CLIENTS | PEOPLE | PERFORMANCE

Barnett Park Sports Ground Pavilion
PRK 1390 BLDG 001 EQ2

Detailed Engineering Evaluation
Quantitative Report

Version FINAL (Rev1)

Redcliffs, Christchurch

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

Barnett Park Sports Ground Pavilion
PRK 1390 BLDG 001 EQ2

Detailed Engineering Evaluation
Quantitative Report
Version FINAL (Rev1)

Redcliffs, Christchurch

Christchurch City Council

Prepared By
Peter O Brien

Reviewed By
Hamish Mackinven

Date
14/03/2014

Contents

Table Index

Figure Index

Appendices

D CERA Building Evaluation Form

Quantitative Report Summary

Barnett Park Sports Ground Pavilion

PRK 1390 BLDG 001 EQ2

Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL (Rev1)

Redcliffs, Christchurch

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 and inspections on the 11th of April 2012 and the 16th of May 2012.

Brief Description

The changing room building, block A, consists of an unreinforced concrete masonry block walled structure forming both internal and external walls. The roof structure consists of lightweight corrugated steel roof cladding on timber purlins supported by timber rafters in the changing room/toilet block and supported by timber roof trusses in the referees changing block/equipment store room.

The referee building, Block B, consists of a reinforced concrete masonry block wall structure with a timber truss roof system. The roof consists of lightweight metal cladding fixed to timber trusses that span between the external walls. The walls reinforcement consists of 14mm vertical bars bars at 450mm centres.

The foundations both blocks consist of slab on grade floor with a perimeter strip foundation under the walls.

Indicative Building Strength

Following a detailed assessment, Block A has been assessed as achieving 37 **%**NBS and Block B has been assessed as achieving 74 **%**NBS. Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines Block A is considered an Earthquake Risk and Block B is considered neither Earthquake Prone nor an Earthquake Risk.

Recommendations

As Block A achieves under 67% NBS, it is considered a potential Earthquake Risk structure in accordance with the NZSEE guidelines. As Block A has not been identified as an Earthquake Prone building and no immediate collapse hazards or critical structural weaknesses have been identified, occupancy of the building can remain.

As Block B has a %NBS greater than 67% it is not deemed Earthquake Prone or an Earthquake Risk and therefore general occupancy is permitted.

1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the Barnett Park Pavilion.

This report is a Quantitative Assessment and is based in general on NZS 1170.5: 2004, NZS 4230: 1990, the New Zealand Society for Earthquake Engineering (NZSEE) guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance (02/2011) and the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes (06/2006).

The quantitative assessment to the building comprises an investigation on in-plane and out-of-plane strength of the unreinforced masonry block walls of block A and the reinforced masonry block walls of block B. The investigation is based on the analysis of the seismic loads that the structure is subjected to, the analysis of the distribution of these forces throughout the structure and the analysis of the capacity of existing structural elements to resist the forces applied. The capacity of the existing structural elements is compared to the demand placed on the element to give the percentage of New Building Standard (%NBS) of each of the structural elements.

Electromagnetic scans have been carried out on site to ascertain the extent of the reinforcement in the walls.

At the time of this report, no finite element modelling of the building structure has been carried out.

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 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 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 1 NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE

Table 1 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

Table 1 %NBS compared to relative risk of failure

4. Building Description

4.1 General

Barnett Park Sports Ground is located at 200A Main Road, Redcliffs, Christchurch. The site consists of two buildings; the first contains public toilets, two changing rooms and a plant room. The second building contains a referee's changing room and equipment storage room. The exact date of construction is unknown but it is evident from a hole in a masonry block that the walls are at least partially unfilled and this would indicate that the building was constructed pre 1976. Ferroscanning indicates that the walls of block A are unreinforced while the walls of Block B are reinforced.

The changing room building, block A, consists of an unreinforced concrete masonry block walled structure forming both internal and external walls. The roof structure consists of lightweight corrugated steel roof cladding on timber purlins supported by timber rafters in the changing room/toilet block and supported by timber roof trusses in the referees changing block/equipment store room.

The referee building, Block B, consists of a reinforced concrete masonry block wall structure with a timber truss roof system. The roof consists of lightweight metal cladding fixed to timber trusses that span between the external walls. The walls reinforcement consists of 14mm vertical bars bars at 450mm centres.

The foundations both blocks consist of slab on grade floor with a perimeter strip foundation under the walls.

The dimensions of the pavilion are approximately 15m long by 5.3m wide and 3.8m tall for the toilet block/changing rooms. The referee's changing room/equipment storage building is approximately 7m by 5.5m and 4m tall. The overall footprint of the building is approximately 125m^2. The nearest waterway to the property is Monks Bay located approximately 120m to the north-east of the site.

No plans or drawings were available for this building.

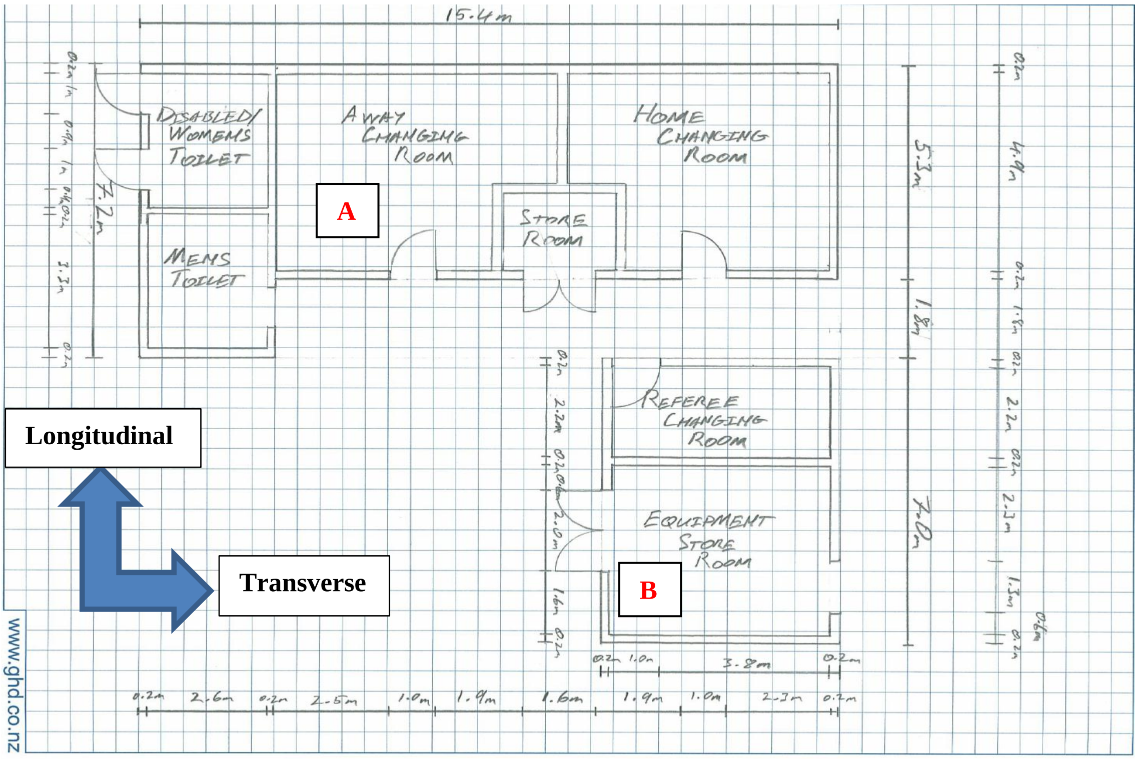


Figure 2 Plan of the building showing key structural elements

4.2 Gravity Load Resisting System

Gravity loads in Block A are transferred from the roof cladding to the timber roof purlins and then on to the timber roof rafters. The rafters then transfer the load to the supporting concrete masonry walls of the pavilion. Loads are transferred through the external masonry walls to the external strip foundation.

The gravity loads of Block B are transferred from the roof cladding to the timber roof purlins and then on to the timber trusses that span between the external reinforced concrete block walls. The loads are then transferred through the concrete block walls to the strip foundation supporting the walls.

