

f'_m = masonry strength

7.3.3 Cleat Connection of Wall Panels

The connection of wall panels was checked. The expansion bolts used to connect the cleat angles to each wall was found to be the controlling capacity of the connection. The bolt connection was checked for both conditions of shear or tension.

8. Geotechnical Consideration

The site is located in the suburb of Linwood, in eastern Christchurch. It is relatively flat, with an elevation in the order of 5m above mean sea level. The site is approximately 250m south of the Avon River, and 6km west of the coast (Pegasus Bay).

8.1 Public Information on Ground Conditions

8.1.1 Published Geology

The geological map of the area[1] indicates that the site is underlain by:

- Holocene alluvial soils of the Yaldhurst Member, sub-group of the Springston Formation, comprising alluvial sand and silt overbank deposits.

Figure 72 (Brown & Weeber) indicates that groundwater levels are likely to be within 1m of the surface.

8.1.2 Environmental Canterbury Logs

Information from Environment Canterbury (ECan) indicates that one borehole with a lithographic log (Ref. M35/2119) is located 150m north of the site. This indicates that the area is silt/clay to 1.8m bgl, overlying gravels to ~10m bgl, which is shown to be underlain by alternating layers of sand/clay, and gravels.

It should be noted that the boreholes were sunk for groundwater extraction and not for geotechnical purposes. Therefore, the amount of material recovered and available for interpretation and recording will have been variable at best and may not be representative. The logs have been written by the well driller and not a geotechnical professional or to a standard. In addition strength data is not recorded.

8.1.3 EQC Geotechnical Investigations

The Earthquake Commission has undertaken geotechnical testing in the area of the site. Information pertaining to this investigation is included in the Tonkin & Taylor Report for Linwood[2]. Three investigation points were considered, as summarised below in Table 8.1.

Table 8-1 ECan Borehole Summary

Bore Name	Dist. From Site	Depth (m)	Log Summary
CPT LWD 02	300m W	0 – 4.5	Soft Silts and Clays
		4.5 – 24.5	Dense Sand
CPT LWD 03	120m N	0 – 2.0	Loose Sands
		2.0 – 2.5	Soft Silt and Clay

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

[2] Tonkin & Taylor Ltd (2011): Christchurch Earthquake Recovery, *Geotechnical Factual Report, Linwood*

Bore Name	Dist. From Site	Depth (m)	Log Summary
		2.5 – 4.0	Dense Sand
CPT LWD 17	400m SW	0 – 5.0	Silts and Clays
		5.0 – 26.0	Sand

Initial observations of the CPT results indicate the soils are composed predominantly of soft silt and clay underlain by dense sands.

8.1.4 Land Zoning

Canterbury Earthquake Recovery Authority (CERA) indicates the site is within the Green Zone, meaning repair and rebuild may take place.

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

Categorised residential properties adjacent to the site are indicated to be TC2 (yellow). This means that minor to moderate land damage from liquefaction is expected in future significant earthquakes.

8.1.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows no signs of liquefaction outside the building footprints or adjacent to the site, as shown in Figure 8.1.

Figure 8.1 Post February 2011 Earthquake Aerial Photography[3]

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

8.2 Seismicity

8.2.1 Nearby Faults

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

Table 8-2 Summary of Known Active Faults[4,5]

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	120 km	NW	~8.3	~300 years
Greendale (2010) Fault	23 km	W	7.1	~15,000 years
Hope Fault	110 km	N	7.2~7.5	120~200 years
Kelly Fault	110 km	NW	7.2	150 years
Porters Pass Fault	60 km	NW	7.0	1100 years

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

8.2.2 Ground Shaking Hazard

This seismic activity has produced earthquakes of Magnitude 6.3 with peak ground accelerations (PGA) up to twice the acceleration due to gravity (2g) in some parts of the city. This has resulted in widespread liquefaction throughout Christchurch.

New Zealand Standard NZS 1170.5:2004 quantifies the Seismic Hazard factor for Christchurch as 0.30, being in a moderate to high earthquake zone. This value has been provisionally upgraded recently (from 0.22) to reflect the seismicity hazard observed in the earthquakes since 4 September 2010.

8.3 Field Investigations

In order to further understand the ground conditions at the site, intrusive testing comprising two piezocone CPT investigations were conducted at the site on 28 June 2012. The locations of the tests are indicated on Figure 8.2 below.

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

[5] GNS Active Faults Database

Figure 8.2 Aerial Photograph depicting CPT Investigation Locations[3]

The coordinates of the test locations are tabulated in Table 8-3.

Table 8-3 Coordinates of Investigation Locations

Investigation	Depth (m bgl)	Easting (NZMG)	Northing (NZMG)
CPT 1	23.07	2482216	5742185
CPT 2	27.89	2482259	5742157

The CPT investigations were undertaken by McMillans Drilling Ltd on 28 June 2012, typically to a target depth of 20m below ground level. However, testing was continued to depths of 23m bgl and 27.9m bgl due to the presence of soft silts and loose sands at 20m. Please refer to Appendix D for CPT logs.

8.4 Ground Conditions Encountered

Interpretation of output graphs6 from the investigation showing Cone Tip Resistance (qc), Friction Ratio (Fr), Inferred Lithology and Inferred Liquefaction Potential are summarised in Table 8.4 and Table 8-5.

A summary of the lithology inferred from the CPT results is outlined in Table 8.4 below.

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

Table 8.4 Summary of CPT-Inferred Lithology

Depth (m)	Inferred Lithology	Cone Tip Resistance q_c (MPa)	Friction Ratio Fr (%)	Relative Density Dr (%)
0 – 6.5	SILT mixtures (with sand lenses)	1 to 8	1 to 6	(Su ≥ 30kPa)
6.5 – 10	SANDS	14 to 25	0.5	80 to 100
10 – 16	SANDS	2 to 18	0.5 to 2	50 to 80
16 – 19	SANDS	12 to 30	0.5	70 to 90
19 – 27	Layers of: • SILT mixtures; and, • SANDS	 1 15 to 30	 ~3 0.5	 (Su ≥ 50kPa) 60 to 80

From the results above, the ground conditions at the site are understood to be predominantly silts to 6.5m, overlying sands to 19m, and layers of sands and silts to depth.

This is considered consistent with the published geology and EQC investigations for the area, from the desktop information reviewed in Sections 8.1.1 and 8.1.3.

Please refer to Appendix D for further detail.

During the CPT investigations, groundwater was inferred to be at 1.2m below ground level. This is slightly lower than, but still consistent with, the inference by Brown & Weeber of groundwater being within 1m of the surface. It is also consistent with site levels in relation to the Avon River.

8.5 Liquefaction Analysis

As the subsoils encountered consisted of sand and silt beneath the site, a more comprehensive liquefaction assessment has been undertaken.

8.5.1 Parameters used in Analysis

Assumptions made for the analysis process are as follows:

- D_{50} particle sizes for the site soil (sands) from CPT soil analysis;
- Importance Level 2, post seismic event (50-year design life); and,
- Peak ground acceleration (PGA) 0.35g.

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

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

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

This typically gave values ranging between 16 and 20 kN/m3 (saturated).

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

8.5.2 Results of Liquefaction Analysis

The results of the liquefaction analysis, as outlined in Table 8-5, indicate that depths to 6.5m, and 10m to 19m, are considered highly liquefiable.

