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Hoon Hay Children's Library
BU 1494-001 EQ2
Detailed Engineering Evaluation
Quantitative Report
Version FINAL

90 Hoon Hay Road, Hoon Hay

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

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Quantitative Report
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90 Hoon Hay Road, Hoon Hay

Christchurch City Council

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Date
7/12/12

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Hoon Hay Children's Library

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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 1170: 2002 Structural Design Actions, NZS 3604:2001 Timber-Framed buildings, NZS 3404:1997 Structural Steel and the New Zealand Society for Earthquake Engineering (NZSEE) guidelines for 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

Hoon Hay Children's Library is located at 90 Hoon Hay Road, Hoon Hay, Christchurch. The library was constructed in 1966.

The library building is of timber frame construction with various cladding materials to the exterior walls and lightweight internal timber walls. The roof has a low pitch of approximately 3 degrees. The roof consists of lightweight corrugated steel sheeting, diagonal timber sarking and timber rafters supported by two steel portal frames. The timber framed walls are lined internally with fibrous plasterboard. External cladding of the timber framed walls is provided by timber cladding to the south-west exterior, red brick cladding to the south-east and north-east. The north western wall is a combination of concrete framing, glazing and concrete infill panels. The floor consists of concrete slab on grade. Perimeter foundations consist of concrete strip footings.

Key Damage Observed

Key damage observed includes:-

Minor cracking in red brick wall cladding in the north corner of the building

Critical Structural Weaknesses

The following potential critical structural weaknesses have been identified in the structure.

- Liquefaction Potential
- Plan Irregularity

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 42% NBS and post-earthquake capacity is also in the order of 42% NBS.

The building has been assessed to have a seismic capacity in the order of 42% NBS and is therefore potentially Earthquake Risk.

Recommendations

As the building has been assessed to have a %NBS greater than 34% NBS but less than 67% NBS, it is deemed to be Earthquake Risk. 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 Risk buildings.

The building has been assessed as being potentially Earthquake Risk. As a result, it is recommended that the library can remain occupied, as per Christchurch City Council's policy regarding occupancy 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 Hoon Hay Children's Library.

This report is a Quantitative Assessment and is based in general on NZS 1170: 2002 Structural Design Actions, NZS 3604: 2011 Timber Framed Buildings, NZS 3404: 1997 Structural Steel and the New Zealand Society for Earthquake Engineering (NZSEE) guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes.

A quantitative assessment of structure involves an analysis of the seismic and gravity loads that a building is subjected to, analysis of the distribution of these forces throughout the structural members and an analysis of the capacity of the existing 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. When the manufacturers' guidelines are not available, values for material strengths are taken from Table 11.1 of the NZSEE guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes. The demand for the building is determined in accordance with NZS 3604: 2011 and the percentage of new building standard (%NBS) is assessed.

Finite element modelling of the building's steel frames was 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 steel 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

Hoon Hay Children's Library is located at 90 Hoon Hay Road, Hoon Hay, Christchurch. The site consists of a single story building and large grass garden. The site is predominantly flat. The library was constructed in 1966.

The library building is of timber frame construction with various cladding materials to the exterior walls and lightweight internal timber walls. The roof has a low pitch of approximately 3 degrees. Roof cladding consists of lightweight corrugated steel sheeting. The cladding is fixed to 25mm diagonal timber sarking which is supported by 200mm x 50mm timber rafters. The timber rafters span between the external walls to the north east and south west and two steel portal frames. The portal frame legs consist of 89mm x 89mm steel square hollow sections with a pinned connection at the base. The portal rafters consist of a 50mm x 50mm steel square hollow sections to top and bottom chords separated by 300mm. There are diagonal 18mm solid steel bars connecting the chords at approximately 600mm centres. The portal frame rafter is connected to the legs by an end plate bolted to the flange of the leg with 2 No. 12mm diameter bolts. The timber framed walls are lined internally with 12mm fibrous plasterboard. External cladding of the timber framed walls is provided by timber cladding to the south-west exterior, red brick cladding to the south-east and north-east. The red brick cladding consists of a single 100mm thick red brick leaf tied back into the timber framing by 3mm steel wire. The timber cladding consists of 12mm horizontal timber slats. The north western wall comprises of a combination of concrete framing, glazing and concrete infill panels. The floor consists of 150mm thick slab on grade reinforced with an 8mm mesh at 150mm centres. Perimeter foundations consist of concrete strip footings.

The site is bordered by Hoon Hay Road to the north-east, and residential properties on all other sides. The dimensions of the library are approximately 12m in the across direction at the south-west and 6.6m in the north-east. The dimensions along the building are approximately 8.6m. The total floor plan area of the library is approximately 53m^2. The height of the building is 2.7m to the south-east and 3.0m to the north-west. The nearest building to the library is the residential property approximately 3m to the north-west whilst the nearest waterway is a tributary of Heathcote River and is located approximately 150m to the north-east of the library.

No plans or drawings were available for this building.

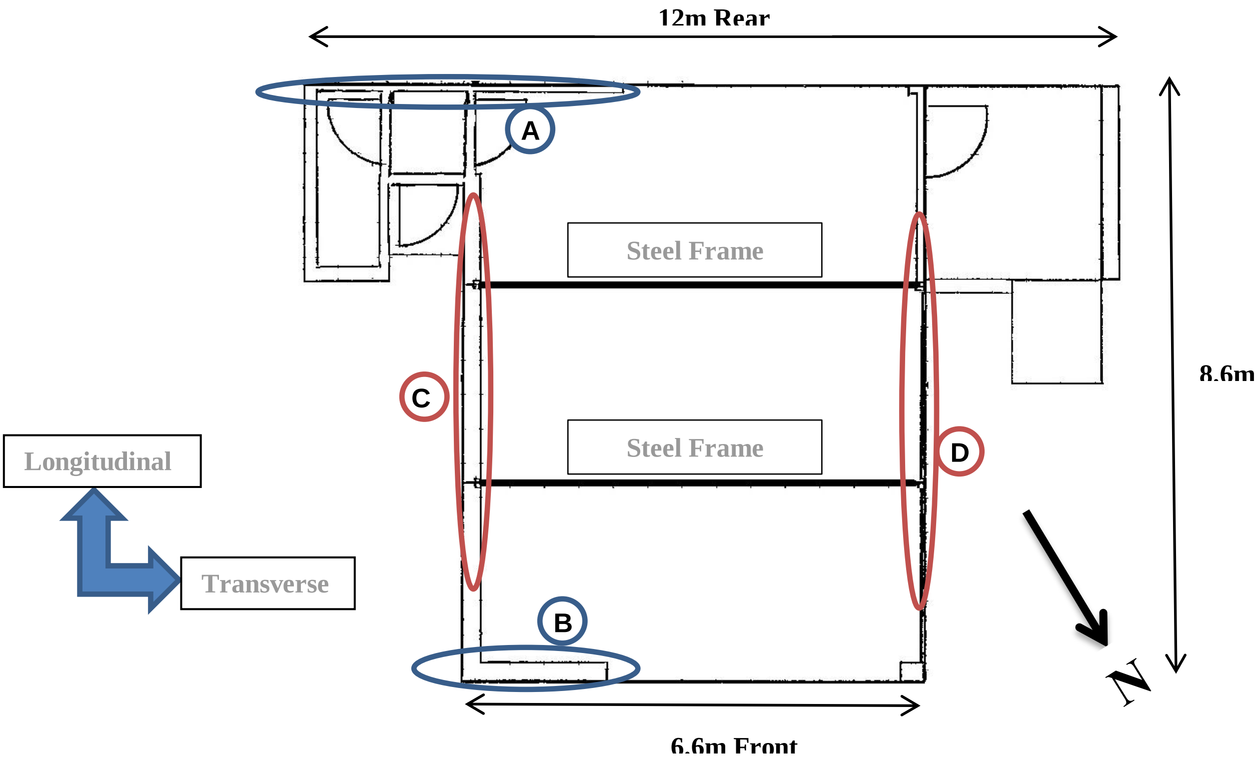


Figure 2 Plan sketch of Hoon Hay Children's Library

4.2 Gravity Load Resisting System

Steel roof trusses support the majority of gravity loads of the timber rafter and light steel corrugate roof. The truss gravity loads are then transferred at each end of the trusses to steel SHS posts, which are supported by concrete edge strip footings. Roof gravity loads at the front and rear of the building would be carried by timber window lintels in lieu of steel trusses. These lintel loads are transferred to timber wall studs that carry the load to foundation level.

The entry porch and side door/toilet areas are more likely to be of NZS 3604-style timber framing roof rafters, purlins, and supporting load bearing timber lintels and walls. Gravity load from loadbearing walls is most likely supported by edge strip foundations, similar to the main part of this building.

4.3 Lateral Load Resisting System

In the ***transverse*** direction, seismic loads are carried by a combination of the plasterboard walls A & B and the steel frames shown in the plan sketch Figure 2. These walls are directly connected to the roof and floor and transfer roof seismic loads to the foundations via their timber framed bracing as plasterboard shear panels.

Longitudinally, seismic loads are primarily braced by the SHS posts, held in place as cantilever frame members by the infill walls below the window sills, see diagram Figure 3 for these areas C & D. The bracing demand from the roof is transferred by these cantilevering posts to the walls below, which in turn brace the transferred load via plasterboard shear panel or masonry shear wall action. Secondary lateral load resistance is provided by the timber frames walls in the area marked C in Figure 3. The SHS posts protruding above the sill have a much longer cantilever length to accommodate the deeper windows on the north-west entry (D) side of the building when compared to the relatively short distance that the steel posts cantilever on the opposite side (C) of the building for the shallow high level windows. Building torsion is likely as these two SHS-braced walls will have different stiffnesses, inherent in the different cantilever lengths. This likely building torsion is function of plan irregularity critical structural weakness.