4.3 Lateral Load Resisting System

In Block A the main resistance to lateral loads acting on the structures is provided by the concrete blockwork walls in both the longitudinal and transverse directions. The loads are transferred from the roof through diaphragm action of the roof structure to the external walls which then transfer the load directly to the foundations.

Lateral loads in Block B are similarly resisted by the concrete block walls however diaphragm action is achieved though the plasterboard lined ceiling of the referee changing room. In the equipment store the roof loads are transferred to the walls through diaphragm action of the truss framed roof structure. The loads are then resisted by the blockwork walls which transfer the loads into the perimeter strip foundation.

5. Assessment

5.1 Quantitative Assessment

The quantitative assessment to the building comprised an investigation of in-plane and out-of-plane strength of the masonry block walls. The investigation was based on the analysis of the seismic loads that the structure is subjected to, distribution of these forces throughout the structure and the analysis of the capacity of existing structural elements to resist the forces applied. The capacity of the existing structural elements was compared to the demand placed on the elements to give the %NBS of each of the structural elements.

5.1.1 Demand

The in-plane shear demand of each wall was assessed by completing a torsion analysis to the building. NZS 1170.5:2004 makes allowance for accidental eccentricity and requires that the earthquake action be applied at an eccentricity of 10% of the building dimension which is perpendicular to the force applied. This results in a torsional action about the centre of resistance of the building, and induces forces in the lateral force resisting (in-plane) walls in addition to the direct shear. As each wall was made of the same material and with the same properties, the direct shear and the force induced in each wall are proportional to the length squared. Cl 5.3.1.2 of NZS 1170.5: 2004 states that for nominally ductile and brittle structures an action set of 100% of the earthquake actions in one direction and 30% in the orthogonal direction must be applied when calculating the demand for any structural member and has such been applied in the analysis.

5.1.2 Seismic Coefficient

The elastic site hazard spectrum for horizontal loading, C(T), for the building was derived from Equation 3.1(1);

$$\mathrm{C(T) = C_h\, Z\, R\, N(T.D)}$$

Where

$C_h(T)$ = the spectral shape factor determined from CL 3.1.2

Z = the hazard factor from CL 3.1.4 and the subsequent amendments which increased the hazard factor to 0.3 for Christchurch

R = the return period factor from Table 3.5 for an annual probability of exceedance of 1/500 for an Importance Level 2 building

N(T,D) = the near-fault scaling facto from CL 3.1.6

The structural performance factor, S_P, was calculated in accordance with CL 4.4.2

$$S_P = 1.33 - 0.3\mu$$

Where μ, the displacement ductility factor, was taken as 2.00 and k_μ of 1.2, for the in-plane assessment of the unreinforced Block A walls in accordance with section 4.3.2.4 of the NZSEE draft document for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance. A displacement ductility factor of 1.5 was assumed for the reinforced Block B walls.

The seismic weight coefficient was then calculated in accordance with Cl 5.2.1.1 of NZS 1170.5: 2011. For the purposes of calculating the seismic weight coefficient a period, T_1, of 0.1 was assumed for the building. The coefficient was then calculated using Equation 5.2(1);

$$C_d(T_1) = \frac{C(T_1)S_P}{k_\mu}$$

Where

$$k_\mu = \frac{(\mu - 1)T_1}{0.7} + 1$$

5.1.3 In-Plane Capacity of the Unreinforced Walls

The in-plane capacity of the unreinforced concrete masonry wall was determined using the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance (06/2006). The NZSEE guidelines recommend checks for 4 different in-plane response modes.

- Diagonal tension failure mode
- Bed-sliding failure mode
- Toe crushing failure mode
- Rocking failure mode

An analysis of each wall was carried out using the methods set out in Section 8 – In-Plane Wall Response, of the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance (06/2006).

5.1.4 In-plane Wall Shear Capacity of the Unreinforced Walls

The in-plane nominal shear capacity of a wall, pier or spandrel was taken as the minimum of the nominal capacity in the diagonal tension failure mode, V_{dt}, the rocking failure mode, V_r, the bed-joint sliding failure mode, V_s, and the toe crushing failure mode, V_{tc}.

$$V_n = \min(V_{dt}, V_s, V_r, V_{tc})$$

5.1.5 Out-of-Plane Capacity of the Unreinforced Walls

The % NBS for out-of-plane flexure of the concrete masonry walls was determined using the methods set out in the University of Auckland DRAFT technical paper titled "Generic procedure for seismic assessment of out-of-plane loaded URM walls" as recommended by the NZSEE following their annual conference in May 2013.

5.1.6 Shear capacity of the Reinforced Walls

The shear capacity of the reinforced filled masonry wall was determined using NZS 4230: 2004. As there are no details as to the level of supervision during the construction stage, the Observation Type was classed in accordance with Table 3.1.

5.1.7 Moment capacity of the Reinforced Walls

The moment capacity of the reinforced filled masonry wall was determined using NZS 4230: 2004 and the user's guide to NZS 4230: 2004. The strength reduction factor, ϕ, for flexure with or without axial tension or compression was taken as 0.85 in accordance with Cl 3.4.7.

6. Damage Assessment

6.1 Surrounding Buildings

Barnett Park Sports Ground Pavilion is located adjacent to residential properties, a sports pitch, a crèche and a car park. There are no buildings that are adjoining the pavilion building. During the inspection of the pavilion there was no apparent damage to the surrounding buildings on either the residential properties or the crèche property.

6.2 Residual Displacements and General Observations

No residual displacements of the structure were noticed during our inspection of the building.

Minor cracking was noted on the east wall of the pavilion. This is not considered significant.

6.3 Ground Damage

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

7. Geotechnical Consideration

The site is located approximately 100m from the Avon/ Heathcote Estuary on relatively flat area in Redcliffs and is approximately 50m from the cliff.

7.1 Published Information on Ground Conditions

7.1.1 Published Geology

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

- Valley fill and slope wash of loess volcanic derived colluvium. The site is close to the boundary of Christchurch formation of sand of fixed dunes and beaches.

7.1.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates six boreholes are located within a 200m radius of the site (see Table 2). Of these boreholes, all of them had a lithographic log which are summarised below.

Bore Name	Distance From Site	Groundwater	Log Summary
M36/10013	200m SE	0.9m	0 – 0.2 m Topsoil 0.2 – 0.8 m Grey Sandy Silt 0.80 – 1.5 m Grey/brown pockets Silty Sand 1.5 – 1.8 m Grey/brown pockets wet sand
M36/10014	200m SE		0 – 0.2 m Topsoil 0.2 -0.4 m Dark Sandy Silt 0.4 – 0.8 m Dark grey Silt and Sand 0.8 – 1.9 m Grey/brown pockets Sand
M36/10015	200m SE		0 – 0.3 m Topsoil 0.3 – 0.7 m Sandy Silt 0.7 – 1.5 m Silt and Sand 1.5 – 1.9 m Sand
M36/10016	200m SE		0 – 0.2 m Topsoil 0.2 – 1.2 m Sandy Silt 1.2 – 1.8 m Grey Sand 1.8 -2.1 m Grey/brown pockets Silty

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

Bore Name	Distance From Site	Groundwater	Log Summary
			Sand
M36/10378	190m SE	1.5m	0 – 0.8 m Topsoil 0.8 – 1.7 m Sand 1.7 – 2.0 m Grey Sand
M36/10379	190m SE		0 - 0.2 m Topsoil 0.2 – 1.7 m Wet Sand 1.7 – 2 m Grey saturated Sand

Table 2 ECan Bore Log Summary Table

It should be noted the quality of soil logging descriptions included on the boreholes is unknown and were likely written by the well driller and not a geotechnical professional or to a recognised geotechnical standard. In addition strength data is not recorded.

7.1.3 EQC Geotechnical Investigations

The Earthquake Commission has not undertaken geotechnical testing in the area of the site.

7.1.4 CERA Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has zoned the site as Green, indicating 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.

The site is classified as Technical Category 2 (TC2). This indicates the site is at risk from minor to moderate land damage from liquefaction is possible in future significant earthquakes.

