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Sydenham Crèche
BU 2194-001 EQ2
Detailed Engineering Evaluation
Quantitative Report
Version FINAL

113 Huxley Street, Sydenham

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

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Quantitative Report
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113 Huxley Street, Sydenham

Christchurch City Council

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Sydenham Crèche

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

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Background

This is a summary of the Quantitative report for the above building structure, and is based in general on NZS 3604:2001 Timber-Framed buildings, the New Zealand Society for Earthquake Engineering (NZSEE) guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance and the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes as well as a full measure of the building carried out on 16 May 2012.

Brief Description

Sydenham Crèche is located at 113 Huxley Road, Sydenham, Christchurch. The site consists of a crèche building, portable office, car park and large outdoor play area. The exact date of construction of the original residential structure is unknown but it is known to have been constructed prior to 1970 and for the purposes of the qualitative analysis is assumed to have been constructed prior to 1965. An extension was constructed in 1971 and a further extension was added during a 1996 conversion of the building for use as a crèche.

The crèche roof is of timber framed construction and has a low pitch of approximately 2°. Roof cladding consists of long-run ribbed steel on timber sarking. The sarking is directly fixed to the timber rafters. The underside of the rafters is lined with a plasterboard ceiling.

After discussion with the tenant it was noted that the general construction of the crèche is unreinforced concrete masonry forming both internal and external walls. Electromagnetic scans of the masonry walls revealed that there is no reinforcement in any of the walls scanned. As a result of opening up works it was discovered that the 1971 extension is of timber framed construction with an unreinforced concrete masonry unit veneer to the exterior. The 1996 extension is of lightweight timber frame construction.

The foundations to the original structure and the 1971 extension consist of suspended timber flooring on timber bearers supported by concrete piles internally and concrete strip footings to the perimeter and internal concrete masonry walls. The foundations to the 1996 extension consist of a reinforced concrete slab floor with reinforced concrete strip footing, on compacted hard fill.

Key Damage Observed

Key damage observed includes:-

- Shear cracking to south-eastern corner
- Cracking to internal walls
- Cracking to timber subfloor bearers

Quantitative Detailed Engineering Evaluation Assessment

Based on the information available, opening up works carried out, the current New Zealand Standards and the guidelines set out by the NZSEE, the building's capacity has been assessed to be in the order of 8% NBS and post-earthquake capacity is also in the order of 8% NBS.

The building has been assessed to have a seismic capacity in the order of 8% NBS and is therefore potentially Earthquake Prone.

Recommendations

As the building has been assessed to have a %NBS less than 33%NBS, it is deemed to be Earthquake Prone. It is recommended that strengthening options be explored to bring the %NBS of the building up to the required 67% in order to comply with Christchurch City Council policy regarding the strengthening of potentially Earthquake Prone buildings.

1. Background

GHD Limited has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of Sydenham Crèche.

This report is a Quantitative Assessment and is based in general on NZS 1170: 2002 Structural Design Actions, NZS 3604: 2011 Timber Framed Buildings and the New Zealand Society for Earthquake Engineering (NZSEE) guidelines for the Assessment and Improvement of Unreinforced Concrete Masonry Buildings for Earthquake Resistance and the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes.

A quantitative assessment of masonry structures involves an analysis of the seismic and gravity loads that a building is subjected to, analysis of the distribution of these forces throughout the structure and an analysis of the capacity of the 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 %NBS of each of the structural elements.

Also, a site measure of the timber framed portion of the building was carried out which was used to determine the buildings bracing capacity in accordance with manufacturers' guidelines where available. The earthquake bracing demand for the building is determined in accordance with NZS 3604: 2011 and the percentage of new building standard (%NBS) is assessed.

At the time of this report, no finite element modelling of the building structure had been carried out. The detailed analysis consisted of a bracing calculation of the timber framed portion of the structure and a capacity check of the structural masonry elements of the building, no further analysis or calculations were 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

Sydenham Crèche is located at 113 Huxley Road, Sydenham, Christchurch. The site consists of a crèche building, portable office, car park and large outdoor play area. The exact date of construction of the original residential structure is unknown but it is known to have been constructed prior to 1970 and for the purposes of the qualitative analysis is assumed to have been constructed prior to 1965. An extension was constructed in 1971 and a further extension was added during a 1996 conversion of the building for use as a crèche.

The crèche roof is of timber framed construction and has a low pitch of approximately 2°. Roof cladding consists of long-run ribbed steel fixed to 25mm timber sarking. The sarking is directly fixed to the 200 x 50mm timber rafters at approximately 450mm centres. The underside of the rafters is lined with a 12.5mm thick plasterboard ceiling.

After discussion with the tenant it was noted that the general construction of the crèche is unreinforced concrete masonry forming both internal and external walls. Electromagnetic scans of the masonry walls revealed that there is no reinforcement in any of the walls scanned. Original external concrete masonry unit walls of the building are unfilled and unreinforced 20 series units. Original internal concrete masonry unit walls are unfilled and unreinforced 15 series units. As a result of opening up works it was discovered that the 1971 extension is of timber framed construction with a 10 series unreinforced concrete masonry unit veneer to the exterior. The 1996 extension is of lightweight timber frame construction. Internal wall linings consist of a plaster finish to masonry walls, along with plasterboard and timber panel lining to the 1971 and 1996 extensions. Exterior cladding of the 1996 extension is provided by a weatherboard plaster cladding system whilst the finish to the existing wall is textured plaster.

The foundations to the original structure and the 1971 extension consist of suspended timber flooring on timber bearers supported by concrete piles internally and concrete strip footings to the perimeter and internal concrete masonry walls. The foundations to the 1996 extension consist of a reinforced concrete slab floor with reinforced concrete strip footing, on compacted hard fill.

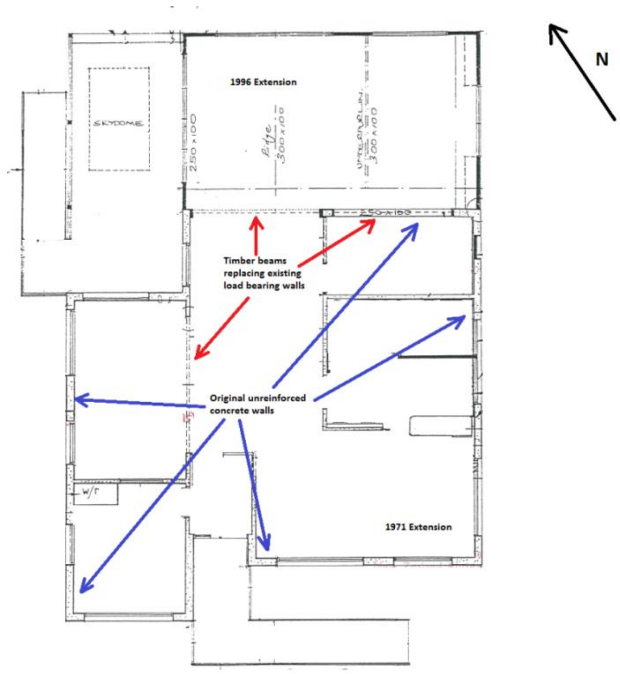


Figure 2 Plan sketch showing key structural elements

The dimensions of the crèche are approximately 15m long by 12m wide and 3m tall. The overall footprint of the building is 160m^2 whilst the nearest waterway to the site is Heathcote River located approximately 450m to the south-east. The site is predominantly flat with a gentle slope from the south-west of the building towards Huxley Street.

Limited plans and drawings were available for this building.

4.2 Gravity Load Resisting System

The gravity loads in the original portion of the structure are transferred from the roof cladding down to the supporting timber sarking. The sarking is supported by the timber rafters at 450mm centres. The rafters span between the internal and external concrete masonry walls and loads from the rafters are transferred back to these supports. Where original masonry walls have been removed, 250 x 100mm timber lintels have been put in place to transfer loads back to the supporting walls. Gravity loads are then transferred down through the masonry walls to the supporting strip foundations and through to the ground below.

Gravity loads acting on the 1996 timber framed extension are transferred down through the roof structure in the same was as for the older portion of the building. The roof spans between a number of 300 x 100mm timber beams and the external walls. The timber beams transfer the loads back to the timber framed external wall of the extension to the northeast and the timber lintel where the original masonry walls were removed to the southwest. Loads are transferred down through the walls to the strip foundations below and through to the ground.

Internal loads are transferred through the suspended timber floor system to the concrete pile foundations and through the slab on grade to the ground below.

4.3 Lateral Load Resisting System

As a result of the square sarking, both the timber floor and timber flat roof form diaphragms to transfer seismic actions to the lateral load resisting walls. The sarking serves the purpose of resisting shear forces in the diaphragm. Most often, the sarking was nailed with 8d or 10d nails, with two or more nails per sarking board at each support. Shear forces perpendicular to the direction of the sarking are resisted by the nail couple. Shear forces parallel to the direction of the sarking are transferred through the nails in the supporting framing members below the sarking joints.

Resistance to lateral loads in the original portion of the building is provided by the unreinforced concrete masonry walls in both the along and across directions of the building.

Lateral load resistance in the 1996 timber framed extension is provided by the wall linings in both directions. In the transverse direction, the linings that provide bracing are British Plaster Board (BPB) Type CP6. In the longitudinal direction, the linings that provide the bracing are HardiFlex Type HF1 and BPB Type CP4. A bracing schedule was carried out in 1996 to calculate the required bracing but only the extension was considered in the calculations.

5. Assessment

5.1 Qualitative Assessment

An initial qualitative assessment has been competed by GHD for the building. This included a visual inspection of the building which was undertaken on 19th of January 2012. Both the interior and exterior of the building were inspected. The main structural elements of the building were the timber framed flat roof with lightweight cladding, the unreinforced concrete masonry walls of the original building and the plasterboard lined timber framed walls of the 1996 extension. No diagonal bracing was visible in the roof.

The visual inspection consisted of observing the building to determine the structural systems and likely behaviour of the building during an earthquake. The site was assessed for damage, including observing the ground conditions, checking for damage in 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 review of available drawings was also carried out.

Based on the information available, and using the NZSEE Initial Evaluation Procedure, the buildings original capacity has been assessed to be in the order of 10% NBS and post-earthquake capacity in the order of 10% NBS. The buildings post-earthquake capacity excluding critical structural weakness is in the order of 21% NBS.

5.2 Quantitative Assessment

5.2.1 Building Demand

Masonry Portion of Structure

Self-weight of the structure was calculated from the Table A1 of Appendix A of NZS 1170.1: 2002.

For nominally ductile and brittle structures a seismic action set comprising 100% of the specified earthquake actions in one direction plus 30% of the specified earthquake actions in the orthogonal direction to this was applied as set out in Cl 5.3.1.2 of NZS 1170.5: 2004. A torsion analysis of the concrete masonry portion of the building was carried out to determine the in-plane shear demand (V*) perpendicular and parallel to each wall. Each wall's maximum base shear demand was calculated and multiplied by the seismic weight coefficient in order to obtain the design base shear.

Timber Framed Portion of the Structure

The earthquake bracing demand on the structure was determined in accordance with Section 5 of NZS 3604: 2011. The bracing unit demand per square metre was determined from Table 5.10. The building is located in Christchurch (zone 2) on class D soils. Therefore a multiplication factor of 0.8 is applied in accordance with Table 5.10 of NZS 3604: 2011.

5.2.2 Seismic Weight Coefficient

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

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

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 structural ductility factor, was taken as 1.00.

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

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

5.2.3 In-Plane Capacity

The in-plane capacity of the concrete masonry wall was determined using the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance. The NZSEE guidelines recommend checks for 5 different in-plane response modes.

1. Diagonal tension failure mode
2. Bed-sliding failure mode
3. Toe crushing failure mode
4. 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. Details of these assessment procedures are included in Appendix D.

5.2.4 In-plane Wall Properties

Properties of in-plane loaded URM walls, piers or spandrels for use in the calculation of nominal in-plane shear capacity are as follows.

Unit Weight of Masonry

The unit weight of masonry, γ_m, shall be calculated in accordance with the equation.

$$\gamma_m = \rho_m \times 9.81$$

Where: acceleration due to gravity is taken as 9.81 ms^{-2} and ρ_m is determined in accordance with Section 2.5.2 of NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance.

Weight of Wall

The weight of the wall, W_w, shall be calculated in accordance with the equation.

$$W_w = \gamma_m \times b_w \times l_w \times h$$

Normal Force at Base of Wall

The normal force acting on the cross section of the base of the wall, N_b, shall be calculated in accordance with the equation.

$$N_b = W_w + N_t$$

Diagonal Tension Strength

The diagonal tension strength of masonry, f_{dt}, shall be calculated in accordance with the equation below for walls, piers and spandrels.

$$f_{dt} = \frac{1}{2}\left(c + \frac{N_t}{A_w} 0.8\mu_f\right)$$

Where: Values for cohesion, c, and coefficient of friction, μ_f, are given in Section 2.5.5 of NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance. The factor of 0.8 is to account for vertical accelerations and other dynamic effects.

Distance to Centre of Inertia of Wall

Distance to the centre of inertia of the wall from the compression toe, a_i, shall be calculated in accordance with the equation for walls with no flanges:

$$a_i = 0.5 \times l_w$$

Average Compressive Stress

Average compressive stress acting on the wall, σ_{ave}, shall be calculated in accordance with the equation

$$\sigma_{avg} = \frac{N_t}{l_w . b_w}$$

5.2.5 Solid In-plane Wall Nominal Shear Capacity

The in-plane nominal shear capacity of a wall, pier or spandrel shall be 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}.

