CHRISTCHURCH CITY COUNCIL
PRK_0348_BLDG_005 EQ2
The Groynes – Dwelling No.1
182 Johns Road

QUANTITATIVE ASSESSMENT REPORT
FINAL

- Rev B
- 26 March 2013

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Sinclair Knight Merz
142 Sherborne Street
Saint Albans
PO Box 21011, Edgeware
Christchurch, New Zealand
Tel: +64 3 940 4900
Fax: +64 3 940 4901
Web: www.skmconsulting.com

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Approver		26/3/2013	Nick Calvert	Senior Structural Engineer

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1. Executive Summary

1.1. Background

A Quantitative Assessment was carried out on the building PRK_0348_BLDG_005 EQ2 located at The Groynes. This building is a two storey timber framed and concrete masonry structure that is used as the Park Ranger's residence. The drawings indicate that this structure was constructed in 1971. An aerial photograph illustrating the building's location is shown below in Figure 1. Detailed descriptions outlining the building's age and construction type are given in Section 5 of this report.

- **Figure 1 Aerial Photograph of PRK_0348_BLDG_005 EQ2 located at The Groynes**

This Quantitative Assessment Report for the building structure is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on the 3 May 2012, intrusive investigations on 7th August 2012, and the Waimairi County Council structural drawings.

1.2. Key Damage Observed

Key damage observed includes:-

- Hairline cracking to concrete ground slab.
- External concrete patio has moved away from the building.
- Hairline cracking to external cladding vertical joints.
- Hairline cracking to internal wall linings.

Further details describing the level of damage and repair recommendations are given in section 5.4 of this report. A building consent is not likely to be required for repairing this damage.

1.3. Critical Structural Weaknesses

No critical structural weaknesses were observed during our site inspection.

1.4. Indicative Building Strength

As described in the Engineering Advisory Group's "Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings" (from July 2011) we have calculated the capacity of the building structure as a percentage of New Building Standard (% NBS) seismic resistance using the quantitative method. Our assessment included consideration of geotechnical conditions, existing earthquake damage to the building and structural engineering calculations to assess both strength and ductility/resilience.

The assessments were based on the following:

- On-site inspection to assess the extent of existing earthquake damage including limited intrusive investigation.
- Qualitative assessment of critical structural weaknesses (CSWs) based on review of available structural drawings and inspection where drawings were not available.
- No geotechnical investigation has been undertaken. We have based this report a desktop geotechnical study.
- Quantitative assessment of the strength and resilience of the existing structural elements taking account of the current condition.

Any building that is found to have a seismic capacity less than 33% of the new building standard is required to be strengthened up to a capacity of at least 67%NBS, as outlined in the Christchurch City Council's Earthquake-Prone Dangerous & Insanitary Buildings Policy 2010.

Based on the information available, and using the Quantitative Assessment Procedure, the building's original capacity has been assessed to be in the order of 70%NBS and as there is no apparent significant damage to structural elements, it's post earthquake capacity is also in the order of 70%NBS. No critical structural weaknesses were found in this building. This assessment has been completed with reference to the structural drawings.

The building has been assessed to have a seismic capacity in the order of 70% NBS and is therefore not potentially earthquake prone.

1.5. Recommendations

Based on the findings of the assessment, which show the building is above 67% NBS no further assessment or strengthening is required under the current Christchurch City Council Earthquake Prone Building Policy.

It is recommended that:

a) There was no damage to the building that would mean it was unsafe to occupy.
b) We consider that barriers around the building are not necessary.

2. Introduction

Sinclair Knight Merz was engaged by Christchurch City Council to carry out a Quantitative Assessment of the seismic performance of building PRK-0348-005 located at The Groynes.

The scope of this quantitative analysis includes the following:

- Analysis of the seismic load carrying capacity of the building compared with current seismic loading requirements or New Buildings Standard (NBS). It should be noted that this analysis considers the building in its damaged state where appropriate.
- Identify any critical structural weaknesses which may exist in the building and include these in the assessed %NBS of the structure.
- Preparation of a summary report outlining the areas of concern in the building as well as identifying strengthening concepts to 67%NBS for any areas which have insufficient capacity if the building is found to be an earthquake prone building.

The recommendations from the Engineering Advisory Group[1] were followed to assess the likely performance of the structures in a seismic event relative to the New Building Standard (NBS). 100% NBS is equivalent to the strength of a building that fully complies with current codes. This includes a recent increase of the Christchurch seismic hazard factor from 0.22 to 0.3[2].

A qualitative assessment was issued on 27 June 2012. This assessment identified that the seismic capacity of the building was likely to be less than 34% of the New Building Standard (NBS). A quantitative assessment was recommended to confirm the initial assessment findings and to determine a more accurate seismic rating of the building.

An intrusive investigation was carried on 7th August 2012. Construction drawings were made available, and these have been used in our evaluation of the building. The building description below is based on a review of the drawings and our visual and intrusive inspections.

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[1] EAG 2011, *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury - Draft*, p 10
[2] http://www.dbh.govt.nz/seismicity-info

3. Compliance

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

3.1. 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

3.2. Building Act

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

3.2.1. 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).

3.2.2. 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.

3.2.3. 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.

3.2.4. 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.

3.2.5. 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.

3.2.6. Section 131 – Earthquake Prone Building Policy

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

3.3. 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. Council recognises that it may not be practicable for some repairs to meet that target. The council will work closely with building owners to achieve sensible, safe outcomes;
- 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 34%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.

3.4. 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:

a) Hazard Factor increased from 0.22 to 0.3 (36% increase in the basic seismic design load)
b) 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.

4. 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 2 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 2: NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE Guidelines**

Table 1 below provides an indication of the risk of failure for an existing building with a given percentage NBS, relative to the risk of failure for a new building that has been designed to meet current Building Code criteria (the annual probability of exceedance specified by current earthquake design standards for a building of 'normal' importance is 1/500, or 0.2% in the next year, which is equivalent to 10% probability of exceedance in the next 50 years).

- **Table 1: %NBS compared to relative risk of failure**

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

5. Building Details

5.1. Building Description

Our evaluation was based on our visual site investigation conducted on 3 May 2012, intrusive site investigations on 7th August 2012 and the limited Waimairi County Council structural drawings.

Building PRK_0348_BLDG_005 EQ2 is a two storey building used as the Park Ranger's residence. The upper storey is constructed from timber framing whereas the lower storey is constructed from concrete block walls. The roof structure consists of timber framing and light weight corrugated steel cladding supported on the upper storey timber framed walls. These timber framed walls are clad with cedar boards externally and plasterboard internally. As noted above the lower story is constructed from concrete block walls that are reinforced around the openings and have a reinforced concrete ring beam located on top. The concrete block walls support both the upper storey and the level 1 suspended timber floor. The building is founded on concrete strip footings and a concrete slab on grade. The original structural drawings are dated June 1971 and as a result we have taken a design period of 1965-1976 for our assessment.

5.2. Gravity Load Resisting System

As detailed above the roof structure consists of timber framing which is supported on the timber framed walls. The level 1 suspended floor also consists of timber framing and is supported on the lower level concrete block walls. The building is founded on concrete strip footings and a concrete slab on grade.

5.3. Seismic Load Resisting System

For the lateral analysis of this building the 'across direction' has been taken as north-south whereas the 'along direction' has been taken as east-west.

Lateral loads acting across and along the building will be resisted at level 1 by the plasterboard linings present on the ceilings and walls as well as the cross bracing present in the walls detailed on the structural drawings. The lateral loads from the upper storey will then be transferred into the lower storey concrete block walls, through the flexible floor diaphragm. The suspended timber floor will act as a timber diaphragm which will also transfer loads into the lower storey block walls. The concrete block walls will act as shear walls to transfer the lateral loads into the foundations.

5.4. Building Damage

SKM undertook inspections on the 3 May 2012. The following was observed during the time of inspection:

5.4.1. External Damage

General

1) No visual evidence of settlement was noted at this site. Therefore a level survey is not required.

2) Vertical joints in cedar cladding boards have opened up. This is possibly existing damage that has been exacerbated by the earthquake. (PHOTO 4 & 5).

3) Vertical joints in the timber fascia board under the guttering have opened up. This is possibly existing damage that has been exacerbated by the earthquake. (PHOTO 6 & 7).

4) Concrete patio has moved away from the building along the northern and eastern sides. This has created a gap between the patio and the building of approximately 10mm (PHOTO 8, 9 & 10).

5) Hairline cracking present to the concrete patio in the north-east corner. Crack widths are approximately 0.3mm wide. Small amount of spalling to the concrete patio next to the north-east corner of the building. (PHOTO 11, 12 & 13).

5.4.2. Internal Damage

Level 0 – Main House Garage

6) Hairline crack present along the ceiling lining joint, located in the eastern half (PHOTO 15 & 16).

7) Crack approximately 0.4mm wide present in concrete slab near the main entrance (PHOTO 17 & 18).

8) Crack approximately 0.2mm wide present in concrete in the south-east corner.

9) Crack approximately 0.3mm wide present in concrete slab. Crack runs north-south and continues through to the small northern storage room (PHOTO 25, 26 & 27).