Table 8-5 Summary of Liquefaction Susceptibility

Depth (m)	Inferred Lithology	Triggering Factor F_L	Liquefaction Susceptibility[10]
0 – 6.5	SILT mixtures (with sand lenses)	0.3 to 0.8	High (Bands)
6.5 – 10	SANDS	>> 1	Negligible
10 – 16	SANDS	0.4 to 2	Severe
16 – 19	SANDS	0.3 to 1	High (Bands)
19 – 27	Layers of: • SILT mixtures; and, • SANDS.	 - 0.5 to 1.8	 *Not Liquefiable* High

(Bands) means that only some bands of soil are indicated to be susceptible within this layer.

While layers at 19m to 27m are indicated to be highly susceptible by the analysis, the severity of liquefaction at this depth is considered significantly reduced due to the greater levels of vertical overburden stress.

Settlement estimates for the CPT points are between 150mm and 270mm for ULS conditions.

Please refer to Appendix D for further details.

[8] Robertson P.K. & Wride C.E. (1998): *Evaluating cyclic liquefaction potential using the cone penetration test.* Canadian Geotechnical Journal, 35: pp. 442–459.

[9] Cubrinovski M., McManus K.J., Pender M.J., McVerry G., Sinclair T., Matuschka T., Simpson K., Clayton P., Jury R. (2010): Geotechnical earthquake engineering practice: *Module 1 – Guideline for the identification, assessment and mitigation of liquefaction hazards*. NZ Geotechnical Society

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

8.6 Interpretation of Ground Conditions

8.6.1 Liquefaction Assessment

Overall, the site is considered to be highly susceptible to liquefaction. This is based on:

- Limited evidence of liquefaction at the surface in the post-earthquake aerial photography;
- Estimated settlements from the CPT results (150mm to 270mm) are well in excess of the 100mm limit for TC2 classification, indicating the site should be considered in line with TC3 guidelines; and,
- The layers of 1m to 6m and 9m to 17m are indicated to be highly susceptible, as outlined in Table 8-5.

8.6.2 Slope Failure and/or Rockfall Potential

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

8.6.3 Foundation Recommendations

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

- The soil class of **D** (in accordance with NZS 1170.5:2004) recommended in Section 8 of the Qualitative DEE/IEP is still believed to be appropriate; and,
- Any rebuilt foundations (or proposed new structures) be undertaken in accordance with DBH's guidelines for **TC3** land, due to the high levels of estimated settlement.

As the buildings this report pertains to are garages, and not inhabited structures, foundations need not meet the same criteria. A floor slab on a gravel raft (hardfill) would be appropriate; this should contain reinforcing steel, and be designed in accordance with the minimum criteria outlined in the CCC and MBIE design guidance.

9. Results of Analysis

The structure was considered for loads in both orthogonal directions. The capacities of each element for each loading direction is given in Table 6 and Table 7 below.

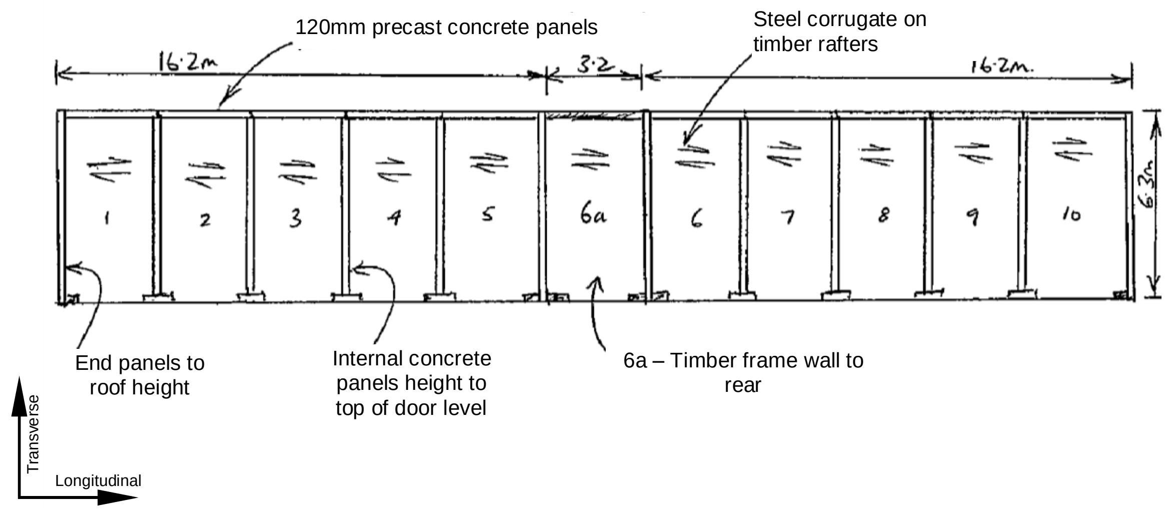


Figure 9.1 Layout of Structure.

The critical structural element was found to be the partition wall for loading in the longitudinal direction. This element was found to have a %NBS of 55% NBS. The undesirable support conditions for the partition walls, whereby no support above ground level on the entrance line is provided, is the reason for the poor performance of these elements.

Table 6 %NBS of Structural Elements for Transverse Loading

Element	% NBS
Partition Walls	>100%
Rear Walls	>100%
Cleat Connection	>100%

Table 7 %NBS for Structural Elements for Longitudinal Loading

Element	% NBS
Partition Walls	55%
Rear Walls	>100%
Cleat Connection	>100%

9.1 Structural Systems

The NBS% values in Table 6 and Table 7 are calculated using no base fixity at the foundation level or any restraint provided by the roof structure. In reality, there will be some base fixity provided by the foundations to the reinforced concrete walls. There will also be a nominal restraint provided by the roof structure which will also potentially improve seismic performance. The nominal diaphragm action of the roof structure will cause torsion in the building, with the remaining elements in the structure capable of resisting these forces. Given the nature of the roof structure, the bracing capacity of the diaphragm is indeterminable.

9.2 Discussion of Results

The results obtained from the analysis are generally consistent with those expected for a building of this size, age and construction type, founded on Class D soils.

Haast Courts – Block I was built in 1983 and was likely designed in accordance with the loading standard current at the time, NZS 4203:1976. The design loads used in accordance with this standard are likely to have been less than those required by the current loading standard, however given the seismic demand is low relative to element capacities, it performs well in most instances. The %NBS controlling the overall building score is derived from the partition walls for loading in the longitudinal direction. The support to these panels are minimal, due to the absence of a reliable roof diaphragm and orthogonal bracing elements along the front entrance line. The panel is required to span inefficiently along its length to achieve stability, hence the element's relative poor performance.

All structural elements except the partition elements have been found to have a %NBS greater than 100%. The partition walls were found to have a %NBS of 55% given the minimal support provided to the elements.

10. Conclusions and Recommendations

The building overall has been assessed as having a seismic capacity of 55% NBS and is therefore classified as being ‘Earthquake Risk’.

Currently the partition walls achieve a %NBS of 55%. No further action is required under the building code, however given that the structure is Earthquake Risk, development of strengthening concepts is recommended.

11. Limitations

11.1 General

This report has been prepared subject to the following limitations:

- Available drawings itemised in 5.2 was used in the assessment.
- The foundations of the building were unable to be inspected beyond those exposed above ground level externally.
- No level or verticality surveys have been undertaken.
- No material testing has been undertaken.

It is noted that this report has been prepared at the request of Christchurch City Council and is intended to be used for their purposes only. GHD accepts no responsibility for any other party or person who relies on the information contained in this report.

11.2 Geotechnical Limitations

The data and advice provided herein relate only to the project and structures described herein and must be reviewed by a competent geotechnical professional before being used for any other purpose. GHD Limited (GHD) accepts no responsibility for other use of the data by third parties.

