CHRISTCHURCH CITY COUNCIL
PRK_0348_BLDG_006 EQ2
The Groynes – Dwelling No.2
182 Johns Road

QUANTITATIVE ASSESSMENT REPORT

FINAL

- Rev B
- 26 February 2014

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1. Executive Summary

1.1. Background

A Quantitative Assessment was carried out on building PRK_0348_BLDG_006 EQ2 (Dwelling no.2) located at The Groynes on 182 Johns Road. This building is a single storey timber framed structure that is used as dwelling by one of the Park Ranger's residence. An aerial photograph illustrating the building location is shown below in Figure 1. Detailed descriptions outlining the building age and construction type are given in Section 5 of this report.

- **Figure 1 Aerial Photograph of Building PRK_0348_BLDG_006 EQ2 Located at The Groynes**

This Quantitative report for the building structure is based on the Engineering Advisory Group's "Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings" (draft) July 2011, visual inspections on the 3rd of May, 2012, and the 8th and 14th of January, 2013 and calculations.

1.2. Key Damage Observed

Key damage observed includes:-

- Differential movement between the house and garage.
- Damage to external timber decks.

- Hairline cracking to external cladding vertical joints.
- Hairline cracking to internal wall linings.

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" (draft) July 2011, we have assessed the capacity of the building as a percentage new building standard 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 investigation 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 on our knowledge of the site and the absence of liquefaction ejecta on the site.
- Assessment of the strength of the existing structures taking account of the current condition.

Any building that is found to have a seismic capacity less than 33% of the new building standard (NBS) is required to be strengthened up to a capacity of at least 67%NBS in order to comply with Christchurch City Council (CCC) policy - 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 100%NBS and since there is no apparent significant damage to structural elements, its post earthquake capacity is also in the order of 100%NBS. No critical structural weaknesses were found in this building.

The building has been assessed to have a seismic capacity in the order of 100% NBS and is therefore not potentially earthquake prone.

1.5. Recommendations

Based on the findings of this assessment indicating the building is in the order of 100%NBS, the building is not likely to be earthquake prone. It is recommended that:

a) We consider that barriers around the building are not necessary.

b) There is no damage to the building that would cause it to be unsafe to occupy.

2. Introduction

Sinclair Knight Merz were engaged by Christchurch City Council to carry out a Quantitative Assessment of the seismic performance of PRK_0348_BLDG_006 EQ2 (Dwelling no.2) located at The Groynes, on182 Johns Road.

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 or 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].

[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 brief 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 Engineering 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 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.

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 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.

- **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

The Dwelling No.2 at the Groynes is a single storey timber building that is used as one of the Park Ranger's residence. The roof structure consists of timber framing and light weight corrugated steel cladding that is supported on the timber framed walls. These timber framed walls are clad with weatherboards. The floor is also constructed from timber framing and is supported on concrete piles and concrete perimeter ring beams.

The garage and adjacent store room appear to have been added on to the house structure after the original construction. The super-structure of the garage is similar to the construction of the house and the sub-structure differs in that it is founded on a strip footings and a concrete floor slab on grade instead of piles. Partial architectural drawings were available, these appear to be for the garage and store room extension and are dated 1984. Due to this a design period of 1965-1976 has been assumed for the original house.

5.2. Gravity Load Resisting system

Our evaluation was based on our site inspections conducted on the 8th and 14th January 2013 and the partial architectural drawings issued by the Waimairi District Council.

The weight of the roof is supported by the timber framed walls that in turn, transfer the loads to the timber floor and the foundations. The foundations of the dwelling consist of concrete piles and concrete perimeter ring beams.

The structure of the garage and its gravity load resisting system is similar than that of the dwelling, except that the garage floor consists of a concrete slab on grade and the superstructure is supported on concrete strip footings.

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.

The seismic loads acting in the across and along direction are resisted by timber walls on concrete perimeter ring beam foundations. Based on the inspection of the subfloor, it is considered that the piles do not have standard connection detail for acting as cantilever piles and that the piles are supporting mainly vertical loads.

The lateral loads from the garage are resisted also by the timber walls which loads are subsequently transferred to the concrete strip footings and concrete floor slab.

5.4. Geotechnical Conditions

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 4.

5.5. Building Damage

SKM undertook inspections on the 3rd of May 2012, and the 8th and 14th of January, 2013. The following was observed during the time of inspection:

5.5.1. External Damage

General

1) No visual evidence of settlement was noted at this site. Therefore a level survey is not required at this stage of assessment.
2) Vertical joints in the weatherboard cladding have opened up (PHOTO 3 & 4).
3) Wall joint between the garage and the house in the north-west corner has opened up (PHOTO 5 & 7).
4) Hairline cracking along the wall joint in the corners along the north elevation (PHOTO 6 & 8).

5) Cracking and damage to concrete path near the north-east corner of the main house. This damage appears to be a result of the tree roots growing under the path and as a result has not be considered as earthquake damage (PHOTO 9 & 10).
6) Northern timber deck has pulled away from the main house structure (PHOTO 11 & 12).
7) Southern timber deck appears to have fallen off the centre row of piles. This has resulted in the deck caving in at the middle and becoming very springy (PHOTO 13 & 14).

5.5.2. Internal Damage

Lounge

1) Hairline cracking present along wall lining joints, located above the west wall opening, the west wall French doors and the east wall window (PHOTO 17, 18, 19, 20 & 21).
2) Hairline crack present along the ceiling lining joint above the west wall window (PHOTO 22).

Dining

1) Hairline cracking present along the ceiling lining joint near the south-east corner (PHOTO 24).
2) Hairline cracking along the wall and ceiling lining joint, present on the east and west walls (PHOTO 25, 26 & 27).
3) Cracking present along the wall lining joint in the north-east corner (PHOTO 27).
4) Hairline cracking present along the wall lining joint above the French doors (PHOTO 28).
5) Hairline cracking along wall lining joint on east wall near south corner (PHOTO 29 & 30).

Garage

1) Hairline cracking present along the wall lining joint above the door on the west wall (PHOTO 31 & 32).

Laundry

1) Hairline cracking present along wall lining joint above the window.
2) Hairline cracking present along the ceiling lining and scotia joint (PHOTO 34 & 35).
3) Hairline cracking along the wall lining and the timber trim present in the north-east and south-east corners (PHOTO 34).