7.1.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 (Figure 1) shows no shows signs of liquefaction with sand boils emminent near the site.

Figure 3 Post February 2011 Earthquake Aerial Photography[2]

7.2 Seismicity

7.2.1 Nearby Faults

There are many faults in the Christchurch region, however only those considered most likely to have an adverse effect on the site are detailed in Table 3 below.

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	140 km	NW	8.3	~300 years
Greendale (2010) Fault	30 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	75 km	NW	7.0	~1100 years

Table 3 Summary of Known Active Faults[3,4]

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

[3] 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.

[4] GNS Active Faults Database

Recent earthquakes since 4 September 2010 have identified the presence of previously unmapped active fault system 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 and average recurrence intervals are yet to be estimated.

7.2.2 Ground Shaking Hazard

The recent 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 now 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.

7.3 Field Investigations

In order to further understand the ground conditions at the site, intrusive testing comprising one piezocone/seismic CPT investigation was conducted at the site on 02 April 2012.

The locations of the tests are tabulated in Table 4.

Investigation	Depth (m bgl)	Easting (NZMG)	Northing (NZMG)
CPT 001	23.0	2489018	5737987

Table 4 Coordinates of Investigation Locations

The CPT investigation was undertaken by McMillans Drilling Ltd on 04 April 2012 to a target depth of 20m below ground level. Please refer to the attached CPT results for detail (Appendix C).

Interpretation of output graphs[5] from the investigation showing Cone Tip Resistance (q_c), Friction Ratio (Fr), Inferred Lithology and Inferred Liquefaction Potential are presented in Table 5.

7.4 Ground Conditions Encountered

7.4.1 Summary of CPT-Inferred Lithology

Depth (m)	Lithology [1]	Cone Tip Resistance q_c (MPa)	Friction Ratio Fr (%)	Relative Density Dr (%)
0 – 13.5	SAND	8 to 30	0.7 to 1	60 to 100
13.5 – 16.0	SILT mixtures	1 to 2	~2	(Su ≥ 40 kPa)
16.0 – 18.0	SAND	10 to 25	~0.6	50 to 90
18.0 – 19.5	SILT mixtures	1 to 10	1.5 to 5	(Su ≥ 40 kPa)

[5] McMillans Drilling CPT data plots, Appendix C

Table 5 Summary of CPT-Inferred Lithology

Please refer to the CPT logs in Appendix C for detail.

Liquefaction Assessment

As the desktop assessment concluded the site is at risk of liquefaction, a more detailed assessment has been conducted.

7.5.1 Parameters used in Analysis

Assumptions made for the analysis process are as follows:

- D50 particle sizes for the site soil (sands) from CPT soil analysis;
- Peak ground acceleration (PGA) 0.35g ULS, and 0.13g SLS (DBH guidelines); and,
- Groundwater levels of 1m bgl.

The following equation has been used to approximate soil unit weight from the CPT investigation data: [6]

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

This typically gave unit weights of 16 to 20 kN/m^3 (saturated).

The liquefaction analysis process has been conducted using the methodology from Stark & Olson[7], and from the NZGS Guidelines[8]. Settlements have been estimated using the methodology proposed by Zhang et al (2002)[9].

7.5.2 Results of Liquefaction Analysis

The results of the liquefaction analysis, as outlined in Table 6, indicate that three distinct bands between 3m and 18m are severely liquefiable.

Depth (m)	Lithology	Triggering Factor F_L	Liquefaction Susceptibility[10]
0 – 3.0	SAND	> 1.5	*Not Liquefiable*
3.0 – 7.5	SAND	0.4 to 1.3	Severe

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

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

[8] 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

[9] Zhang G., Robertson P.K., & Brachman R.W.I. (2002): *Estimating liquefaction-induced ground settlements from CPT for level ground*. Canadian Geotechnical Journal, Vol 39, pp. 1168-1180

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

7.5 – 13.5	SAND	> 1.5	*Not Liquefiable*
12.8 – 13.5	SAND	0.4 to 1.4	Severe
13.5 – 16.0	SILT mixtures	-	*Not Liquefiable*
16.0 – 18.0	SAND	0.4 to 2.5	Severe
18.0 – 23.0	SILT mixtures	-	*Not Liquefiable*

Table 6 Summary of Liquefaction Susceptibility

Liquefaction-induced settlement at the site is estimated to be in the order of 103mm for a ULS design earthquake, and 11mm for SLS.

Please refer to the 'Soil Liquefaction Susceptibility Assessment' spreadsheets in Appendix C for detail.

7.6 Interpretation of Ground Conditions

7.6.1 Liquefaction Potential

The site is considered to have a minor to moderate liquefaction potential during future earthquakes as evidenced by:

- Evidence of liquefaction at the site following the February (Mw 6.3, 2.0g) and June (Mw 6.0-6.3, 1.5g) events;
- Results of liquefaction assessment showing one sand layer between 3m and 7.5m bgl as being moderately susceptible to liquefaction;
- Settlement estimates are in the order of 103mm (ultimate) and 11mm (serviceability); and,
- CERA TC2 classification indicates the site is at risk from minor to moderate land damage from liquefaction is possible in future significant earthquakes.

7.6.2 Slope Failure and/or Rockfall Potential

The site is located within Redcliffs, a hill suburb in eastern Christchurch. Although the site itself is situated on relatively flat ground, it is surrounded by hills and there is the potential for rockfall in this area (to the west of the car park). The park and tracks are currently shut due to this hazard.

7.6.3 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 DEE/IEP is still believed to be appropriate;
- If repair or rebuild work is undertaken for the structure's foundations, this should be in accordance with DBH and CERA guidelines for TC2 land. While the ULS settlement estimate is slightly over the 100mm criteria, SLS is very low, and hence TC2 is still considered appropriate; and,
- If new foundations are constructed, it is recommended that preferential consideration be given to deep foundations.

8. Survey

No level or verticality surveys have been undertaken for this building at this stage.

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

9. Initial Capacity Assessment

9.1 Seismic Parameters

The seismic design parameters based on current design requirements from NZS1170:2002 and the NZBC clause B1 for this building are:

- Site soil class assumed to be: D, NZS 1170.5:2004, Clause 3.1.3, Soft Soil;
- Site hazard factor, Z = 0.3, NZBC, Clause B1 Structure, Amendment 11 effective from 1 August 2011;
- Return period factor R_u = 1.0, NZS 1170.5:2004, Table 3.5, Importance Level 2 structure with a 50 year design life.

9.2 Wall Investigation

The position of each wall is indicated in the plans below and each wall is named accordingly.