Where drill hole or test pit logs, cone tests, laboratory tests, geophysical tests and similar work have been performed and recorded by others under a separate commission, the data is included and used in the form provided by others. The responsibility for the accuracy of such data remains with the issuing authority, not with GHD.

The advice tendered in this report is based on information obtained from the desk study investigation location test points and sample points. It is not warranted in respect to the conditions that may be encountered across the site other than at these locations. It is emphasised that the actual characteristics of the subsurface materials may vary significantly between adjacent test points, sample intervals and at locations other than where observations, explorations and investigations have been made. Subsurface conditions, including groundwater levels and contaminant concentrations can change in a limited time. This should be borne in mind when assessing the data.

It should be noted that because of the inherent uncertainties in subsurface evaluations, changed or unanticipated subsurface conditions may occur that could affect total project cost and/or execution. GHD does not accept responsibility for the consequences of significant variances in the conditions and the requirements for execution of the work.

The subsurface and surface earthworks, excavations and foundations should be examined by a suitably qualified and experienced Engineer who shall judge whether the revealed conditions accord with both the assumptions in this report and/or the design of the works. If they do not accord, the Engineer shall modify advice in this report and/or design of the works to accord with the circumstances that are revealed.

An understanding of the geotechnical site conditions depends on the integration of many pieces of information, some regional, some site specific, some structure specific and some experienced based. Hence this report should not be altered, amended or abbreviated, issued in part and issued incomplete in any way without prior checking and approval by GHD. GHD accepts no responsibility for any circumstances which arise from the issue of the report which have been modified in any way as outlined above.

Appendix A

Photographs

Photo 1. Eastern elevation with garage door openings.

Photo 2. Interior of Garages.

Photo 3. Minimal piers along garage door elevation.

Photo 4. Cleat connection between rear and partition walls.

Photo 5. Minor cracking in reinforced concrete ground slab.

Photo 6. Roof joists spanning between partition walls.

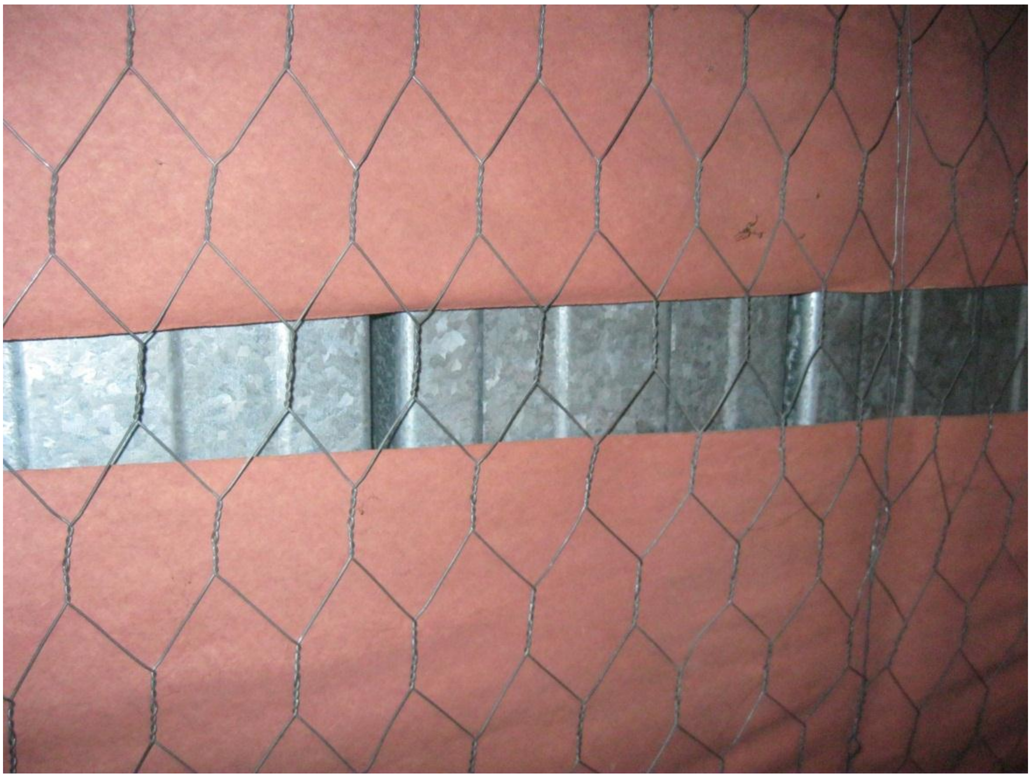
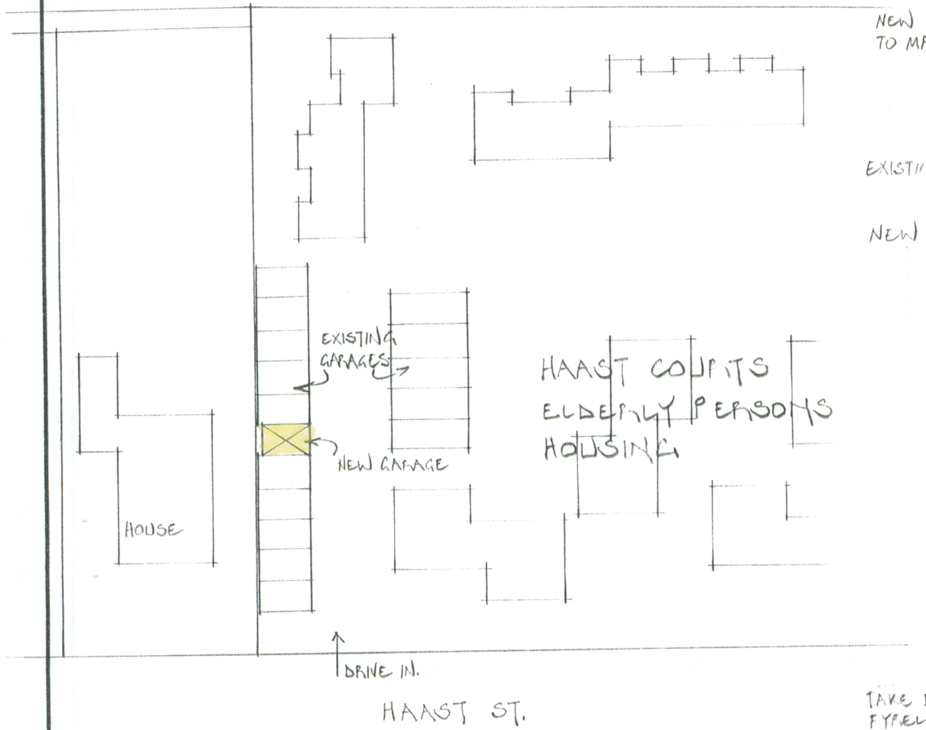


Photo 7. Corrugate metal sheets roof structure.