Kitchen

1) Hairline cracking present along the ceiling lining joints (PHOTO 37).
2) Hairline cracking present along the wall lining joints on the south wall, east corner (PHOTO 38).

Store Room – (Located Between Laundry & Garage)

1) Separation gap present between the store floor and laundry floor. Kitchen and laundry door has raked and as a result is not closing properly (PHOTO 39).

Hallway

1) Hairline cracking present to ceiling lining (PHOTO 41 & 42).
2) Hairline cracking present along wall lining joints above the lounge, bedrooms, toilet, bathroom and front doors (PHOTO 43).
3) Hairline cracking present along the ceiling and wall lining joint on the west wall.

North-East Bedroom (Bedroom 3)

1) Hairline cracking present along wall lining joint above the window (PHOTO 45).
2) Hairline cracking present along ceiling lining joints (PHOTO 46).
3) Hairline cracking present along the wall and ceiling lining joint (PHOTO 47).

North-West Bedroom (Bedroom 2)

1) Hairline cracking present along the wall and ceiling lining joint (PHOTO 49).
2) Hairline cracking present along the wall lining joint above the window (PHOTO 50).

Master Bedroom (Bedroom 1)

1) Hairline cracking present along the ceiling lining joints (PHOTO 52).
2) Hairline cracking present along the wall lining joints above all windows (PHOTO 54).
3) Cracking present along the wall lining joint in the south-west corner (PHOTO 53 & 55).
4) Cracking present along the wall lining joint under the north-west window (PHOTO 56 & 57).
5) Cracking present along the wardrobe architrave and wall lining joint (PHOTO 58 & 59).
6) Hairline cracking present along the wall lining joint under south wall windows (PHOTO 60 & 61).

Toilet

1) Cracking present along wall lining joints (PHOTO 62 & 63).
2) Hairline cracking present along the ceiling lining and scotia joint (PHOTO 62 & 64).
3) Hairline cracking present along wall lining and timber trim joint in the corners (PHOTO 64).
4) Cracking present along the skirting board and wall lining joint (PHOTO 65 & 66).

Bathroom

1) Hairline cracking present along ceiling lining joints (PHOTO 67 & 68).

6. Available Information and Assumptions

6.1. Available Information

Following our inspections on the 8th and 14th January 2013, SKM carried out a seismic review on the structure. This review was undertaken using the available information which was as follows:

- SKM site measurements and inspection findings of the building.
- Partial architectural drawings were available for this building.

6.2. Survey

There was no visible settlement of the structure, nor were there any significant ground movement issues around the building. The site has been assesses as 'Rural and Unmapped' on the CERA 'Land Zone Technical Categories Map' for residential properties. However the worst areas near this site are classed as TC2. Due to these factors we do not recommend that any survey be undertaken at this stage of the assessment.

6.3. Assumptions

The 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.
- The soil on site is class [D] as described in AS/NZS1170.5:2004, Clause 3.1.3, Soft Soil. The soil of this site was classified like this in the Geotechnical Desktop Study attached in Appendix 4.
- Standard design assumptions for typical office and factory 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.
  - 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

- The following ductility criteria used in the building:
- **Table 2: Assumed Building Ductility**

Building	Ductility of Building in Current State	Ductility of Building in Strengthened State
Dwelling No.2	2	2

The ductility noted above has been used considering it is a value inherent to timber buildings with sheets linings at the walls that provide a bracing capacity to the structure.

- The subfloor assessment is based in the assumption that the piles don't have standard connections to work as cantilever piles and hence not able to resist lateral loads. Then, the lateral loads are resisted by the perimeter foundation beam assuming that a suitable connection between the perimeter beam and the walls exists.

The detailed engineering analysis is a post construction evaluation. Since it is not a full design and construction monitoring, 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.

6.4. 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
    a. Determine the building's status following any rapid assessment that have been done

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

b. 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

c. Review the foundations and any geotechnical information available. This will include determining the zoning of the land and the likely soil behaviour, a site investigation may be required

d. Investigate possible Critical Structural Weaknesses (CSW) or collapse hazards

e. Assess the original and post earthquake strength of the building (this assessment is subsequently superseded by the quantitative assessment)

2) Quantitative procedure

a. Carry out a geotechnical investigation if required by the qualitative assessment

b. 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 33 %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]. This timeframe is likely to be adjusted by CERA and Table 6 below contains the likely new recommendations.

[4] NZSEE 2006, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes*, p 2-2

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

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.

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

- AS/NZS 1170 parts 0, 1 and 5 Structural Design Actions
- NZS 3101:2006 Concrete Structures Standard
- NZS 3404:1997 Steel Structures Standard
- NZS 2606:1993 Timber Structures Standard
- NZS 4230:1990 Design of Reinforced Concrete Masonry Structures

7. Results and Discussions

7.1. Critical Structural Weaknesses

No critical structural weaknesses for the building were observed during our visual inspection.

7.2. Analysis Results

The equivalent static force method was used to analyse the seismic capacity of the building. The results of the analysis are reported in the following table and the building appears to be over 100%NBS. The results below are calculated for the building in its damaged state. It should be noted that 100%NBS does not imply that the building is fully complied with the current code requirements, as there could be concealed details which were not picked up during the inspections. The building results have been broken down into their seismic resisting elements. The results obtained are based in the assumptions described in Section 6.3 of this report.

(%NBS = the reliable strength / new building standards)

- **Table 4: DEE Results**

Building	Seismic Resisting Element	Action	Seismic Rating %NBS
Dwelling No.2	Gib lined timber framed walls (across direction)	Shear	100%
	Gib lined timber framed walls (along direction)	Shear	100%
	Floor diaphragm (across and along direction)	Shear	100%
	Subfloor concrete perimeter ring beam (along and across direction)	Shear	100%

8. Conclusion

SKM carried out a quantitative assessment on the Dwelling No.2 located at the Groynes. This assessment concluded that the building is classified as Low Risk Building.

Table 5: Quantitative assessment summary

Description	Grade	Risk	%NBS	Structural performance
Dwelling No.2	A+	Low	>100%	Acceptable.

The building is not likely to be earthquake prone. It is recommended that:

a) We consider that barriers around the building are not necessary.

b) There is no damage to the building that would cause it to be unsafe to occupy.