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

Capacity in Diagonal Tension Failure Mode

Nominal shear capacity corresponding to diagonal tension failure, V_{dt}, shall be calculated in accordance with the equation below for walls where no perpendicular flanges are present

$$V_{dt} = 0.54. b_w. l_w. \zeta. f_{dt}. \sqrt{\left(1 + \frac{\sigma_{avg}}{f_{dt}}\right)}$$

Where: ζ is a factor to correct for nonlinear stress distribution (See Table 2)

	ζ
Slender walls, where $h/l_w > 2$	1.5
Stout walls, where $h/l_w < 0.5$	1.0
1. Linear interpolation may be used for values of h/l_w	

Table 2 Shear stress factor for inclusion in diagonal tension failure mode equation

Capacity in Rocking Failure Mode

Nominal shear capacity corresponding to the rocking failure mode, V_r, shall be calculated in accordance with the equation

$$V_r = \frac{N_b}{h}. \left[a_i - \frac{l_{er}}{3}\right]$$

Where: l_{er} is the effective length of the wall in rocking, taken as 0.1 x l_w.

Capacity in Bed-joint Sliding Failure Mode

Bed-joint sliding failure is not an expected behaviour of URM walls subjected to seismic loading. The bed-joint sliding capacity of an in-plane loaded wall need only be assessed when conditions suit the initiation of bed-joint sliding, specifically, when either or both the brick compressive strength and mortar compressive strength fall in the bounds of "soft".

Ultimate shear capacity corresponding to bed-joint sliding failure, *Vs*, shall be calculated in accordance with the equation

$$V_s = l_w. b_w. c + 0.8. \mu_f. N_t$$

Where: Values for cohesion, *c*, and coefficient of friction, μ_f, are given in Section 2.5.5 of NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance. The factor of 0.8 is to account for vertical accelerations and other dynamic effects.

Capacity in Toe Crushing Failure Mode

The toe crushing failure mode is not an expected response of typical New Zealand URM walls to in-plane seismic loading. For this reason toe crushing failure need only be assessed when the wall has been retrofitted with unbonded post-tensioning or a seismic improvement intervention that inhibits the formation of the diagonal tension failure mode.

Nominal shear capacity corresponding to toe crushing failure, V_{tc}, shall be calculated in accordance with the below equation for walls where no perpendicular flanges are present.

$$V_{tc} = \frac{N_b}{h}.\left[\frac{1}{2}.l_w - \frac{1}{3}.l_{etc}\right]$$

where no perpendicular flanges are present

$$l_{etc} = \frac{2.N_b}{1.3.f'_m.b_w}$$

5.2.6 Out-of-Plane Capacity

The % NBS for out-of-plane flexure of the concrete masonry walls was determined using the methods set out in NZSEE guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes Section 10.3.4 (b). The following steps are those required to assess the displacement response capability and the displacement demand, from which the adequacy of the walls can be determined.

The wall panel is assumed to form hinge lines at the points where effective horizontal restraint is assumed to be applied. The centre of compression on each of these hinge lines is assumed to form a pivot point. The height between these pivot points is the effective panel height *h*. At mid-height between these pivots, height *h/2* from either, a third pivot point is assumed to form.

Step 1

Divide the wall panel into two parts, a top part bounded by the upper pivot and the mid-height between the top and bottom pivots, and a bottom part bounded by the mid-height pivot and the bottom pivot.

Step 2

Calculate the weight of the wall parts, W_b of the bottom part and W_t of the top part, and the weight acting at the top of the storey, *P*.

Step 3

From the nominal thickness of the wall, t_{nom}, calculate the effective thickness, *t.*

$$t = t_{nom}\left(0.975 - 0.025\frac{P}{W}\right)$$

Step 4

Assess the maximum distance, e_p, from the centroid of the top part of the wall to the line of action of *P*, and similarly e_b, e_t and e_o. Usually, the eccentricities e_b and e_p will each vary between 0 and *t/2* (where *t* is the effective thickness of the wall). Exceptionally they may be negative.

Step 5

Calculate the mid-height deflection, Δ_i, which would cause instability under static conditions. The following formula may be used to calculate this deflection.

$$\Delta_i = \frac{b\mathrm{h}}{2a}$$

Where

$$b = W_b e_b + W_t(e_0 + e_b + e_t) + P(e_0 + e_b + e_t + e_p) - \Psi(W_b y_b + W_t y_t)$$

And

$$a = W_b y_b + W_t\left(\frac{h}{2} + y_t\right) + Ph$$

And

$$\Psi = Initial\ slope\ of\ wall$$

Step 6

Assign a maximum usable deflection, Δ_m, as 0.6 Δ_i.

Step 7

Calculate the period of the wall, T_p, as four times the duration for the wall to return from a displaced position measured by Δ_m to the vertical. The period may be calculated from the following equation.

$$T_p = 6.27\sqrt{\frac{J}{a}}$$

Where J is the rotational inertia of the masses associated with *W* and *P* and any ancillary masses, and is given by the following equations.

Case number	0	1	2	3
e_p	0	0	$t/2$	$t/2$
e_b	0	$t/2$	0	$t/2$
b	$(W/2+P)t$	$(W+3P/2)t$	$(W/2+3P/2)t$	$(W+2P)t$
a	$(W/2+P)h$	$(W/2+P)h$	$(W/2+P)h$	$(W/2+P)h$
$\Delta_i = bh/(2a)$	$t/2$	$\frac{(2W+3P)t}{(2W+4P)}$	$\frac{(W+3P)t}{(2W+4P)}$	t
J	$\{(W/12)[h^2+7t^2]+Pt^2\}/g$	$\{(W/12)[h^2+16t^2]+9Pt^2/4\}/g$	$\{(W/12)[h^2+7t^2]+9Pt^2/4\}/g$	$\{(W/12)[h^2+16t^2]+4Pt^2\}/g$
C_m	$(2+4P/W)t/h$	$(4+6P/W)t/h$	$(2+6P/W)t/h$	$4(1+2P/W)t/h$

Step 8

Calculate the seismic coefficient ($C_p(T_p)$) for an elastically responding part (μ_p = 1) with this period (T_p), that applies at this elevation in the building.

$$C_p(T_p) = C(0)C_{Hi}C_i(T_p)$$

Where

C(0) = the site hazard coefficient for T = 0 determined from NZS 1170.5 Section 3.1, using the values for the modal response spectrum method and numerical integration time history methods

C_{Hi} = the floor height coefficient for level I, from NZS 1170.5 Section 8.3.

$C_i(T_p)$ = the part spectral shape factor at level I, from NZS 1170.5 Section 8.4

Step 9

Calculate γ the participation factor for the rocking system. This factor may be taken as

$$\gamma = \frac{Wh^2}{8J}$$

This may be taken as a maximum of 1.5.

Step 10

From $C_p(T_p)$, T_p, R_p *and* γ calculate the displacement response, D_{ph} from;

$$D_{ph} = \gamma\left(\frac{T_p}{2\pi}\right)^2 \times C_p(T_p) \times R_p \times g$$

Where R_p is from NZS 1170.5 Table 8.1

Step 11

Calculate the % NBS

$$\%NBS = 0.72\frac{\Delta_i}{D_{ph}}$$

5.2.7 Timber Framed Wall Bracing Capacity

The extension to the building was constructed in 1996 and a bracing schedule for the extension was carried out and included in the specifications for the consent application. Two different bracing systems were used for the construction, HardiFlex and BPB. Bracing capacities for the different systems for the design period were available from the manufacturers. Bracing capacity in both directions of the extension were calculated and compared with the bracing demand to calculate the % NBS.

$$\%\mathrm{NBS} = \frac{\mathrm{BU_{provided}}}{\mathrm{BU_{demand}}} \text{ x } \%100$$

6. Geotechnical Consideration

The site is relatively flat at approximately 7m above mean sea level. It is approximately 700m west of the Heathcote River, and 5km west of Pegasus Bay.

6.1 Published Information on Ground Conditions

6.1.1 Published Geology

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

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

6.1.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that a number of boreholes are within 200m of the site (see Table 2). Of those considered relevant, three lithographic logs indicate that the ground conditions are predominantly layers of sand and silt. Varying amounts of gravel and clay are also indicated to be present, along with varying depths of fill material (rubble).

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M36/8732	1.6m	N/A	140m N
M36/8761	3m	N/A	100m W
M36/9706	6m	N/A	170m NW

Table 3 ECan Borehole Summary

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

6.1.3 EQC Geotechnical Investigations

The Earthquake Commission has undertaken geotechnical testing in the area of the site. Information pertaining to this investigation is included in Tonkin and Taylor Report[2]. Within 300m of the property, two investigation points were undertaken, the results of which are detailed below in Table 3.

[1] Brown, L. J. and Weeber J.H. (1992): *Geology of the Christchurch Urban Area*. Institute of Geological and Nuclear Sciences
[2] Tonkin and Taylor . September 2011: Christchurch Earthquake Recovery, Geotechnical Factual Report, Sydenham

Bore Name	Orientation from Site	Log Summary
CPT-SYD-01	100m NW	0 – 1.0m Pre-drilled 1.0 – 2.2m Dense Sands 2.2 – 2.7m Sand and Silt 2.7m Refusal (Likely Gravel)
CPT-SYD-14	280m S	0 – 3.5m Firm to stiff Sandy Silt 3.5 – 5.5m Loose to medium dense SAND 5.5 – 7.0m Clayey SILT 7.0 – 10.8m Dense SAND 10.8 – 25.2m Layers of Firm to stiff SILT, and Medium dense to dense SAND

Table 4 EQC Geotechnical Investigation Summary Table

Initial observations of the CPT results indicate the soils at site -01 are typically dense sand, overlying gravel at 2.5 to 3m. Site -14 is indicated as being underlain by layers of typically medium dense sand and typically firm silt. This would infer that liquefaction is possible in a significant seismic event, but is not likely to be severe.

6.1.4 CERA Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has zoned the site Green, indicating that 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, yellow) - that minor to moderate land damage from liquefaction is possible in future significant earthquakes.

6.1.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows no signs of liquefaction outside the building footprint or adjacent to the site, as shown in Figure 3**.**

Figure 3 Post February 2011 Earthquake Aerial Photography[3]

6.1.6 Assessment of Ground Conditions from Desk-top Study

From the Ecan borehole information and EQC geotechnical investigations, the ground conditions adjacent to the site comprise of predominantly layers of sand and silt. Varying amounts of gravel and clay are also indicated to be present.

6.2 Seismicity

6.2.1 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 in Table 4 below.

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	130 km	NW	8.3	~300 years
Greendale (2010) Fault	22 km	W	7.1	~15,000 years
Hope Fault	110 km	N	7.2~7.5	120~200 years

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

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Kelly Fault	110 km	NW	7.2	~150 years
Porters Pass Fault	75 km	NW	7.0	~1100 years

Table 5 Summary of Known Active Faults[4,5]

Recent earthquakes since 4 September 2010 have identified the presence of a previously unmapped active fault system underneath Canterbury, 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 established.

6.2.2 Ground Shaking Hazard

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.

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.

Liquefaction was not observed during the site inspection on 19 February 2011. However, based on the desktop study, due to the anticipated presence of predominantly sand and silt beneath the site, it was considered that liquefaction was likely to occur at this site. It was considered that possible liquefaction could occur where sands and silts are present.

6.3 Field Investigations

In order to further understand the ground conditions at the site, intrusive testing comprising two piezocone CPT investigations was conducted at the site on 02 April 2012.

The location of the test is tabulated in Table 5.

Investigation	Depth (m bgl)	Easting (NZMG)	Northing (NZMG)
CPT 20A	1.0	2481194	5739386
CPT 20B	1.0	2481193	5739385

Table 6 Coordinates of Investigation Locations

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

The CPT investigations were undertaken by McMillan Drilling Service on 02 April 2012, typically to a target depth of 20m below ground level. However, refusal was reached at depth of 1.0m due to the presence of dense gravels.

Interpretation of output graphs[6] from the investigation showing Cone Tip Resistance (q_c), Friction Ratio (Fr), Inferred Lithology and Inferred Liquefaction Potential are presented in

CPT	**Depth (m)**	**Lithology [1]**	**Cone Tip Resistance qc (MPa)**	**Friction Ratio Fr (%)**
20 A	0 – 1.0	SAND	~5	~1
	>1.0	Gravel - Refusal	> 40	-
20 B	0 – 1.0	SAND	~5	~1
	>1.0	Gravel - Refusal	> 40	-

Table 7.

6.4 Ground Conditions Encountered

CPT	**Depth (m)**	**Lithology [1]**	**Cone Tip Resistance qc (MPa)**	**Friction Ratio Fr (%)**
20 A	0 – 1.0	SAND	~5	~1
	>1.0	Gravel - Refusal	> 40	-
20 B	0 – 1.0	SAND	~5	~1
	>1.0	Gravel - Refusal	> 40	-

Table 7 outlines a summary of the ground conditions encountered.

CPT	**Depth (m)**	**Lithology [1]**	**Cone Tip Resistance q_c (MPa)**	**Friction Ratio Fr (%)**
20 A	0 – 1.0	SAND	~5	~1
	>1.0	Gravel - Refusal	> 40	-
20 B	0 – 1.0	SAND	~5	~1
	>1.0	Gravel - Refusal	> 40	-

Table 7 Summary of CPT-Inferred Lithology

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

6.5 Interpretation of Ground Conditions

6.5.1 Liquefaction Assessment

From the CPT site investigations carried out, the ground conditions underlying the site are understood to be predominantly sand overlying dense gravels.

Although other investigations in the general area show variable sand, silt and gravel subsurface conditions, the crèche site is considered to have negligible susceptibility to liquefaction, based on the following evidence:

- Dense gravel being encountered at shallow depth by the two CPT investigations at the site.
- Post- 22 February earthquake aerial photography not showing signs of liquefaction at the site.

6.5.2 Slope Failure and/or Rockfall Potential

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

6.5.3 Foundation Recommendations

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

- In accordance with NZS 1170.5:2004, a soil class of **D** (Deep or Soft Soil) should be adopted for the site;
- Should the building be considered for rebuild, standard foundations for TC2 zoned land, as per the DBH guidelines, can be used for this structure; and,
- Ground improvement works for this particular building are not considered necessary.

7. Survey

A level survey will not be required as there is no evidence of significant liquefaction or ground settlement.