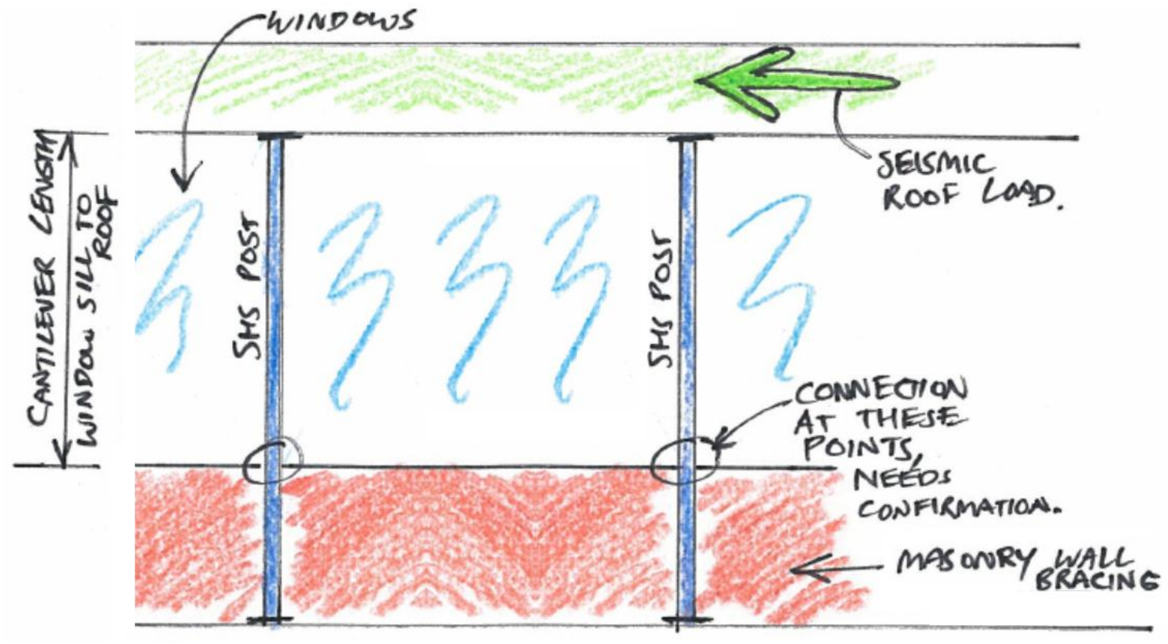


Figure 3 Sketch of cantilever SHS post system

5. Assessment

5.1 Qualitative Assessment

An inspection of the property was undertaken on the 27th of January 2012. Both the interior and exterior of the building was inspected. The main structural elements identified were the timber framed, plasterboard lined walls, the low pitch roof and the structural steel frames. The steel frame structural components of the roof were all able to be viewed due to the exposed nature of the structure. No inspection of the foundations of the structure was able to be undertaken.

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.

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 22% NBS and post-earthquake capacity in the order of 22% NBS. The buildings post-earthquake capacity excluding critical structural weakness is in the order of 45% NBS.

5.2 Quantitative Assessment

5.2.1 Building Demand

Steel Framed Portion of the Structure

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

For ductile and limited ductility structures with seismic-resisting systems located along two perpendicular directions, the specified actions may be assumed to act separately along each of these two horizontal directions as set out in Cl 5.3.1.1 of NZS 1170.5: 2004.

Individual member bending moment, shear force and axial force demands were extracted from a finite element analysis of the frame.

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) of NZS 1170.5;

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

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

$$S_P = 1.3 - 0.3\mu$$

Where μ, the structural ductility factor, was taken as 2.00.

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 Member Bending Moment Capacity (Section 5.1 of NZS 3404:1997)

A member bent about the section major principle axis shall satisfy:

$$M_x^* \leq \Phi M_{sx} \text{ and}$$

$$M_x^* \leq \Phi M_{bx}$$

Where

$M_x^* = the\ design\ bending\ moment\ from\ analysis$

$\Phi = the\ strength\ reduction\ factor\ from\ Table\ 3.3\ of\ NZS\ 3604{:}\ Part\ 1\ 1997$

$M_{sx} = the\ nominal\ section\ capacity\ in\ bending, as\ specified\ in\ Clause\ 5.2\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

$M_{bx} = the\ nominal\ member\ capacity\ in\ bending, as\ specified\ in\ Clause\ 5.3\ or\ 5.6\ of$

$NZS\ 3404{:}\ Part\ 1\ 1997$

For hollow sections, the nominal member capacity in bending, M_{bx}, always equals the nominal section capacity in bending, M_{sx}, according to clause 5.6.1.4 of NZS 3404: Part 2 1997.

5.2.4 Member Shear Capacity (Section 5.9 of NZS 3404:1997)

A member web subjected to shear force, V^*, shall satisfy:

$$V^* \leq \Phi V_v$$

Where

$V^* = the\ design\ shear\ force\ from\ analysis$

$\Phi = the\ strength\ reduction\ factor\ from\ Table\ 3.3\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

$V_v = the\ nominal\ section\ capacity\ of\ the\ web, as\ determined\ in\ Clause\ 5.11.2\ of$

$NZS\ 3404{:}\ Part\ 1\ 1997$

5.2.5 Member Shear and Bending Moment Interaction (Section 5.12 of NZS 3404:1997)

A member subjected to bending moment, M_x^*, and shear force, V^*, shall have its nominal web shear capacity, V_{vm}, calculated using the equations set out in Clause 5.12.2 of NZS 3404: Part 1 1997. The web design shear capacity in the presence of bending moment shall satisfy

$$V^* \leq \Phi V_{vm}$$

Where

$V^* = the\ design\ shear\ force\ from\ analysis$

$\Phi = the\ strength\ reduction\ factor\ from\ Table\ 3.3\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

$V_{vm} = the\ nominal\ section\ capacity\ of\ the\ web, modefied\ for\ the\ presence\ of\ bending\ as$

$determined\ in\ Clause\ 5.12.2\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

5.2.6 Member Axial Capacity (Section 6 of NZS 3404:1997)

A concentrically loaded member subject to a design axial compressive force, N^*, shall comply with both:

$$N^* \leq \Phi N_s$$

$$N^* \leq \Phi N_c$$

Where

$N^* = the\ design\ axial\ force\ from\ analysis$

$\Phi = the\ strength\ reduction\ factor\ from\ Table\ 3.3\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

$N_s = the\ nominal\ section\ capacity\ , as\ determined\ in\ Clause\ 6.2.1.1\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

$N_c = the\ nominal\ member\ capacity\ , as\ determined\ in\ Clause\ 6.3\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

5.2.7 Member Combined Axial and Bending Moment Capacity (Section 8 of NZS 3404:1997)

A member subject to uniaxial bending and axial actions need not be checked for combined actions when the axial force is not significant as defined by Cl 8.1.4 of NZS 3404:1997. The design axial force shall be considered significant unless it complies with:

$$N^* \leq 0.05\Phi N_s \; if\ the\ member\ is\ subject\ to\ uniaxial\ bending\ and\ is\ an\ I\ or\ channel\ section$$

$$N^* \leq 0.05\Phi N_c \; if\ the\ member\ is\ subject\ to\ uniaxial\ bending\ and\ is\ any\ other\ cross\ section$$

Where axial force is considered significant, the following general design provision should be satisfied:

$$M_x^* \leq \Phi M_{rx}$$

Where

$M_x^* = the\ design\ bending\ moment\ from\ analysis$

$\Phi = the\ strength\ reduction\ factor\ from\ Table\ 3.3\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

$M_{rx} = the\ nominal\ section\ moment\ capacity, reduced\ by\ axial\ force\ as\ specified\ in\ Clause$

$8.3.2.1\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

5.2.8 Connection Bolt Shear Capacity (Section 9 of NZS 3404:1997)

A bolt subject to a design shear force shall satisfy (Cl 9.3.2.1 of NZS 3404:1997);

$$V_f^* \leq \Phi V_f$$

Where

$V_f^* = the\ design\ shear\ action\ from\ analysis$

$\Phi = the\ strength\ reduction\ factor\ from\ Table\ 3.3\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

$V_f = the\ nominal\ bolt\ shear\ capacity, as\ specified\ in\ Clause$

$9.3.2.1\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

5.2.9 Connection Bolt Tension Capacity (Section 9 of NZS 3404:1997)

A bolt subject to a design tensile force shall satisfy (Cl 9.3.2.2 of NZS 3404:1997);

$$N_{tf}^* \leq \Phi N_{tf}$$

Where

$N_{tf}^* = the\ design\ tensile\ action\ from\ analysis$

$\Phi = the\ strength\ reduction\ factor\ from\ Table\ 3.3\ of\ NZS\ 3604{:}\ Part\ 1\ 1997$

$N_{tf} = the\ nominal\ bolt\ tensile\ capacity, as\ specified\ in\ Clause$

$9.3.2.2\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

5.2.10 Connection Bolt Combines Shear andTension Capacity (Section 9 of NZS 3404:1997)

A bolt required to resist both design shear and design tension forces at the same time shall satisfy (Cl 9.3.2.3 of NZS 3404:1997);

$$\left(\frac{V_f^*}{\Phi V_f}\right)^2 + \left(\frac{N_{tf}^*}{\Phi N_{tf}}\right)^2 \leq 1.0$$

Where

$N_{tf}^* = the\ design\ tensile\ action\ from\ analysis$

$\Phi = the\ strength\ reduction\ factor\ from\ Table\ 3.3\ of\ NZS\ 3604{:}\ Part\ 1\ 1997$

$N_{tf} = the\ nominal\ bolt\ tensile\ capacity, as\ specified\ in\ Clause\ 9.3.2.2\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

$V_f^* = the\ design\ shear\ action\ from\ analysis$

$V_f = the\ nominal\ bolt\ shear\ capacity, as\ specified\ in\ Clause\ 9.3.2.1\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

5.2.11 Timber Framed Wall Bracing Capacity

The building was constructed in 1966 and as such, no bracing capacities for the wall linings were available for the calculations. Therefore the capacities are taken in accordance with Table 11.1 of the NZSEE guidelines.