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. Additional Photos

Photo 1: North Elevation

Photo 2: South Elevation

Photo 3: Cracking along Weatherboard Joints

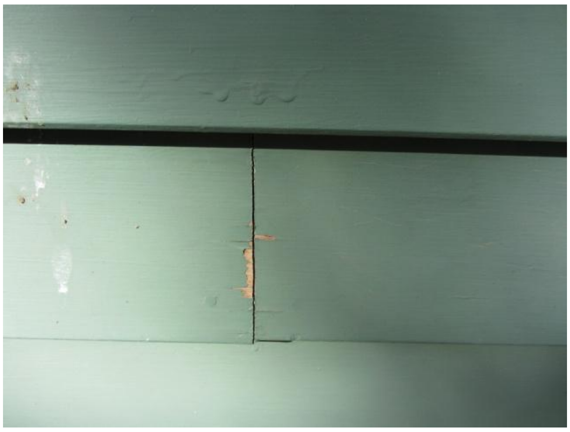


Photo 4: Close Up of Photo 3

	
Photo 5: Hairline Cracking along Cladding Joints	Photo 6: Hairline Cracking along Cladding Joints

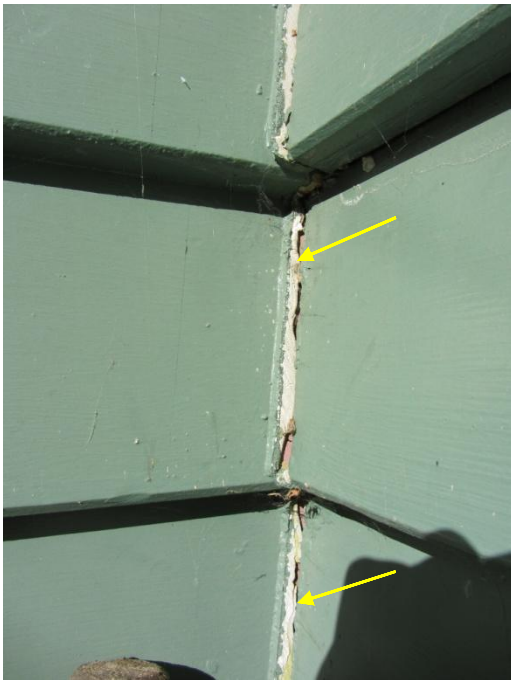



Photo 7: Close Up of Photo 5	Photo 8: Close Up of Photo 6

	
Photo 9: Damage to Pathway on North Side of Dwelling	Photo 10: Close Up of Photo 10
	
Photo 11: Northern Deck has Pulled Away from House Structure	Photo 12: Close Up of Photo 11

Photo 13: Damage to Southern Deck

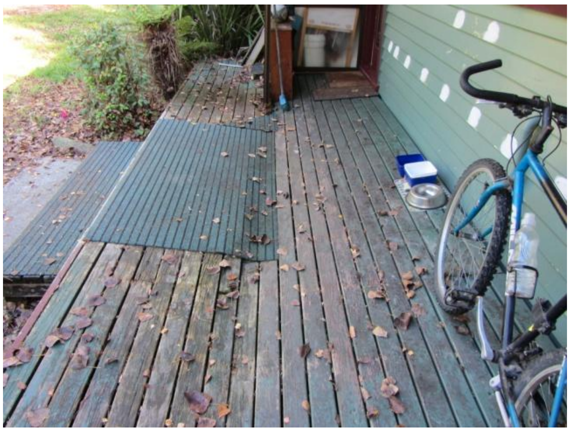


Photo 14: Close Up of Photo 13

Photo 15: Lounge Looking North-East

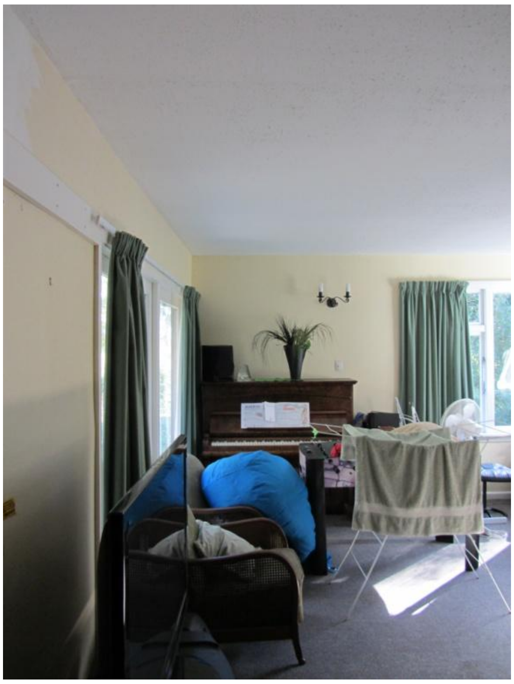


Photo 16: Lounge Looking North

	
Photo 17: Hairline Cracking to Wall Lining Joints above the French Door and Opening on the West Wall	Photo 18: Close Up of Photo 17

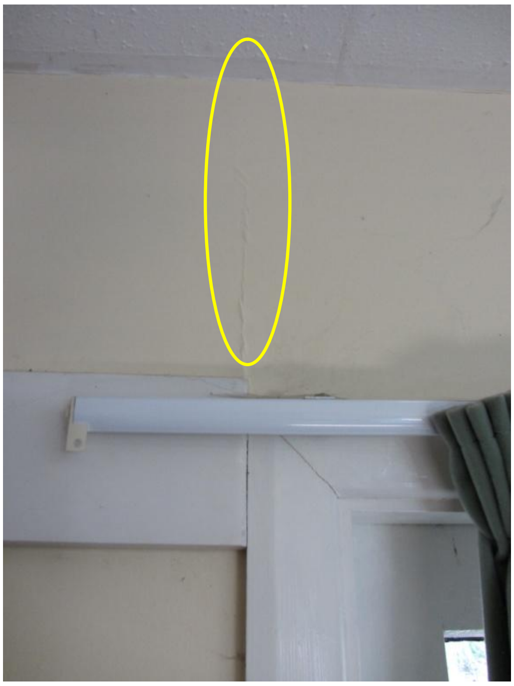
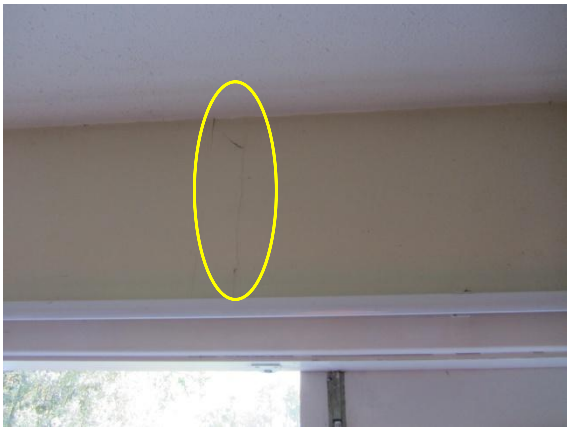