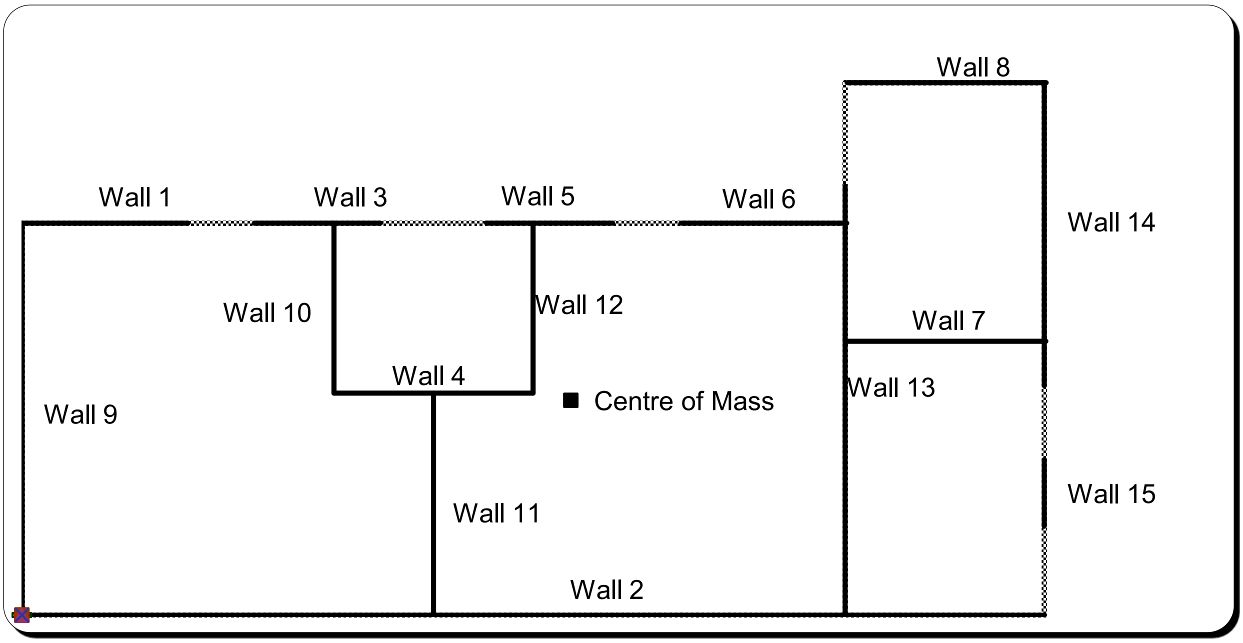


Figure 4 Plan Details and Wall Locations of Block A

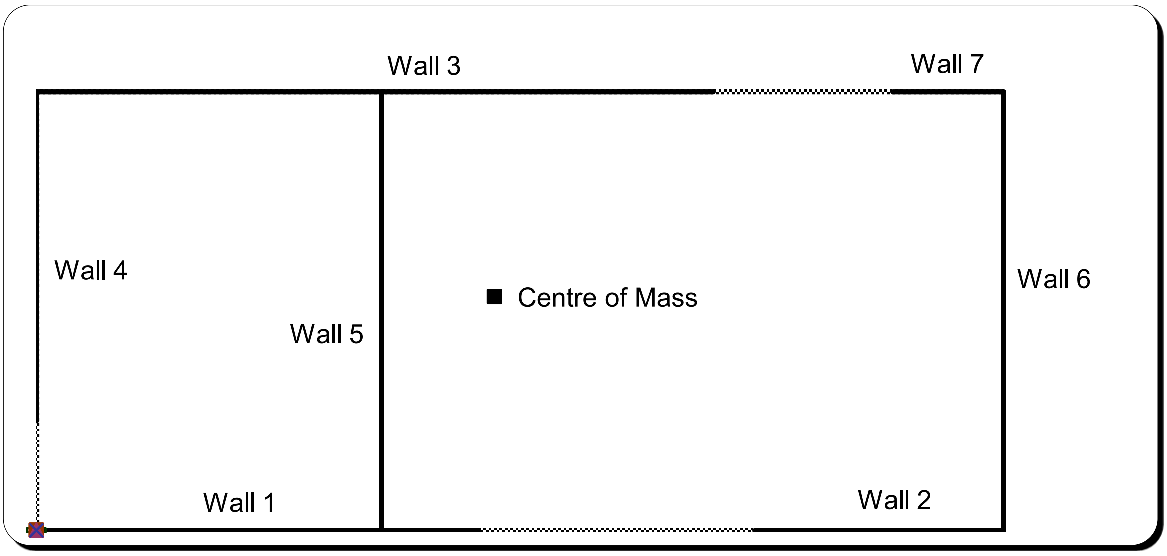


Figure 5 Plan Details and Wall Locations of Block B

9.3 Block A Analysis Results

The results of the in plane analysis and subsequent earthquake designation under the NZSEE guidelines are listed below in Table 7.

Wall number	V* kN	ϕV_n kN	%NBS	Earthquake Status	M* kNm	ϕM_n kNm	%NBS	Earthquake Status
1	6.9	5.2	75%	Not Risk or Prone	16.6	15.1	91%	Not Risk or Prone
2	129.4	74.6	58%	Risk	310.6	572.4	184%	Not Risk or Prone
3	4.0	3.0	75%	Not Risk or Prone	9.6	8.7	91%	Not Risk or Prone
4	6.8	7.5	111%	Not Risk or Prone	16.2	21.7	134%	Not Risk or Prone
5	4.0	3.0	75%	Not Risk or Prone	9.6	8.7	91%	Not Risk or Prone
6	6.9	5.2	75%	Not Risk or Prone	16.6	15.1	91%	Not Risk or Prone
7	8.3	6.6	80%	Not Risk or Prone	19.8	19.1	96%	Not Risk or Prone
8	12.0	7.5	62%	Risk	28.9	21.7	75%	Not Risk or Prone
9	62.5	23.4	37%	Risk	150.0	67.8	45%	Risk
10	16.7	6.6	40%	Risk	40.0	19.1	48%	Risk
11	9.2	3.9	42%	Risk	22.0	11.2	51%	Risk
12	14.5	6.6	45%	Risk	34.8	19.1	55%	Risk
13	66.2	24.6	37%	Risk	158.8	71.3	45%	Risk
14	37.7	14.0	37%	Risk	90.4	40.6	45%	Risk
15	1.8	0.7	37%	Risk	4.4	2.0	45%	Risk

Table 7 Block A In Plane Analysis Results

The results of the out of plane displacement response capability analysis and subsequent earthquake designation under the NZSEE guidelines are listed in Table 8.

Wall number	Δ_{ins}	D_{ph}	%NBS	Earthquake Status
1-6 8-9 14-15	0.166	0.107	93%	Not Risk or Prone
7 10-13	0.184	0.151	73%	Not Risk or Prone

Table 8 Block A Out-Of-Plane Analysis Results

9.4 Block B Analysis Results

The results of the in plane analysis and subsequent earthquake designation under the NZSEE guidelines are listed below in Table 9.

Wall	V* kN	ϕV_n kN	%NBS	Earthquake Status	M* kNm	ϕM_n kNm	%NBS	Earthquake Status
1	82.36	458.23	618.17%	Not Risk or Prone	26.87	19.85	73.85%	Not Risk or Prone
2	26.06	150.89	643.33%	Not Risk or Prone	8.50	6.28	73.85%	Not Risk or Prone
3	159.65	806.32	561.16%	Not Risk or Prone	52.09	46.53	89.33%	Not Risk or Prone
4	80.02	600.36	833.67%	Not Risk or Prone	26.11	29.48	112.92%	Not Risk or Prone
5	111.55	868.78	865.38%	Not Risk or Prone	36.39	52.40	144.00%	Not Risk or Prone
6	114.98	868.78	839.54%	Not Risk or Prone	37.51	52.40	139.70%	Not Risk or Prone
7	4.26	65.69	1715.18%	Not Risk or Prone	1.39	1.24	89.33%	Not Risk or Prone

Table 9 Block B In Plane Analysis Results

The results of the in plane analysis and subsequent earthquake designation under the NZSEE guidelines are listed below in Table 10.

Wall	M* kNm	ϕM_n kNm	%NBS	Earthquake Status
1	4.31	9.76	226.27%	Not Risk or Prone
2	4.31	9.03	209.30%	Not Risk or Prone
3	4.31	10.64	246.60%	Not Risk or Prone
4	4.31	10.12	234.68%	Not Risk or Prone
5	4.31	10.79	250.15%	Not Risk or Prone
6	4.31	10.79	250.15%	Not Risk or Prone
7	4.31	8.50	197.05%	Not Risk or Prone

Table 10 Block B Out Of Plane Analysis Results

9.5 Discussion of Results

Following a detailed assessment, Block A has been assessed as achieving 37 **%**NBS and Block B has been assessed as achieving 74 **%**NBS. Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines Block A is considered an Earthquake Risk and Block B is considered neither Earthquake Prone nor an Earthquake Risk.

9.6 Occupancy

As Block A achieves greater than 33% NBS, it is considered a potentially Earthquake Risk structure in accordance with the NZSEE guidelines. As no immediate collapse hazards or critical structural weaknesses have been identified for the building, occupancy of the building is permitted.

As Block B has a %NBS greater than 67% it is not deemed Earthquake Prone or an Earthquake Risk and therefore general occupancy is permitted.

10. Strengthening

As the building has not been identified as Earthquake Prone, no further action is required by Christchurch City Council to comply with the Earthquake Prone, Dangerous and Insanitary Buildings Policy (2010).