Level 0 – Laundry and Toilet

10) Hairline cracking present along ceiling lining joints (PHOTO 19 & 20)

Level 0 – Western Garages / Storage Rooms

11) Hairline cracking present along ceiling lining joints in the north-west garage/ storage room (PHOTO 21 & 22).

12) Crack approximately 0.2-0.3mm wide present in slab (PHOTO 23 & 24).

Level 1 – Living Area

13) Hairline cracking present to wall lining in the north-east corner. Cracking continues along the wall lining and ceiling lining joint on the north wall (PHOTO 29 & 30).

14) Hairline cracking present around the original flu penetration near north-west corner.

Level 1 - Kitchen

15) Hairline cracking present in wall lining joint above east wall window (PHOTO 32 & 33).

Level 1 – Dining Area

16) Vertical hairline crack present at wall lining joint under the eastern corner of the south wall window (PHOTO 34 & 35).

17) Hairline crack present at wall lining joint on the south wall. Crack located above the kitchen partition wall (PHOTO 34, 36 & 37).

Level 1 - Hallway

18) Joints between the door architrave and door jamb have opened up on the eastern door (PHOTO 39).

19) Hairline crack present in the ceiling lining joint in the south-west corner (PHOTO 40, 41 & 42).

Level 1 - Bathroom

20) Hairline cracking present along wall lining joint above the southern corner of the bathroom doorway (PHOTO 43 & 44)

21) Hairline cracking along wall lining joint in the north-west corner (PHOTO 43 & 45)

22) Hairline cracking present along the top joints of the door architrave (PHOTO 50 & 51)

Level 1 - Toilet

23) Hairline cracking present along the wall lining joints each side of the window (PHOTO 46 & 47).

24) Hairline cracking present along the scotia and wall lining joint in the south-east and south-west corners (PHOTO 48 & 49).

25) Hairline cracking present along the top joints of the door architrave (similar to PHOTO 50 & 51).

Level 1 – South-West Bedroom

26) Hairline cracking present along the wall lining joint above south corner of doorway (PHOTO 52 & 53).

Level 1 – Master Bedroom

27) Hairline cracking present along the wall lining joint above the north corner of the doorway (PHOTO 55 & 56).

28) Hairline cracking presnt along the wall lining joints above all north wall window corners (PHOTO 57 & 58).

29) Hairline cracking present along the wall lining and ceiling lining joint. Occurs on the western and southern walls (PHOTO 59 & 60).

30) Hairline cracking present along the ceiling lining joint near the west wall window (PHOTO 61 & 62).

Photo's detailing the damage note above can be found in Appendix 1 – Photos

6. Available Information and Assumptions

6.1. Available Information

Following our visual inspection on the 3 May 2012, SKM carried out a quantitative assessment of the building structure. This review was undertaken using the available information which was as follows:

- Structural drawings of the building dated 1971.
- SKM intrusive site investigation carried out on building on 7th August 2012.
- SKM qualitative assessment report.

6.2. Survey

The building was not surveyed.

6.3. Design Assumptions

The following design assumptions made in undertaking the assessment include:

- The building was built according to the drawings and according to good practice at the time. We have reviewed the building and from our visual inspection the structure appears to be built in accordance with the drawings.
- A geotechnical desktop study was carried out for this site. The main conclusions from this report are:
  - The site has been assessed as NZS1170.5 Class D (deep or soft soil) from adjacent borehole logs.
  - Liquefaction risk appears to be low to moderate
  - In general the structures on site appear to be relatively light construction supported on shallow footings. There is relatively good agreement on the geology of the soil below a depth of 5m from the available ground investigation data. However, as no geotechnical parameters are available, in order to perform a quantitative assessment, additional investigations recommended to estimate shallow soil properties are:
  - Two CPTs near larger buildings such as the ranger's office and dwelling 2 are recommended. For small structures such as the kiosk and office building, two hand augers to infer the composition of shallow soils would be adequate.

  The full geotechnical desktop study can be found in Appendix 3.It should be noted that since the completion of the geotechnical desktop study, Christchurch City Council no longer requires additional geotechnical investigations in the quantitative assessment. This contradicts what we have recommended for the desktop geotech investigation, but is acceptable for this site, noting the following limitations. No subsurface investigations took place in the geotechnical desktop study therefore the information presented in the study has not been verified on site. In preparing the desktop study SKM has relied upon, and presumed accurate, any information (or confirmation of the absence thereof) provided by our Client, and from other sources as described in the report.

- Masonry blocks are only filled where there is reinforcing present, all other masonry blocks are unfilled

6.4. Design Criteria

The following design criteria made in undertaking the assessment include:

- Standard design criteria for typical residential buildings as described in AS/NZS1170.0:2002:
  - 50 year design life, which is the default NZ Building Code design life.
  - Structure Importance Level 2. This level of importance is described as 'normal' with medium or considerable consequence for loss of human life, or considerable economic, social or environmental consequence of failure.
  - 1.5kPa live load.
- The building has a short period less than 0.4 seconds.
- Site hazard factor, Z = 0.3, NZBC, Clause B1 Structure, Amendment 11 effective from 1 August 2011
- A ductility level of 1 was used in the analysis of this building; this is a conservative approach as it may be possible that the ductility level is 1.25.
- The following material properties were used in the analyses:
- **Table 2: Material Properties**

Material	Nominal Strength	Structural Performance	Reference
Masonry in shear	f'_{ms} = 0.25 Mpa	S_p = 1.0	AS 3700 section 3.3.4
Tensile strength of mortar joint	f'_{mt} = 0.20 Mpa	Sp = 1.0	AS 3700 section 3.3.3
Lateral modulus of rupture of masonry	f'_{ut} = 0.8 Mpa	S_p = 1.0	AS 3700 section 3.2

Material	Nominal Strength	Structural Performance	Reference
Gib Lining	Bracing capacity = 2.1kN/m	S_p = 1.0	NZSEE 2006 AISBE guidelines Table 11.1

The detailed engineering analysis is a post construction evaluation therefore it has the following limitations:

- It is not likely to pick up on any concealed construction errors (if they exist)
- Other possible issues that could affect the performance of the building such as corrosion and modifications to the structure will not be identified unless they are visible and have been specifically mentioned in this report.
- The detailed engineering evaluation deals only with the structural aspects of the structure. Other aspects such as building services are not covered.

6.5. The Detailed Engineering Evaluation (DEE) process

The DEE is a procedure written by the Department of Building and Housing's Engineering Advisory Group and grades buildings according to their likely performance in a seismic event. The procedure is not yet recognised by the NZ Building Code but is widely used and recognised by the Christchurch City Council as the preferred method for preliminary seismic investigations of buildings[3].

The procedure of the DEE is as follows:

1) Qualitative assessment procedure
2) Determine the building's status following any rapid assessment that have been done
3) Review any existing documentation that is available. This will give the engineer an understanding of how the building is expected to behave. If no documentation is available, site measurements may be required
4) Review the foundations using conservative geotech parameters.
5) Investigate possible Critical Structural Weaknesses (CSW) or collapse hazards
6) Assess the original and post earthquake strength of the building (this assessment is subsequently superseded by the quantitative assessment)

[3] http://resources.ccc.govt.nz/files/EarthquakeProneDangerousAndInsanitaryBuildingsPolicy2010.pdf

7) Quantitative procedure
8) Carry out a desktop geotechnical investigation for the quantitative assessment
9) Analyse the building according to current building codes and standards. Analysis accounts for damage to the building.

The DEE assessment ranks buildings according to how well they are likely to perform relative to a new building designed to current earthquake standards, as shown in Table 3. The building rank is indicated by the percent of the required New Building Standard (%NBS) strength that the building is considered to have. Earthquake prone buildings are defined as having less than 34 %NBS strength which correlates to an increased risk of approximately 20 times that of 100% NBS[4]. Buildings that are identified to be earthquake prone are required by law to be strengthened within 30 years of the owner being notified that the building is potentially earthquake prone[5].

Table 3: DEE Risk classifications

Description	Grade	Risk	%NBS	Structural performance
Low risk building	A+	Low	> 100	Acceptable. Improvement may be desirable.
	A		100 to 80	
	B		80 to 67	
Moderate risk building	C	Moderate	67 to 33	Acceptable legally. Improvement recommended.
High risk building	D	High	33 to 20	Unacceptable. Improvement required.
	E		< 20	

The DEE method rates buildings based on the plans (if available) and other information known about the building and some more subjective parameters associated with how the building is detailed and so it is possible that %NBS derived from different engineers may differ.

This assessment describes only the likely seismic Ultimate Limit State (ULS) performance of the building. The ULS is the level of earthquake that can be resisted by the building without catastrophic failure. The DEE does also consider Serviceability Limit State (SLS) performance of the building and or the level of earthquake that would start to cause damage to the building but this result is secondary to the ULS performance.