8. Initial Capacity Assessment

8.1 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: D, NZS 1170.5:2004, Clause 3.1.3, Soft Soil
- Return period factor R_u = 1.0, NZS 1170.5:2004, Table 3.5, Importance Level 2 structure with a 50 year design life.

8.2 In-Plane Demand

The base shear demand of each wall highlighted in Figure 4 was assessed by completing a torsion analysis of the building which spread the seismic load between the walls of the building based on the dimensions of each wall. Loads were calculated both in and out of plane and the design base shear for each wall was calculated in accordance with Cl 5.3.1.2 of NZS 1170.5: 2004. Following the analysis it was found that Wall 7 has the lowest %NBS. The maximum base shear, V*, for Wall 7 is:

V* = 13.3kN

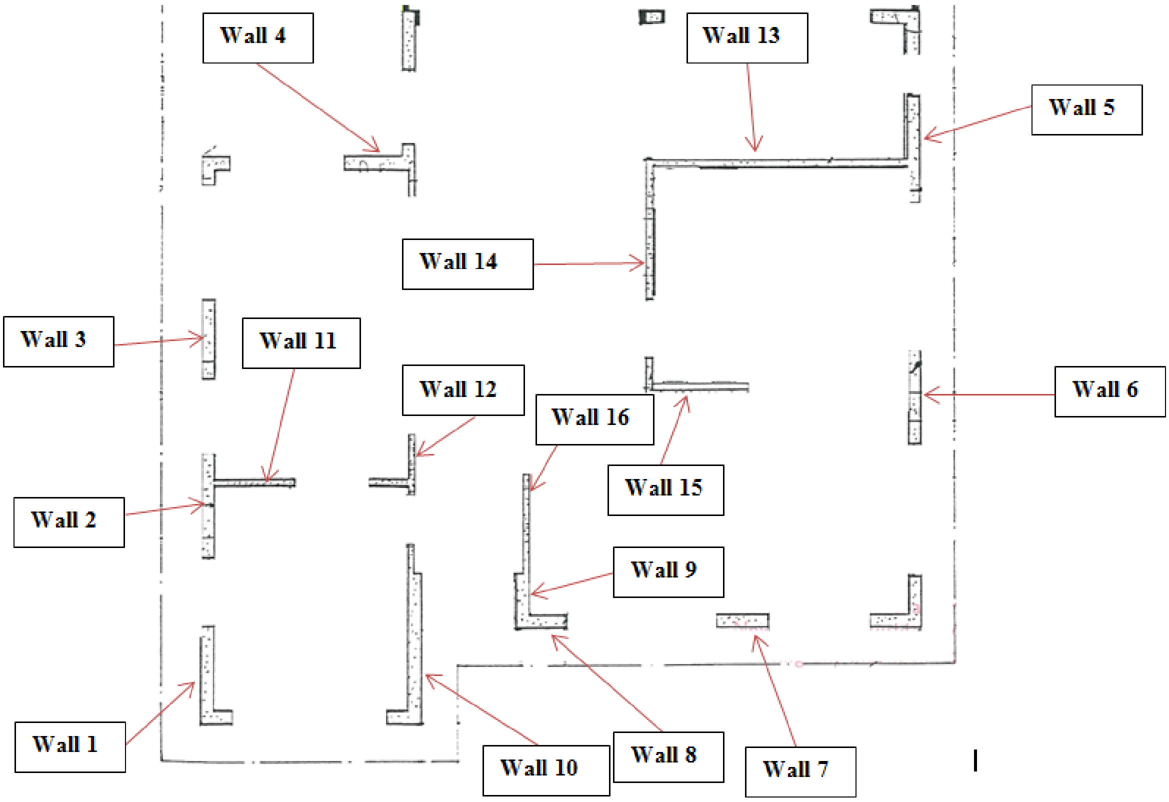


Figure 4 Walls assessed

8.3 In-Plane Capacity

The in-plane capacity of each wall highlighted in Figure 4 was calculated and the critical wall was assessed as being Wall 7. The in-plane capacities for Wall 7 are given in Table 8.

NZSEE Failure Mode	Component	kN
8.4.1 Diagonal Tension Failure Mode	$V_{dt} = 0.54.b_w.l_w.\zeta.f_{dt}.\sqrt{\left(1+\frac{\sigma_{avg}}{f_{dt}}\right)}$	18.1
8.4.2 Rocking Failure Mode	$V_r = \frac{N_b}{h}.\left[a_i - \frac{l_{er}}{3}\right]$	1.2
8.4.3 Bed-joint Sliding Failure Mode	$V_s = l_w.b_w.c + 0.8.\mu_f.N_t$	40.9
8.4.4 Toe Crushing Failure Mode	$V_{tc} = \frac{N_b}{h}.\left[\frac{1}{2}.l_w - \frac{1}{3}.l_{etc}\right]$	1.3
	Capacity Critical	1.2

Table 8 In-Plane Capacities

8.4 In-Plane % NBS

Following detailed calculations being carried out, the in-plane %NBS was found to be in the order of 9%.

8.5 Out-of-Plane Flexure % NBS

The out-of-plane flexural capacities of all the walls were calculated using the process outlined in the NZSEE Guidelines Section 10.3. The out-of-plane flexural capacity of the internal walls was found to be in the order of 8% NBS. The out-of-plane flexural capacity of the external walls was found to be in the order of 11% NBS.

8.6 Wall Bracing Demand

In accordance with Table 5.10 of NZS 3604: 2011, for a light roof, light cladding with a pitch between 0°-25° then a bracing demand of 6 BU/m^2 is taken.

In accordance with Table 5.10 for Earthquake Zone 2 which covers Christchurch and for soil class D, the bracing demand is reduced by a factor of 0.8 and so the total building demand for the building is;

$$\mathrm{BU_{demand}} = (0.8 \text{ x } 6 \ \mathrm{BU/m^2} \text{ x Floor area })$$

$$= 201.6 \text{ BU}$$

8.7 Wall Bracing Capacity

The bracing capacity of the building was assessed using strengths from the manufacturer's technical data. The results of the bracing capacity analysis can be seen in Table 9 and Table 10.

Bracing Line	Bracing Capacity (BU)
1	178
Total bracing capacity =	178 BU

Table 9 Bracing capacity along the timber framed extension

Bracing Line	Bracing Capacity (BU)
1	226
2	13
Total bracing capacity =	239 BU

Table 10 Bracing capacity across the timber framed extension

8.8 Wall Bracing % NBS

The % NBS of the timber framed extension is outlined, in both the across and along directions of the building, in table 11.

Direction	%NBS
Across	100
Along	88

Table 11 %NBS results from detailed wall bracing calculations for timber framed extension

8.9 Discussion of Results

Following a detailed assessment, the building has been assessed as achieving 8**%** New Building Standard (NBS). Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the building is considered potentially an Earthquake Prone building as it does not achieve above 33% NBS.

The results obtained are consistent with those expected for a building of unreinforced concrete masonry construction designed pre-1970s. As the walls of the building are unreinforced the flexural capacity of the walls is significantly lower than required to meet the 67% NBS required.

8.10 Occupancy

As the building has been assessed to have a %NBS less than 34%NBS, it is deemed to be a potentially Earthquake Prone. It is recommended that strengthening options are implemented to increase seismic capacity along and across the building to achieve a minimum of 67% NBS across and along the building. As the building has achieved such a low % NBS and failure of the structural systems is likely to be in a brittle manor, it is recommended that general occupancy of the building is not permitted.

9. Strengthening

As the %NBS of the building has been assessed at 8%, additional works are required to increase the in and out of plane capacity to achieve the minimum 67% as recommended by The New Zealand Society for Earthquake Engineering (NZSEE).

10. Recommendations

As the building has been assessed to have a %NBS less than 33%NBS, it is deemed to be Earthquake Prone. It is recommended that strengthening options be explored to bring the %NBS of the building up to the required 67% in order to comply with Christchurch City Council policy regarding the strengthening of potentially Earthquake Prone buildings.

11. Limitations

11.1 General

This report has been prepared subject to the following limitations:

- No intrusive geotechnical investigations have been undertaken.
- No level or verticality surveys have been undertaken.
- No calculations, other than the wall bracing calculations, shear and moment capacity checks included in this report 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.

11.2 Geotechnical Limitations

This report presents the results of a geotechnical appraisal prepared for the purpose of this commission, and prepared solely for the use of Christchurch City Council and their advisors. The data and advice provided herein related 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.

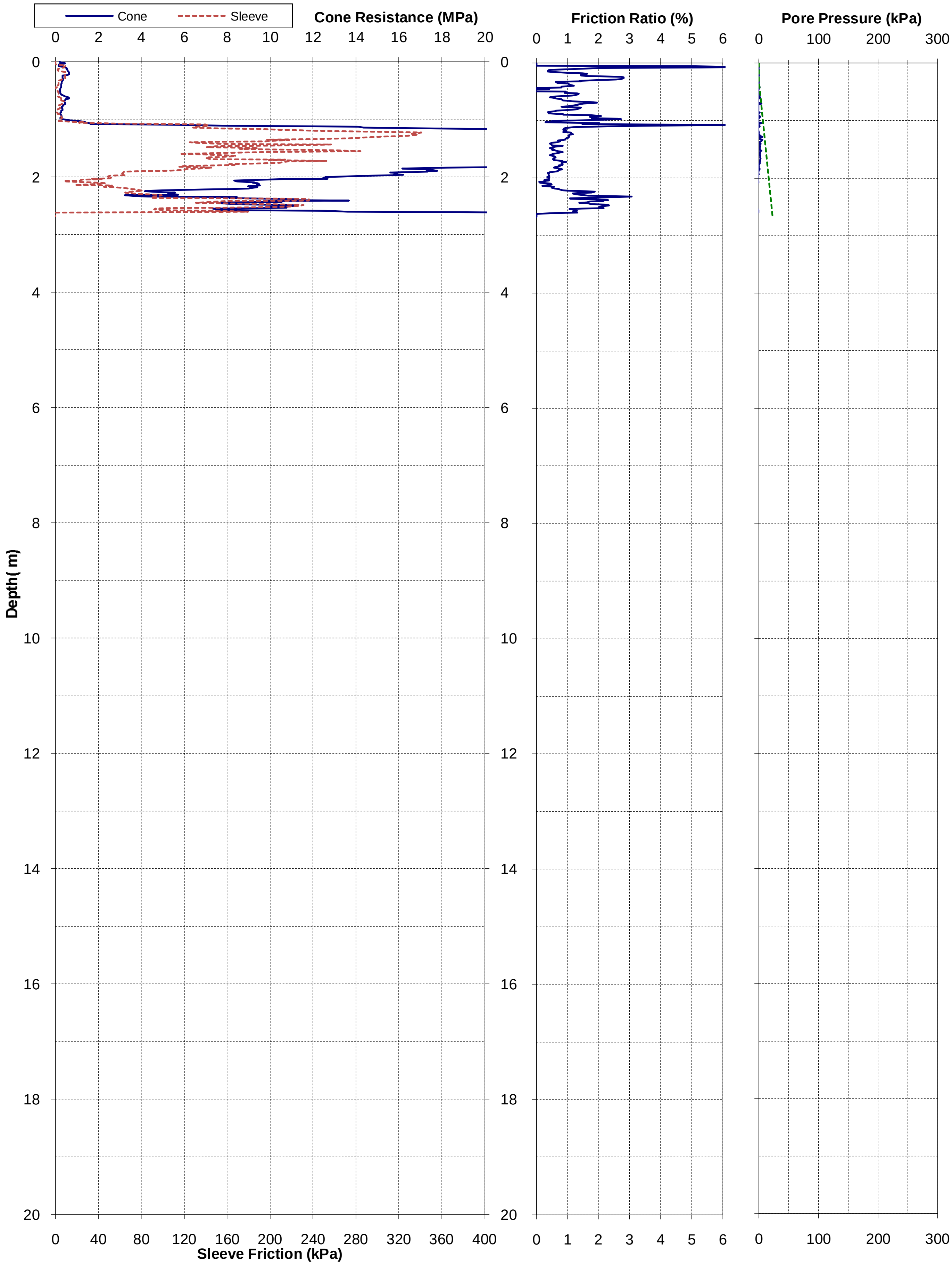
The advice tendered in this report is based on a visual geotechnical appraisal. No subsurface investigations have been conducted. An assessment of the topographical land features have been made based on this information. It is emphasised that geotechnical conditions may vary substantially across the site from where observations have been made. Subsurface conditions, including groundwater levels can change in a limited distance or time. In evaluation of this report cognisance should be taken of the limitations of this type of investigation.

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.