It is however recommended that the building is strengthened to 67% NBS in line with the NZSEE guidelines.

11. Recommendations

As Block A has been assessed to have a %NBS less than 67% NBS, it is deemed to be Earthquake Risk. It is recommended that strengthening options be explored and implemented to bring the %NBS of the building up to a minimum of 67% NBS.

As Block B has been assessed to have a %NBS greater than 67% NBS, it is not deemed to be Earthquake Prone nor an Earthquake Risk. As no immediate collapse hazards or critical structural weaknesses have been identified for the building, occupancy can continue.

12. Limitations

12.1 General

This report has been prepared subject to the following limitations:

- Drawings of the building were unavailable. As a result the information contained in this report has been inferred from visual inspections of the building and site only.
- No intrusive structural investigations have been undertaken.
- No level or verticality surveys have been undertaken.
- No material testing has been undertaken.
- No calculations, other than those detailed in Section 5 have been carried out on the structure.

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.

12.2 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 engineer 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

Photograph 1 West elevation of the pavilion.

Photograph 2 South elevation of the pavilion.

Photograph 3 North elevation of the pavilion.

Photograph 4 East face of pavilion building.

Photograph 5 Timber roof trusses of referee changing room/equipment storage building.

Photograph 6 Gap between sections of the building.

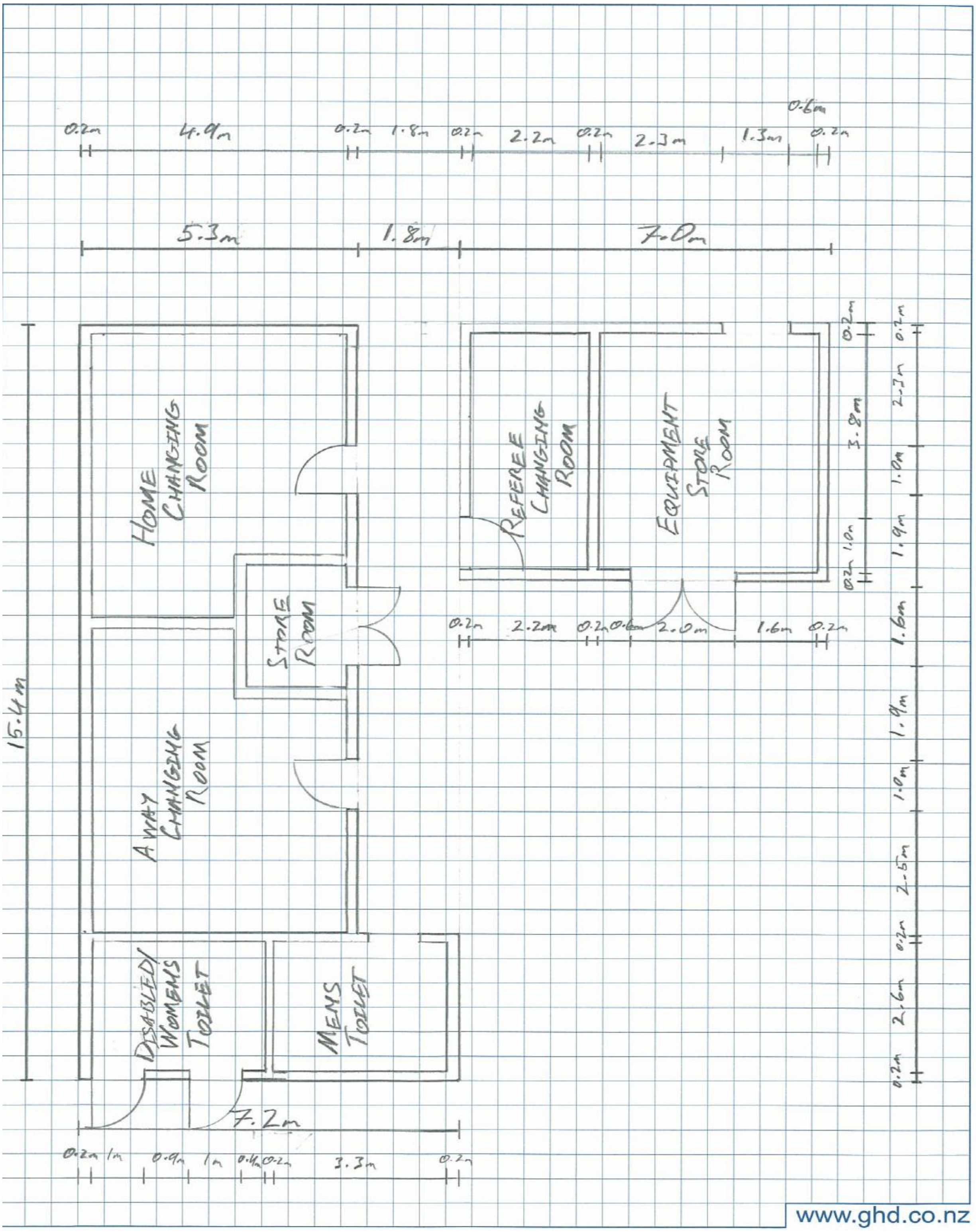
Photograph 7 Roof of toilet block/changing rooms.

Photograph 8 Roof cladding, timber purlins and timber rafters in the roof of the toilet block/changing rooms.

Photograph 9 Masonry walls extending the full height of the building to roof apex level.

Appendix B
Existing Drawings

No drawings have been made available for this building. Shown below is a sketch of the building showing key structural elements.



Appendix C

Geotechnical Results

CPT ANALYSIS NOTES

Soil Type

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

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

Liquefaction Screening

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

- high susceptibility
- medium susceptibility
- low susceptibility

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

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

Low susceptibility is all other cases.

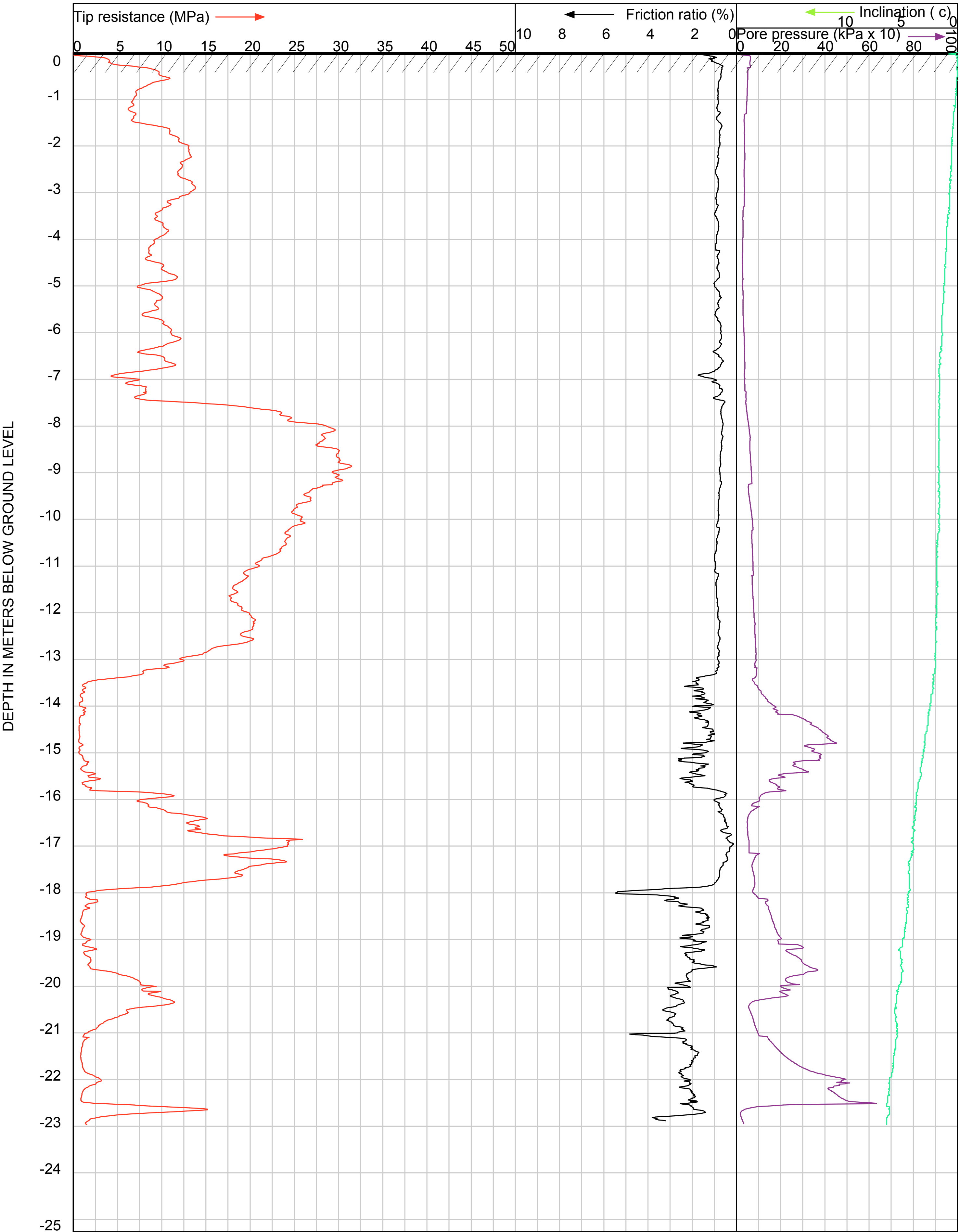
Relative Density (D_R)