[4] NZSEE 2006, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes*, p 2
[5] http://resources.ccc.govt.nz/files/EarthquakeProneDangerousAndInsanitaryBuildingsPolicy2010.pdf

The NZ Building Code describes that the relevant codes for determining %NBS are primarily:

- AS/NZS 1170 Structural Design Actions
- NZS 3101:2006 Concrete Structures Standard
- NZS 3404:1997 Steel Structures Standard
- NZS4230:2004 Design of Reinforced Concrete Masonry Structures
- NZS 3603:1993 Timber Structures Standard
- NZS 3604:2011 Timber Framed Buildings

7. Results and Discussions

7.1. Critical Structural Weaknesses

This building has no critical structural weaknesses

7.2. Analysis Results

The equivalent static force method as defined in NZS1170.5 was used to calculate the loads acting on the building. The capacities of the structural elements were then calculated and compared to the demands to ascertain the % NBS. The 1st floor was analysed using the Gib Ezy Brace spreadsheet, and adjusting the capacities of the existing brace elements in accordance with the NZSEE document "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes". The load demands in the ground floor masonry walls were distributed by a centre of stiffness and centre of mass analysis, and using a nominal amount of redistribution from the 1st floor diaphragm the loads demands on each masonry wall could be evaluated. The results of the analysis are reported in the following table as %NBS. The results below are calculated for the building in its damaged state. The building results have been broken down into their seismic resisting elements.

(%NBS = probable strength / new building standards)

Table 4: DEE Results

Seismic Resisting Element	Action	Seismic Rating %NBS
1st Floor Gib lined walls, longitudinal direction	Shear	100%
1st Floor Gib lined walls, transverse direction	Shear	100%
Ground Floor masonry walls acting in longitudinal direction	Shear	70%
Ground Floor masonry walls acting in transverse direction	Shear	70%
Out of plane bending of masonry wall	Flexural	100%

7.3. Recommendations

The quantitative assessment carried out on Groynes Dwelling no.1 indicates that the building has a seismic capacity greater than 67% of NBS and is therefore classed as being in the category of 'Low Risk Buildings'. The policy Christchurch City Council has adopted will mean strengthening of this building is not required.

8. Conclusion

SKM carried out a quantitative assessment on Groynes Dwelling No.1 located at 182 Johns Road. This assessment concluded that the building is not Earthquake Prone.

- **Table 5: Quantitative assessment summary**

Grade	Risk	%NBS	Structural Performance
B	Low	70	Acceptable. Improvement may be desirable.

It is recommended that:

a) There was no damage to the building that would mean it was unsafe to occupy.
b) We consider that barriers around the building are not necessary.

9. Limitation Statement

This report has been prepared on behalf of, and for the exclusive use of, SKM’s client, and is subject to, and issued in accordance with, the provisions of the contract between SKM and the Client. It is not possible to make a proper assessment of this report without a clear understanding of the terms of engagement under which it has been prepared, including the scope of the instructions and directions given to, and the assumptions made by, SKM. The report may not address issues which would need to be considered for another party if that party's particular circumstances, requirements and experience were known and, further, may make assumptions about matters of which a third party is not aware. No responsibility or liability to any third party is accepted for any loss or damage whatsoever arising out of the use of or reliance on this report by any third party.

Without limiting any of the above, in the event of any liability, SKM's liability, whether under the law of contract, tort, statute, equity or otherwise, is limited in as set out in the terms of the engagement with the Client.

It is not within SKM’s scope or responsibility to identify the presence of asbestos, nor the responsibility of SKM to identify possible sources of asbestos. Therefore for any property pre-dating 1989, the presence of asbestos materials should be considered when costing remedial measures or possible demolition.

Should there be any further significant earthquake event, of a magnitude 5 or greater, it will be necessary to conduct a follow-up investigation, as the observations, conclusions and recommendations of this report may no longer apply Earthquake of a lower magnitude may also cause damage, and SKM should be advised immediately if further damage is visible or suspected.

10. Appendix 1 – Photos

	
Photo 1: East Elevation	Photo 2: North Elevation
	
Photo 3: West Elevation	Photo 4: Hairline Cracking along Joints in External Cladding

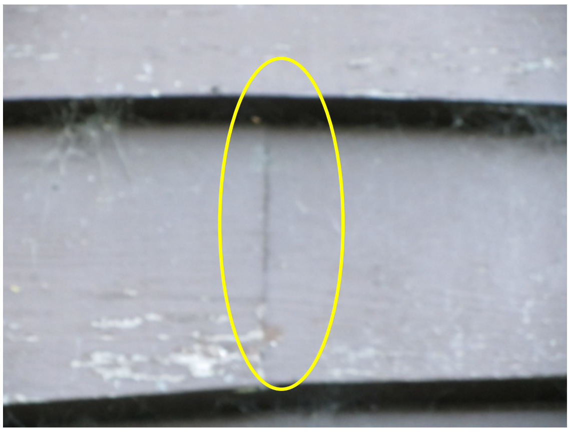


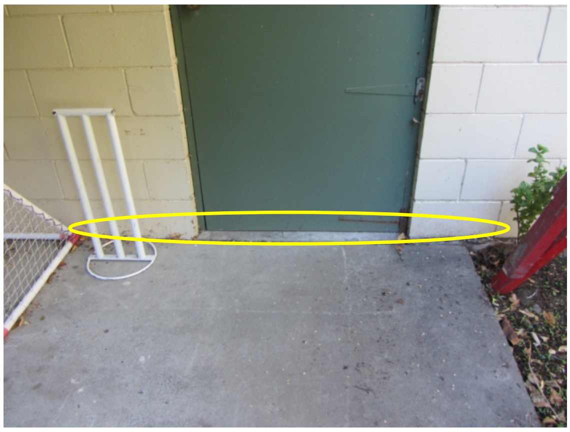
Photo 5: Close Up of Photo 4	Photo 6: Hairline Cracking along Fascia BoardJoints
	
Photo 7: Close Up of Photo 6	Photo 8: Gap Between Concrete Patio and Building along North Side of Building
	
Photo 9: Close Up of Photo 8	Photo 10: Close Up of Photo 8

	
Photo 11: Hairline Cracking to Concrete Patio in North-East Corner	Photo 12: Close Up of Photo 11
	
Photo 13: Close Up of Photo 11	Photo 14: Main House Garage

	
Photo 15: Hairline Cracking to Ceiling Lining Joint in Main Garage	Photo 16: Close Up of Photo 15
	
Photo 17: Hairline Cracking to Concrete Slab in Main Garage	Photo 18: Close Up of Photo 17

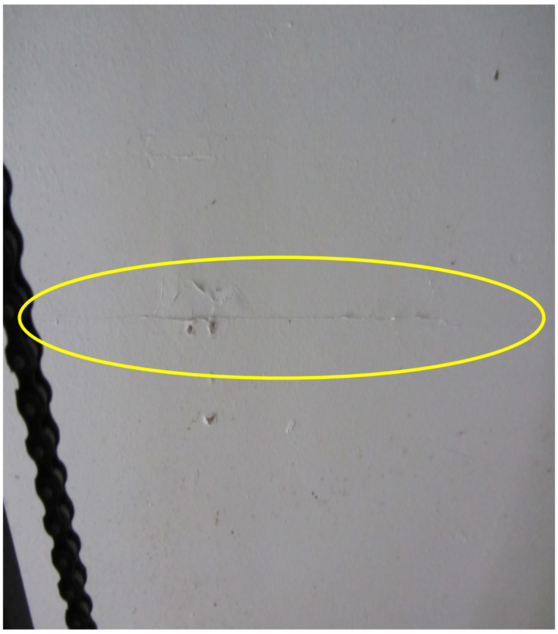
	
Photo 19: Hairline Cracking to Ceiling Lining Joints in Laundry	Photo 20: Close Up of Photo 19
	
Photo 21: North-West Garage / Storage Room	Photo 22: Hairline Cracking to Ceiling Lining Joint in NW Garage / Storage Room

Photo 23: Hairline Cracking to Concrete Slab in NW Garage / Storage Room

Photo 24: Close Up of Photo 23

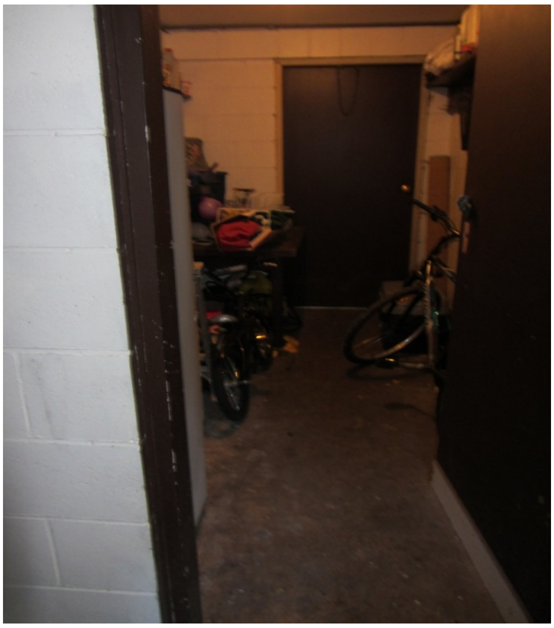


Photo 25: Continuation of Hairline Crack running N/S in Main Garage Floor Slab

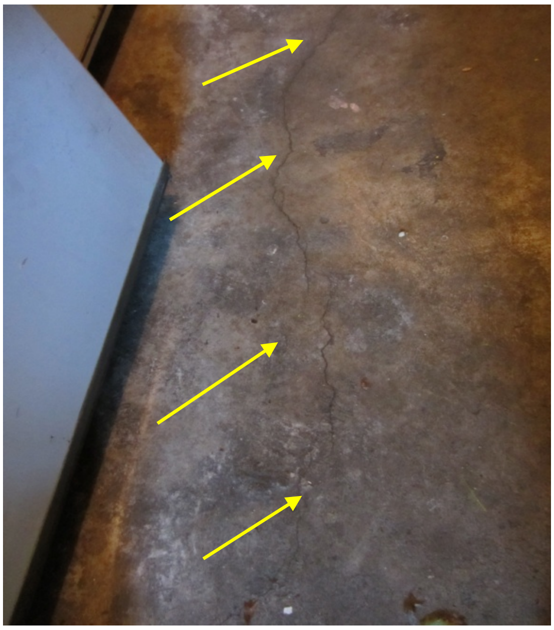


Photo 26: Close Up of Photo 25

	
Photo 27: Close Up of Photo 26	Photo 28: Living Area
	
Photo 29: Hairline Cracking to Wall Lining in NE Corner	Photo 30: Hairline Cracking Present along Wall and Ceiling Lining Joint on North Wall