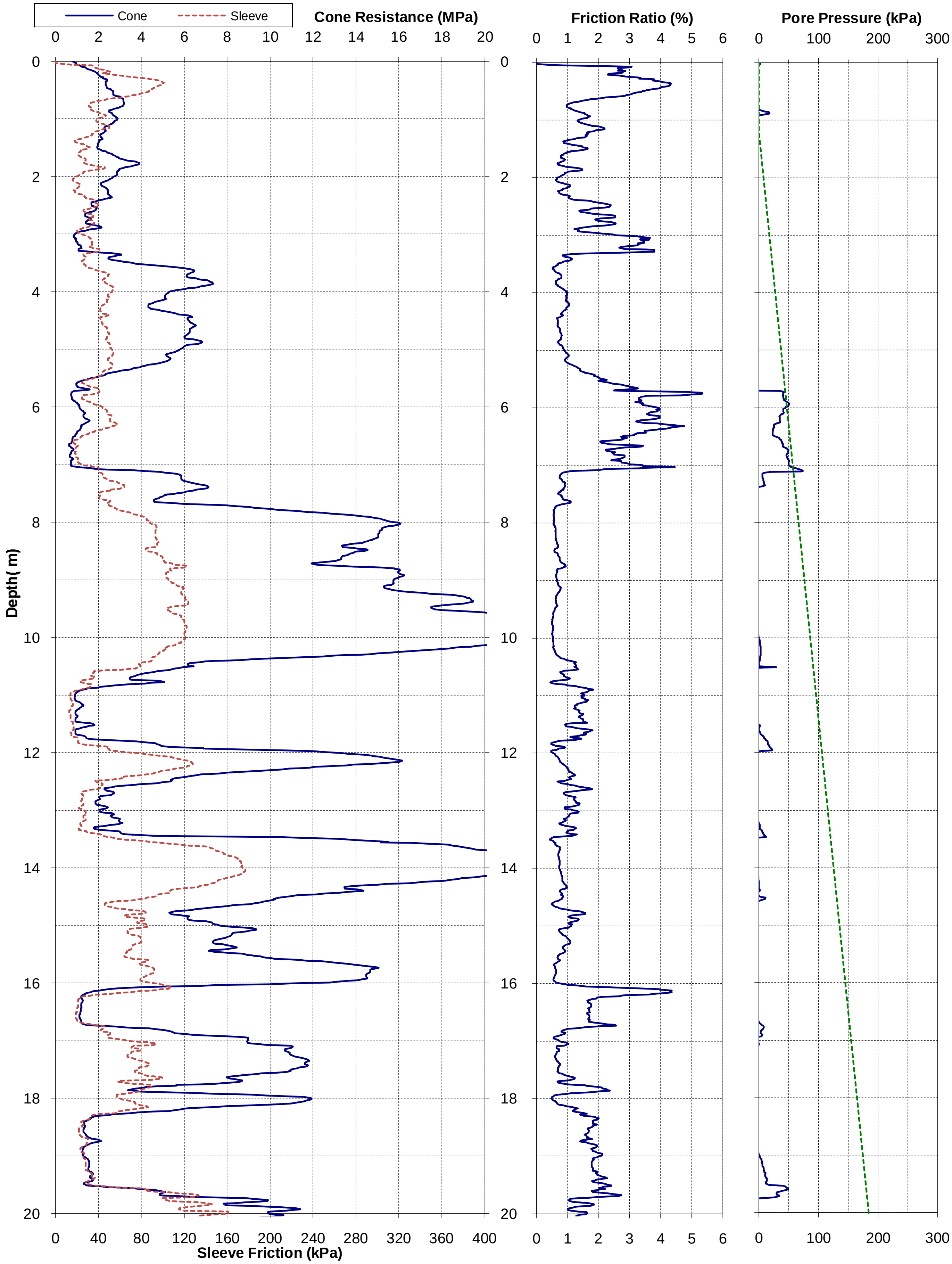
Appendix A

Geotechnical Investigation Reports and Analysis

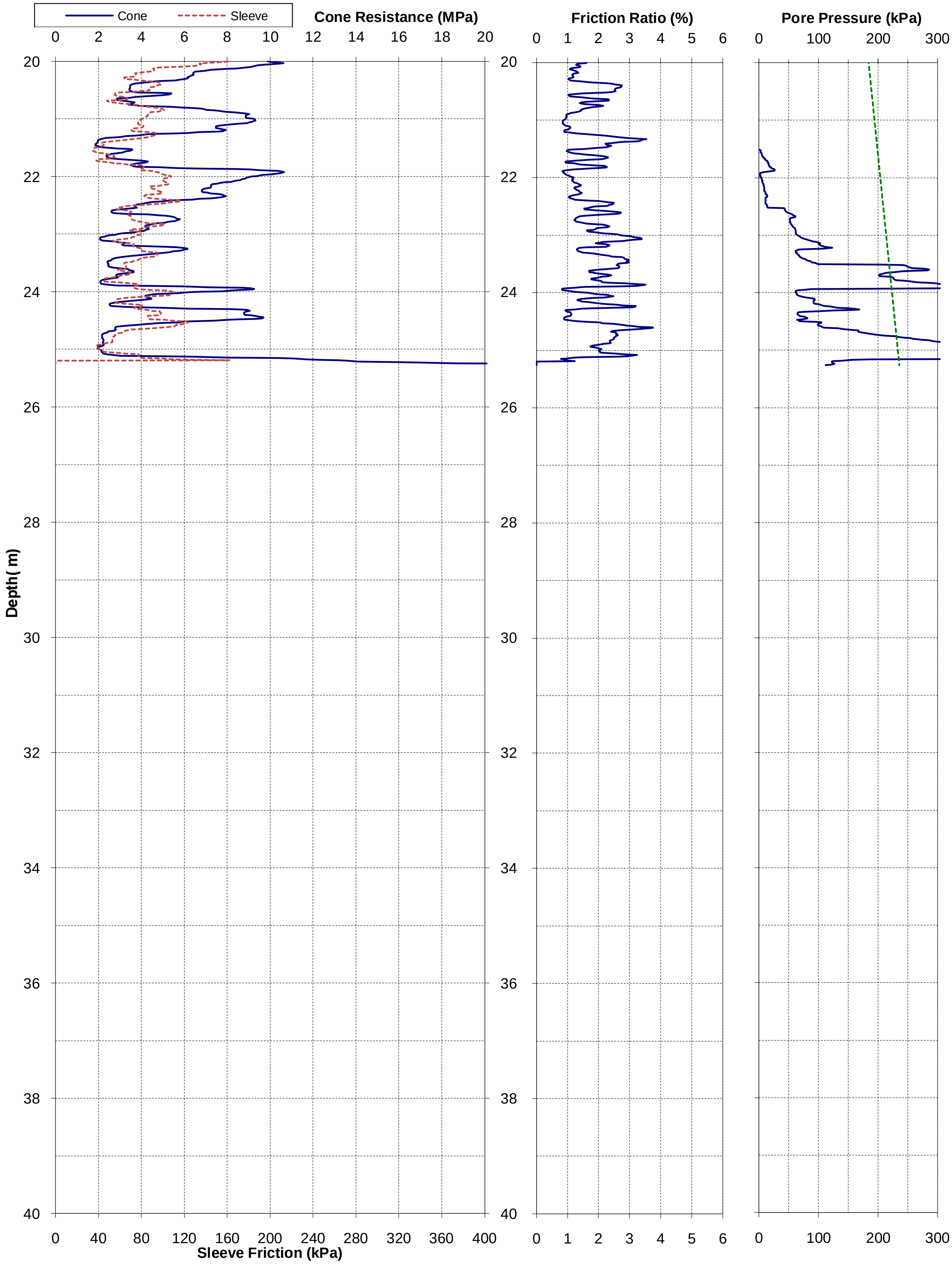
Project:	Christchurch 2011 Earthquake - EQC Ground Investigations			Page: 1 of 1		CPT-SYD-01
Test Date:	6-May-2011	Location:	Sydenham	Operator:	Perry	
Pre-Drill:	1.2m	Assumed GWL:	0.3mBGL	Located By:	Survey GPS	
Position:	2481110mE	5739438.3mN	8.225mRL	Coord. System:	NZMG & MSL	
Other Tests:				Comments:		



Project:	Christchurch 2011 Earthquake - EQC Ground Investigations			Page:	1 of 2	CPT-SYD-14
Test Date:	8-Aug-2011	Location:	Sydenham	Operator:	Opus	
Pre-Drill:	1.2m	Assumed GWL:	1.2mBGL	Located By:	Survey GPS	
Position:	2481297.9mE	5739126mN	6.774mRL	Coord. System:	NZMG & MSL	
Other Tests:				Comments:		



Project:	Christchurch 2011 Earthquake - EQC Ground Investigations			Page: 2 of 2		CPT-SYD-14
Test Date:	8-Aug-2011	Location:	Sydenham	Operator:	Opus	
Pre-Drill:	1.2m	Assumed GWL:	1.2mBGL	Located By:	Survey GPS	
Position:	2481297.9mE	5739126mN	6.774mRL	Coord. System:	NZMG & MSL	
Other Tests:				Comments:		



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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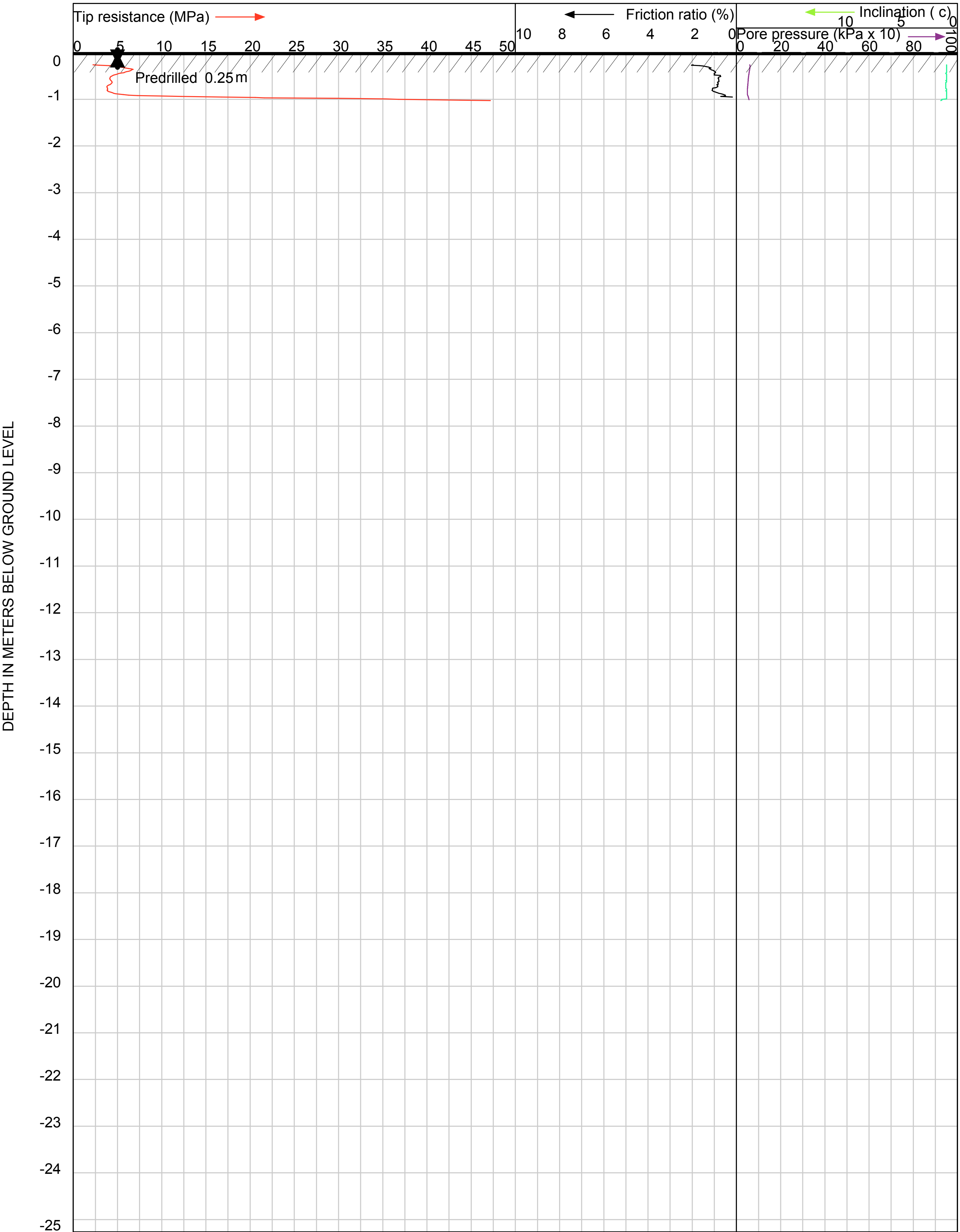
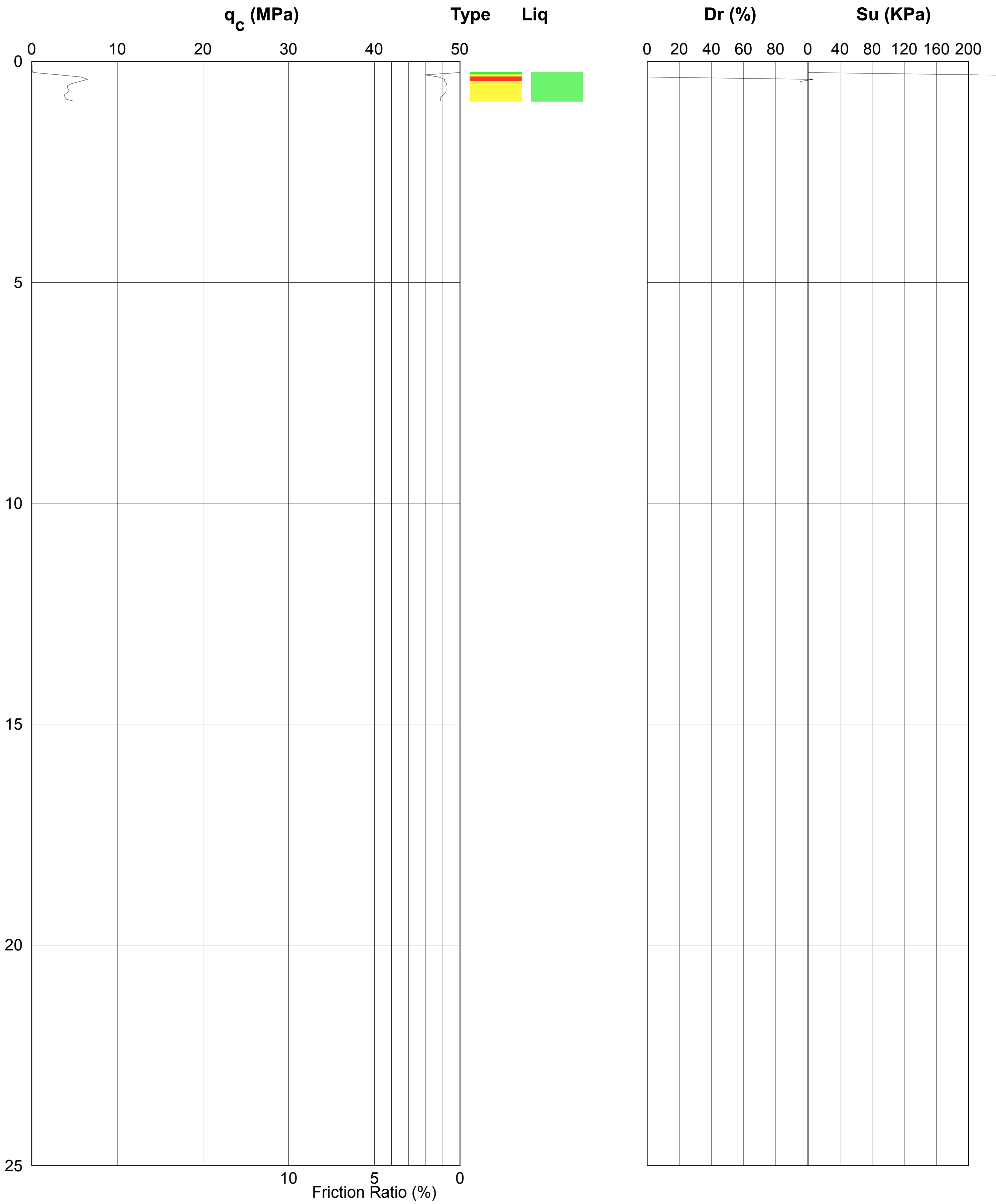