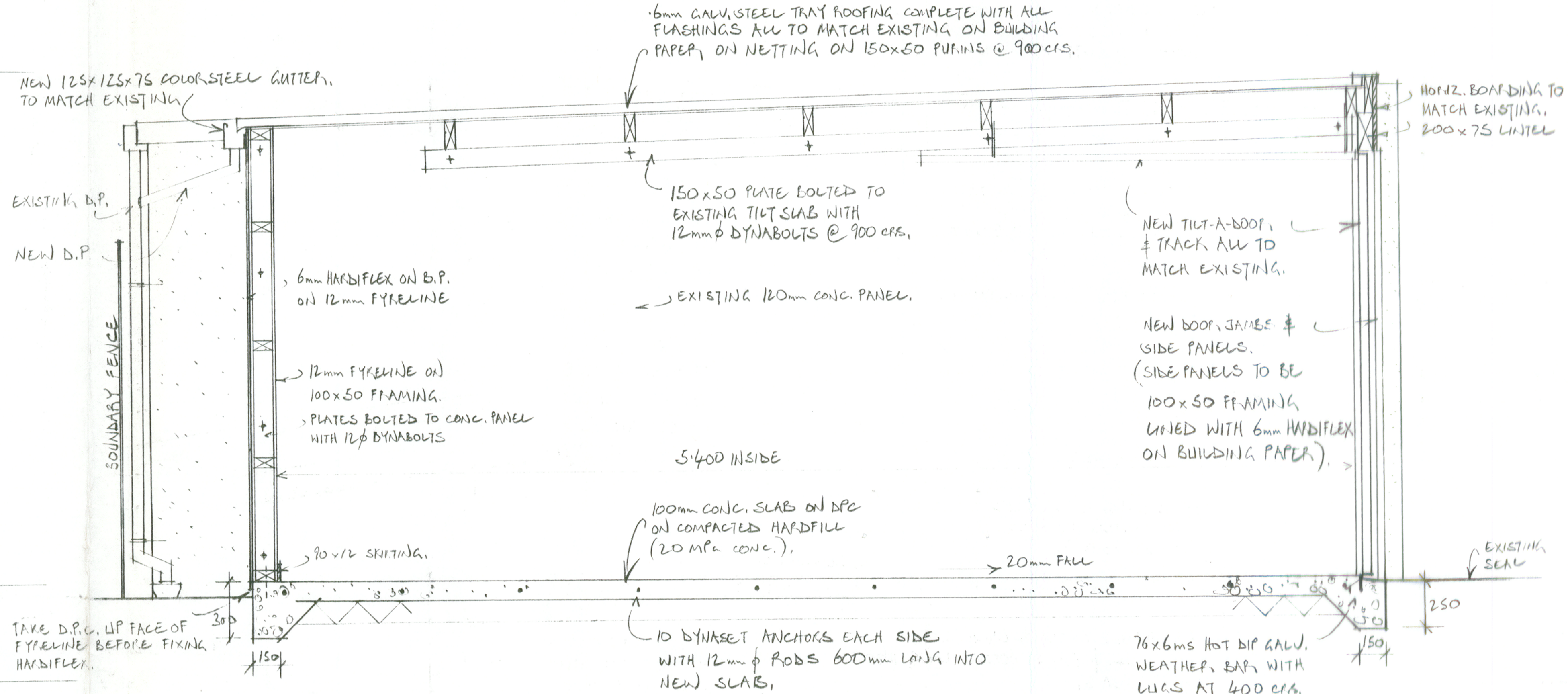
Appendix B

Existing Drawings

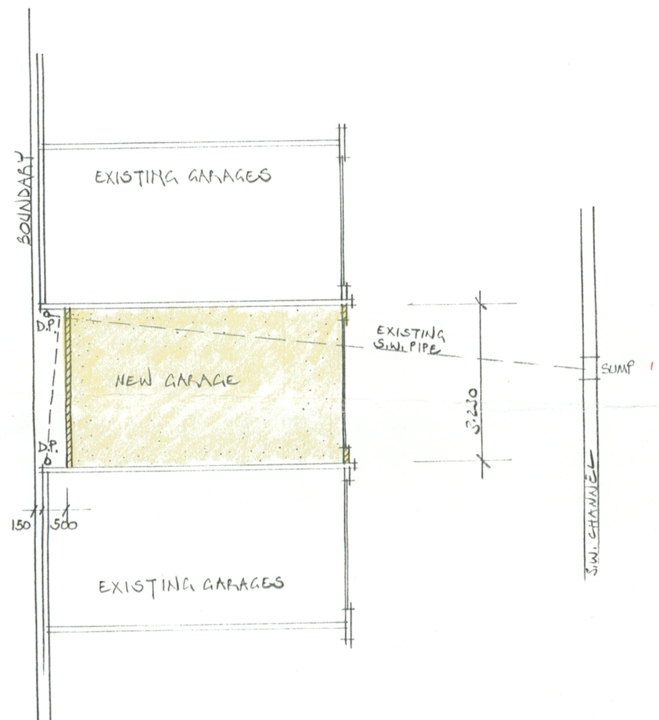
CONTRACTOR SHALL VERIFY ALL DIMENSIONS BEFORE STARTING WORK



SITE PLAN 1:500



TYPICAL SECTION 1:20



FLOOR PLAN 1:100

MAIN CONTRACTOR - ARRANGE FOR & CARRY OUT ALL OF THE WORKS AS DRAWN & TO COMPLY WITH NZS 3604 1990.
- ALLOW FOR A CONTINGENCY SUM OF $500 WHICH MAY BE REQUIRED FOR ALTERING THE EXISTING STORM WATER DRAINS. THIS WILL BE CONFIRMED ON SITE WHEN THE AREA IS EXCAVATED. ALLOW FOR SOME ADDITIONAL DIGGING IN YOUR QUOTATION FOR FINDING THE CORRECT POSITIONS OF THE S.W. PIPES.
*- THE EXISTING RUBBISH BINS, FENCE SURROUNDS & PLANTS IN THE AREA ALLOCATED FOR THE GARAGE, WILL BE REMOVED BY THE C.C.C.
- THE MAINTENANCE PERIOD IS 3 MONTHS.

PAINTING

NEW ROOF - DEGREASE/GALV. IRON PRIMER/2 COATS P.V.A. ROOF PAINT TO MATCH EXISTING.

HARDIFLEX REAR WALL & EACH SIDE OF TILT-A-DOOR - SEAL COAT/2 COAT PVA TO MATCH EX.

TIMBER DOOR JAMBS - PRIME/UNDERCOAT/2 COATS ENAMEL TO MATCH.

HORIZONTAL BOARDING - PRIME/2 COATS PVA TO MATCH EXISTING.

CONTRACT PERIOD - 3 WEEKS.

BCP 9207310

CHRISTCHURCH CITY COUNCIL
Approved Subject to the By-Laws
- 3 NOV 1992
for Engineer

CHRISTCHURCH CITY COUNCIL
CITY ARCHITECTS DIVISION : CITY WORKS & PLANNING DEPARTMENT

NEW GARAGE - 43 HAAST ST. (HAAST COURTS E.P.H.)