	
Photo 31: Kitchen	Photo 32: Hairline Cracking along Wall Lining Joint above Window
	
Photo 33: Close Up of Photo 32	Photo 34: Dining Area

	
Photo 35: Hairline Cracking Present along Wall Lining Joint under South Wall Window	Photo 36: Hairline Cracking Present along Wall Lining Joint on South Wall above Kitchen Partition

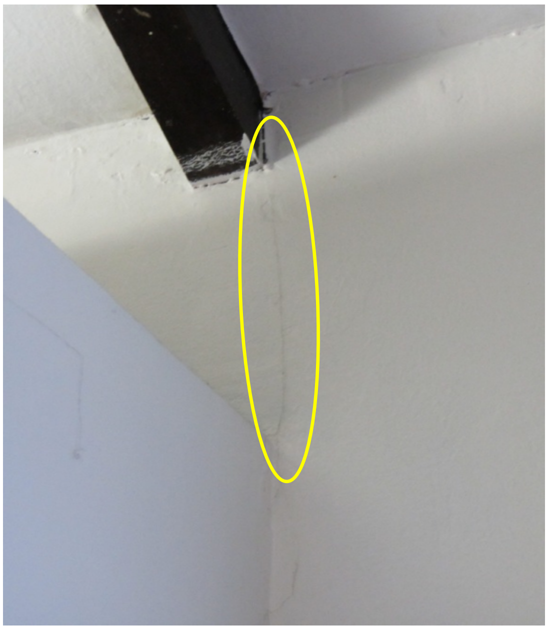
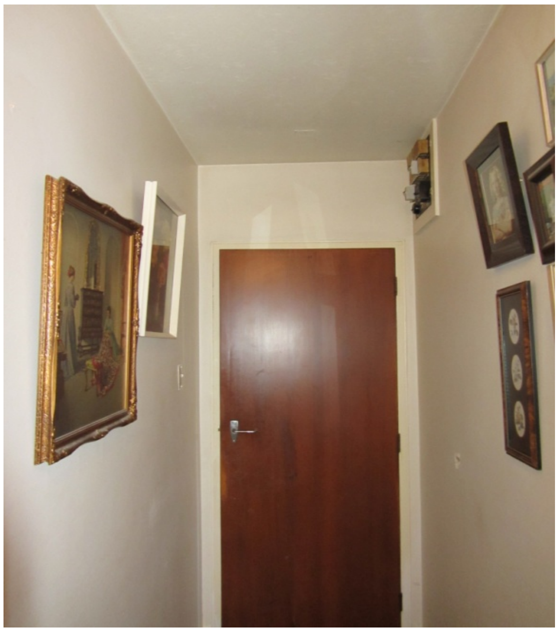

	

Photo 37: Close Up of Photo 36	Photo 38: Hallway – Looking east

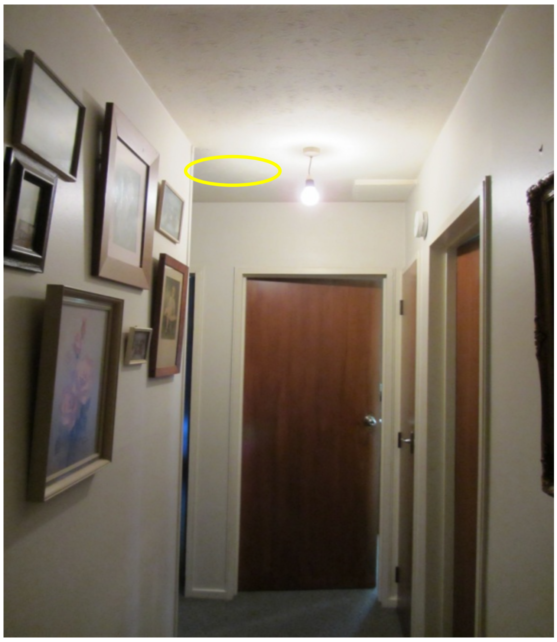
	
Photo 39: Gaps between Door Architrave and Door Jamb on East Wall have Opened Up	Photo 40: Hallway – Looking West
	
Photo 41: Hairline Cracking to Ceiling Lining in SW Corner of Hallway	Photo 42: Close Up of Photo 41

	
Photo 43: Bathroom	Photo 44: Hairline Cracking to Wall Lining above Bathroom Doorway
	
Photo 45: Hairline Cracking along Wall Lining Joint in NW Corner of Bathroom	Photo 46: Toilet

Photo 47: Hairline Cracking along Wall Lining Joint Each side of Toilet Window

Photo 48: Hairline Cracking along Scotia and Wall Lining Joint

Photo 49: Close Up of Photo 48

Photo 50: Hairline Cracking to Toilet & Bathroom Door Architrave Joints

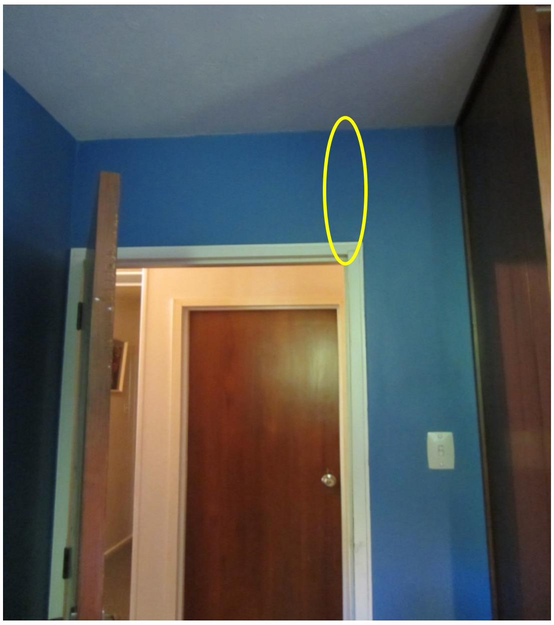
	
Photo 51: Close Up of Photo 50	Photo 52: Hairline Cracking along Wall Lining Joint above Doorway in SW Bedroom
	
Photo 53: Close Up of Photo 52	Photo 54: Master Bedroom

Photo 55: Hairline Cracking along Wall Lining Joint above Doorway in Master Bedroom

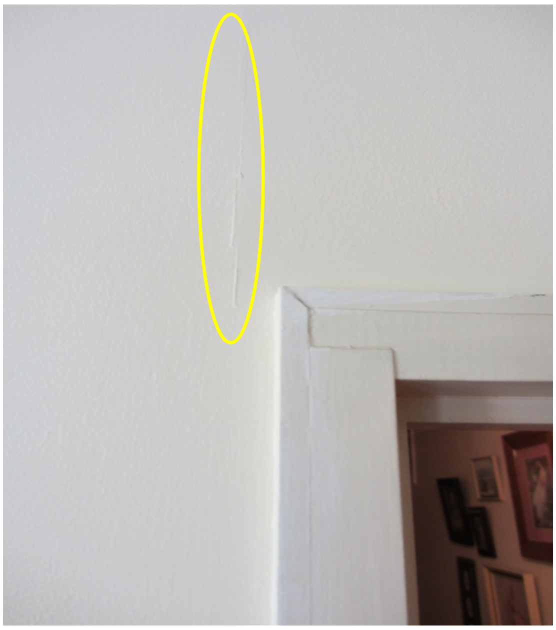


Photo 56: Close Up of Photo 55

Photo 57: Hairline Cracking along Wall Ling Joints above Windows in Master Bedroom