CLIENT : GHD
LOCATION : Christchurch Various (CCC Properties)
DATE : 2-4-2012
OPERATOR : H. Pardoe
REMARK 1 : CPTu20/A
REMARK 2 : Effective Refusal

JOB # : 10386
TEST # : 20

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

CPT No: CPTu20/A

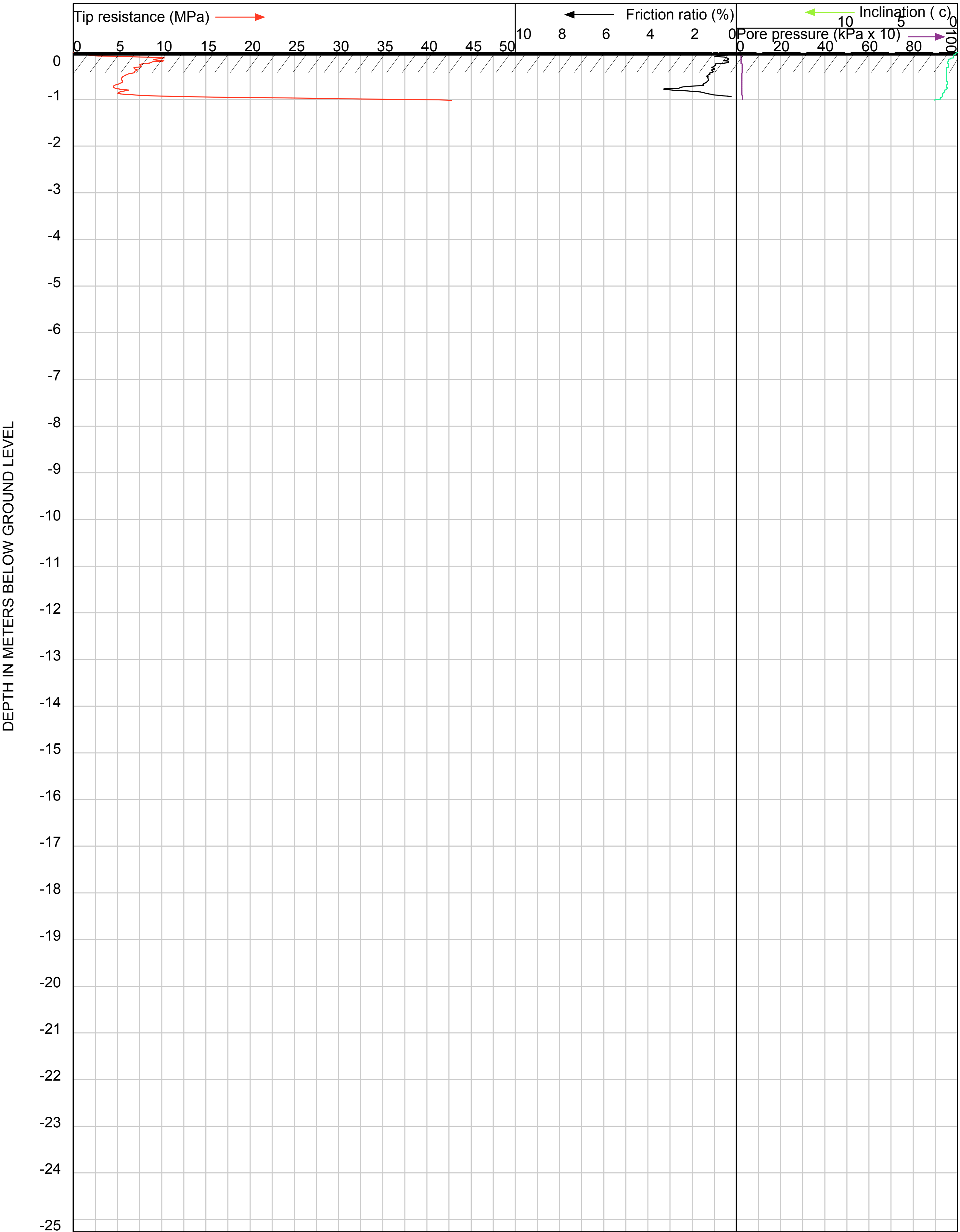
Project: GHD

Location: Christchurch Various (CCC Properties)

Date: 2-4-2012

Operator: H. Pardoe

Remark: Effective Refusal



CLIENT	: GHD
LOCATION	: Christchurch Various (CCC Properties)
DATE	: 2-4-2012
OPERATOR	: H. Pardoe
REMARK 1	: CPTu20/B
REMARK 2	: Effective Refusal

JOB # : 10386
TEST # : 120

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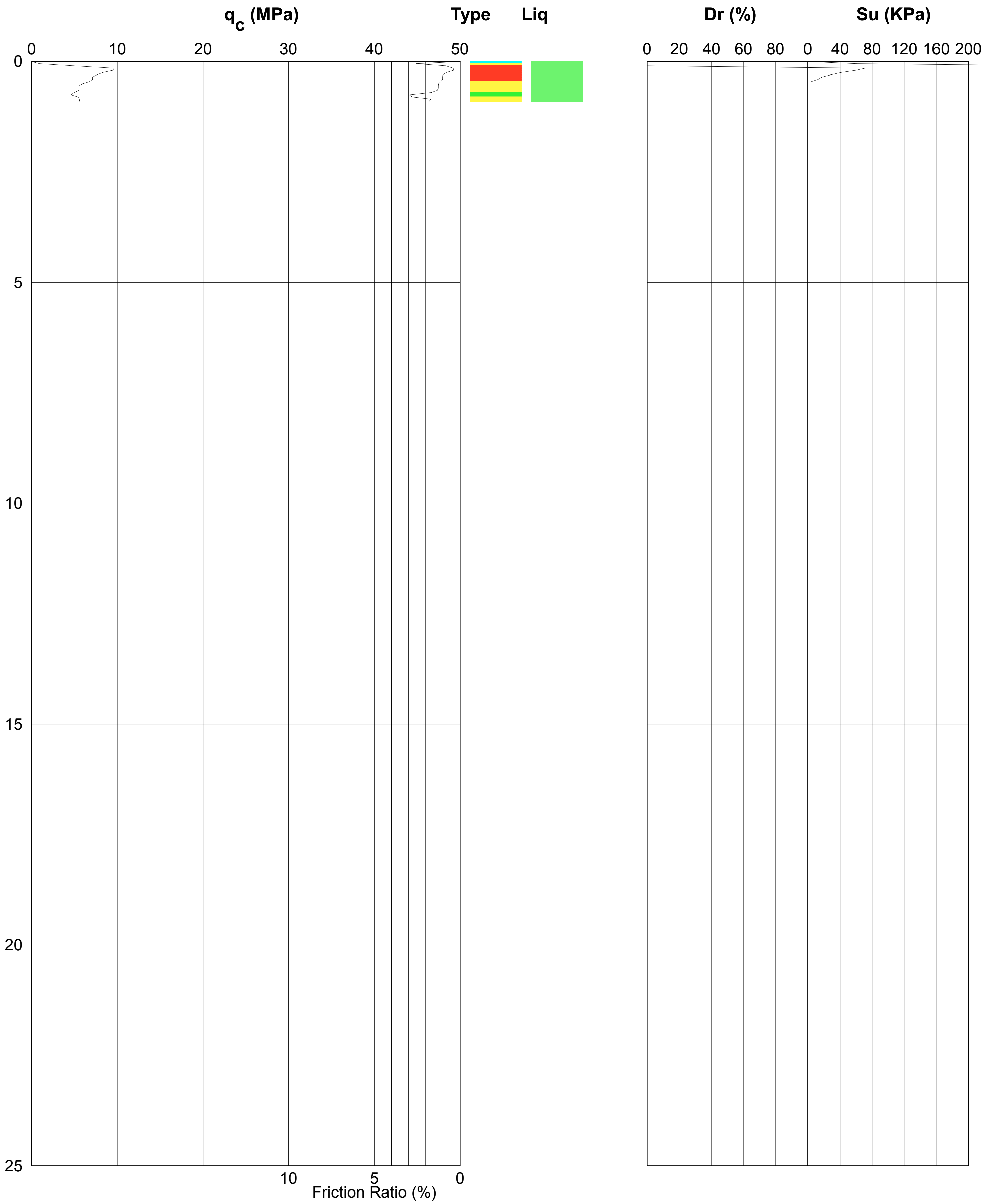
PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



Job No:	10386	Date:	2-4-2012
CPT No:	CPTu20/B	Operator:	H. Pardoe
Project:	GHD	Remark:	Effective Refusal
Location:	Christchurch Various (CCC Properties)		

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Appendix B

Photographs

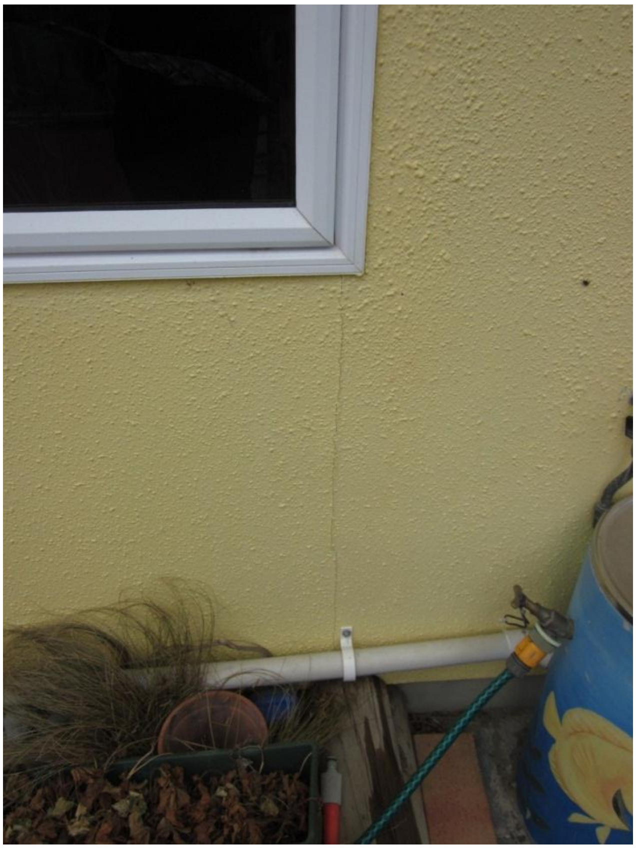
Photograph 1 South elevation showing portable office, original building and 1971 extension.

Photograph 2 North elevation showing 1996 extension.

Photograph 3 Cracking below window at construction joint between the original building and the 1971 extension on the south-west.

Photograph 4 Horizontal cracking traversing around the southernmost corner of the building, above and below window level.

Photograph 5 Cracking along the construction joint between the original building and 1971 extension on the south-east wall.



Photograph 6 Cracking below windows of the 1996 extension to the north-east of the building.

Photograph 7 Cracking of masonry wall where external walls connect to internal .

Photograph 8 Cracking to plasterboard lining around purlin, above opening to 1996 extension.

Photograph 9 Subfloor view looking South East along strip foundation to internal masonry wall.

Photograph 10 As above, showing concrete piles.

Photograph 11 Section of wall removed to expose steel reinforcement at the southernmost corner of the building.

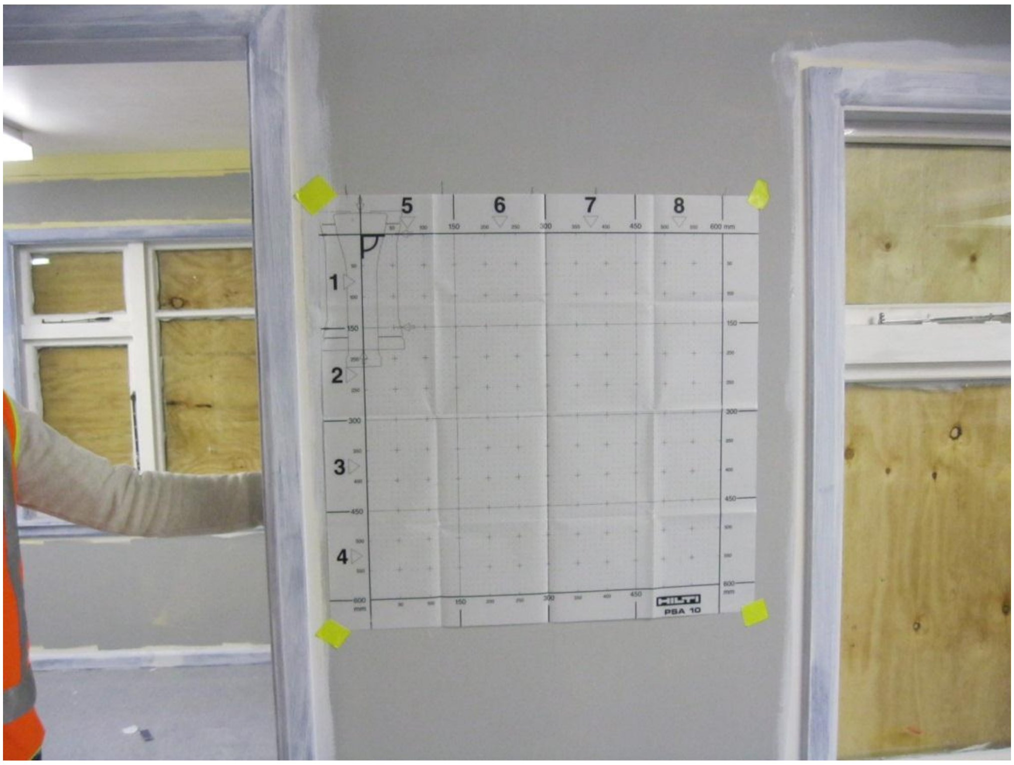
Photograph 12 Cracking penetrates through the wall along the mortar joint.