SCALE 1:500 1:100, 1:20	DESIGN DRAWN GAL TRACED	JOB NO 507.13	SHEET NUMBER A.1.
DATE 27-10-92		FILE NO	OF SHEETS

Appendix C

CERA Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Haast Courts Block I	Reviewer:	Stephen Lee
Building Address:	Unit: / No: 43 / Street: Haast Street	CPEng No:	1006840
	Garages 1 to 10	Company:	GHD
Legal Description:	LOT 1 DP 47661	Company project number:	513059647
		Company phone number:	(03) 3780900
GPS south:	Degrees 43 / Min 31 / Sec 39.12	Date of submission:	
GPS east:	Degrees 172 / Min 39 / Sec 21.27	Inspection Date:	26/11/12
		Revision:	
Building Unique Identifier (CCC):	BU 0792-009 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):	10.00

Building

No. of storeys above ground:	1 (single storey = 1)	Ground floor elevation (Absolute) (m):	10.00
Ground floor split?	no	Ground floor elevation above ground (m):	0.00
Storeys below ground	0		
Foundation type:	strip footings	if Foundation type is other, describe:	Strip footings with concrete floor slab
Building height (m):	2.25	height from ground to level of uppermost seismic mass (for IEP only) (m):	2.25
Floor footprint area (approx):	213		
Age of Building (years):	30	Date of design:	1976-1992
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	parking	Brief strengthening description:	
Use (upper floors):	parking		
Use notes (if required):	Garages		
Importance level (to NZS1170.5):	IL1		

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	
Floors:	concrete flat slab	slab thickness (mm)	
Beams:	none	overall depth x width (mm x mm)	
Columns:			
Walls:	load bearing concrete	#N/A	

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	concrete shear wall		note total length of wall at ground (m):	35.6
Ductility assumed, μ:	1.25		wall thickness (m):	0.12
Period along:	0.40	0.00 from parameters in sheet	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	concrete shear wall		note total length of wall at ground (m):	75.6
Ductility assumed, μ:	1.25		wall thickness (m):	0.12
Period across:	0.40	0.00 from parameters in sheet	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

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

Non-structural elements

Stairs:			
Wall cladding:	exposed structure	describe	Painted shear walls
Roof Cladding:	Other (specify)	describe	Corrugated Steel
Glazing:			
Ceilings:	none		
Services(list):			

Available documentation

Architectural	partial	original designer name/date	Christchurch City Cuncil, 1992
Structural	none	original designer name/date	
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:	Good overall	Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:			
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):	Insignificant		
Across	Damage ratio:	0%		
	Describe (summary):	Insignificant		
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:		Describe:	
	Building Consent required:		Describe:	
	Interim occupancy recommendations:		Describe:	
Along	Assessed %NBS before:	55%	If IEP not used, please detail assessment methodology:	Detailed Calculation
	Assessed %NBS after:	55%		
Across	Assessed %NBS before:	100%		
	Assessed %NBS after:	100%		

Appendix D

Geotech

CPT ANALYSIS NOTES

Soil Type

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

- sand (and gravel)
- silt-sand
- silt
- clay-silt
- clay
- peat

Liquefaction Screening

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

- high susceptibility
- medium susceptibility
- low susceptibility

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

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

Low susceptibility is all other cases.

Relative Density (D_R)

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

Undrained Shear Strength (S_U)

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

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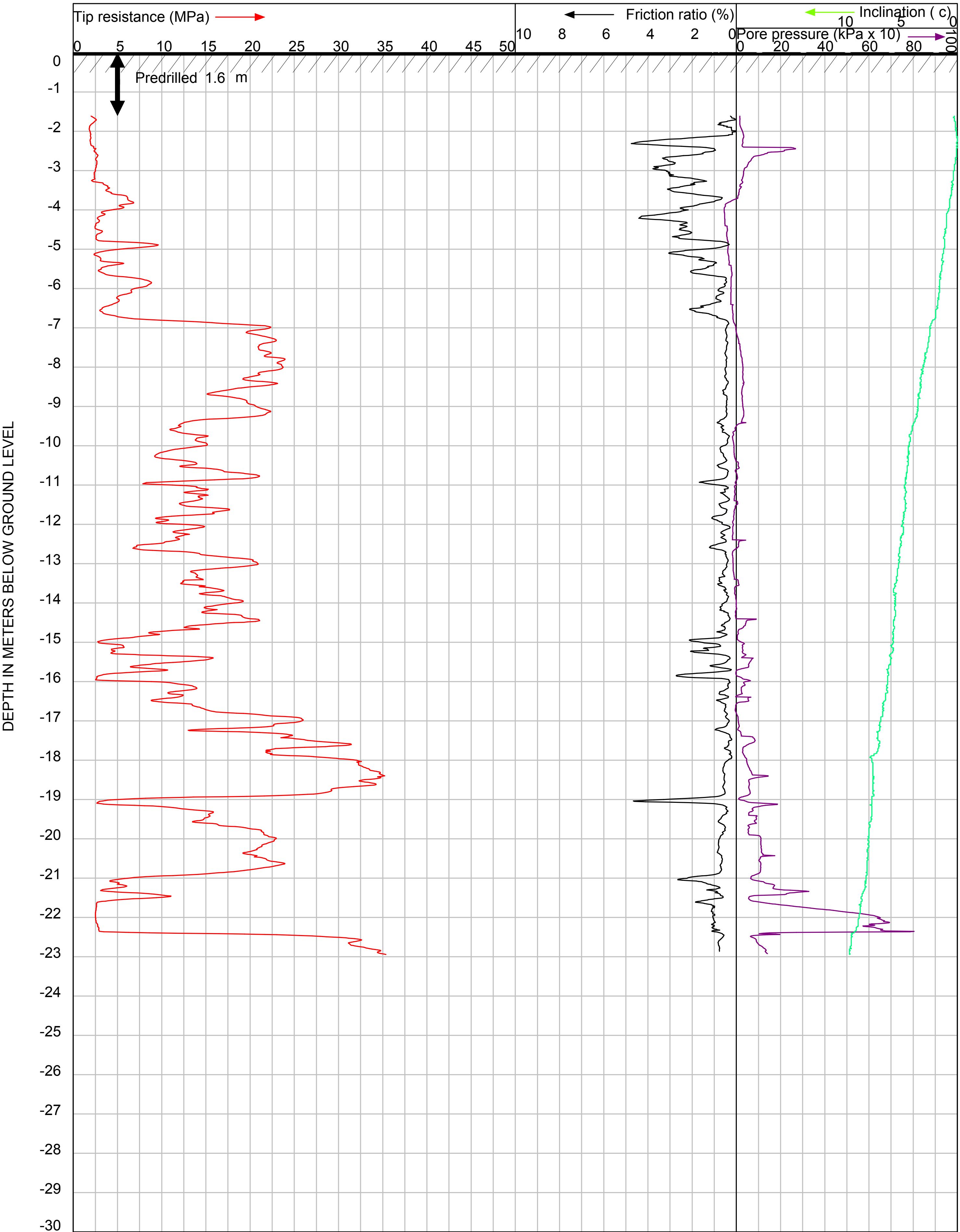
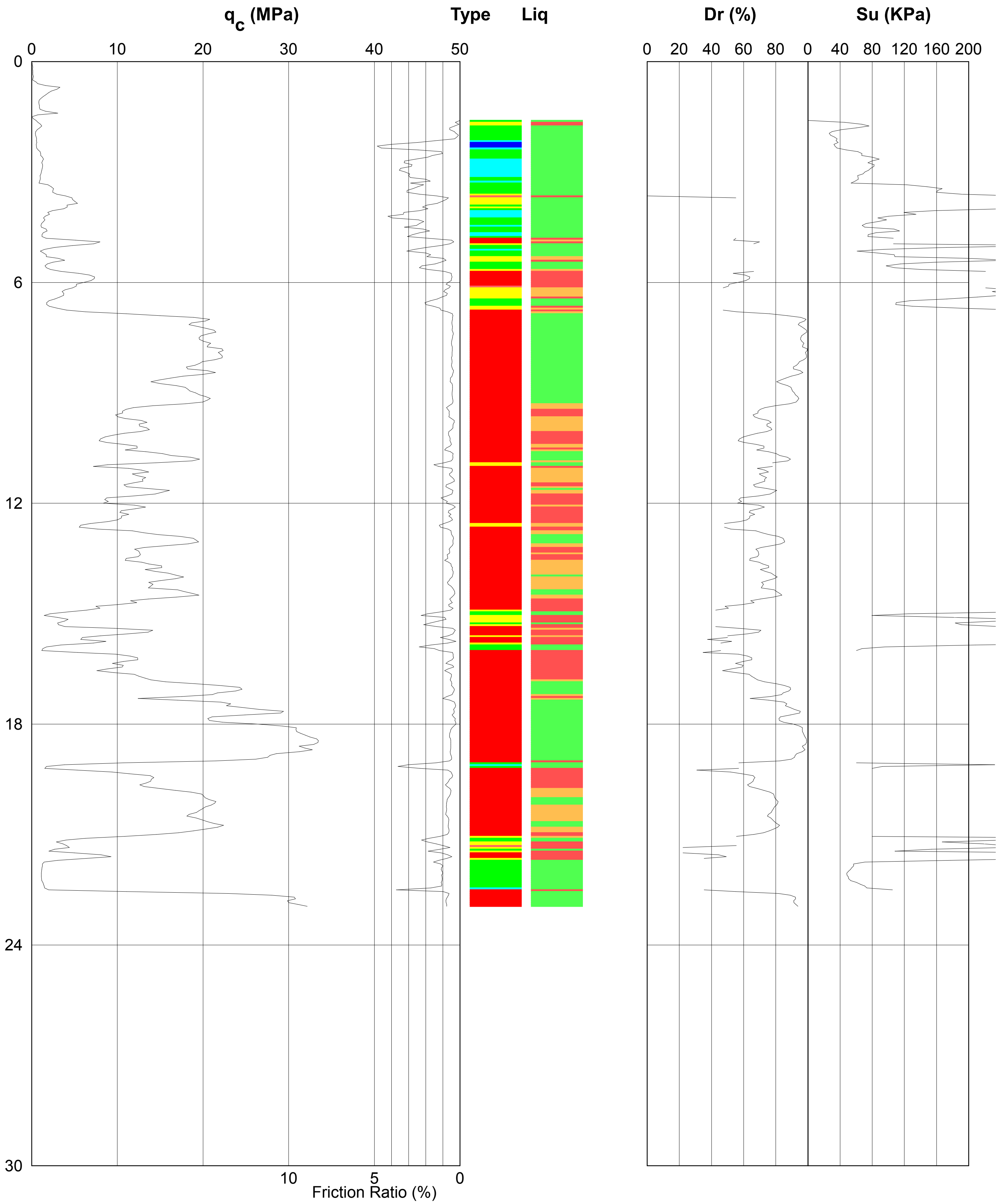