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

Undrained Shear Strength (S_U)

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

McMILLAN
DRILLING SERVICES

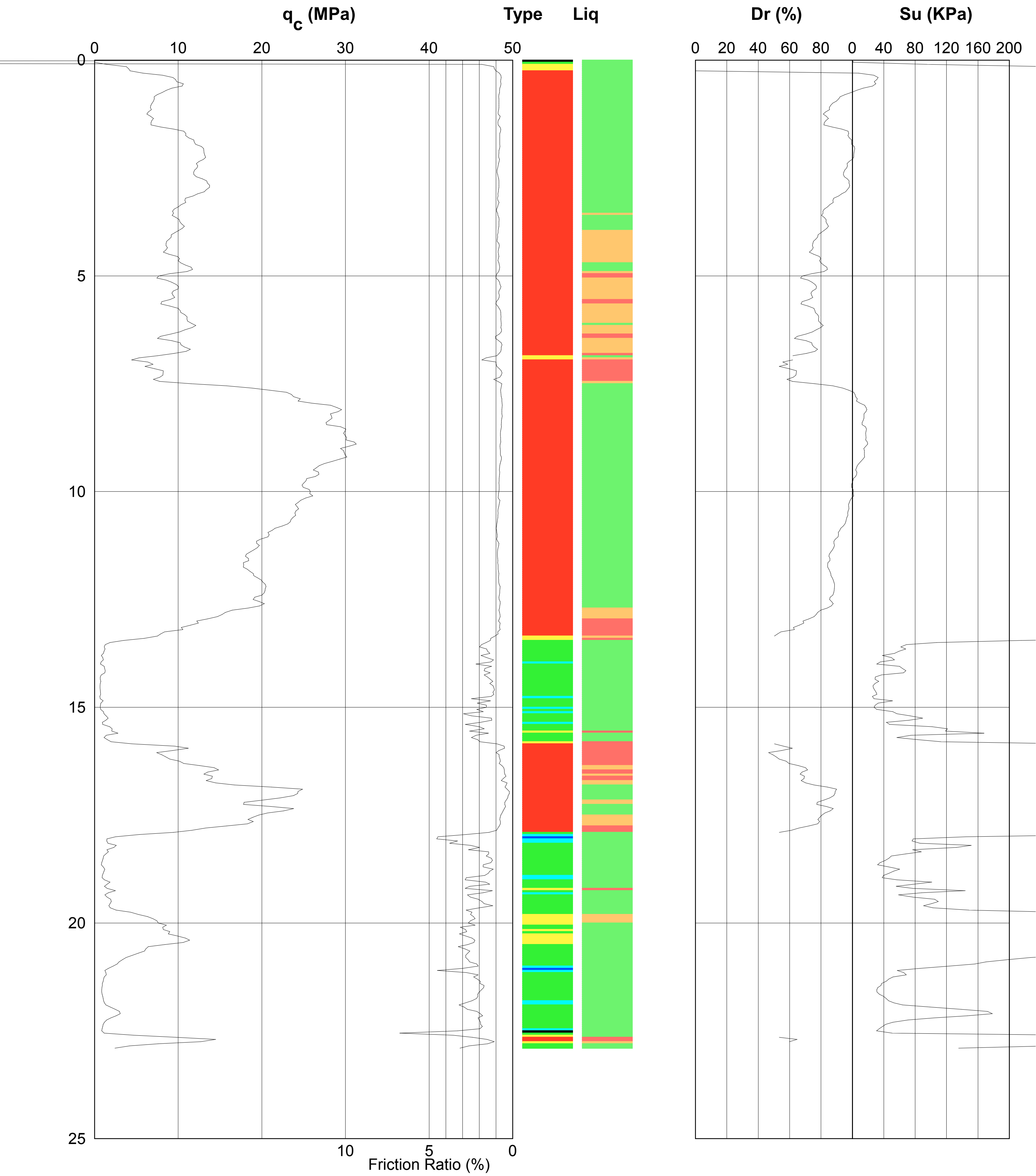


CLIENT : GHD
LOCATION : Christchurch Various (CCC Properties)
DATE : 2-4-2012
OPERATOR : H. Pardoe
REMARK 1 : CPTu30
REMARK 2 : Target Depth

JOB # : 10386
TEST # : 30

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

PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



Job No:	10386	Date:	2-4-2012
CPT No:	CPTu30	Operator:	H. Pardoe
Project:	GHD	Remark:	Target Depth
Location:	Christchurch Various (CCC Properties)		