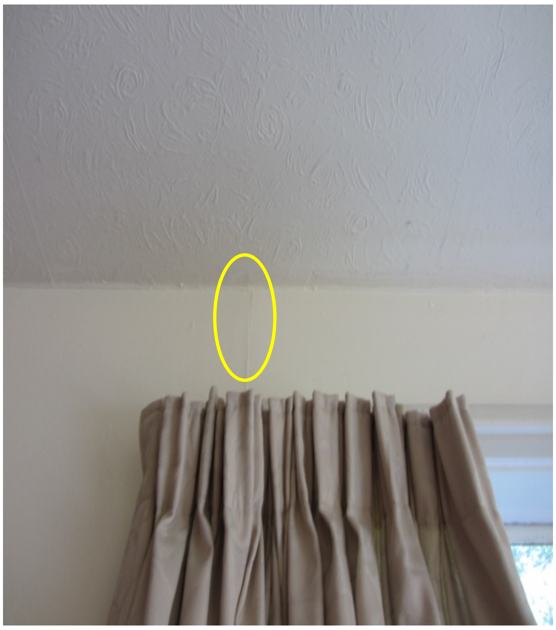


Photo 58: Close Up of Photo 57

	
Photo 59: Hairline Cracking along the Wall and Ceiling Lining Joint in the Master Bedroom – Occurs on West & South walls	Photo 60: Close Up of Photo 59
	
Photo 61: Hairline Cracking along Ceiling Lining Joint above West Wall Window in Master Bedroom	Photo 62: Close Up of Photo 60

11. Appendix 2 – CERA Standardised Report Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	PRK_0348_BLDG_005 EQ2	Reviewer:	J. Carter
	Unit No: Street	CPEng No:	1017618
Building Address:	Dwelling No.1 - The Groynes 182 Johns Road	Company:	SKM
Legal Description:		Company project number:	ZB01276.066
		Company phone number:	03 940 4900
	Degrees Min Sec		
GPS south:		Date of submission:	26-Mar
GPS east:		Inspection Date:	3/05/2012
		Revision:	B
Building Unique Identifier (CCC):	PRK_0348_BLDG_005 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	mixed	Soil Profile (if available):	The regional geological map shows the site as underlain by river alluvium, comprising gravel, sand and silt, beneath plains or low level terraces.
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:	2 single storey = 1	Ground floor elevation (Absolute) (m):	10.10
Ground floor split?	no	Ground floor elevation above ground (m):	0.10
Storeys below ground	0		
Foundation type:	strip footings	if Foundation type is other, describe:	
Building height (m):	6.50	height from ground to level of uppermost seismic mass (for IEP only) (m):	6.5
Floor footprint area (approx):	100		
Age of Building (years):	41	Date of design:	1965-1976
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	other (specify)	Brief strengthening description:	
Use (upper floors):	other (specify)		
Use notes (if required):	residential		
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	met-rib steel cladding on75x50 timber purlins on 125x50 timber rafters on 200x75 timber spine beam
Floors:	timber	joist depth and spacing (mm)	200x50 timber joists at 450crs
Beams:	timber	type	floor and roof structure as detailed above
Columns:	load bearing walls	typical dimensions (mm x mm)	190mm thk concrete block
Walls:	partially filled concrete masonry	thickness (mm)	190

Lateral load resisting structure

Lateral system along:	partially filled CMU	**Note: Define along and across in detailed report!**	note total length of wall at ground (m):	13
Ductility assumed, μ:	1.00		wall thickness (m):	0.19
Period along:	0.40	0.40 from parameters in sheet	estimate or calculation?	estimated
Total deflection (ULS) (mm):	25		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):	5		estimate or calculation?	estimated
Lateral system across:	partially filled CMU		note total length of wall at ground (m):	7
Ductility assumed, μ:	1.00		wall thickness (m):	0.19
Period across:	0.40	0.40 from parameters in sheet	estimate or calculation?	estimated
Total deflection (ULS) (mm):	25		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):	5		estimate or calculation?	estimated

Separations:

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

Non-structural elements

Stairs:	steel	describe supports	bolted to deck support beam
Wall cladding:	other light	describe	cedar boards
Roof Cladding:	Metal	describe	met-rib steel cladding (light-weight)
Glazing:	timber frames		
Ceilings:	plaster, fixed		
Services(list):	lights, insulation etc		

Available documentation

Architectural	partial	original designer name/date	Waimairi District Council
Structural	partial	original designer name/date	Waimairi District Council
Mechanical		original designer name/date	
Electrical		original designer name/date	
Geotech report	partial	original designer name/date	Geotechnical Desktop study by SKM dated 20 April 2012

Damage

Site: (refer DEE Table 4-2)

Site performance:	1	Describe damage:	no damage observed during site inspection
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	no structural damage noted during site inspection
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	yes	Describe:	hairline cracking to concrete slab and wall linings

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

Recommendations

	Level of repair/strengthening required:	minor non-structural	Describe:	
	Building Consent required:	no	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	
Along	Assessed %NBS before:	70%	%NBS from IEP below	If IEP not used, please detail assessment methodology: Calculations by NLC, refer to SKM Quantitative Report
	Assessed %NBS after:	70%		
Across	Assessed %NBS before:	70%	%NBS from IEP below	
	Assessed %NBS after:	70%		

12. Appendix 3 – Geotech Desk Study

Christchurch City Council - Structural Engineering Service

Geotechnical Desk Study

SKM project number	ZB01276
SKM project site number	063-080 inclusive
Address	Groynes, 182 Johns Road
Report date	20 April 2012
Author	Ross Roberts / Ananth Balachandra
Reviewer	Leah Bateman
Approved for issue	Yes

1. Introduction

This report outlines the geotechnical information that Sinclair Knight Merz (SKM) has been able to source from our database and other sources in relation to the property listed above. We understand that this information will be used as part of an initial qualitative DEE, and will be supplemented by more detailed information and investigations to allow detailed scoping of the repair or rebuild of the building.

2. Scope

This geotechnical desk top study incorporates information sourced from:

- Published geology
- Publically available borehole records
- Liquefaction records
- Aerial photography
- Council files
- A preliminary site walkover

3. Limitations

This report was prepared to address geotechnical issues relating to the specific site in accordance with the scope of works as defined in the contract between SKM and our Client. This report has been prepared on behalf of, and for the exclusive use of, our Client, and is subject to, and issued in accordance with, the provisions of the contract between SKM and our Client. The findings presented in this report should not be applied to another site or another development within the same site without consulting SKM.

The assessment undertaken by SKM was limited to a desktop review of the data described in this report. SKM has not undertaken any subsurface investigations, measurement or testing of materials from the site. In preparing this report, SKM has relied upon, and presumed accurate, any information (or confirmation of the absence thereof) provided by our Client, and from other sources as described in the report. Except as otherwise stated in this report, SKM has not attempted to verify the accuracy or completeness of any such information.

This report should be read in full and no excerpts are to be taken as representative of the findings. It must not be copied in parts, have parts removed, redrawn or otherwise altered without the written consent of SKM.

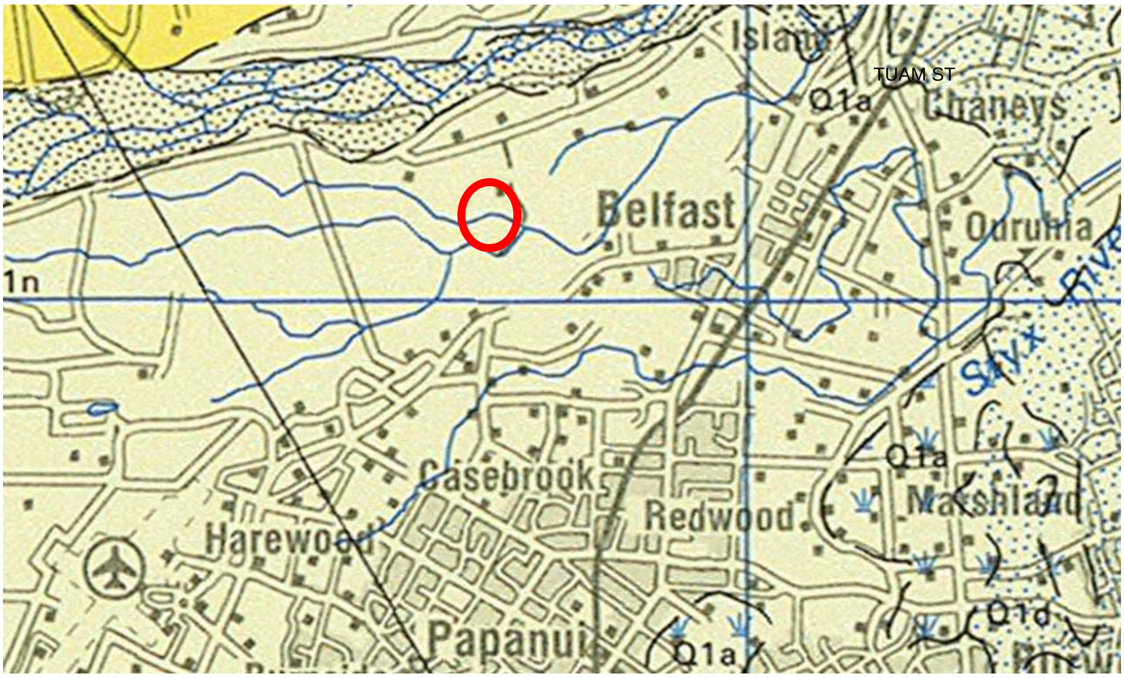
4. Site location

- **Figure 1 – Site location (courtesy of LINZ http://viewers.geospatial.govt.nz)**

These structures are located on 182 Johns Road.