CLIENT : GHD NZ Ltd
LOCATION : 149 Stanmore Road, Christchurch
DATE : 26-6-2012
OPERATOR : S.Cardona
REMARK 1 : CPTu001
REMARK 2 : Effective Refusal

JOB # : 10677
TEST # : 1
CONE TYPE/SERIAL #: I-CFXYP20-10/ 071017T

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



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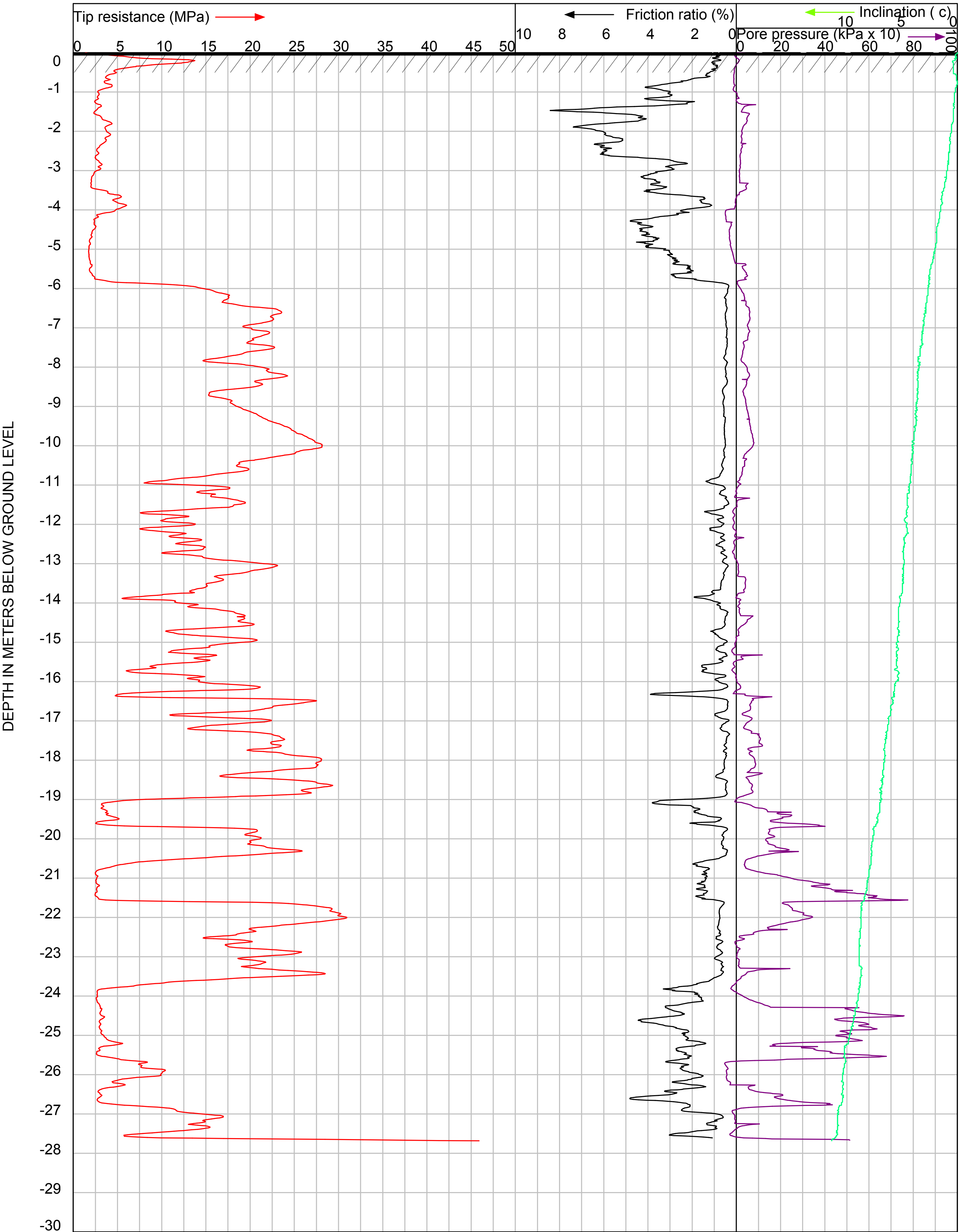
CPT No:	CPTu001	Operator:	S.Cardona
Project:	GHD NZ Ltd	Remark:	Effective Refusal
Location:	149 Stanmore Road, Christchurch		

McMILLAN DRILLING SERVICES



CLIENT	:	GHD NZ Ltd
LOCATION	:	149 Stanmore Road, Christchurch
DATE	:	26-6-2012
OPERATOR	:	S.Cardona
REMARK 1	:	CPTu002
REMARK 2	:	Effective Refusal