McMILLAN DRILLING SERVICES

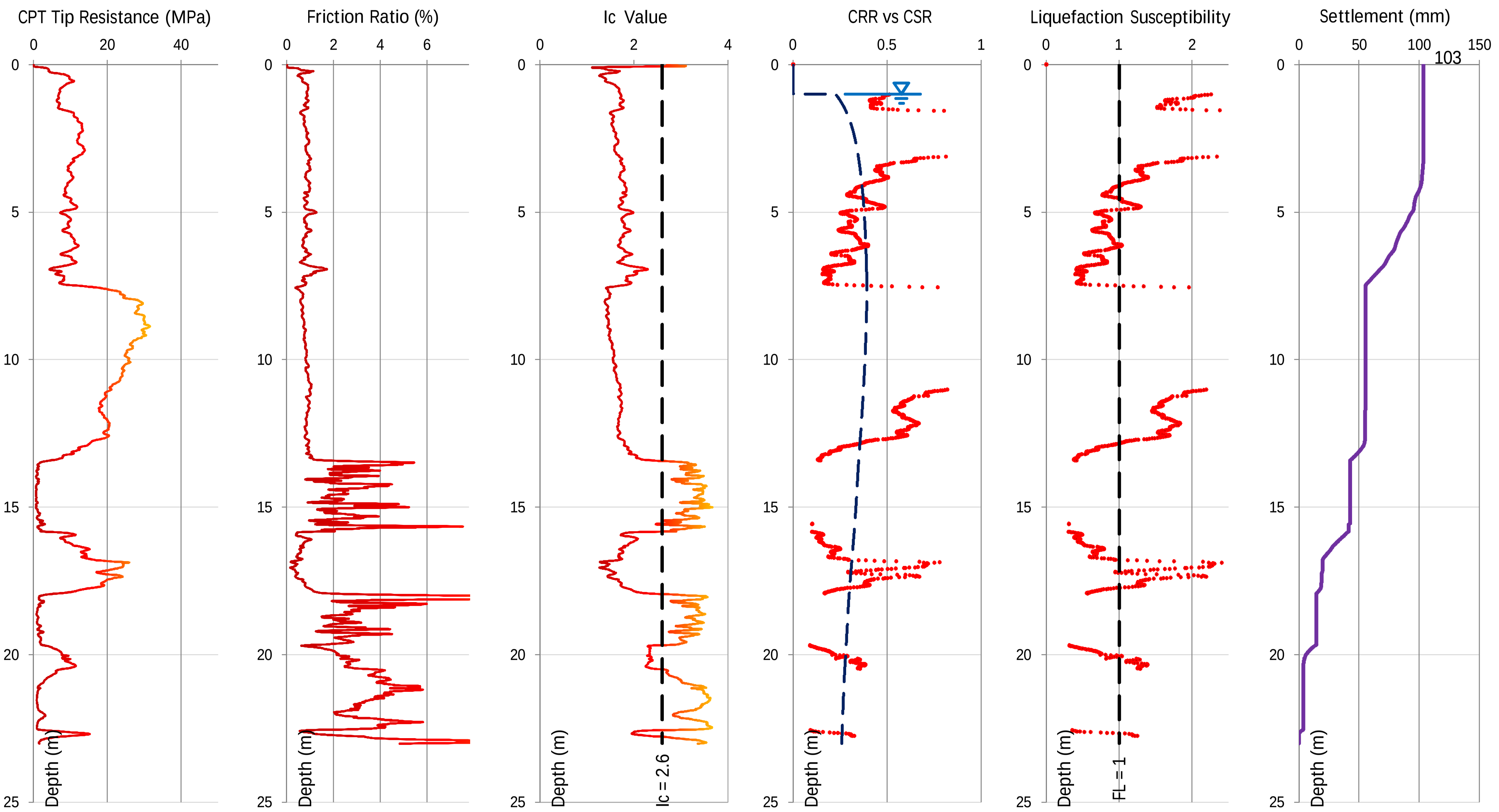
SOIL LIQUEFACTION SUSCEPTIBILITY ASSESSMENT

LOCATION :	Barnett Park Pavilion	SHEET :	1
PROJECT :	CCC Structural Engineering Services	CALCULATED :	HNN
JOB NO :	51 30596 30	CHECKED BY :	
TEST DATE :	4 Apr 2012	DATE :	27 Jul 2012

PGA (a_{max}):	0.35 g	Groundwater Level (m bgl):	1.0	Bore depth (m):	23.01
EQ Magnitude:	7.5	Atmospheric Pressure (kPa):	101	Test data step (m):	0.01

Total Estimated Settlement (mm)

103

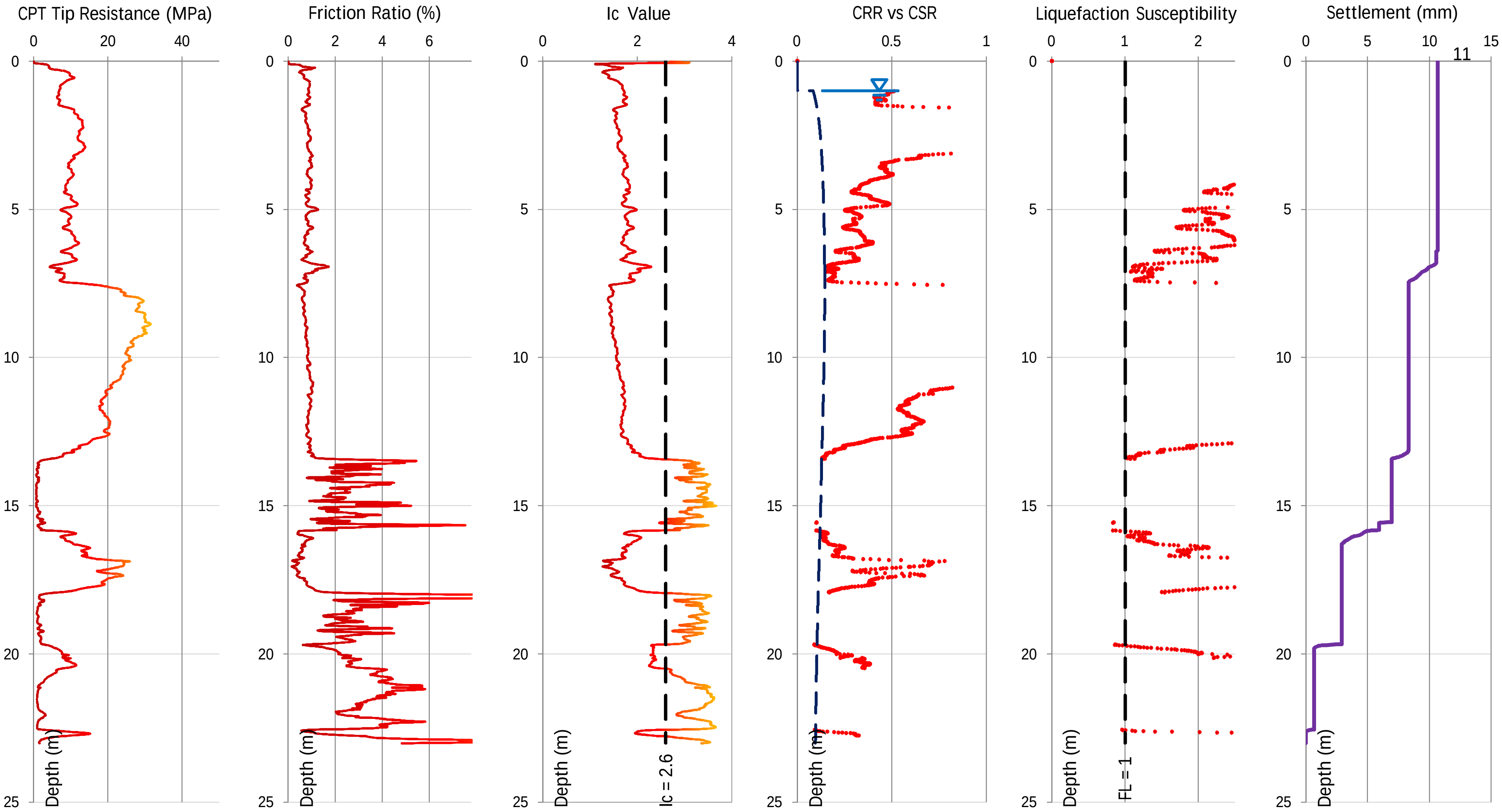


SOIL LIQUEFACTION SUSCEPTIBILITY ASSESSMENT

LOCATION :	Barnett Park Pavilion	SHEET :	1
PROJECT :	CCC Structural Engineering Services	CALCULATED :	HNN
JOB NO :	51 30596 30	CHECKED BY :	
TEST DATE :	4 Apr 2012	DATE :	27 Jul 2012

PGA (a_{max}): **0.13 g**
EQ Magnitude: **7.5**
Groundwater Level (m bgl): **1.0**
Atmospheric Pressure (kPa): **101**
Bore depth (m): **23.01**
Test data step (m): **0.01**

Total Estimated Settlement (mm)
11



N:\NZ\Wellington\Projects\51\30596\30 Barnett Park Sports Ground\Investigation\Geotech Investigation\Liquefaction Analysis\Liquefaction and Settlement Analysis SLS.xlsx

Appendix D

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Barnett Park Sports Pavilion block A			Reviewer:	Hamish Mackinven
	Unit	No:	Street	CPEng No:	1003941
Building Address:	A	200	Main Road, Redcliffs	Company:	GHD
Legal Description:				Company project number:	513059630
				Company phone number:	(03) 3780900
	Degrees	Min	Sec		
GPS south:	43	33	54.61	Date of submission:	14/03/2014
GPS east:	172	44	24.57	Inspection Date:	
				Revision:	
Building Unique Identifier (CCC):	PRK_1390_BLDG_001 EQ2			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	0
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):	2.00

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	2.00
Ground floor split?	no		Ground floor elevation above ground (m):	0.20
Storeys below ground	0			
Foundation type:	mat slab		if Foundation type is other, describe:	
Building height (m):	3.80		height from ground to level of uppermost seismic mass (for IEP only) (m):	3.8
Floor footprint area (approx):	124			
Age of Building (years):			Date of design:	1965-1976
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	public		Brief strengthening description:	
Use (upper floors):				
Use notes (if required):				
Importance level (to NZS1170.5):	IL2			