5. Review of available information

5.1 Geological maps



- **Figure 2 – Regional geological map (Forsyth et al, 2008). Site marked in red.**

The local geological map of the Christchurch area does not extend to the location of the site.

The regional geological map shows the site as underlain by river alluvium, comprising gravel, sand and silt, beneath plains or low level terraces.

5.2 Liquefaction map

Following the 22 February 2011 event drive through reconnaissance was undertaken from 23 February until 1 March by M Cubrinovsko and M Taylor of Canterbury University. However, the reconnaissance did not extend to the location of the site.

5.3 Aerial photography

- **Figure 3 – Aerial photography from 24 Feb 2011 (http://viewers.geospatial.govt.nz/)**

- **Figure 4 Aerial photograph showing liquefied material ejected near road way (http://viewers.geospatial.govt.nz/)**

The aerial photographs appears to show some evidence of liquefaction occurring on site due to the 22 February earthquake, with localised sand boils and liquefied material ejected near the road way visible in figure 4.

5.4 CERA classification

A review of the LINZ website (http://viewers.geospatial.govt.nz/) shows that the site is:

- Zone: Green
- DBH Technical Category: N/A (Rural & Unmapped) – the residential area south of the site is classified as TC2

5.5 Historical land use

Reference to historical documents (eg Appendix A) shows that parts of the site were classified as swamp or marshland. The area classified appears to be larger than lakes currently present on site. This could indicate that adjacent land on site could be underlain by soft or liquefiable deposits. With a number of creeks running through the site, it is possibly that much of the area would be underlain by soft river deposits.

5.6 Existing ground investigation data

- **Figure 5 – Local boreholes from Project Orbit and SKM files (https://canterburyrecovery.projectorbit.com/)**

Where available logs from these investigation locations are attached to this report (Appendix B), and the results are summarised in Appendix C.

5.7 Council property files

Council documents and drawings relating to applied building permits, project memorandums, building consents and resource consent were available for this site. However, records including drawings and documents for only some of the structures were available.

In general the proposed drawings for the toilets blocks indicate a 100mm thick concrete floor slab on a layer of compacted hardfill and reinforced concrete footings around the perimeter was used as the foundation solution. Footings varying between 170mm to 300mm wide and 500mm to 740mm deep, depending on the ground profile near the structure, were indicated in the council drawings. A minimum embedment depth of 300mm increasing up to 450mm was noted with two D12 rods indicated as the reinforcement proposed for the footings.

Likewise, the drawings for the yacht building and toilets show a 100mm thick on grade concrete slab and 300mm deep reinforced concrete footings below the internal walls of the structure. The width of the footing is shown to vary between 170mm to 300mm.

The drawing for the proposed kiosk structure shows the structure was to be supported by 150mm diameter timber posts around the perimeter of the building. Approximately 300mm of the pile is shown to be above ground level. However, the embedment depth of the pile is not clear from available drawings. 100mm by 50mm bearers are used to distribute the loading from the structure to the identified timber posts.

The proposed drawings for the carport storage sheds show 200mm by 200mm concrete “piles” to be the foundation solution for the structure. However, no further information was available from the drawing or relevant council documents. There is some uncertainty on which building in the site inspection this record refers to. No map showing the location of the building on site was found.

The proposed drawing for the garage/ workshop indicates that a 100mm thick concrete slab on grade was proposed as the floor for the structure. A reinforced concrete footing that is 200mm wide was proposed beneath the walls of the structure. A minimum embedment depth of 300mm and height of 200mm above ground level is specified in the drawings for the footing. The recorded foundation information does not appear to match the garage/ workshop building inspected. No detailed map showing the location of the building was found in the available council records. It is expected that the exact location of the building would need to be verified to use this information.

The Ranger's office (dwelling 1) structure, labelled as the "relocated office" in the council records is indicated to be supported on 150mm diameter piles spaced at 1.4m centres over the footprint of the structure. The piles are indicated to be 525mm long with a minimum of 225mm of its length being embedded. Concrete corner foundations are also indicated for the office building. No other details about the foundation solution for the building were found during the review of available council records.

Drawing showing the extension to the dwelling 1 structure labelled as extension to the "information centre" indicates that short timber piles approximately 150mm in diameter below the bearer timber beam, embedded in 300mm by 350mm concrete footings was used as the foundation solution. The piles are shown to be approximately 900mm long. A minimum cover of 150mm above the concrete block to ground level and 300mm from ground level to the bearer beams is identified. The 125mm by 75mm bearers are shown to be tied into the foundations of the existing information centre structure.

In addition, some of the council documents indicate the presence of a septic tank near the toilet block structure. It is not clear where the respective toilet block is located. It is possible that additional septic tanks are present near toilet blocks spread throughout the foot print of the site.

No other ground investigation data or record of any excavation was found during the review of available council records.

5.8 Site walkover

A site walkover was conducted by a SKM engineer in the week commencing 9 April 2012. A site plan showing the located of the inspected building is provided in Appendix D.

PRK_0348_BLDG_007 EQ2

The small timber frame building was noted to be constructed using fibre board clad, slab on grade foundation and sheet metal roof. Minor damage was noted with the roof iron lifting but this damage possibly could have occurred before the earthquake. The structure itself is located on level ground with no land damage noted during the external site inspection.

PRK_0348_BLDG_005 EQ2

The building was noted as being rangers' office. The structure was a timber frame building on timber pole piles, sheet metal clad and sheet metal roof. The building was noted to be on level land but driveway to the north slopes up towards the road. No apparent building or land damage was noted during the external site inspection.

PRK_0348_BLDG_012 EQ2

The structure was observed to comprise a concrete base and concrete perimeter footing. The building was timber frame construction with sheet metal clad and roof. The structure appears to in a state of disrepair; however this is not as a consequence of the recent earthquake. The structure was located on a water way but no evidence of liquefaction, lateral spreading or other form of land damage was observed during the external site inspection.

PRK_0348_BLDG_008 EQ2

The structure was a masonry block building with sheet metal roof and slab on grade foundation. The building is located on flat ground close to a waterway to the east. No evidence of any land or building damage was observed during the external site inspection.

PRK_0348_BLDG_011 EQ2

The building was observed to be a farm shed type construction comprising timber pole with timber frame and sheet metal clad roof. No access was available to the site on the day of the inspection. However, the site is adjacent to a waterway to the west and there was no evidence of any land damage in the surrounding vicinity.

PRK_0348_BLDG_006 EQ2

The dwelling was located within an enclosed area. Therefore it was difficult to ascertain the construction type for the structure. However, the structure was likely to be weatherboard clad with sheet metal roof. A confirmation of the type foundation was not able to be made. The building was located adjacent to a waterway to the east. However, no evidence of land damage was visible during the external site inspection.

PRK_0348_BLDG_010 EQ2

The building was a masonry block construction with sheet metal roof and slab on grade. It was located on relatively flat ground with no building or land damage noted during the site inspection.

PRK_0348_BLDG_004 EQ2

The building was a masonry block construction with timber A frame, sheet metal roof and slab on grade foundations. The structure is located close to water ways. The ground was observed to be undulating in the area. However, no evidence of any liquefaction was noted near the site. Therefore it is possible that the undulations may not have been caused by the earthquake. No damage to the building was noted during the external site inspection.

PRK_0348_BLDG_014 EQ2

The building was noted to be a timber frame construction with sheet metal clad / sheet metal roof. The foundation appears to be either a timber floor or no foundation/floor present for the building. During the external site inspection, there does not appear to be any building damage. The site is adjacent to a lake, with a wooden jetty that runs adjacent and perpendicular to the building. No significant damage to the perpendicular jetty was apparent. The jetty which is adjacent to the building however slopes toward the lake to the west of the building. It is not clear if this was a consequence of the earthquake. There was no clear evidence that any lateral spread or liquefaction occurred on site during the site walkover. However, some undulations of the ground were observed in the area.

PRK_0348_BLDG_017 EQ2

The structure was a masonry block building with sheet metal roof and slab on grade foundation. The slab has approximately 400 mm thickness exposed above ground level. The building is located on flat ground, with no evidence of any land or building damage observed during the external site walkover.

PRK_0348_BLDG_002 EQ2

The building is a masonry block construction with sheet metal roof and slab on grade foundation. The structure is located on level ground. There does not appear to be any significant building damage from the external site inspection, however, cracking of the paving slabs to the west of the building was observed. The cracking was noted to be around the downpipe and across the pavement and looks to be relatively fresh (cracks range from 5-20mm). Settlement of the paving slab of up to 30mm was also noted.

PRK_0348_BLDG_009 EQ2

The structure was a timber pole information kiosk. No significant land damage was observed during the site walkover.

PRK_0348_BLDG_013 EQ2

The building was a timber frame construction with sheet metal walls and roof though the front of the building was mainly made up of 2 roller doors. Foundations appear to be railway sleepers. There was no building or land damage noted during the external site inspection.