Section 11.4 of the NZSEE guidelines states that shear panels can utilise their full bracing capacity for aspect ratios (height-to-width) up to 2:1. For aspect ratios greater than 2:1 and up to 3.5:1 a limiting factor can be applied in accordance with the NEHRP Recommended Provisions (BSSC, 2000) as follows;

$$\text{Aspect ratio factor} = \frac{\text{2x Width}}{\text{Height}}$$

Any sections of wall with an aspect ratio greater than 3.5:1 were not included for the purpose of the bracing calculations.

6. Damage Assessment

6.1 Surrounding Buildings

Hoon Hay Childrens Library is located in a residential area. The library is not connected to any of the buildings situated on the adjacent properties. There was no apparent damage noted to the surrounding buildings or adjoining properties

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 to the external red brick cladding in the corner to the north of the building. These cracks are not considered significant.

6.3 Ground Damage

Upon inspection of the library it was noted that there was minor liquefaction on the property boundary to the west.

7. Geotechnical Consideration

The subject site is located in the south-western area of Christchurch within the suburb of Hoon Hay. The site is surrounded by residential properties, and bordered to the north-east by Hoon Hay Road. The site is located on predominantly flat land and is approximately 10m above mean sea level. It is also situated approximately 200 m southwest of the Heathcote River.

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;

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

7.1.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that one borehole with a lithographic log is located within a 200m radius of the site. The log indicates silt/sandy silt to ~1.1m bgl, with groundwater present at 1.1m bgl

It should be noted that 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 (EQC) has undertaken geotechnical testing in the area of the site. Information pertaining to this investigation is included in Tonkin and Taylor Report[2]. Within 200 m of the property two investigation points were undertaken, the results of which are detailed below in Table 2.

Bore Name	Grid Reference	Log Summary
CPT – HNH -16	2477872 - 5738502	0 – 8.5 m Silts and sands (WT at 0.7m bgl)
BH – HNH - 02	2477860 - 5738487	0 – 1.2 m Fill 1.2 – 5.2 m Sand 5.2 – 7.8 m Silt 7.8 – 10.0 m Gravelly Sand 10.0 – 11.3 m Sand 11.3 – 14.85 m Gravel

Table 2 EQC Geotechnical Investigation Summary Table

[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. IGNS Limited: Lower Hutt.

[2] Tonkin & Taylor Ltd. (September 2011): Christchurch Earthquake Recovery, *Geotechnical Factual Report, Hoon Hay*.

Initial observations of the CPT results indicate the soils are fine to coarse grained, and are soft/loose.

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[3] 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 3 (TC3), which indicates potential for moderate to significant land damage from liquefaction in future significant earthquakes.

7.1.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake (Figure 4) shows signs of signifanct liquefaction on Hoon Hay Road immediately north east of the property.

Figure 4 Post February 2011 Earthquake Aerial Photography[4]

7.1.6 Summary of Ground Conditions

From the ECan borehole information and that obtained from EQC indicates that the site is underlain by silt, sand and gravel layers with groundwater at approximately 1m below ground level.

[3] CERA Technical Category Maps, http://cera.govt.nz/maps/technical-categories

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

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 (km)	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	127 NW	8.3	~300 years
Greendale (2010) Fault	18 W	7.1	~15,000 years
Hope Fault	110 N	7.2~7.5	120~200 years
Kelly Fault	110 NW	7.2	~150 years
Porters Pass Fault	60 NW	7.0	~1100 years

Table 3 Summary of Known Active Faults[5,6]

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. Average recurrence intervals are yet to be estimated.

7.2.2 Ground Shaking Hazard

This 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 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 CPT investigation was conducted at the site on 16 April 2012.

The locations of the tests are tabulated in Table 4.

Investigation	Depth (m bgl)	Easting (NZMG)	Northing (NZMG)
CPT 001	20	2477888	5738608

Table 4 Coordinates of Investigation Locations

[5] 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.

[6] GNS Active Faults Database

The CPT investigation was undertaken by PRO-DRILL Specialist Drilling Engineers on 16 April 2012 to a target depth of 20m below ground level. Please refer to the attached CPT results for detail (Appendix A)

Interpretation of output graphs[7] from the investigation showing Cone Tip Resistance (q_c), Friction Ratio (Fr), Inferred Lithology and Inferred Liquefaction Potential are presented in Table 5 and Table 6.

Please refer to Appendix A for further detail.

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 (%)
0 – 5.2	Silty SAND to SILT	1– 3	0.5 – 1
5.2 – 7.2	Silty CLAY to CLAY	<1	2– 4
7.2 – 8.4	Silty SAND	~4	0.6 – 1
8.4 – 11.7	SAND to Gravelly SAND	15– 45	<1

Table 5 Summary of CPT-Inferred Lithology

7.5 Liquefaction Assessment

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
- Hazard factor for Christchurch Z = 0.30
- Importance Level 2, post seismic event (50-year design life)- R = 1.0
- Spectral shape factor C = 1.12 (for class D, E)
- PGA 0.35g (DBH guidelines)

The following equation has been used to approximate soil unit weight from the CPT investigation data: [8]

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

This unit weight values ranging between 16 and 17 kN/m^3 (saturated) to 6.5m bgl, and ~18 kN/m^3 below this.

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

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

The liquefaction analysis process has been conducted using the methodology from Stark & Olson[9], and from the NZGS Guidelines[10].

7.5.1 Results of Liquefaction Analysis

The results of the liquefaction analysis, as outlined in **Error! Reference source not found.**, indicate hat depths of 10m to 20m are considered moderately liquefiable.

Depth (m)	Lithology	Triggering Factor F_L	Liquefaction Susceptibility [11]
0 – 5.2	Silty SAND to SILT	0.19 – 0.43	Severe
5.2 – 7.2	Silty CLAY to CLAY	-	*Not liquefiable*
7.2 – 8.4	Silty SAND	0.27 – 0.61	Severe
8.4 – 11.7	SAND to Gravelly SAND	-	*Not liquefiable*

Table 6 Summary of Liquefactions Susceptibility

Please refer to Appendix A for further detail.

7.6 Interpretation of Ground Conditions

7.6.1 Liquefaction Assessment

Overall, the site is considered to be highly susceptible to liquefaction based on;

- Previous liquefaction and settlement at the site following the February (Mw 6.3, 2.0g) and June (Mw 6.0-6.3, 1.5g) events. The inspection undertaken on the 27th January 2012 of the library noted that there was minor liquefaction remaining on the property at the boundary to the north-west;
- CERA TC3 classification indicates the site is at risk from moderate to significant land damage from future significant earthquakes; and,
- The ground conditions encountered saturated sand layers considered to be severely liquefiable.

7.6.2 Slope Failure and/or Rockfall Potential

The site is located within Hoon Hay, a flat suburb 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.

[9] Olson, S.M. & Stark, T.D. (2002). *Liquefied strength ratio from liquefaction flow failure case histories.* Canadian Geotechnical Journal, 39 (3), 629–647pp.

[10] 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

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

7.6.3 Foundation Recommendations

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

- A site subsoil class of **D**, Deep or Soft Soil, should be adopted for the site (in accordance with NZS 1170.5:2004);
- A allowable bearing capacity of 100kPa can be used for standard shallow foundation solutions using timber and concrete floors, in accordance with New Zealand Building regulations and NZS 3604; and,
- If re-build is deemed necessary a deep investigation specific to the new building footprint should be undertaken with bearing capacity investigation.

8. Survey

No level or verticality surveys have been undertaken for this building at this stage as indicated by Christchurch City Council guidelines.

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: 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.

9.2 Bending Moment Capacity

The bending moment capacities of members highlighted in Figure 5 were calculated in accordance with NZS 3404:1997 Section 5.1. The capacities for members are given in Table 7.