Photo 19: Close Up of Photo 17	Photo 20: Hairline Cracking above East Wall Window

	
Photo 21: Close Up of Photo 20	Photo 22: Hairline Cracking along Ceiling Lining Joint above East Window

	
Photo 23: Dining Room	Photo 24: Hairline Cracking along Ceiling Lining
	
Photo 25: Hairline Cracking along Wall and Cieling Lining Joint	Photo 26: Close Up of Photo 25

	
Photo 27: Cracking along Wall Lining in North-East Corner	Photo 28: Hairline Cracking along Wall Lining Joint above French Doors

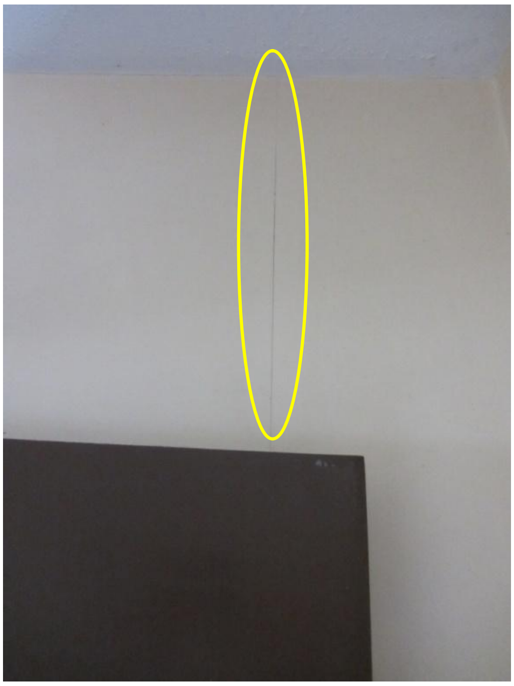
	
Photo 29: Hairline Cracking along Wall Lining Joint on East Wall near South Corner	Photo 30: Close Up of Photo 29

	
Photo 31: Cracking along Wall Lining Joints in Garage on East Wall	Photo 32: Close Up of Photo 31

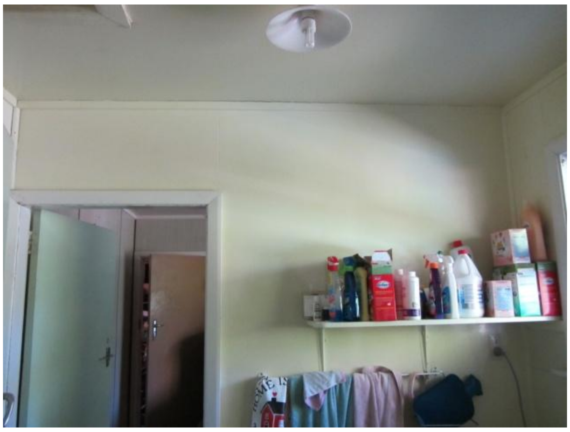


Photo 33: Laundry	Photo 34: Cracking along lining and scotia joints and lining and timber trim joints

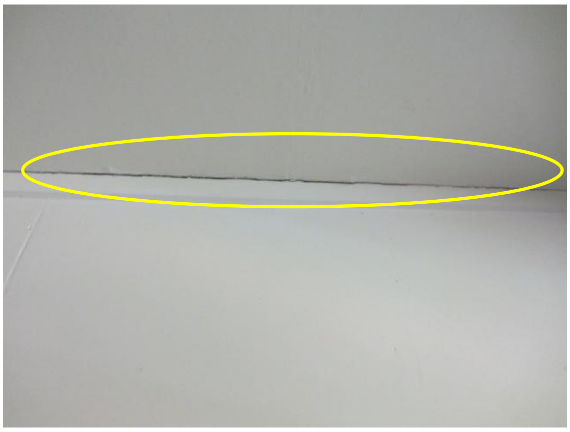
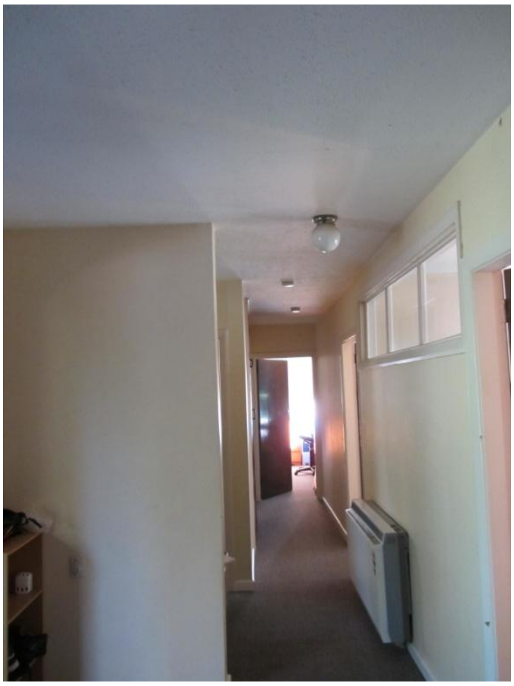



Photo 35: Close Up of Photo 34	Photo 36: Kitchen

	
Photo 37: Hairline Cracking along Ceiling Lining Joint	Photo 38: Hairline Cracking to Wall Lining Joints on the South Wall, East Corner