JOB # : 10677

TEST # : 2

CONE TYPE/SERIAL #: I-CFXYP20-10/ 071017T

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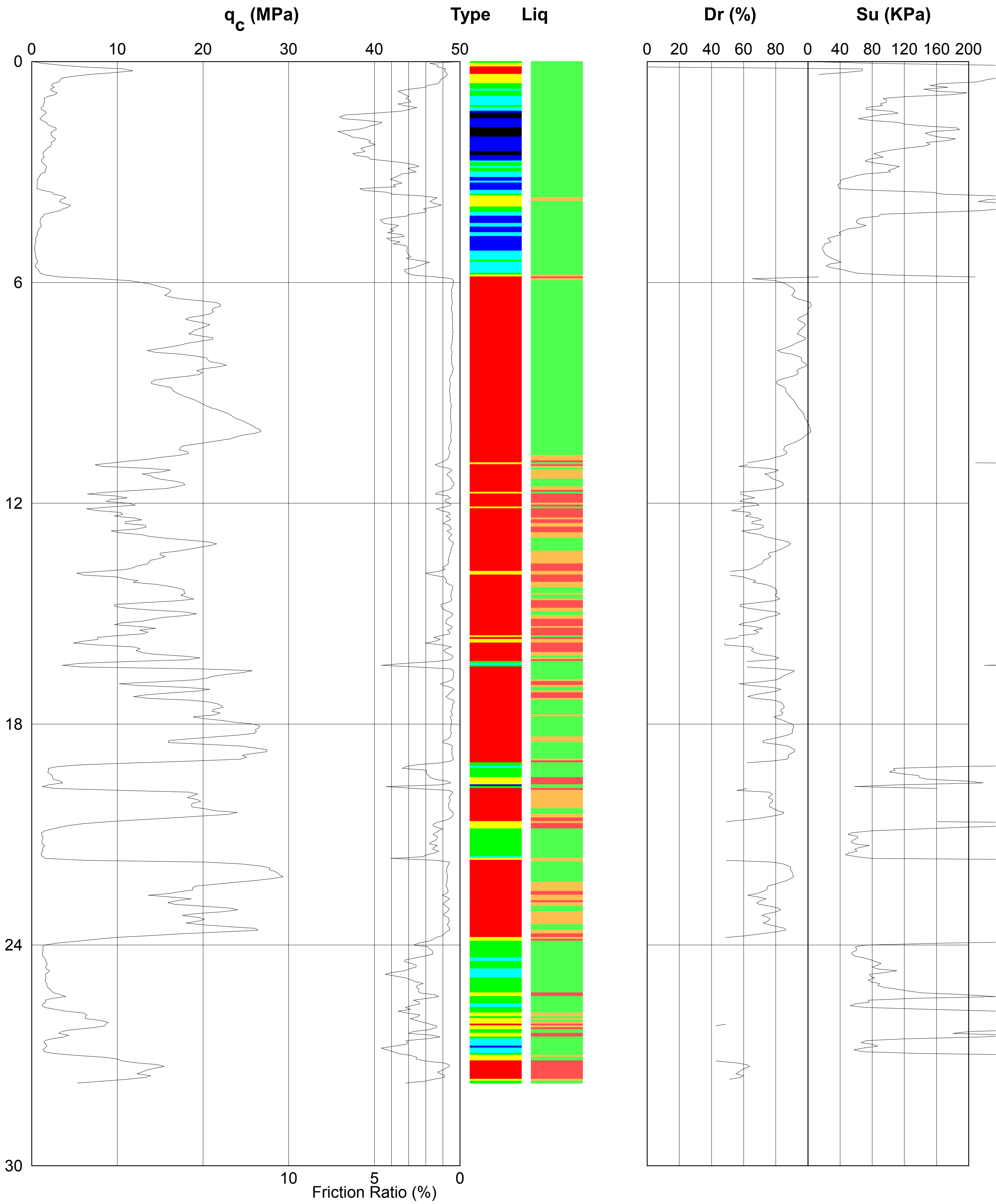
PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



Job No:	10677	Date:	26-6-2012
CPT No:	CPTu002	Operator:	S.Cardona
Project:	GHD NZ Ltd	Remark:	Effective Refusal
Location:	149 Stanmore Road, Christchurch		

McMILLAN DRILLING SERVICES

CIVIL CONSTRUCTION OVERVIEW

- 5 x Piling Rigs (20 to 80 tonne);
- 4 x Tieback/Micro-Piling Rigs (0.5 to 20 tonne);
- Sheet Piling & Injection Grouting;
- Dewatering;
- 26 x Drilling Rigs Company wide.

A NEW ZEALAND FIRST METHOD – INTRODUCED TO THE MARKET BY MCMILLAN'S:

Provisionally Patented Vibration Free Stone Column Method:

- Can be used next to sensitive buildings;
- No mess (dry);
- Cost effective (minimal setup times);
- Further savings possible for building construction – i.e. ground beams, deep rafts, pile starters, boxing to piles;
- No corrosion issues, all natural materials;
- Reliance on individual piles, and the risk of differential settlement is reduced.

Fully Instrumented Continuous Flight Auger / Displacement Auger Piling:

- Cost effective;
- Sizes 350mm to 900mm and 19m depth;
- Fast (150m of 600mm diameter reinforced concrete pile can be installed per day);
- Lateral load capacity of RC piles exceed some other piling methods;
- Quiet & vibration free;
- Fully reinforced concrete piles, with no corrosion issues.

McMILLAN'S ALSO OFFER THE FOLLOWING SERVICES:

- Screw Piles;
- Conventional Bored Concrete Piles;
- Mini & Micro Piles;
- Retaining Walls;
- Sheet Piling;
- Anchors & Tiebacks.