Section Size	Capacity kNm	Demand kNm
89mm x 89mm SHS (Post)	15.5 (ΦM_{sx})	12.36 (Member 1)
50mm x 50mm SHS (Truss)	2.54 (ΦM_{sx})	1.29 (Member 4)
Critical Section: 89mm x 89mm SHS		%NBS = 100

Table 7 Member Bending Capacities and Demands

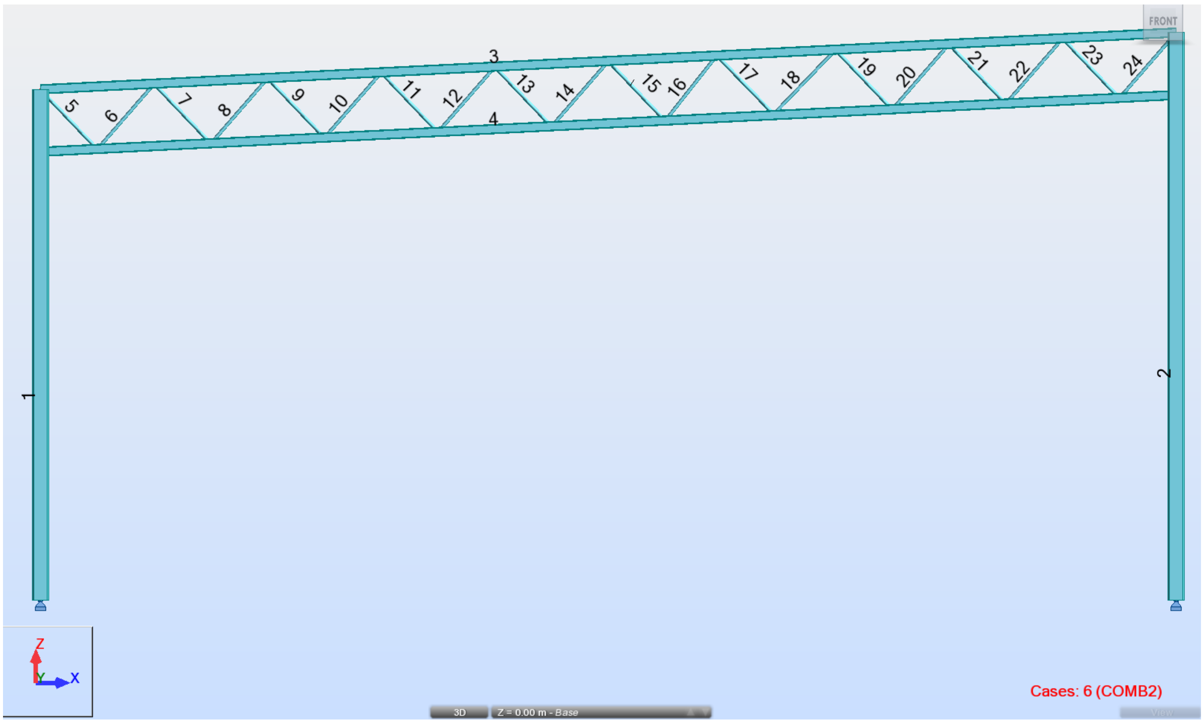


Figure 5 Portal Frame Members

9.3 Shear Force Capacity

The shear force capacities of members highlighted in Figure 5 were calculated in accordance with NZS 3404:1997 Section 5.9. The capacities for members are given in Table 8.

Section Size	Capacity kN	Demand kN
89mm x 89mm SHS (Post)	137.9 (ΦV_v)	1.84 (Member 2)
50mm x 50mm SHS (Truss)	42.5 (ΦV_v)	3.27 (Member 4)
	Critical Section: 50mm x 50mm SHS	%NBS = 100

Table 8 Member Shear Force Capacities and Demands

9.4 Axial Force Capacity

The axial force capacities of members highlighted in Figure 5 were calculated in accordance with NZS 3404:1997 Section 6. The capacities for members are given in Table 9.

Section Size	Capacity kN	Demand kN
89mm x 89mm SHS (Post)	321.13 (ΦN_c)	6.51 (Member 1)
50mm x 50mm SHS (Truss)	16.23 (ΦN_c)	34.64 (Member 4)
	Critical Section: 50mm x 50mm SHS	%NBS = 47

Table 9 Member Axial Force Capacities and Demands

9.5 Connection Bolt Shear Force Capacity

The shear force capacities of the connection bolts were calculated in accordance with NZS 3404:1997 Section 9. The capacities for the bolts are given in Table 10.

Section Size	Capacity kN	Demand kN
12mm Diameter Category 4.6/S	44.64 (ΦV_f)	4.3
		%NBS = 100

Table 10 Connection Bolt Shear Force Capacities and Demands

9.6 Connection Bolt Tensile Force Capacity

The tensile force capacities of the connection bolts were calculated in accordance with NZS 3404:1997 Section 9. The capacities for the bolts are given in Table 11.

Section Size	Capacity kN	Demand kN
12mm Diameter Category 4.6/S	30.336 (ΦN_{tf})	46.4
		%NBS = 65

Table 11 Connection Bolt Tension Force Capacities and Demands

9.7 Connection Bolt Tensile Force Capacity

The combined shear and tensile force capacities of the connection bolts were calculated in accordance with NZS 3404:1997 Section 9. The capacities for the bolts are given in Table 12.

Combined Actions	Maximum Value	Calculated Value
$\left(\frac{V_f^*}{\Phi V_f}\right)^2 + \left(\frac{N_{tf}^*}{\Phi N_{tf}}\right)^2$	1.0	2.349
		%NBS = 42

Table 12 Connection Bolt Combined Shear and Tension Force Capacities and Demands

9.8 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;

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

= 255 BU

9.9 Wall Bracing Capacity

The bracing capacity of the building was assessed using strengths from the NZSEE guidelines (Table 11.1). Table 11.1 applies a reduction factor of 30% on the bracing capacity due to unknown fixing details of walls constructed prior to 1990. The results of the bracing capacity analysis can be seen in Table 9.3 and Table 9.4.

Section 11.4 of the NZSEE guidelines states that shear panels can utilise their full bracing capacity for aspect ratios (height-to-width) up to 2:1. For aspect ratios greater than 2:1 and up to 3.5:1 a limiting factor is to be applied in accordance with NEHRP Recommended Provisions (BSSC, 2000) as follows;

$$\text{Aspect ratio factor } = \frac{2\text{x Width}}{\text{Height}}$$

Any sections of wall with an aspect ratio greater than 3.5:1 were not included for the purpose of the bracing calculations.

Bracing Line	Bracing Capacity (BU)
A	96
B	74
C	299
D	28
Total bracing capacity =	497 BU

Table 13 Bracing capacity along the timber framed extension

Bracing Line	Bracing Capacity (BU)
1	160
2	20
3	82
Total bracing capacity =	262 BU

Table 14 Bracing capacity across the timber framed extension

9.10 Wall Bracing % NBS

The % NBS of the building is outlined, in both the across and along directions, in Table 12.

Direction	%NBS
Across	100
Along	100

Table 15 %NBS results from detailed wall bracing calculations for timber framed extension

9.11 Discussion of Results

Following a detailed assessment, the building has been assessed as achieving 42**%** New Building Standard (NBS). Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the building is considered potentially an Earthquake Risk building as it achieves greater than 34% NBS but does not achieve above 67% NBS.

9.12 Occupancy

As the building has been assessed to have a %NBS greater than 34% but less than 67%, it is deemed to be a potentially Earthquake Risk. It is recommended that strengthening options are implemented to increase seismic capacity along and across the building to achieve a minimum of 67% NBS. As the building has been assessed as being potentially Earthquake Risk, occupancy of the building is permitted in accordance with Christchurch City Councils policy regarding the occupancy of Earthquake Prone Buildings.

10. Strengthening

As the %NBS of the building has been assessed at 42%, additional works are required to increase the %NBS of the building to achieve the minimum 67% as recommended by The New Zealand Society for Earthquake Engineering (NZSEE).

The connection detail has been assessed as having a %NBS of 42%. Strengthening of the truss to post connection is required in order for the % NBS of the connection to comply with the current design standards. In order to increase the strength of the connection, it is recommended that a fillet weld around the connection plate be added to reduce the combined forces demand on the connection bolts.

The bottom chord of the steel truss has been assessed as having a %NBS of 47%. The critical component effecting the %NBS of the member is the distance between the lateral restraints of the horizontal elements of the steel truss. The member axial capacity is severely affected by the effective length of the member. Provision of lateral restraints at third points along the member would reduce the effective length of the member and increase the member capacity above the demand for the member.

A number of design options for provision of lateral restraint could be implemented. Strengthening options can be explored upon consultation with the Christchurch City Council and their advisors.

11. Recommendations

As the building has been assessed to have a %NBS greater than 34% but less than 67%, it is deemed to be Earthquake Risk. 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.

12. Limitations

12.1 General

This report has been prepared subject to the following limitations:

- No level or verticality surveys have been undertaken.
- No calculations, other than the wall bracing calculations, shear force, axial force 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.

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.