	
Photo 39: Separation Bewteen the Kitchen and Laundry Floor	Photo 40: Hallway

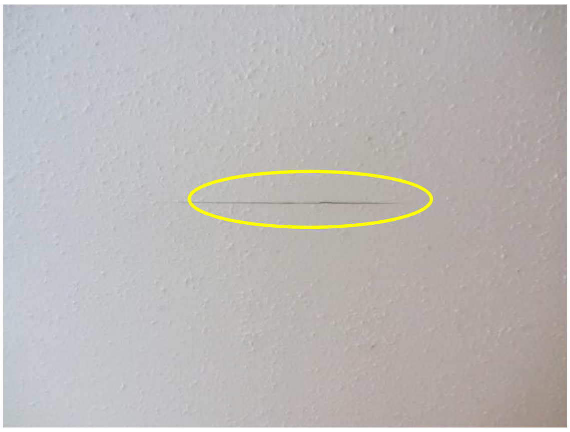
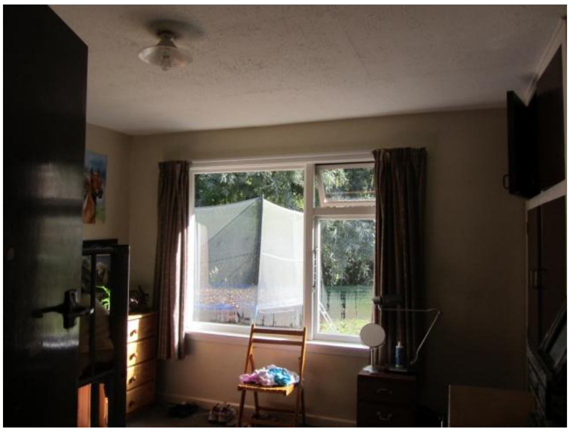
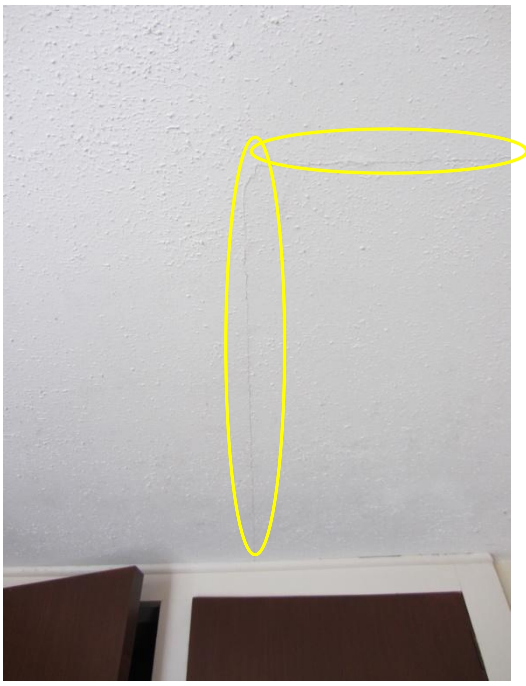



Photo 41: Cracking to Ceiling Lining	Photo 42: Cracking to Ceiling Lining

	
Photo 43: Hairline Cracking to Wall Lining Joint above Doorawy (Typical)	Photo 44: Bedroom 3

	
Photo 45: Hairline Cracking alon Wall Lining Joint above Window	Photo 46: Hairline Cracking along Ceiling Lining Joints

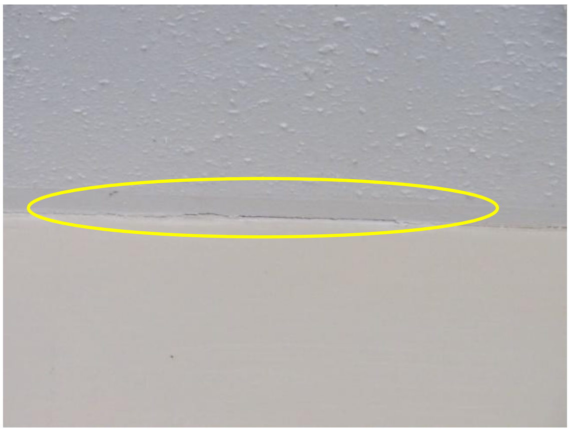



Photo 47: Hairline Cracking along Wall and Ceiling Lining Joint	Photo 48: Bedroom 2

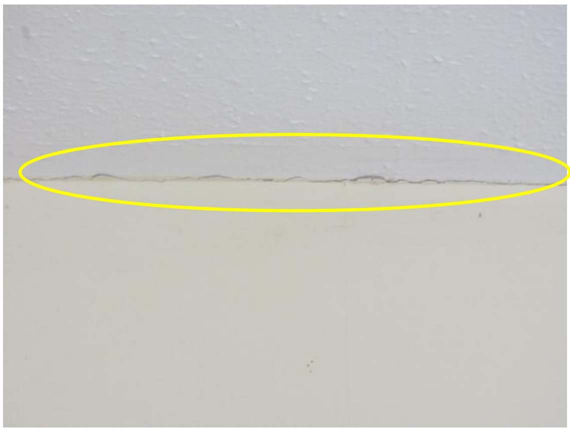


Photo 49: Hairline Cracking along Wall and Ceiling Lining Joint	Photo 50: Hairline Cracking along Wall Lining Lining Joint above Window
	
Photo 51: Master Bedroom	Photo 52: Hairline Cracking along Ceiling Lining Joints

	
Photo 53: Cracking along Wall Lining Joint in South-West Corner and above Window	Photo 54: Hairline Cracking along Wall Ling Joint above Window (Typical)

	
Photo 55: Close Up of Photo 53	Photo 56: Hairline Cracking along Wall Lining Joint under North-West Window

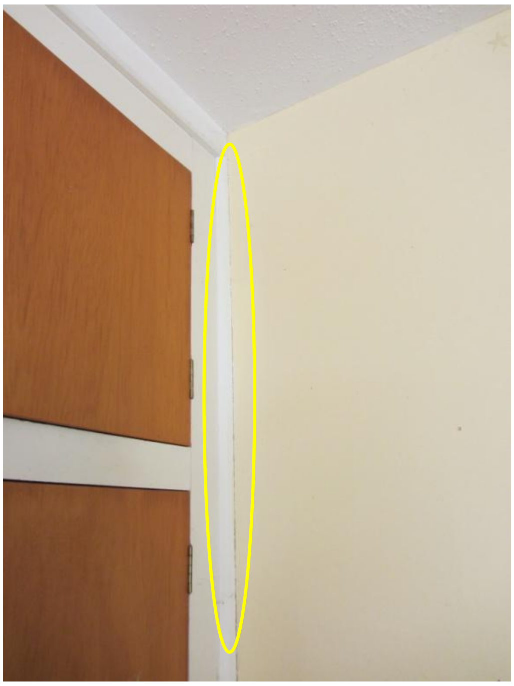
	
Photo 57: Close Up of Photo 56	Photo 58: Cracking along Wardrobe Architrave and Wall Lining Joint