Photograph 13 Section of wall removed to expose a timber frame internal wall.

Photograph 14 Open section at southern corner.

Photograph 15 Second section opened to expose steel reinforcement closer to window as shown in the CCC plans.

Photograph 16 No evidence of expected steel reinforcement as shown in CCC plans.



Photograph 17 The position of the first electromagnetic wall scan. (See Appendix C for scan image)

Photograph 18 The position of the second electromagnetic wall scan in the office/staffroom. (See Appendix C for scan image)

Photograph 19 Opening up around the connection of the 1996 extension beam to the roof support lintel. Connection detail differs to detail on record.

Photograph 20 Cracking noted to suspended timber ground floor bearers following opening up of a section of the floor.

Photograph 21 View inside the roof space close to the intersection of the old roof and the 1996 extension showing old and new timbers and timber roof sarking.

Photograph 22 View inside the roof space from showing timber sarking and roof joists.

Appendix C

Existing Drawings

See attached diagrams and calculations. Refer also to Carpentry and Plasterboard Linings sections of this Specification. Bracing elements are shown on the elevations.

Principle: The building extension only is considered in the calculations below although some benefit will be obtained from the inherent bracing action of the attached existing building.

Spacing of bracing lines increased from the standard 5.00 metres because of the use of plywood sarking to the roof of the extension.

Wind Zone: M (48 units/m)
Earthquake Zone: B (2.0 units /sq.m.)
Roof Pitch: Less than 25 degrees.

Length of Extension:	8.40 m	Bracing Units across = 8.4 x 48 = 403.2
Width of Extension:	5.00 m	Bracing units along = 5.0 x 48 = 240.0
Extension Area:	42 sq.m.	Bracing units each way = 42 x 2 = 84.0

Individual bracing elements along length of building extension must contribute 240 bracing units in total: Assume existing building wall line contributes 25% of this amount then new wall must contribute 180 units.

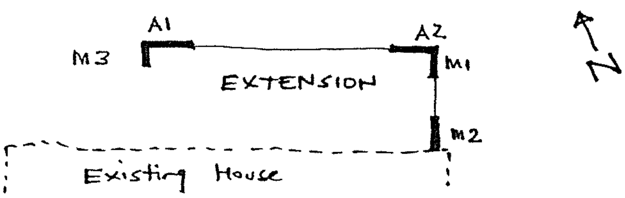
A1	Type CP6: 0.9m x 100 units/m =	90
A2	Type CP6: 0.9m x 100 units/m =	90
	Total =	180 OK.

Individual bracing elements across width of building must contribute 403.2 bracing units in total:

M1	Type HF1: 1.5m x 132 units/m =	198
M2	Type HF1: 1.5m x 132 units/m =	198
M3	Type CP4: 0.6m x 97 units/m =	58
	Total =	456 OK.

Sub-floor bracing provided by reinforced concrete ring foundation and floor.

Continued.



The following systems are used to provide required bracing action to parts of exterior walls:

CP4: As for CP6 but length of bracing unit to be between 0.6m and 1.2m.

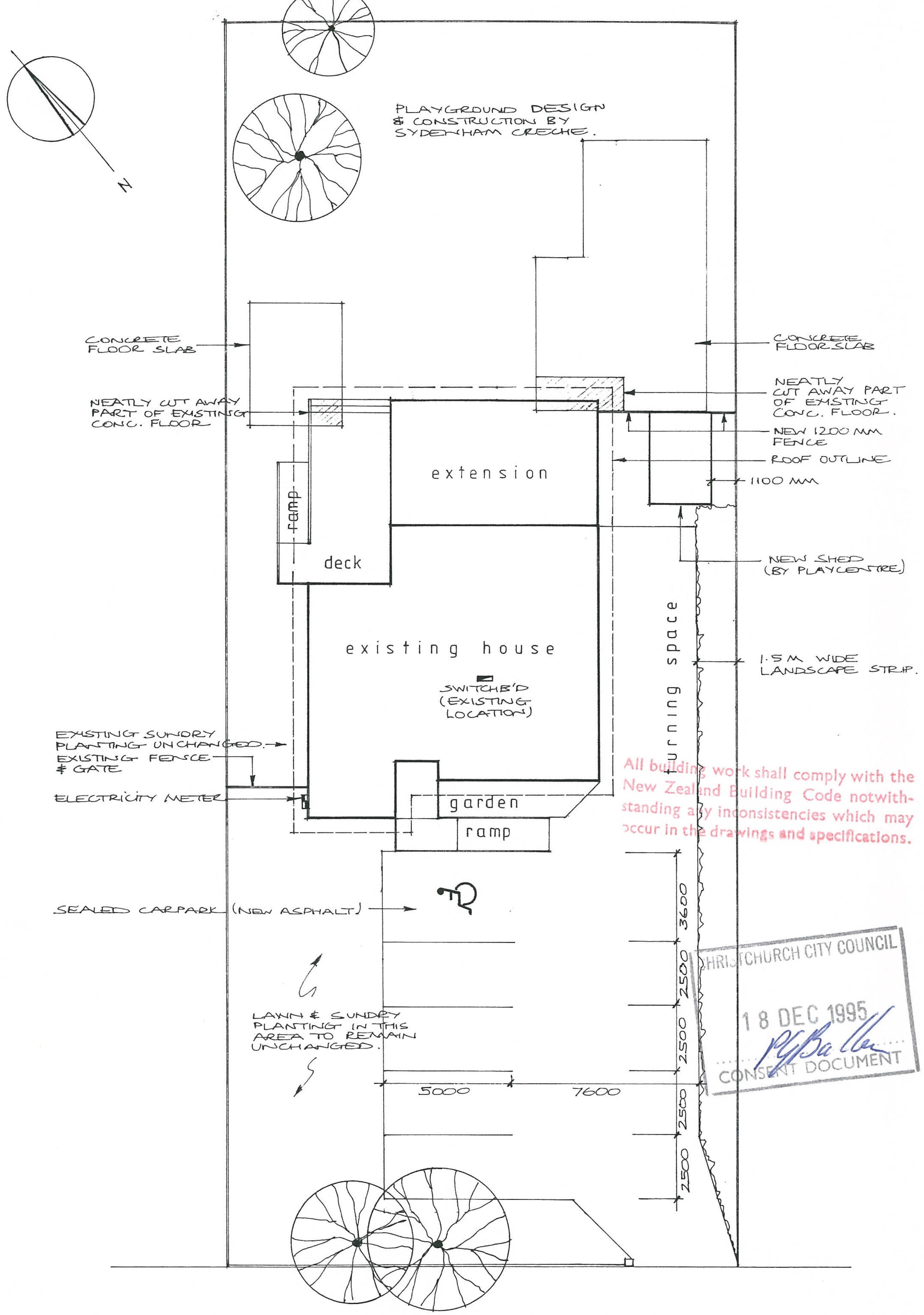
CP6: 7.5mm C-D grade construction ply checked into outside face of timber frame and standard gibraltarboard to the inside face. Fixings shall be in accordance with Clause K4.5 or Clause E13.2 (Figure E8) of NZS 3604:1990. Length of bracing unit to be between 0.9m and 1.8m.

HK1: 7.5mm Hardiflex board on outer face of framing. Fixings shall be in accordance with Clause K4.5 or Clause E13.2 (Figure E8) of NZS 3604:1990. Length of bracing unit to be between 1.2m and 2.4m.

APPENDIX 4: Certificate of Title: Attached.

APPENDIX 5: Engineer's Producer Statement: To be supplied through the Architect's office before work begins on site.

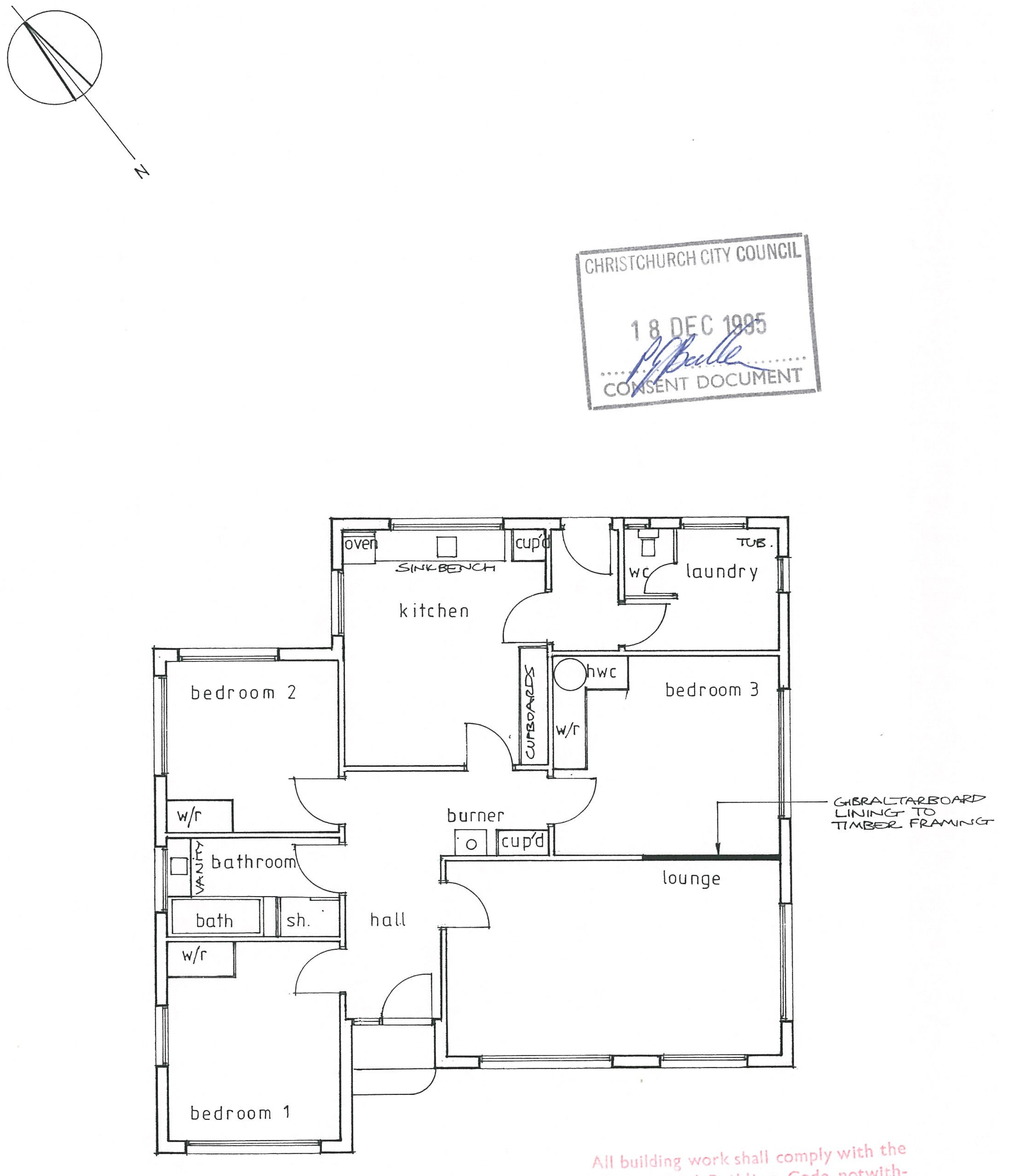
Refer to the Producer Statement for inspections required by the Engineer.