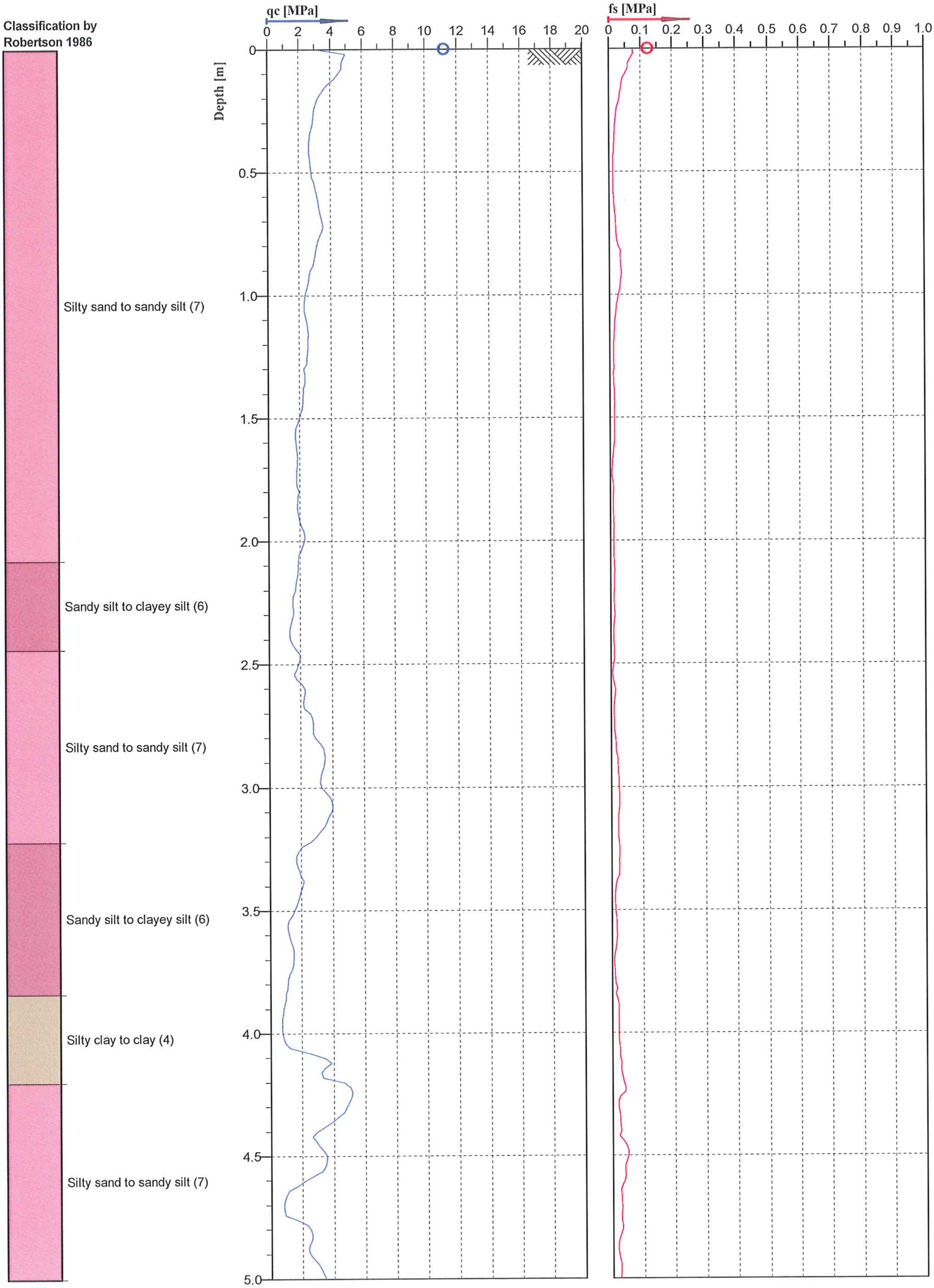
It should be noted that because of the inherent uncertainties in subsurface evaluations, changed or unanticipated subsurface conditions may occur that could affect total project cost and/or execution. GHD does not accept responsibility for the consequences of significant variances in the conditions and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

Geotechnical Investigation Reports and Analysis



PRO-DRILL
SPECIALIST DRILLING ENGINEERS
0800 477 637

U2

Cone No: 4485
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: 90 Hoon Hay Rd	Position: X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no: 1
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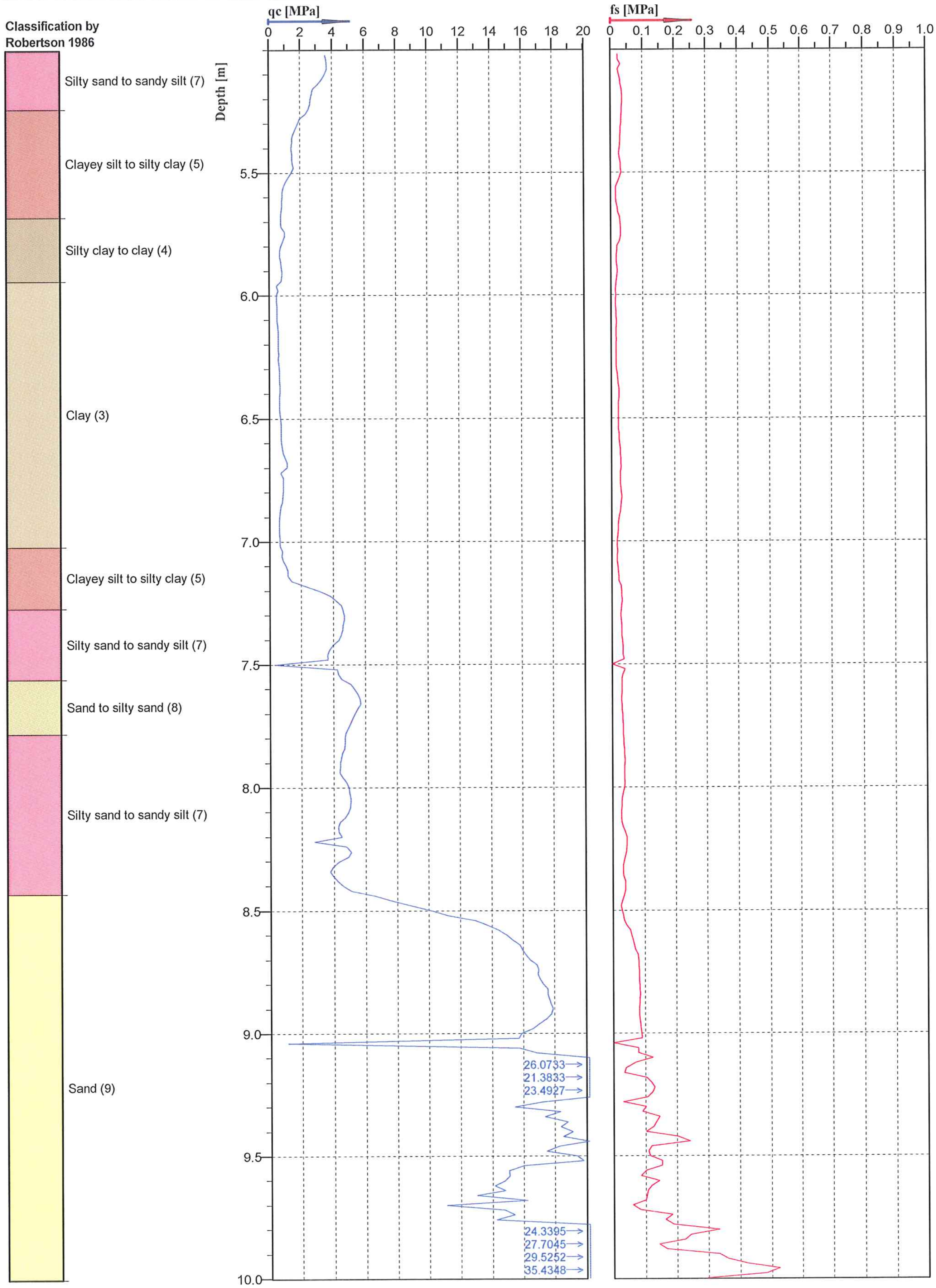

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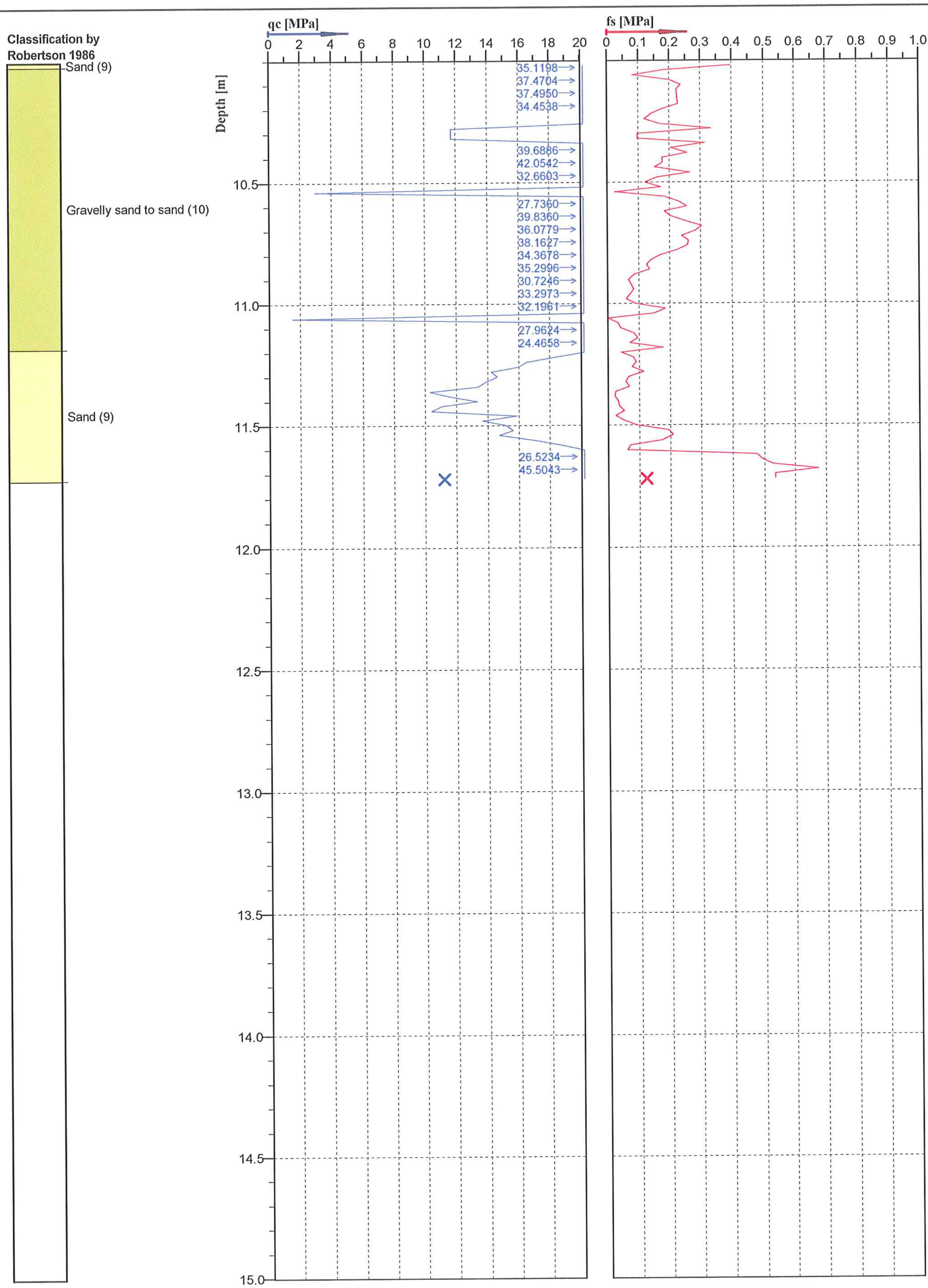
Project: 90HOONHAYRD		Page: 2/3	Fig:
		File: 90HoonHayRdCPT1.cpt	