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DRILLING SERVICES

SOIL LIQUEFACTION SUSCEPTIBILITY ASSESSMENT

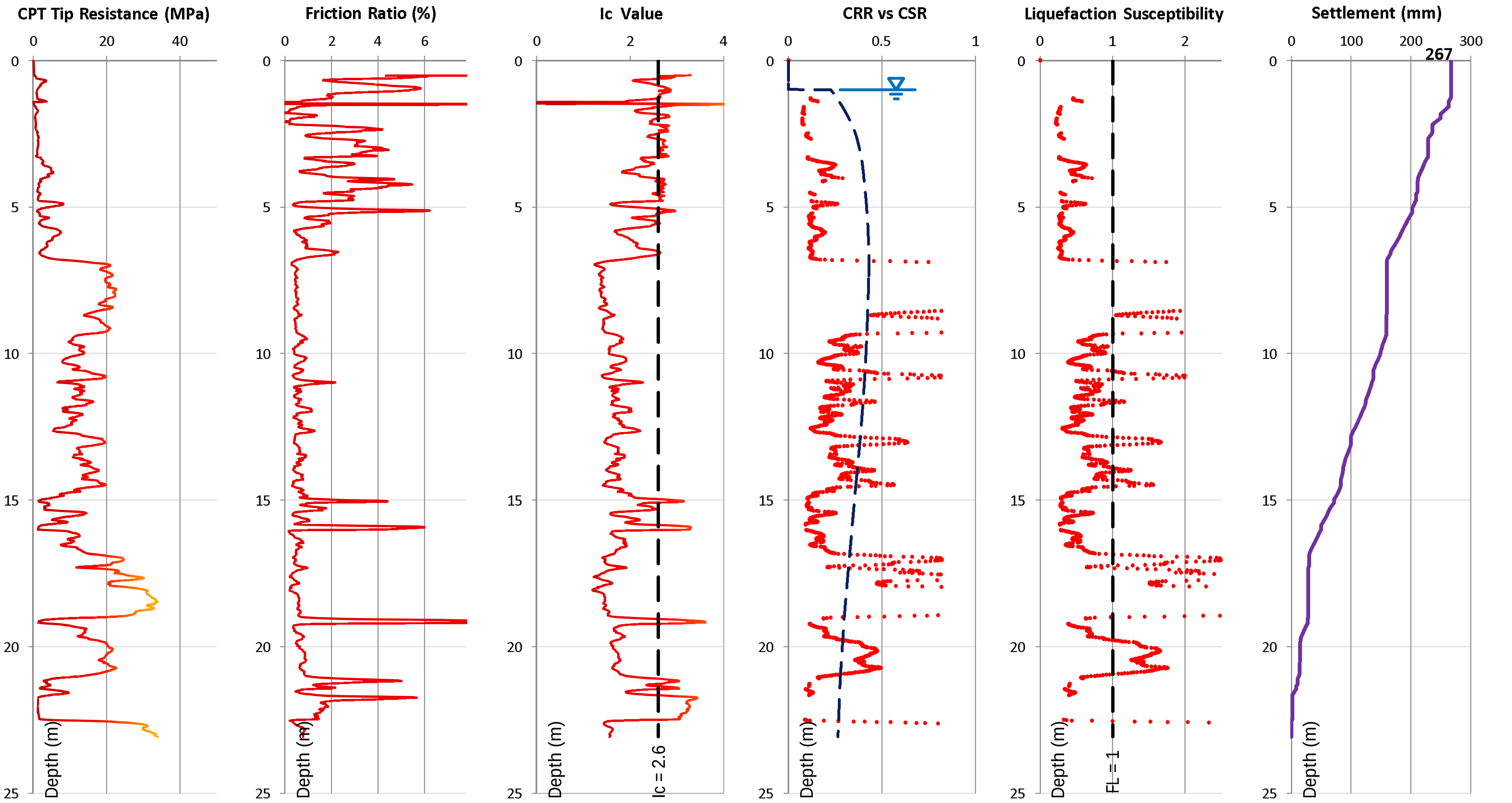
LOCATION :	CPT 1	SHEET :	1
PROJECT :	Haast Courts - 149 Stanmore Road	CALCULATED :	HNN
JOB NO :	51 30596 41	CHECKED BY :	
TEST DATE :	28 Jun 2012	DATE :	4 Jul 2012

PGA (a_{max}): **0.35 g**
EQ Magnitude: **7.5**

Groundwater Level (m bgl): **1.0**
Atmospheric Pressure (kPa): **101**

Bore depth (m): **23.07**
Test data step (m): **0.01**

Total Estimated Settlement (mm)
267



N:\NZ\Wellington\Projects\51\30596\41 Haast Courts Block A U 1-4-43\Investigation\Geotech Investigation\Liquefaction and Settlement Analysis CPT 1.xlsx

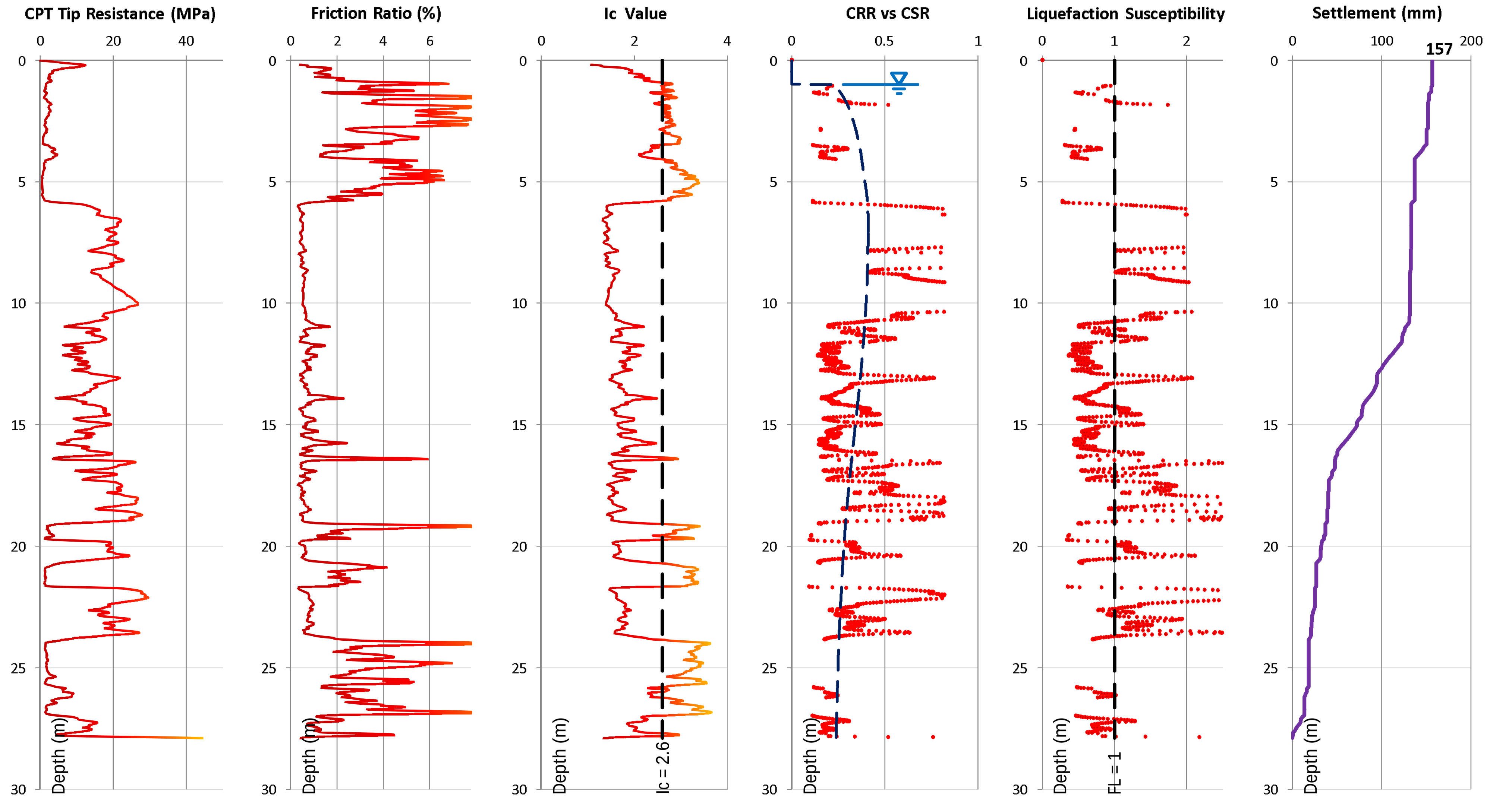
SOIL LIQUEFACTION SUSCEPTIBILITY ASSESSMENT

LOCATION :	CPT 2	SHEET :	2
PROJECT :	Haast Courts - 149 Stanmore Road	CALCULATED :	HNN
JOB NO :	51 30596 41	CHECKED BY :	
TEST DATE :	28 Jun 2012	DATE :	4 Jul 2012

PGA (a_{max}):	**0.35 g**	Groundwater Level (m bgl):	**1.0**	Bore depth (m):	**27.89**
EQ Magnitude:	**7.5**	Atmospheric Pressure (kPa):	**101**	Test data step (m):	**0.01**

Total Estimated Settlement (mm)

157



GHD

Level 11, Guardian Trust House
15 Willeston street, Wellington 6011
T: 64 4 472 0799   F: 64 4 472 0833   E: wgtnmail@ghd.com

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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
FINAL	Paul Clarke	Stephen Lee		Nick Waddingtom		20/12/12