	
Photo 59: Close Up of Photo 58	Photo 60: Hairline Cracking along Wall Lining Joint under South Wall Window

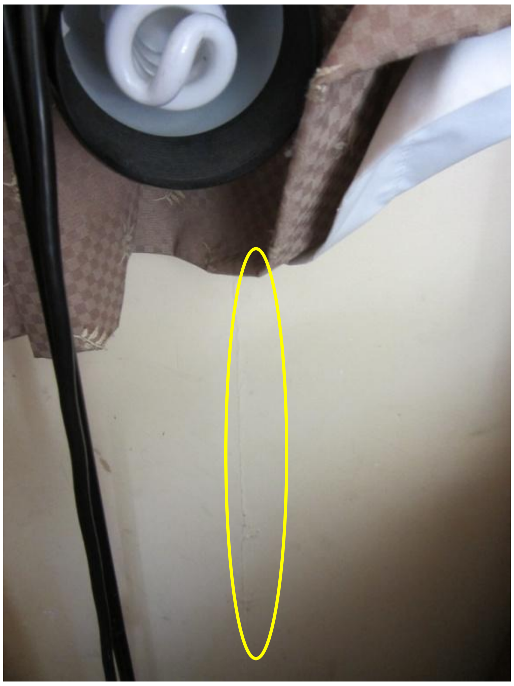
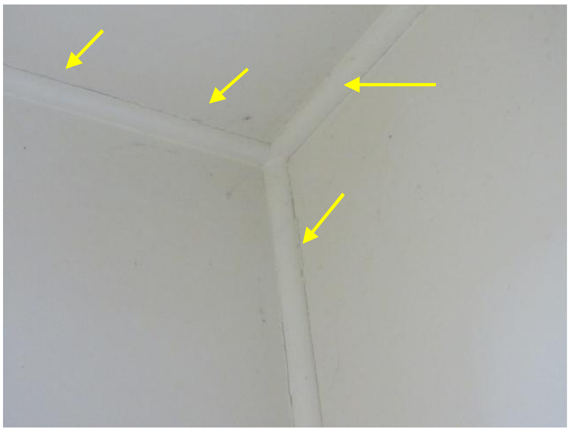
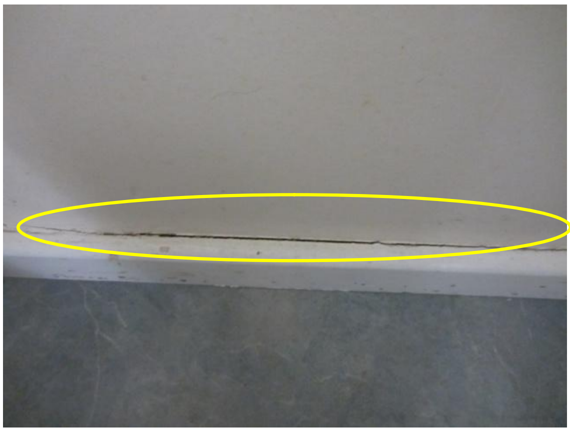


Photo 61: Close Up of Photo 60	Photo 62: Cracking along Wall Lining Joints in Toilet
	
Photo 63: Close Up of Photo 62	Photo 64: Hairline Cracking along Ceiling Lining and Scotia Joint and along Wall Lining and Timber Trim Joint

	
Photo 65: Cracking along Wall Lining and Scotia Joint	Photo 66: Close Up of Photo 65

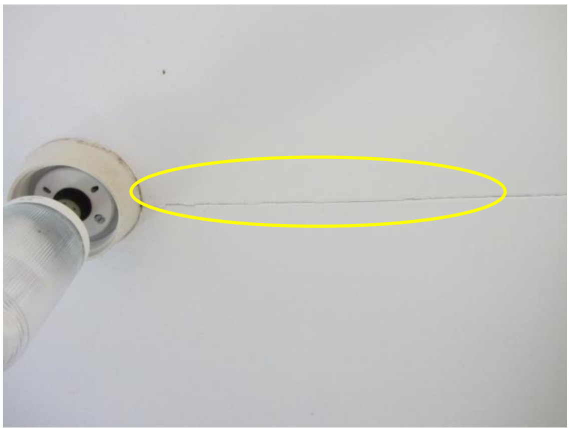



Photo 67: Hairline Cracking along Ceiling Lining Joints in Bathroom	Photo 68: Hairline Cracking along Ceiling Lining and Scotia Joint

	
Photo 69: View of subfloor concrete perimeter ring beam.	Photo 70: View of foundations: concrete perimeter ring beam and piles.
	
Photo 71: View of subfloor piles	Photo 72: Close up of photo 3.

	
Photo 73: View of roof trusses.	Photo 74: View of rafters and flat purlins at the roof.

11. Appendix 2 – CERA Standardised Report Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Dwelling No. 2	Reviewer:	N Calvert
	Unit / No: 182 / Street: Johns Road	CPEng No:	242062
Building Address:	The Groynes	Company:	SKM
Legal Description:		Company project number:	ZB01276.067
	Degrees / Min / Sec	Company phone number:	03 940 4900
GPS south:		Date of submission:	26-Feb
GPS east:		Inspection Date:	14/01/2012
		Revision:	B
Building Unique Identifier (CCC):	PRK 0348 BLDG 006	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):	

Building

No. of storeys above ground:	1 (single storey = 1)	Ground floor elevation (Absolute) (m):	0.40
Ground floor split?	no	Ground floor elevation above ground (m):	0.40
Storeys below ground	0		0
Foundation type:	other (describe)	if Foundation type is other, describe:	Concrete perimeter ring beam and concrete piles
Building height (m):	4.00	height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	180		
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):			
Use notes (if required):	residential		
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber truss	truss depth, purlin type and cladding	corrugated steel cladding on 25x140mm flat timber purlins on timber trusses, truss are triangular with max depth approxiamtely 1.2m deep
Floors:	timber	joist depth and spacing (mm)	50x200 timber joists at 600crs (assumed)
Beams:	timber	type	rafters 50x100 at 600mm
Columns:	load bearing walls	typical dimensions (mm x mm)	50x150 timber stud (assumed)
Walls:		Timber framed walls	