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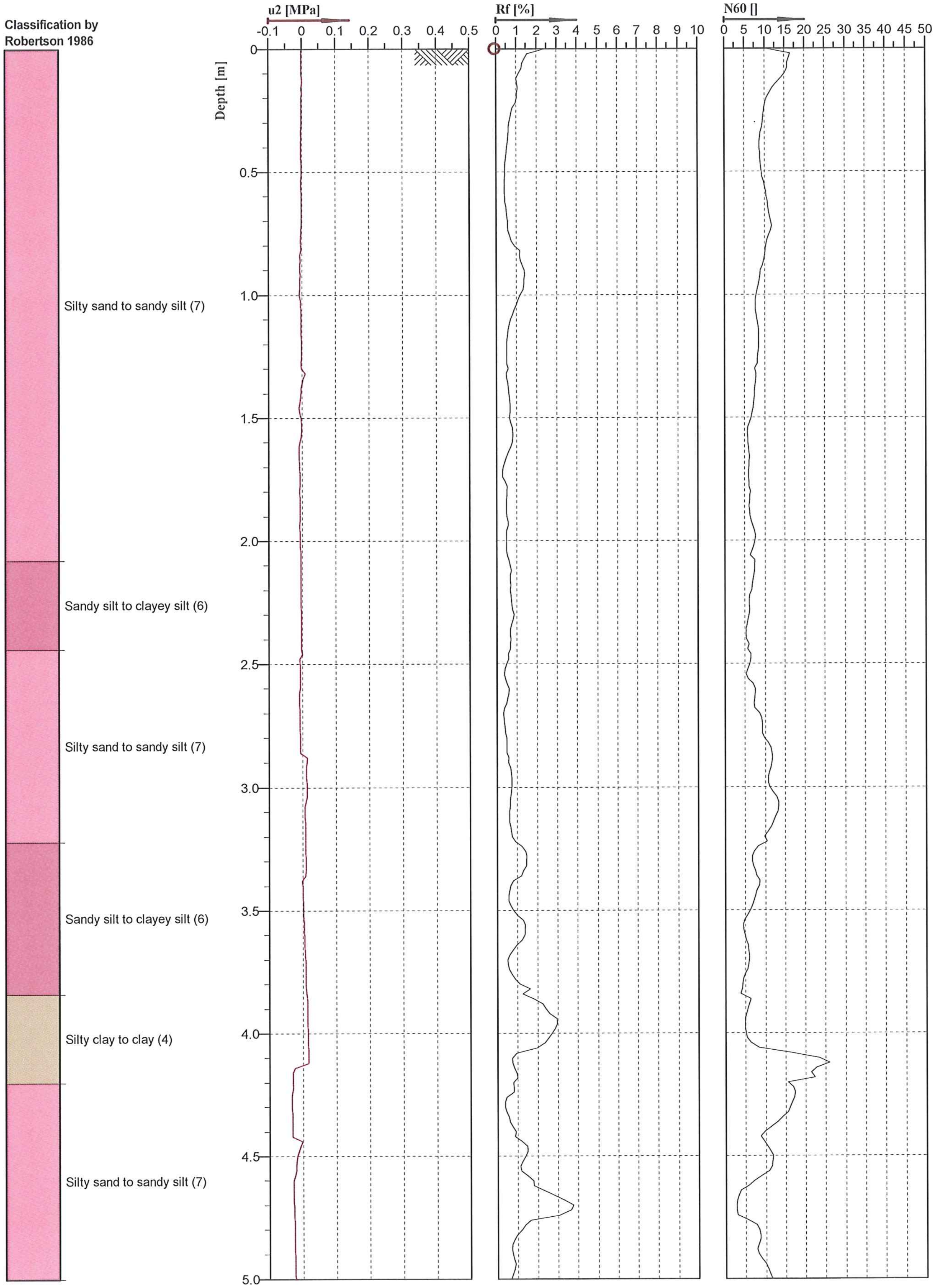
		File: 90HoonHayRdCPT1.cpt	



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Cone No: 4485
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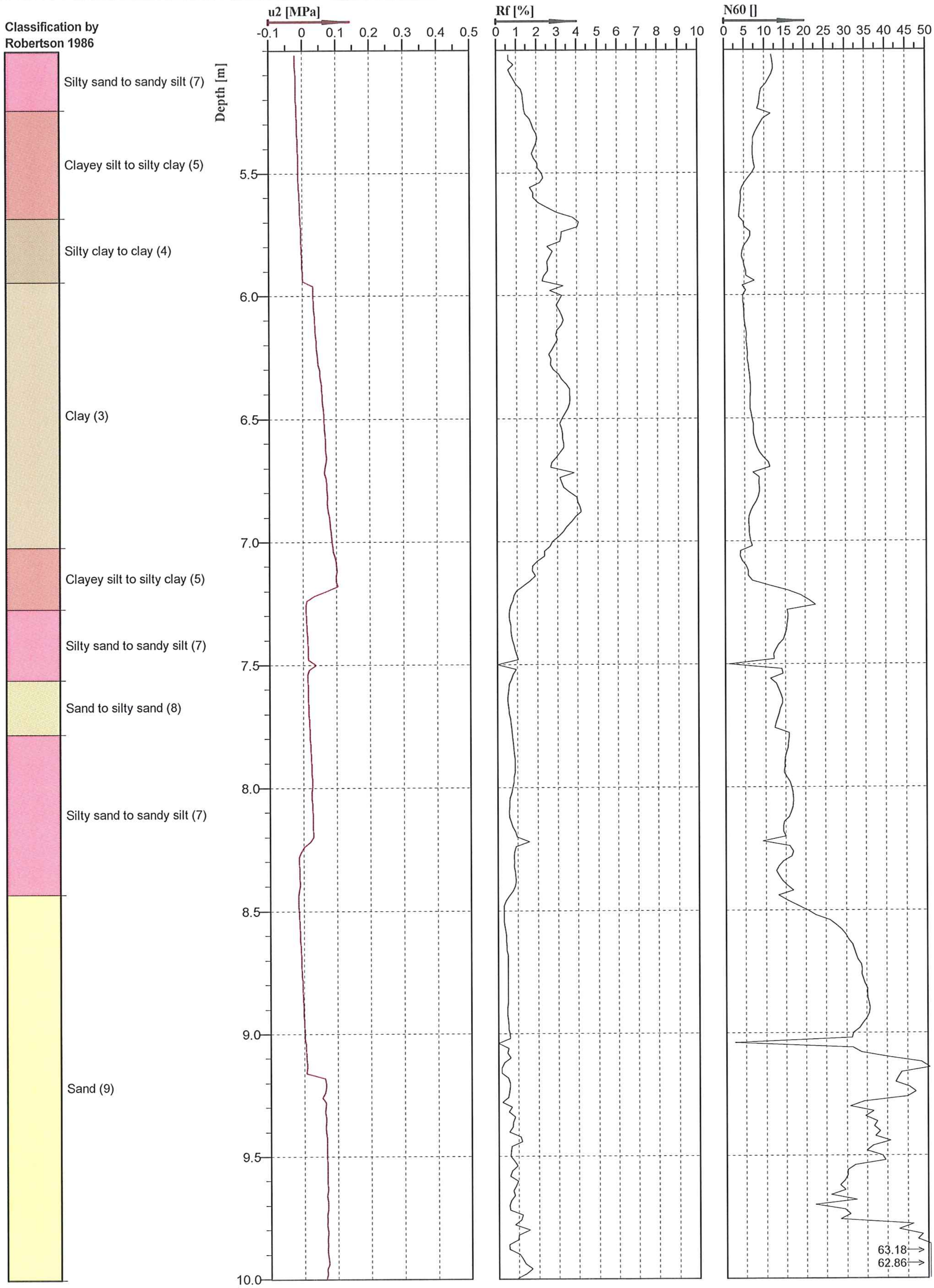
Project ID:	Client: GHD	Date: 4/16/2012	Scale: 1 : 22
Project: 90HOONHAYRD		Page: 1/3	Fig:
		File: 90HoonHayRdCPT1.cpt	



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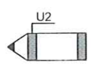