PRK_0348_BLDG_016 EQ2

The structure was a small timber frame shed with plywood clad, with no apparent foundations other than a timber floor or possibly timber slats and sheet metal roof. No building or land damage was noted.

PRK_0348_BLDG_003 EQ2

The building was a masonry block construction with sheet metal roof and slab on grade foundation. The building was located on level ground but ground behind to the west slopes up an embankment (approximately 1.2m high). No land or building damage was noted during the external site walkover.

6. Conclusions and recommendations

6.1 Site geology

An interpretation of the most relevant local investigation suggests that the site is underlain by:

Depth range (mBGL)	Soil type
0 - 4	Fill / peat and soft clay
4 - 15	Soft clay
15+	Sandy gravels from the riccarton formation

The water table was inferred to be approximately 2m below ground level from nearby boreholes.

6.2 Seismic site subsoil class

The site has been assessed as NZS1170.5 Class D (deep or soft soil) from adjacent borehole logs.

As described in NZS1170, the preferred site classification method is from site periods based on four times the shear wave travel time through material from the surface to the underlying rock. The next preferred methods are from borelogs including measurement of geotechnical properties or by evaluation of site periods from Nakamura ratios or from recorded earthquake motions. Lacking this information, classification may be based on boreholes with descriptors but no geotechnical measurements. The least preferred method is from surface geology and estimates of the depth to underlying rock.

In this case the third preferred method has been used to make the assessment. As boreholes including measurement of geotechnical properties was not available for this desk study, site specific study in the future could result in a revision to the site subsoil class.

6.3 Building Performance

In general the existing foundations for the structures are adequate for their current purpose.

6.4 Ground performance and properties

Liquefaction risk appears to be low to moderate. Some evidence of liquefaction occurring on site was observed from the aerial photographs. However, no significant land damage or evidence of liquefaction was noted during the site walkover of the structures located on site. It should be noted, however, that the site walkover was conducted more than a year after the 22nd February earthquake and so it is possible that some liquefaction did occur but the evidence is no longer apparent. The clay layer inferred to lie between 4m to 15m is unlikely to be susceptible to liquefaction. Likewise, the lenses of sand that may be present in the sandy gravel layer below 15m may be susceptible to liquefaction but it is unlikely that any surface effects of this liquefaction would be observed. Therefore, any observed liquefied ejecta could be due to shallow silt or loose sand content.

As no geotechnical parameters were measured in the available ground investigation data, an estimation of the shallow ground properties has not been made in this desk study. Additional investigations are required, in order to assess the likely shallow ground properties.

6.5 Further investigations

In general the structures on site appear to be relatively light constructions supported on shallow footings. There is relatively good agreement on the geology of the soil below a depth of 5m from the available ground investigation data. However, as no geotechnical parameters are available, in order to perform a quantitative DEE, additional investigations are required. Additional investigations recommended are:

- Two CPTs near larger buildings such as the ranger's office and dwelling 2 are recommended. For small structures such as the kiosk and office building, two hand augers to infer the composition of shallow soils would be adequate.

If investigation is required for more than one asset it is advised to carry these out at the same time as scope may be able to be reduced by carrying out a site wide investigation.

7. References

Brown LJ, Weeber JH, 1992. Geology of the Christchurch urban area. Scale 1:25,000. Institute of Geological & Nuclear Sciences geological map 1.

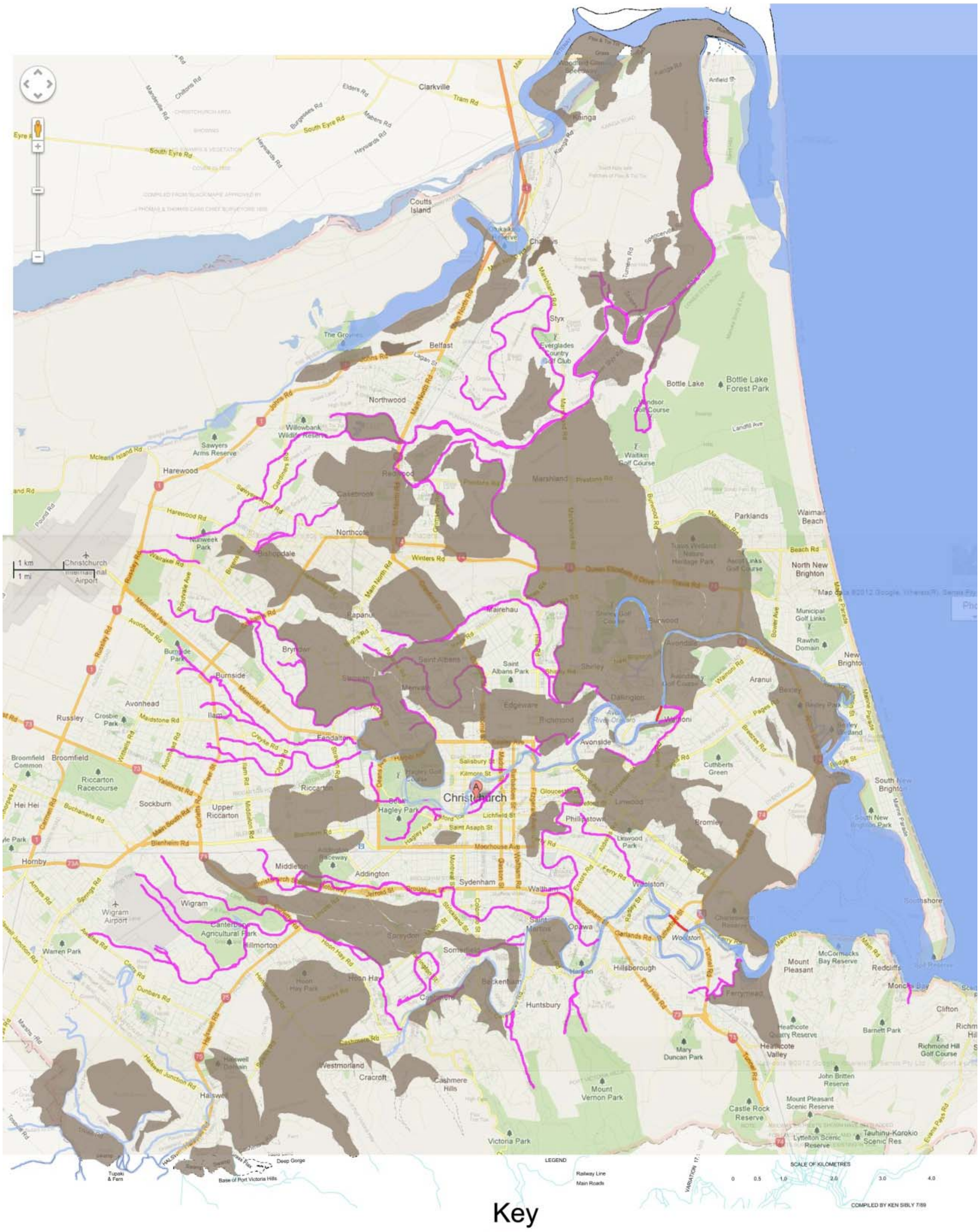
Cubrinovski & Taylor, 2011. Liquefaction map summarising preliminary assessment of liquefaction in urban areas following the 2010 Darfield Earthquake.

Forsyth PJ, Barrell DJA, Jongens R, 2008. Geology of the Christchurch area. Institute of Geological & Nuclear Sciences geological map 16.

Land Information New Zealand (LINZ) geospatial viewer (http://viewers.geospatial.govt.nz/)

EQC Project Orbit geotechnical viewer (https://canterburyrecovery.projectorbit.com/)

Appendix A – Christchurch 1856 land use



The swamps and previous creeks/rivers from 1856 have been overlayed onto a map of Christchurch in 2012

Key

- Previous creeks/rivers
- Existing creeks/rivers
- New creeks/rivers
- Swamp/Marshland

Appendix B – Existing ground investigation logs

Borelog for well M35/5250

Gridref: M35:7810-5045 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 11.2 +MSD
Driller : A M Bisley & Co
Drill Method : Cable Tool
Drill Depth : -24.9m Drill Date : 25/06/1985