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	lightweight timber framed walls	note typical wall length (m)	2.5
Ductility assumed, μ:	1.25		
Period along:	0.10 (0.00)	estimate or calculation?	estimated
Total deflection (ULS) (mm):	10	estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):		estimate or calculation?	
Lateral system across:	lightweight timber framed walls	note typical wall length (m)	2.5
Ductility assumed, μ:	1.25		
Period across:	0.10 (0.00)	estimate or calculation?	estimated
Total deflection (ULS) (mm):	10	estimate or calculation?	estimated
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:			n/a
Wall cladding:	other light	describe	weatherboard
Roof Cladding:	Metal	describe	corrugated steel
Glazing:	timber frames		
Ceilings:	plaster, fixed		
Services(list):	lights, insulation etc		

Available documentation

Architectural	partial	original designer name/date	Waimairi District Council
Structural		original designer name/date	
Mechanical		original designer name/date	
Electrical		original designer name/date	
Geotech report		original designer name/date	

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 internal 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:	100%	%NBS from IEP below	If IEP not used, please detail assessment methodology: Quantitative Assesment
	Assessed %NBS after:	100%		
Across	Assessed %NBS before:	100%	%NBS from IEP below	
	Assessed %NBS after:	100%		

12. Appendix 5 – Geotechnical Desktop Study

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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.

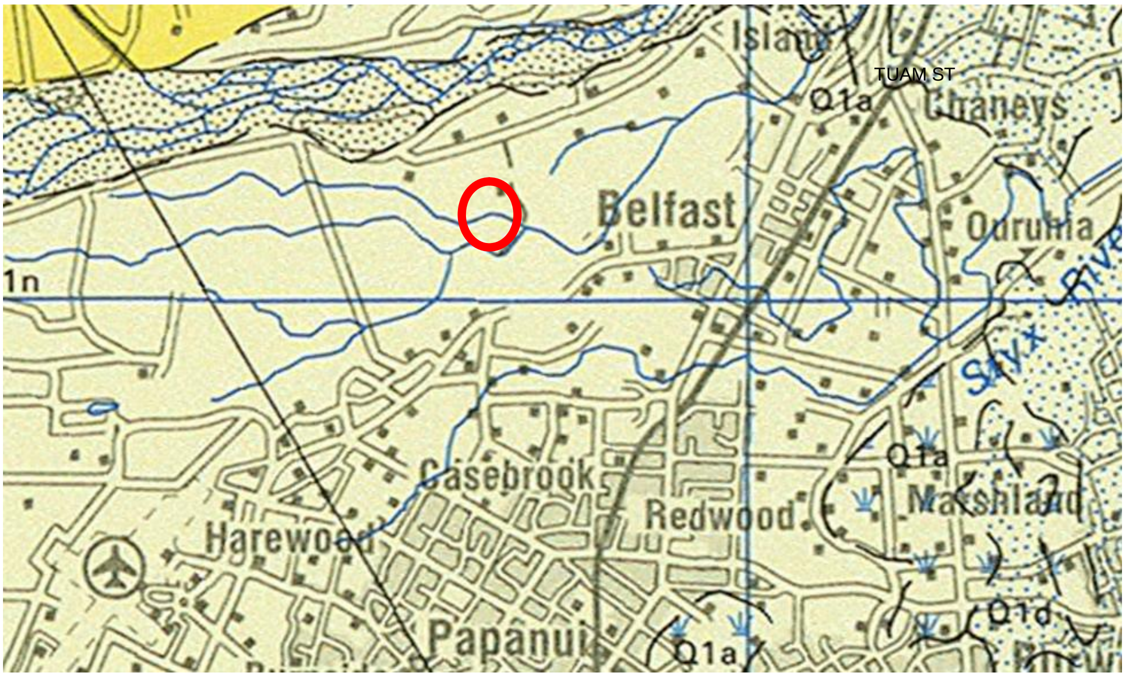
4. Site location

- **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_020 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.

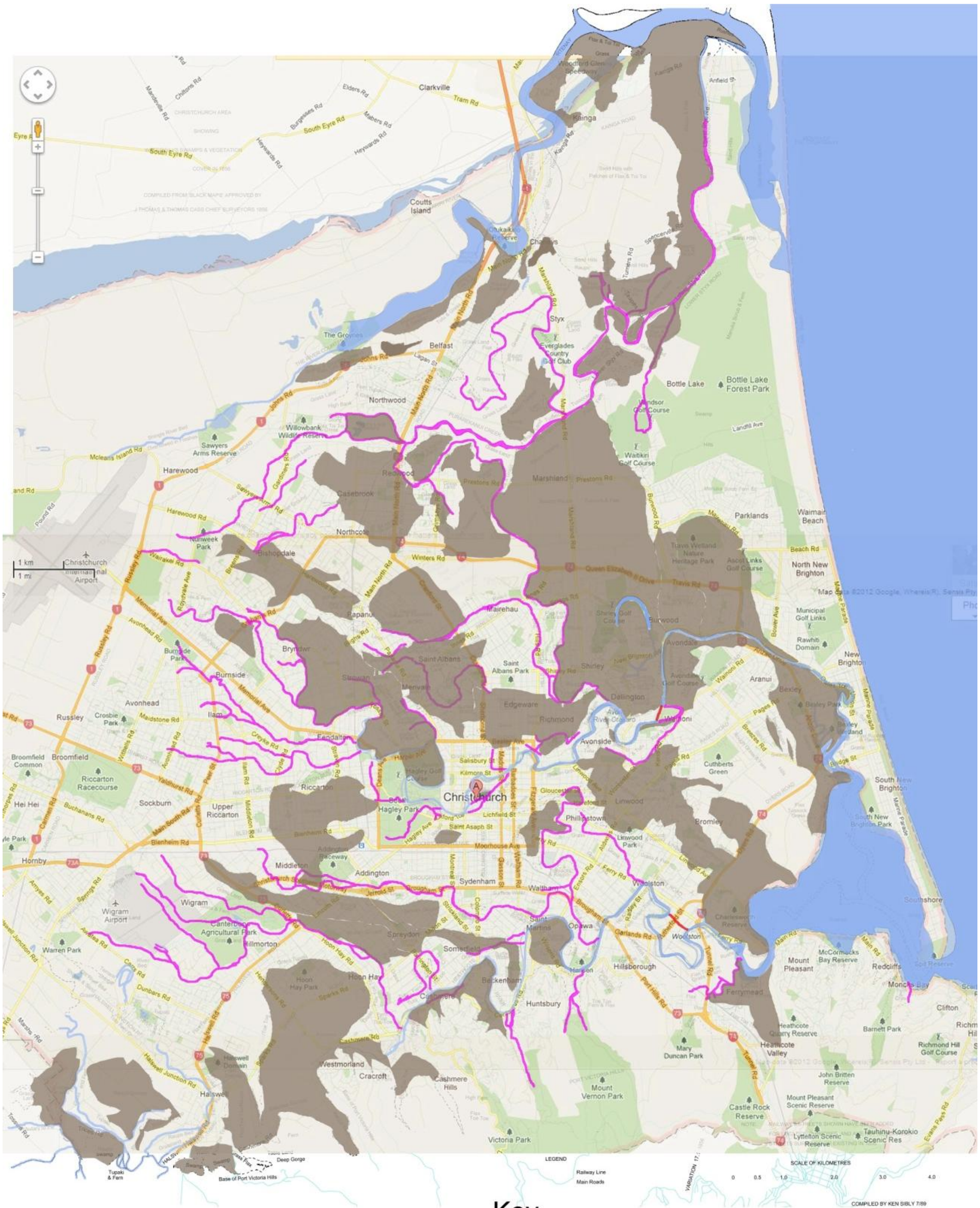
Cubrinovski & Taylor, 2011. Liquefaction map summarising preliminary assessment of liquefaction in urban areas following the 2010 Darfield Earthquake.