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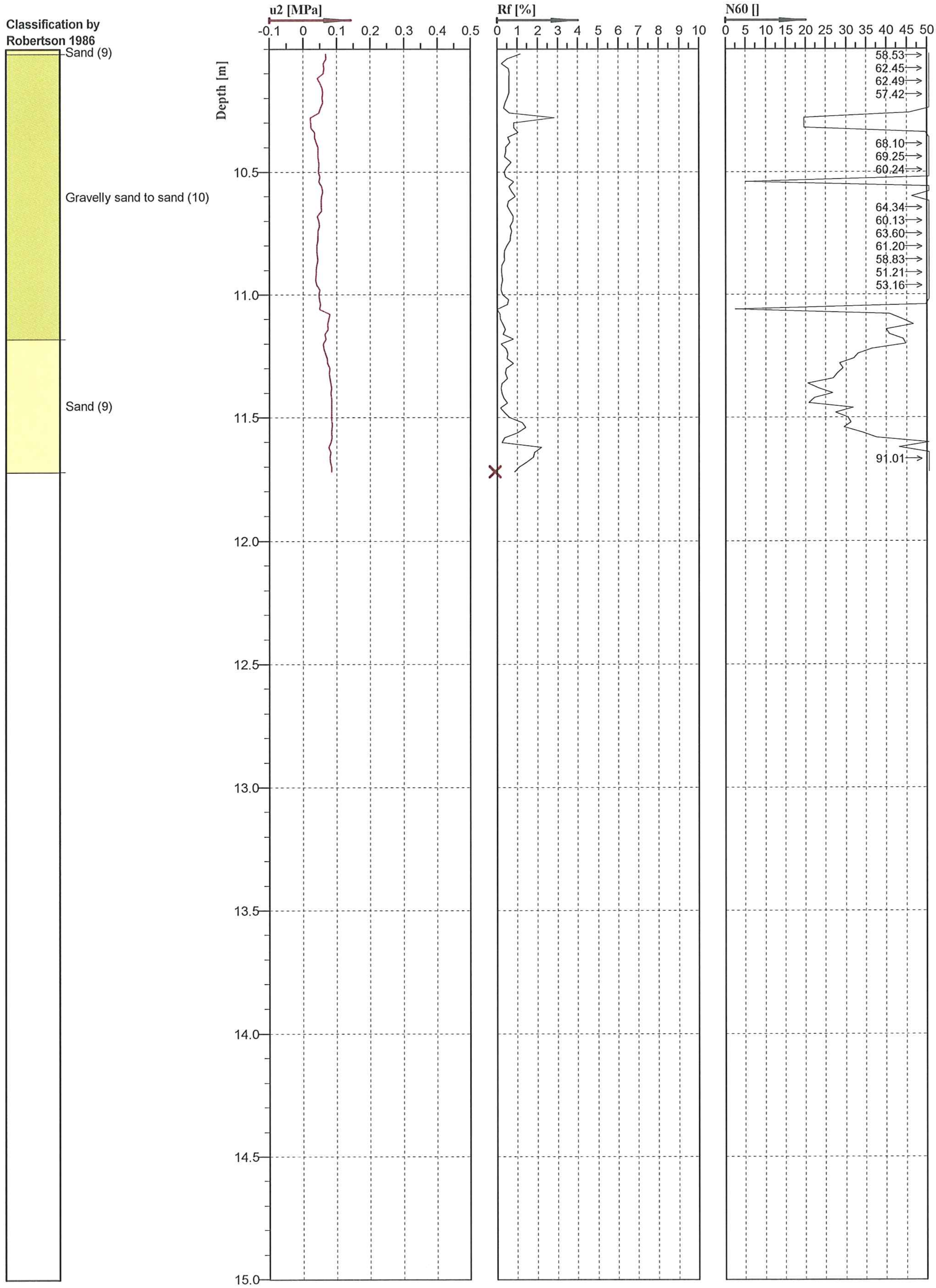
Project ID:	Client: GHD	Date: 4/16/2012	Scale: 1 : 22
Project: 90HOONHAYRD		Page: 2/3	Fig:
		File: 90HoonHayRdCPT1.cpt	



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Project ID:	Client: GHD	Date: 4/16/2012	Scale: 1 : 22
Project: 90HOONHAYRD		Page: 3/3	Fig:
		File: 90HoonHayRdCPT1.cpt	

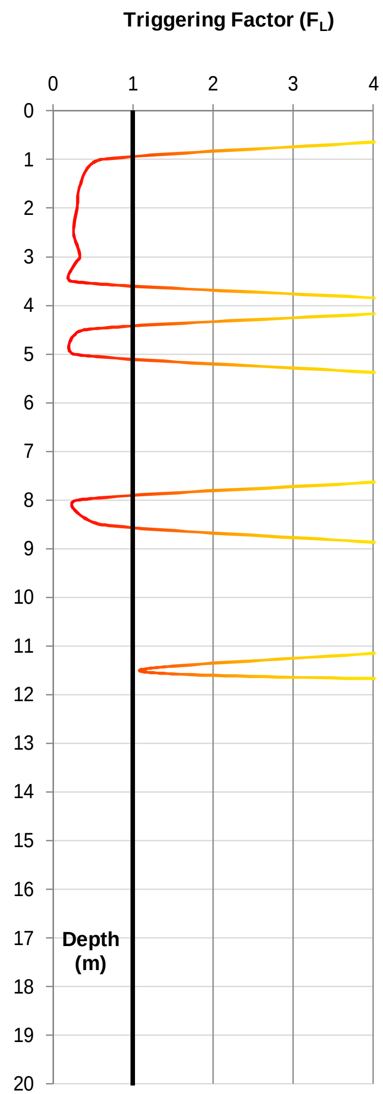
SOIL LIQUEFACTION SUSCEPTIBILITY ASSESSMENT

LOCATION :	**CPT 01**	SHEET :	**1**
PROJECT :	**Hoon Hay Childrens Library**	CALCULATED BY :	**MH**
JOB NO :	**51 30596 12**	CHECKED BY :	
DATE :	**26 April 2012**		

CPT No:	CPT 01	**p_{atm}:**	101 kPa
GWL:	0.7 m	**Bore depth:**	11.7 m
a_{max}:	0.34 g	**EQ Magnitude:**	7.5

Soil Level From	Soil Level To	Thickness (m)	γ, Soil Unit Weight (kN/m³)	Cone Resistance q_c (MPa)	Sleeve Friction Stress, f_s (MPa)	Soil Total Stress σ_{vo} (kPa)	Soil Effective Stress σ'_{vo} (kPa)	Cyclic Stress Ratio, CSR	Cyclic Resistance Ratio, CRR	Soil Behavior (Robertson, 2010)	Triggering Factor F_L	Liquefaction Potential
0	0.5	0.5	16.2	2.77	0.01	8.1	8.1	0.00		SANDS: clean sand to silty sand		NL
0.5	1	0.5	17.1	2.37	0.03	16.7	13.7	0.26	0.16	SAND MIXTURES: silty sand to sandy silt	**0.59**	**Severe**
1	1.5	0.5	16.2	1.97	0.01	24.8	16.9	0.32	0.11	SAND MIXTURES: silty sand to sandy silt	**0.34**	**Severe**
1.5	2	0.5	16.0	2.26	0.01	32.8	20.0	0.35	0.10	SANDS: clean sand to silty sand	**0.30**	**Severe**
2	2.5	0.5	15.7	1.84	0.01	40.7	23.0	0.38	0.10	SAND MIXTURES: silty sand to sandy silt	**0.25**	**Severe**
2.5	3	0.5	17.1	3.32	0.02	49.2	26.7	0.39	0.13	SANDS: clean sand to silty sand	**0.33**	**Severe**
3	3.5	0.5	16.0	1.53	0.01	57.2	29.8	0.41	0.10	SAND MIXTURES: silty sand to sandy silt	**0.24**	**Severe**
3.5	4	0.5	16.4	0.76	0.02	65.4	33.1	0.42		SILT MIXTURES: sandy silt to silty clay		NL
4	4.5	0.5	18.0	3.54	0.05	74.4	37.1	0.42	0.16	SAND MIXTURES: silty sand to sandy silt	**0.38**	**Severe**
4.5	5	0.5	17.1	3.46	0.02	83.0	40.8	0.43	0.12	SANDS: clean sand to silty sand	**0.27**	**Severe**
5	5.5	0.5	17.1	1.40	0.03	91.5	44.4	0.43		SILT MIXTURES: sandy silt to silty clay		NL
5.5	6	0.5	15.8	0.45	0.01	99.4	47.4	0.44		CLAYS: silty clay to clay		NL
6	6.5	0.5	16.4	0.67	0.02	107.6	50.7	0.44		CLAYS: silty clay to clay		NL
6.5	7	0.5	16.1	0.63	0.02	115.6	53.8	0.44		CLAYS: silty clay to clay		NL
7	7.5	0.5	12.5	0.26	0.00	121.9	55.2	0.46		CLAYS: silty clay to clay		NL
7.5	8	0.5	17.8	4.95	0.04	130.8	59.2	0.45	0.13	SANDS: clean sand to silty sand	**0.30**	**Severe**
8	8.5	0.5	17.7	10.20	0.03	139.6	63.1	0.45	0.27	SANDS: clean sand to silty sand	**0.61**	**Severe**
8.5	9	0.5	19.2	15.85	0.09	149.2	67.8	0.44		SANDS: clean sand to silty sand		NL
9	9.5	0.5	19.6	19.38	0.12	159.0	72.7	0.44		SANDS: clean sand to silty sand		NL
9.5	10	0.5	20.9	32.90	0.30	169.5	78.2	0.43		SANDS: clean sand to silty sand		NL
10	10.5	0.5	19.9	36.14	0.12	179.4	83.3	0.42		Gravelly sand to dense sand		NL
10.5	11	0.5	19.5	32.63	0.09	189.2	88.1	0.41		Gravelly sand to dense sand		NL
11	11.5	0.5	19.2	15.19	0.09	198.8	92.8	0.41	0.44	SANDS: clean sand to silty sand	**1.07**	**Moderate**
11.5	11.7	0.2	21.7	49.12	0.53	203.1	95.2	0.40		SANDS: clean sand to silty sand		NL

Liquefaction Susceptibility



Appendix B

Photographs

Photograph 1 North-east elevation. (Front)

Photograph 2 View of the north west wall of the library.