Environment Canterbury
Your regional council

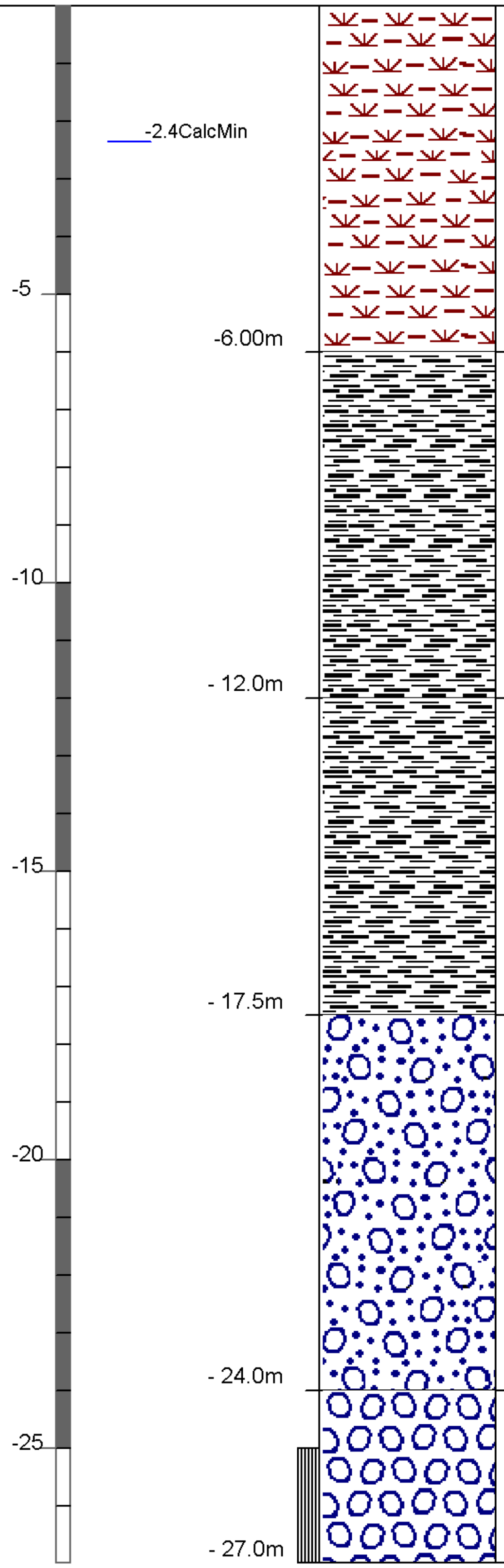
Water Level: -2.2CalcMin

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

Depth(m)	Full Drillers Description	Formation Code
-4.00m	Fill	fi
-5.00m	Grey gravel & sand	sp
- 15.8m	Grey clay	sp
- 24.9m	Grey stained gravel & sand.Water-bearing	ri

Borelog for well M35/7885

Gridref: M35:7844-5053 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 13 +MSD
Driller   : East Coast Drilling
Drill Method : Rotary Rig
Drill Depth   : -27m   Drill Date : 6/01/1998



Depth(m)	Full Drillers Description	Formation Code
-6.00m	Topsoil, peat, soft Grey clay	sp
- 12.0m	Soft blue Grey clay	sp
- 17.5m	Soft blue Grey clay	sp
- 24.0m	Sandy firm gravels	ri
- 27.0m	Free large round gravels	ri

Borelog for well M35/10305

Gridref: M35:78627-50764 Accuracy : 2 (1=high, 5=low)
Ground Level Altitude : 12.74 +MSD
Driller : Texco Drilling Ltd
Drill Method : Unknown
Drill Depth : -5.8m Drill Date : 6/07/2004

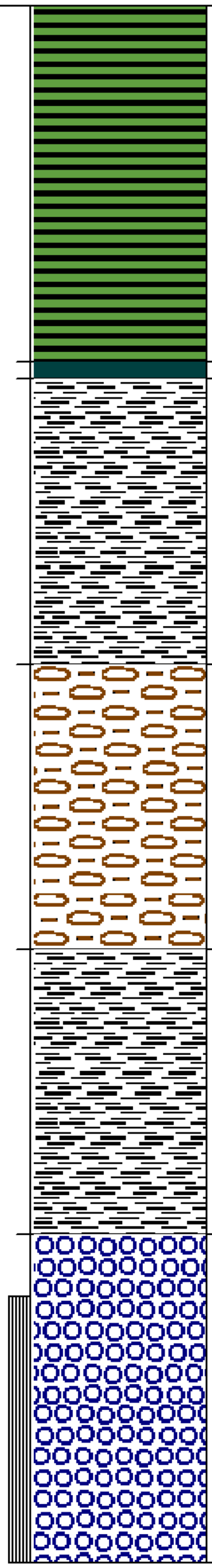
Environment Canterbury Regional Council

Water Level: -2.3CalcMin

Depth(m)	Full Drillers Description	Formation Code
-0.30m	Brown topsoil	
-0.70m	Yellowish brown homogeneous unweathered fine sand. Loosely packed moist.	
-1.90m	Yellowish brown homogeneous unweathered silt. Soft firm moist plastic.	
-4.70m	1.1 of blueish grey silt (as above) and peat in layers 0.4m thick. Gradual transition between them.	
-5.80m	Blueish grey homogeneous unweathered silt and trace of fine sand. Soft firm moist plastic.5.4m collapse of pit walls due to water ingress.	

Borelog for well M35/3475

Gridref: M35:785-505 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 13.1 +MSD
Driller : Smith, J R & I G
Drill Method : Cable Tool
Drill Depth : -17.5m Drill Date : 29/11/1983

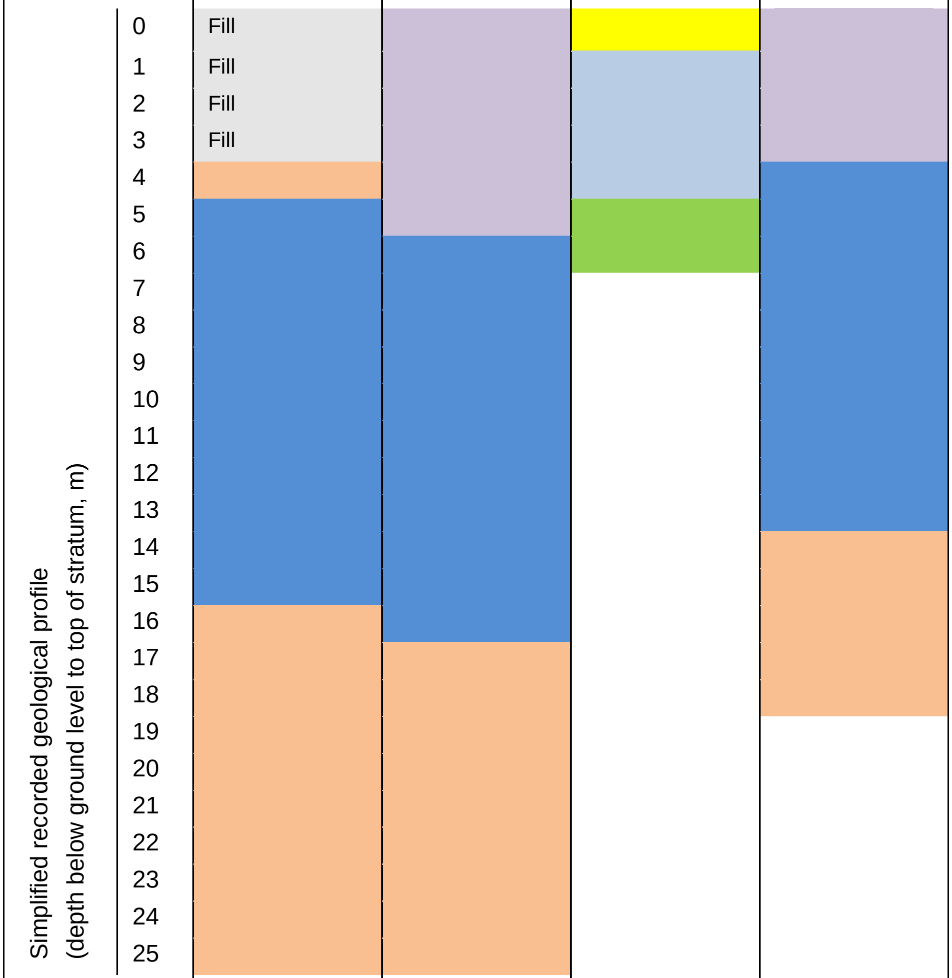


Scale(m)	Water Level	Depth(m)	Full Drillers Description	Formation Code
	-2.3CalcMin		Peat,small layer of clay,Blue gravel	
		-4.00m	Peat & wood	sp?
-5		-4.19m	Grey pug clay	sp?
		-7.40m	Grey pug clay.wood @ 10m	sp?
-10		- 10.6m	Grey pug clay	sp?
-15		- 13.8m	Free gravel at 14m	sp?
		- 17.5m		ri

Appendix C – Geotechnical Investigation Summary

- **Table 1 Summary of most relevant investigation data**

ID	1	2	3	4
Type *	BH	BH	BH	BH
Ref	M35-5250	M35-7885	M35-10305	M35-3475
Depth (m)	24.9	27	5.8	17.5
Distance from site (m)	*30*	*150*	*200*	*160*
Ground water level (mBGL)	2.2	2.4	2.3	2.3



Greater depths

*BH: Borehole, HA: Hand Auger, WW: Water Well, CPT: Cone Penetration Test

Sensitive or organic clay/silt; Clay to silty clay; Clayey silt to silt; Silty sand to silt; Clayey sand; Sand; Gravelly sand or gravel

VL = very loose, L = loose, MD = medium dense, D = dense, VD = very dense
VS = very soft, So = soft, F = firm, St = stiff, VS = very stiff, H = hard

Note the shortest distance from the site boundary to the investigation location is provided in the table due to the very large footprint of the site

Appendix D – Site Plan outlining the location of the building as named in the external site walkover

Could not find – Toilets Kimihia? Or Toilets – CLOSED (behind toilet block?)