1·2

PROPOSED SITE PLAN

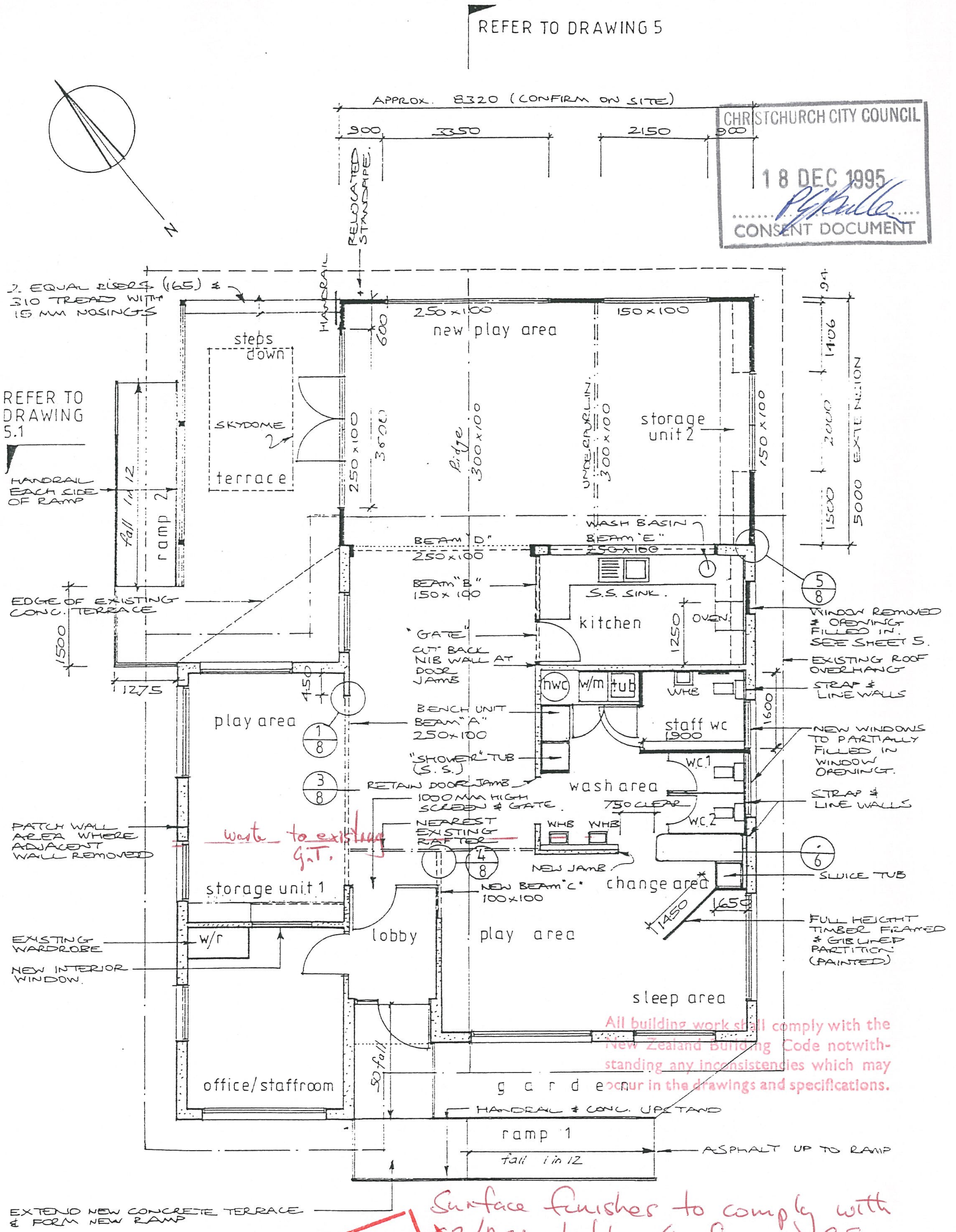
ROPOSED SYDENHAM CRECHE, 113 HUXLEY STREET 1:200 9507



2

EXISTING PLAN
1:100 9507

PROPOSED SYDENHAM CRECHE



CHRISTCHURCH CITY COUNCIL
18 DEC 1995
P G Bulle
CONSENT DOCUMENT

All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.

Surface finishes to comply with E3/AS1 table 4 for CS Purpose Group.

AMENDED
C.C.C.

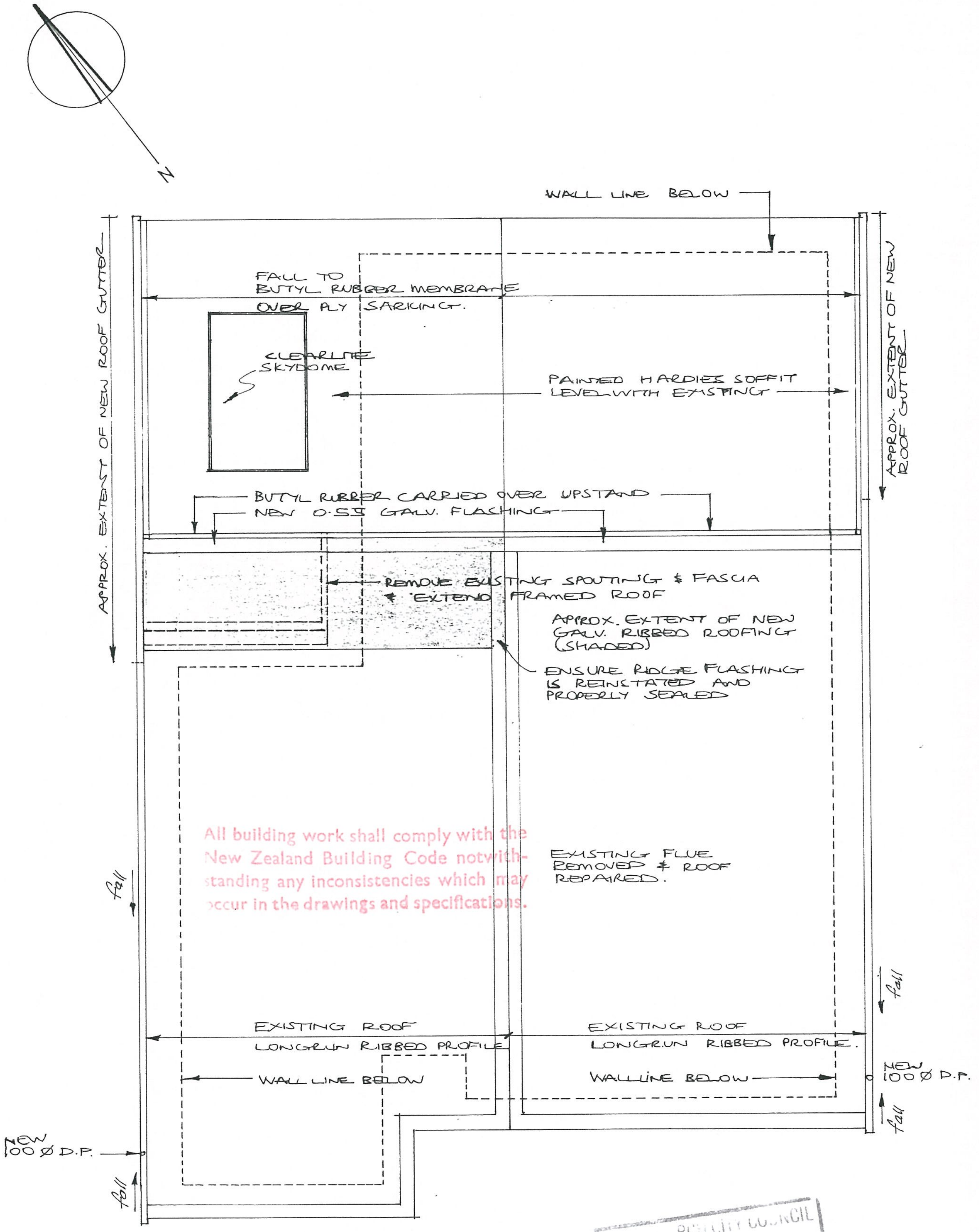
Amended

R 11.11.95 BEAM SIZES ADDED

3R

FLOOR PLAN

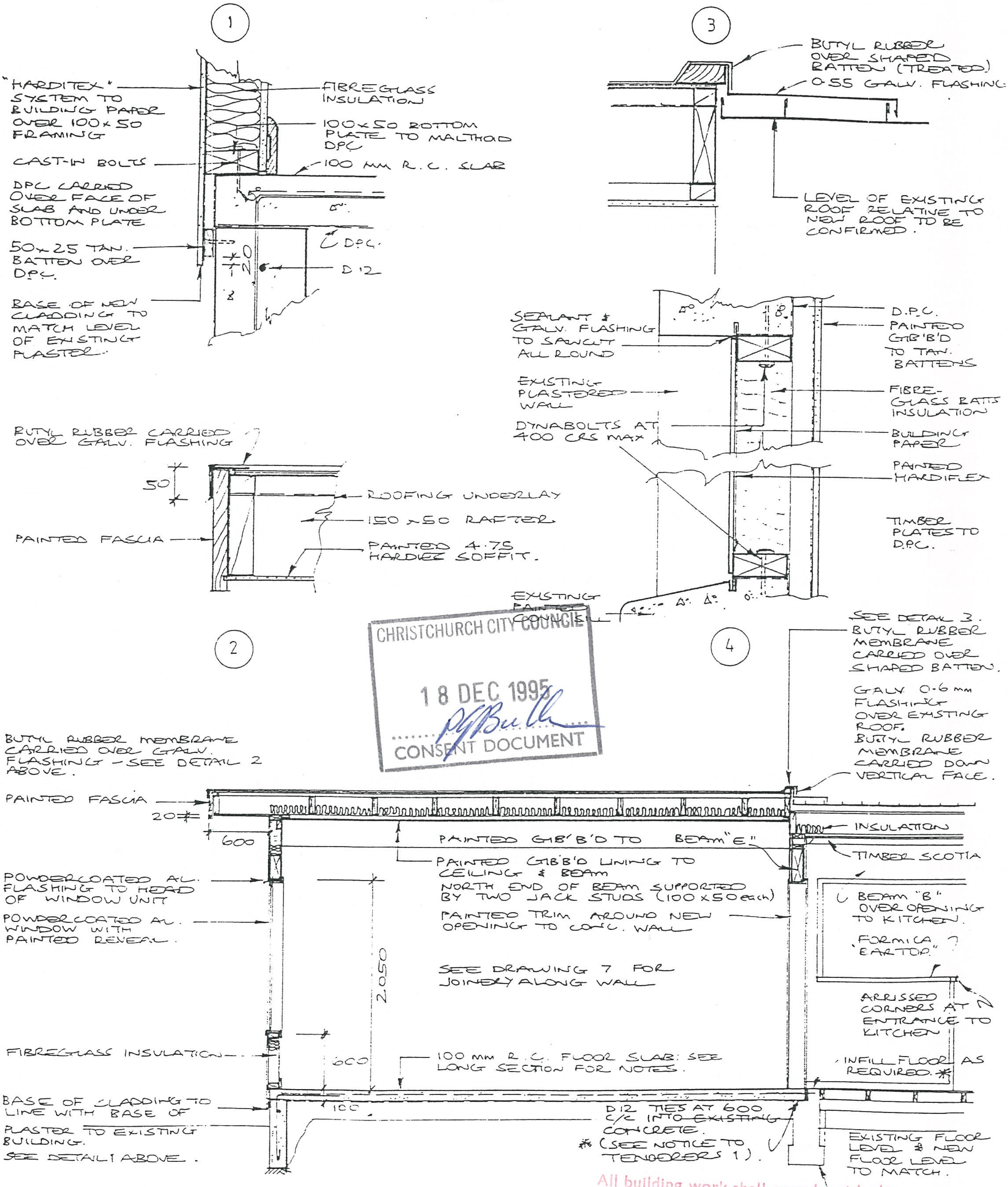
PROPOSED SYDENHAM CRECHE 1:100 9507



3.1
ROOF PLAN
1 100 9507

PROPOSED SYDENHAM CRECHE

Amended



CROSS SECTION 1:50

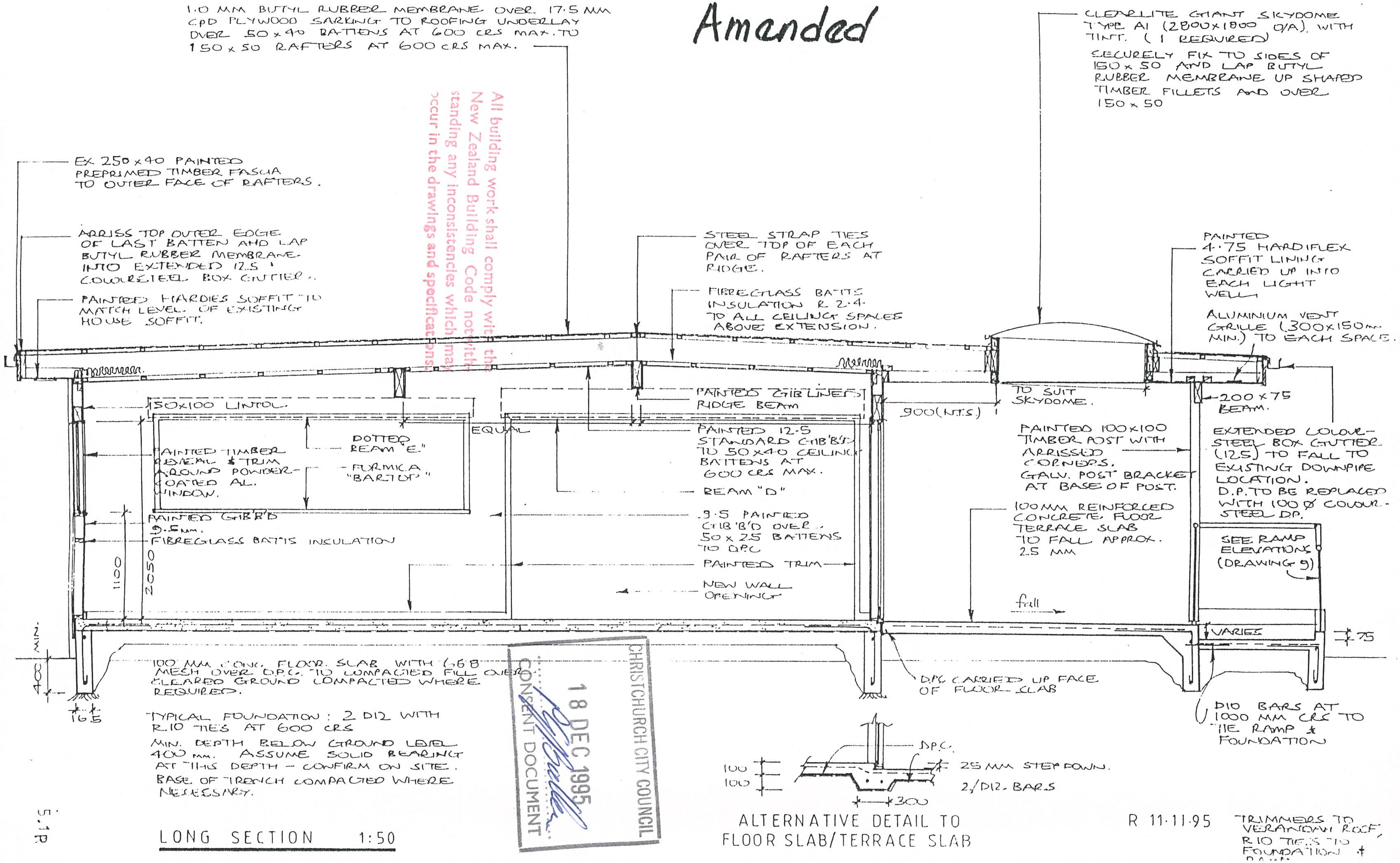
R 11.11.95 CAST IN BOLTS (DETAIL 1), DYNABOLTS (DETAIL 4)