Photograph 3 View of the library from the south-west. (Rear)

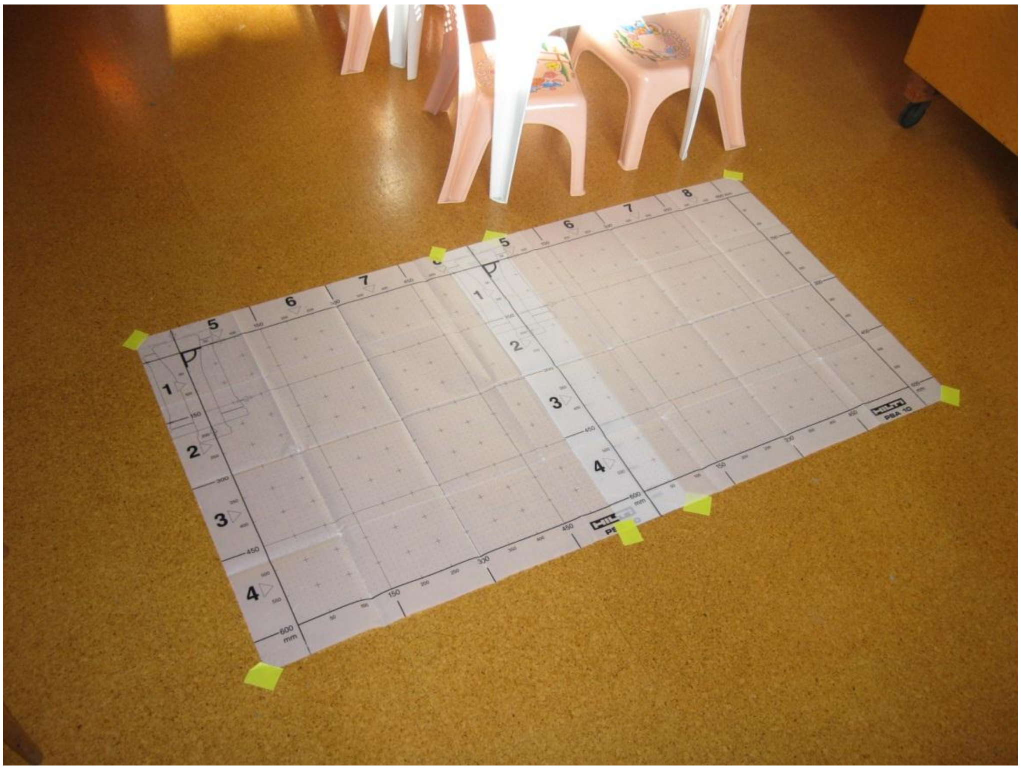
Photograph 4 View of the south-east wall.

Photograph 5 Minor cracking to red brick cladding in north-west corner.

Photograph 6 Roof support trusses spanning from north-west to south-east.

Photograph 7 Bolted connection between steel truss and column.

Photograph 8 Minor liquefaction at western border of property.



Photograph 9 Location of scan of floor slab.

Photograph 10 Hole cut in plasterboard ceiling lining to allow visual verification of ceiling diaphragm.

Photograph 11 Diagonal timber roof sarking supported by timber rafters with lateral bracing.

Photograph 12 Wall ties connecting red brick masonry cladding to the timber frame.

Photograph 13 Pinned base connection of the leg of the frame showing a 100mm wide base plate bolted to a raised concrete footing.

Photograph 14 Pinned connection at the base of the frame leg.

Appendix C

Existing Drawings

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
- NZS 3404:1997 Steel Structures
- New Zealand Society of Earthquake Engineering Guidelines for Assessment and Improvement of the Structural Performance of Buildings in Earthquakes

Materials

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

- Seel (f_y): 350 MPa
- Plasterboard: 3kN/m

Assessment Load Criteria

Basic Assessment Information:

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

Height of building:	2.7 to 3 m
Dimensions of the building:	6m x 8m (see floor plan – Drawing)
Site Location:	90 Hoon Hay Road, Hoon Hay, 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:

Corrugated Steel Roof Cladding (1.5mm thick)	0.12 kN/m^2
Timber Roof Sarking (25mm thick)	0.15 kN/m^2
Ceiling Fibrous Plasterboard Lining (10mm thick)	0.09 kN/m^2
Timber Rafters (200mm x 50mm @ 400c/c)	0.115 kN/m^2
Steel Rafters (50mm x 50mm)	0.035 kN/m
Steel Bars (18mm diameter)	0.029 kN/m
Red Brick Cladding (100mm)	1.9 kN/m^2
Plasterboard Wall Linings (12mm thick)	0.11 kN/m^2
Glazing (5mm thick)	0.1275 kN/m^2
Concrete Panel Infill (100mm thick)	2.4 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

Snow Load is not considered 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)	2.00

Performance Factor (Sp)	0.7
Gravitational Constant (g)	9.81 m/sec^2
Liquefaction Potential	moderate

Site Description

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

Ground Conditions

To be updated by Geotechnical Engineer.

Appendix E

CERA Report Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Hoon Hay Childrens Library			Reviewer:	David Lee
	Unit	No:	Street	CPEng No:	112052
Building Address:		90	Hoon Hay Road	Company:	GHD
Legal Description:				Company project number:	513059612
				Company phone number:	(03) 378 0900
	Degrees	Min	Sec		
GPS south:	43	33	33.42	Date of submission:	1/08/2012
GPS east:	172	36	8.61	Inspection Date:	16/05/2012
				Revision:	Draft
Building Unique Identifier (CCC):	BU 1494-001 EQ2			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	0
Soil type:	silty sand	Soil Profile (if available):	Gravel, Sand and Silt
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	10.00

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	10.20
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):	3.40		height from ground to level of uppermost seismic mass (for IEP only) (m):	3.4
Floor footprint area (approx):	102			
Age of Building (years):	56		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):	Childrens Library			
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	frame system		
Roof:	steel truss	truss depth, purlin type and cladding	Steel Truss 50x50mm SHS T&B Chords
Floors:	concrete flat slab	slab thickness (mm)	150
Beams:	timber	type	
Columns:	structural steel	typical dimensions (mm x mm)	50x50
Walls:	non-load bearing	0	

Lateral load resisting structure

Lateral system along:	lightweight timber framed walls		**Note: Define along and across in detailed report!**		
Ductility assumed, μ:	2.00			note typical wall length (m)	
Period along:	0.10	0.00		estimate or calculation?	estimated
Total deflection (ULS) (mm):				estimate or calculation?	
maximum interstorey deflection (ULS) (mm):				estimate or calculation?	
Lateral system across:	welded and bolted steel moment frame				
Ductility assumed, μ:	2.00			note typical bay length (m)	6
Period across:	0.10	0.00		estimate or calculation?	estimated
Total deflection (ULS) (mm):				estimate or calculation?	
maximum interstorey deflection (ULS) (mm):				estimate or calculation?	

Separations:

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

Non-structural elements

Stairs:			
Wall cladding:	brick or tile	describe (note cavity if exists)	Red brick, timber and concrete
Roof Cladding:	Membrane	substrate	
Glazing:	timber frames		
Ceilings:	plaster, fixed		
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:		Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	0-2 $m^2/100m^3$	notes (if applicable):	Approximately 0.25m^2
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/None/Insignificant	
Across	Damage ratio:	0%	
	Describe (summary):	Minor/None/Insignificant	
Diaphragms	Damage?:	no	Describe:
CSWs:	Damage?:	no	Describe:
Pounding:	Damage?:	no	Describe:
Non-structural:	Damage?:	no	Describe:

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

Recommendations

	Level of repair/strengthening required:	minor structural	Describe:	Strengthening of steel truss to post connection. Provision of lateral restraints at third points of truss chords.
	Building Consent required:	yes	Describe:	Altering Structure
	Interim occupancy recommendations:	full occupancy	Describe:	In line with CCC policy
Along	Assessed %NBS before e'quakes:	100%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology: Timber bracing schedule to NZS 3604: 2011 and steel capacity checks to NZS 3404:1997
	Assessed %NBS after e'quakes:	100%		
Across	Assessed %NBS before e'quakes:	42%	##### %NBS from IEP below	
	Assessed %NBS after e'quakes:	42%		

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: 3.4m

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):	0.1	0.1
(%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	1.00
Note 2: for RC buildings designed between 1976-1984, use 1.2	1.0
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)	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: 1.00

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

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)	1.00	1.00
Ductility scaling factor: =1 from 1976 onwards; or =k_μ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	0.00	0.00

2.6 Structural Performance Scaling Factor:

Sp:	1.000	1.000
Structural Performance Scaling Factor **Factor E**:	1	1

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: 1

3.2. Vertical irregularity, Factor B: 1

3.3. Short columns, Factor C: 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 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!**

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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
FINAL	Peter O'Brien	David Lee		Nick Waddington		7/12/12