<u>Gravity Structure</u>

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	
Floors:	concrete flat slab	slab thickness (mm)	150
Beams:	none	overall depth x width (mm x mm)	
Columns:			
Walls:	partially filled concrete masonry	thickness (mm)	200

Lateral load resisting structure

Lateral system along:	partially filled CMU		**Note: Define along and across in detailed report!**	note total length of wall at ground (m):	33.8
Ductility assumed, μ:	1.00			wall thickness (m):	0.2
Period along:	0.40	0.40	from parameters in sheet	estimate or calculation?	
Total deflection (ULS) (mm):				estimate or calculation?	
maximum interstorey deflection (ULS) (mm):				estimate or calculation?	
Lateral system across:	partially filled CMU			note total length of wall at ground (m):	17.8
Ductility assumed, μ:	1.00			wall thickness (m):	0.2
Period across:	0.40	0.40	from parameters in sheet	estimate or calculation?	
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:			
Roof Cladding:	Metal	describe	Lightweight corrugated steel
Glazing:			
Ceilings:	none		
Services(list):			

Available documentation

Architectural	none	original designer name/date	
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	Describe damage:	None
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:	green	
Along	Damage ratio:	0%	Describe how damage ratio arrived at:
	Describe (summary):	minor cracking to east wall	
Across	Damage ratio:	0%	
	Describe (summary):	no damage visible	
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:	significant structural and strengthening	Describe:
	Building Consent required:		Describe:
	Interim occupancy recommendations:	do not occupy	Describe:
Along	Assessed %NBS before:	58%	##### %NBS from IEP below
	Assessed %NBS after:	58%	
Across	Assessed %NBS before:	37%	##### %NBS from IEP below
	Assessed %NBS after:	37%	

If IEP not used, please detail assessment methodology: Calculations detailed in report

IEP **Use of this method is not mandatory - more detailed analysis may give a different answer, which would take precedence. Do not fill in fields if not using IEP.**

Period of design of building (from above): 1965-1976

h_n from above: 3.8m

Seismic Zone, if designed between 1965 and 1992:

not required for this age of building

not required for this age of building

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

Note:1 for specifically design public buildings, to the code of the day: pre-1965 = 1.25; 1965-1976, Zone A =1.33; 1965-1976, Zone B = 1.2; all else 1.0

Note 2: for RC buildings designed between 1976-1984, use 1.2

Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)

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

2.2 Near Fault Scaling Factor

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

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

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:

Z_{1992}, from NZS4203:1992

Hazard scaling factor, **Factor B:** #DIV/0!

2.4 Return Period Scaling Factor

Building Importance level (from above): 2
Return Period Scaling factor from Table 3.1, **Factor C**: 1.00

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)		
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	0.00	0.00

2.6 Structural Performance Scaling Factor:

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

2.7 Baseline %NBS, (NBS%)b = (%NBS)nom x A x B x C x D x E

%NBSb: **#DIV/0!** **#DIV/0!**

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

3.1. Plan Irregularity, factor A: insignificant 1

3.2. Vertical irregularity, Factor B: insignificant 1

3.3. Short columns, Factor C: insignificant 1

3.4. Pounding potential
Pounding effect D1, from Table to right 1.0
Height Difference effect D2, from Table to right 1.0

Therefore, Factor D: 1

3.5. Site Characteristics insignificant 1

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

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

3.6. Other factors, Factor F

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

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

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

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

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

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Barnett Park Sports Pavilion block B			Reviewer:	Hamish Mackinven
	Unit	No:	Street	CPEng No:	1003941
Building Address:	B	200	Main Road, Redcliffs	Company:	GHD
Legal Description:				Company project number:	513059630
				Company phone number:	(03) 3780900
	Degrees	Min	Sec		
GPS south:	43	33	54.61	Date of submission:	
GPS east:	172	44	24.57	Inspection Date:	28/02/2014
				Revision:	
Building Unique Identifier (CCC):	PRK_1390_BLDG_001 EQ2			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	0
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):	2.00

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	2.00
Ground floor split?	no		Ground floor elevation above ground (m):	0.20
Storeys below ground	0			
Foundation type:	mat slab		if Foundation type is other, describe:	
Building height (m):	3.80		height from ground to level of uppermost seismic mass (for IEP only) (m):	3.8
Floor footprint area (approx):	124			
Age of Building (years):			Date of design:	1965-1976
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	public		Brief strengthening description:	
Use (upper floors):				
Use notes (if required):				
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	
Floors:	concrete flat slab	slab thickness (mm)	150
Beams:	none	overall depth x width (mm x mm)	
Columns:			
Walls:	partially filled concrete masonry	thickness (mm)	200

Lateral load resisting structure

Lateral system along:	partially filled CMU	**Note: Define along and across in detailed report!**	note total length of wall at ground (m):	33.8
Ductility assumed, μ:	1.25		wall thickness (m):	0.2
Period along:	0.40	0.40 from parameters in sheet	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	partially filled CMU		note total length of wall at ground (m):	17.8
Ductility assumed, μ:	1.25		wall thickness (m):	0.2
Period across:	0.40	0.40 from parameters in sheet	estimate or calculation?	
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:			
Roof Cladding:	Metal	describe	Lightweight corrugated steel
Glazing:			
Ceilings:	none		
Services(list):			

Available documentation

Architectural	none	original designer name/date	
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	Describe damage:	None
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: green

Along — Damage ratio: 0% — Describe (summary): minor cracking to east wall

Describe how damage ratio arrived at:

Across — Damage ratio: 0% — Describe (summary): no damage visible

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

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

Recommendations

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

Along — Assessed %NBS before: 74% ##### %NBS from IEP below

Assessed %NBS after: 74%

If IEP not used, please detail assessment methodology:

Across — Assessed %NBS before: 100% ##### %NBS from IEP below

Assessed %NBS after: 100%

IEP **Use of this method is not mandatory - more detailed analysis may give a different answer, which would take precedence. Do not fill in fields if not using IEP.**

Period of design of building (from above): 1965-1976

h_n from above: 3.8m

Seismic Zone, if designed between 1965 and 1992:

not required for this age of building

not required for this age of building

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

Note:1 for specifically design public buildings, to the code of the day: pre-1965 = 1.25; 1965-1976, Zone A =1.33; 1965-1976, Zone B = 1.2; all else 1.0

Note 2: for RC buildings designed between 1976-1984, use 1.2

Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)

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

2.2 Near Fault Scaling Factor

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

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

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:

Z_{1992}, from NZS4203:1992

Hazard scaling factor, **Factor B:** #DIV/0!

2.4 Return Period Scaling Factor

Building Importance level (from above): 2

Return Period Scaling factor from Table 3.1, **Factor C**:

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)		
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	0.00	0.00

2.6 Structural Performance Scaling Factor:

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

2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom} \times A \times B \times C \times D \times E$

$\%NBS_b$:	**#DIV/0!**	**#DIV/0!**

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

3.1. Plan Irregularity, factor A:	insignificant	1
3.2. Vertical irregularity, Factor B:	insignificant	1
3.3. Short columns, Factor C:	insignificant	1

3.4. Pounding potential

Pounding effect D1, from Table to right 1.0

Height Difference effect D2, from Table to right 1.0

Therefore, Factor D: 1

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

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

3.5. Site Characteristics	insignificant	1

3.6. Other factors, Factor F

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

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)

List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

3.7. Overall Performance Achievement ratio (PAR)	**0.00**	**0.00**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	**#DIV/0!**	**#DIV/0!**
4.4 Percentage New Building Standard (%NBS), (before)		**#DIV/0!**

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

Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
Draft	Kevan O Brien	Stephen Lee		Nick Waddington		02/08/12
Draft (Rev 1)	Kevan O Brien	Stephen Lee		Nick Waddington		07/08/12
Final	Kevan O Brien	Stephen Lee		Donna Bridgman		16/09/13
Final (Rev 1	Peter O'Brien	Hamish Mackinven		Donna Bridgman		14/03/14