5R

DETAILS / CROSS SECTION

1:10 1:50 9507

PROPOSED SYDENHAM CRECHE

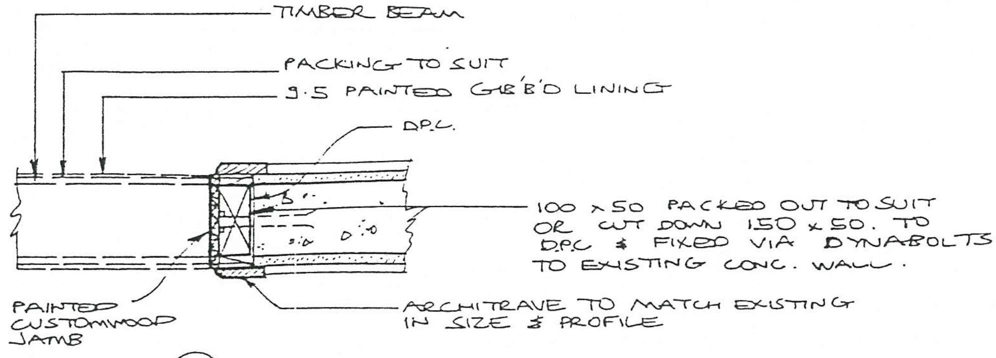


LONG SECTION 1:50

ALTERNATIVE DETAIL TO FLOOR SLAB/TERRACE SLAB

R 11.11.95 TRIMMERS TO VERANDAH ROOF, R10 TIES TO FOUNDATION & RAMP

5.1P



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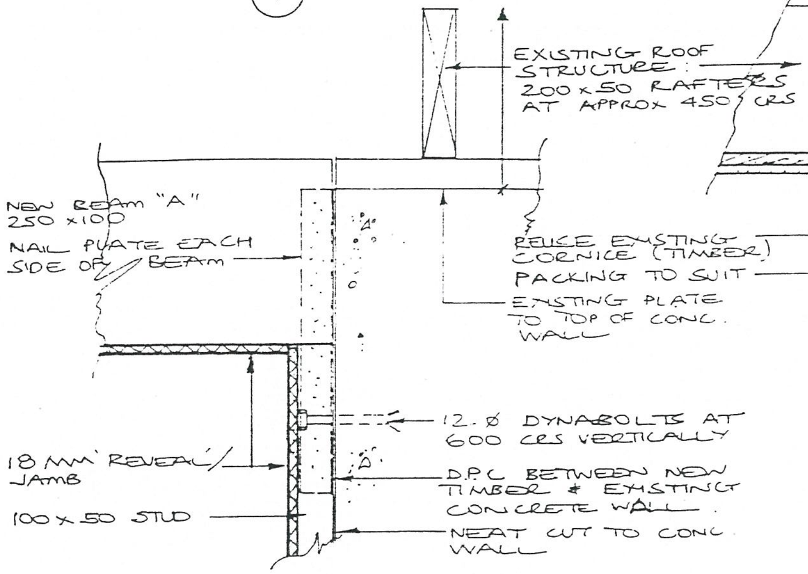
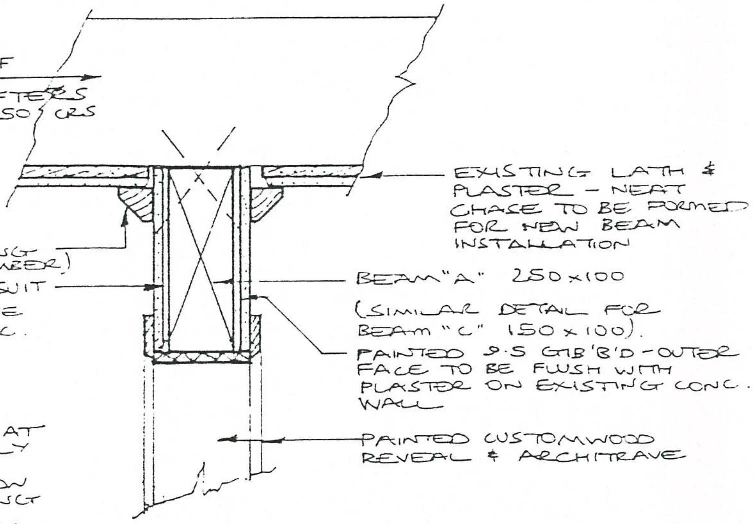
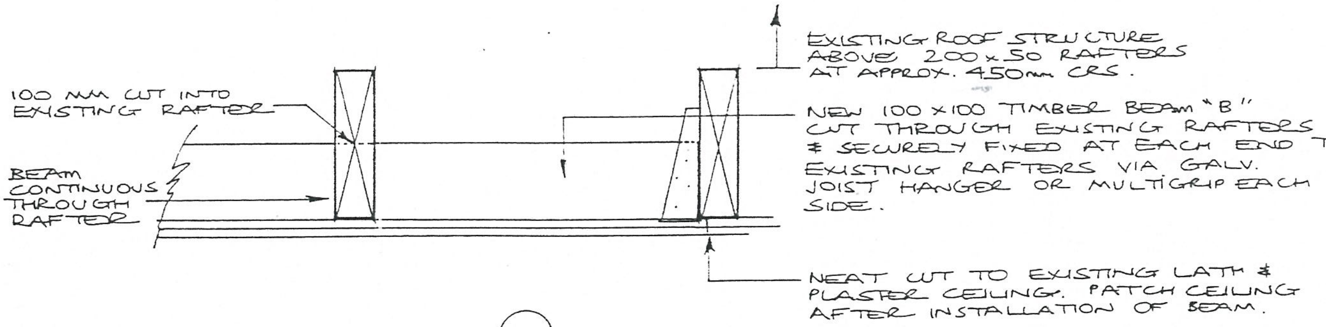




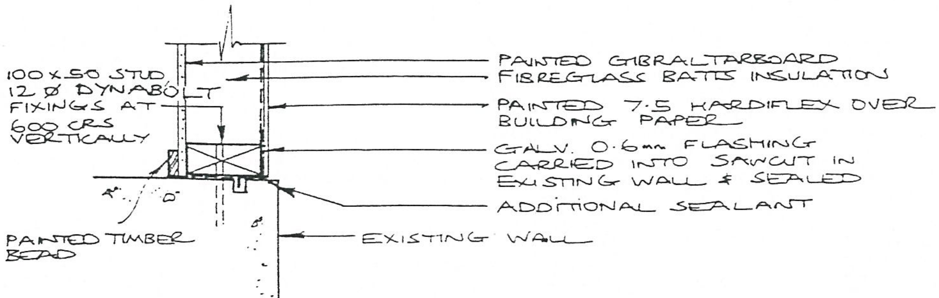
3

All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.

2



4



CHRISTCHURCH CITY COUNCIL
18 DEC 1995
CONSENT DOCUMENT

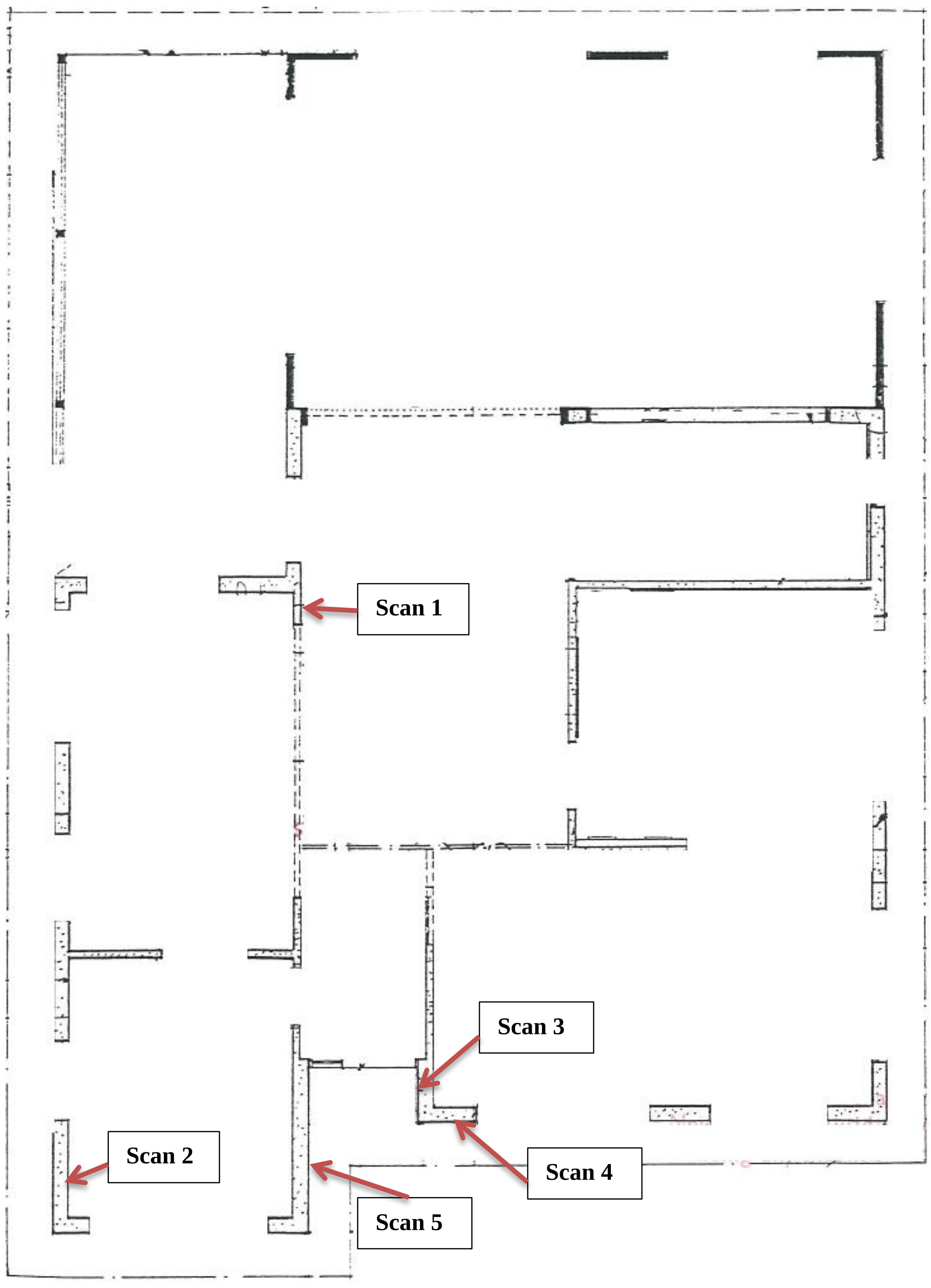
Amended

R 11.11.95 DELETE REFERENCE TO TRUBOLTS AND ADD DYNABOLTS.

8R

SYDENHAM CRECHE | DETAILS 1:10 | 9507

Positions of Wall Scans



Appendix D

Basis of Design

Basis of Design

General

The basic assumptions, design codes and references, practice advisory, material strengths and properties, and loading data used in the analysis and design are presented below.

Codes, Standards and Design manual

New Zealand Standard

- NZS 1170.0:2002 Structural Design Actions Part 0: General Principles
- NZS 1170.1:2002 Structural Design Actions Part 1: Permanent, Imposed and Other Actions
- NZS 1170.5:2004 Structural Design Actions Part 5: Earthquake Actions – New Zealand and the NZBC Clause B1 Structure
- NZS 3604:2011 Timber-framed Buildings
- New Zealand Society of Earthquake Engineering Guidelines for Assessment and Improvement of the Structural Performance of Buildings in Earthquakes
- Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance

Materials

The material strengths and properties used in the analysis of the existing structures are as follows:

Concrete Compressive Strength

- Masonry (f'm): 12.8 MPa

Assessment Load Criteria

Basic Assessment Information:

Properties of the structure that will be used in the structural assessment are:

Height of building:	2.4 m
Dimensions of the building:	12m x 16m (see floor plan – Drawing)
Site Location:	113 Huxley Street, Sydenham, Christchurch, New Zealand
Importance level:	2 (Office type)

Dead Loads

Dead load to be considered as specified in New Zealand Code (NZS 1170.1:2002)

The weights of various materials being considered in the assessment are as follows:

Floor Dead Load	
Timber floor	0.35 kN/m^2
Steel sheet, flat galvanized	
Per millimetre thickness	0.08 kN/m^2
140mm thick masonry wall	1.83 kN/m^2
190mm thick masonry wall	2.10 kN/m^2
Unit weight of timbers @ 12% moisture content	4.60 kN/m^2

Live Loads

Live loads to be considered as indicated in New Zealand Code (NZS 1170.1:2002)

Roof Live Load (maintenance and repair)	0.25 kN/m^2 or 1.80/A + 0.12

Snow Load

There is no snow load used in the analysis.

Wind Load

Wind loading is not considered in the analysis.

Seismic Load

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)	1/500 (ULS) Importance Level 2
Annual Probability of Exceedance	
(Table 3.3, NZS 1170.0:2002)	1/25 (SLS)
Return Period Factor (Ru)	
(Table 3.5, NZS 1170.5:2004)	1.0 (ULS)
Return Period Factor (Rs)	
(Table 3.5, NZS 1170.5:2004 and NZBC Clause B1 Structure)	0.33 (SLS)
Ductility Factor (μ)	1.25
Performance Factor (Sp)	0.925
Gravitational Constant (g)	9.81 m/sec^2
Liquefaction Potential	moderate

Site Description

The site is located within sydenham, a flat suburb in south-central Christchurch.

Ground Conditions

To be updated by Geotechnical Engineer.

GHD

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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
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