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

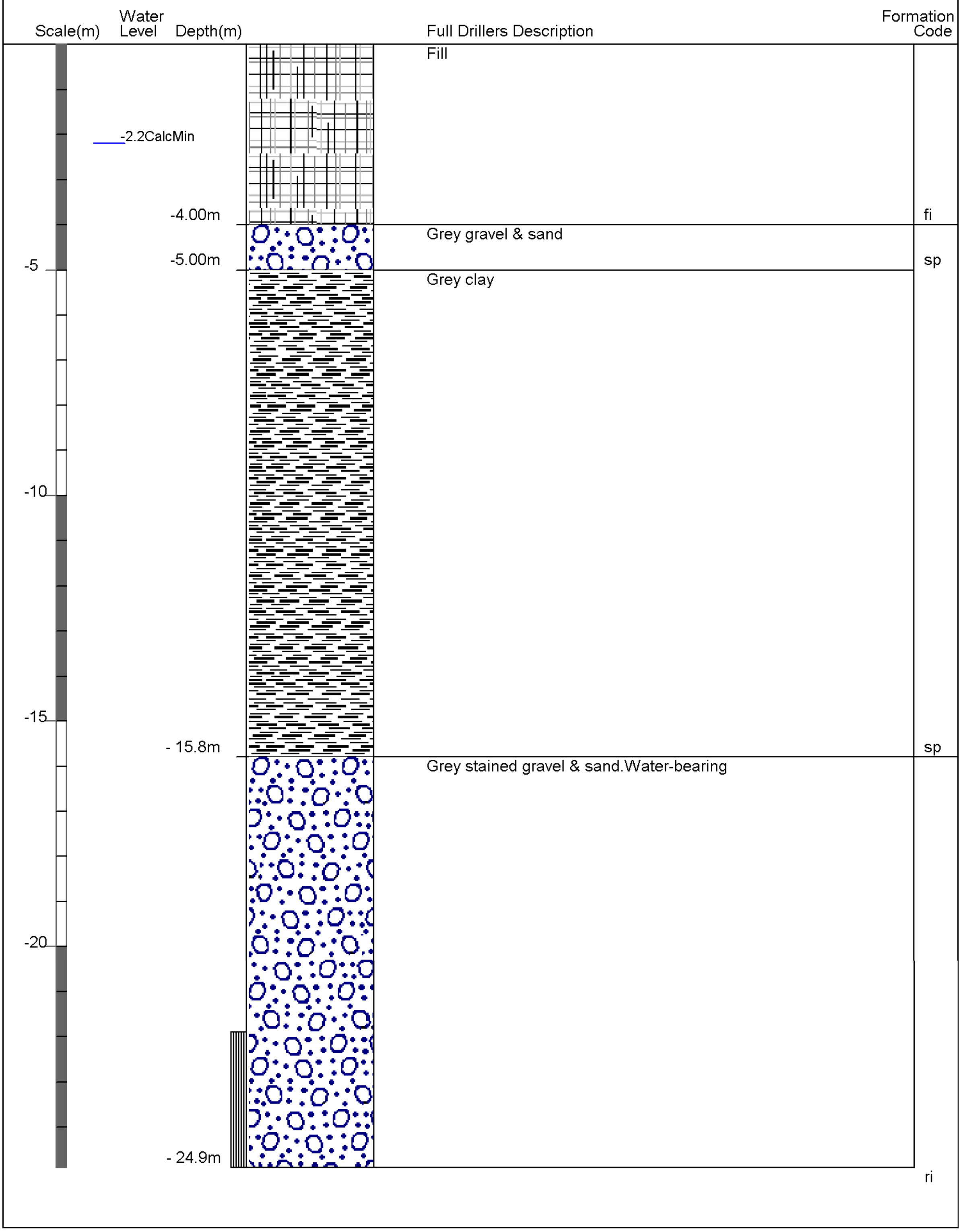
Key

- 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



SINCLAIR KNIGHT MERZ

SKM

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

Environment Canterbury
Your regional council

Water Level: -2.4CalcMin

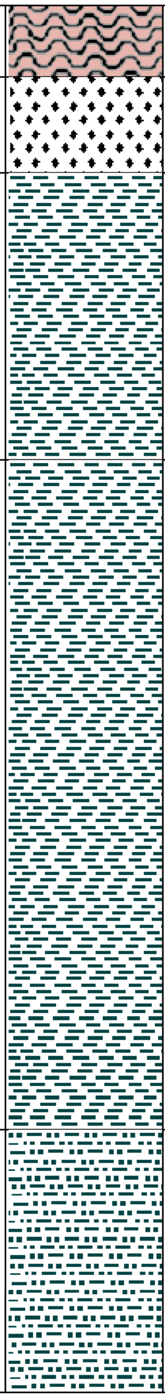
Scale(m): -5, -10, -15, -20, -25

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

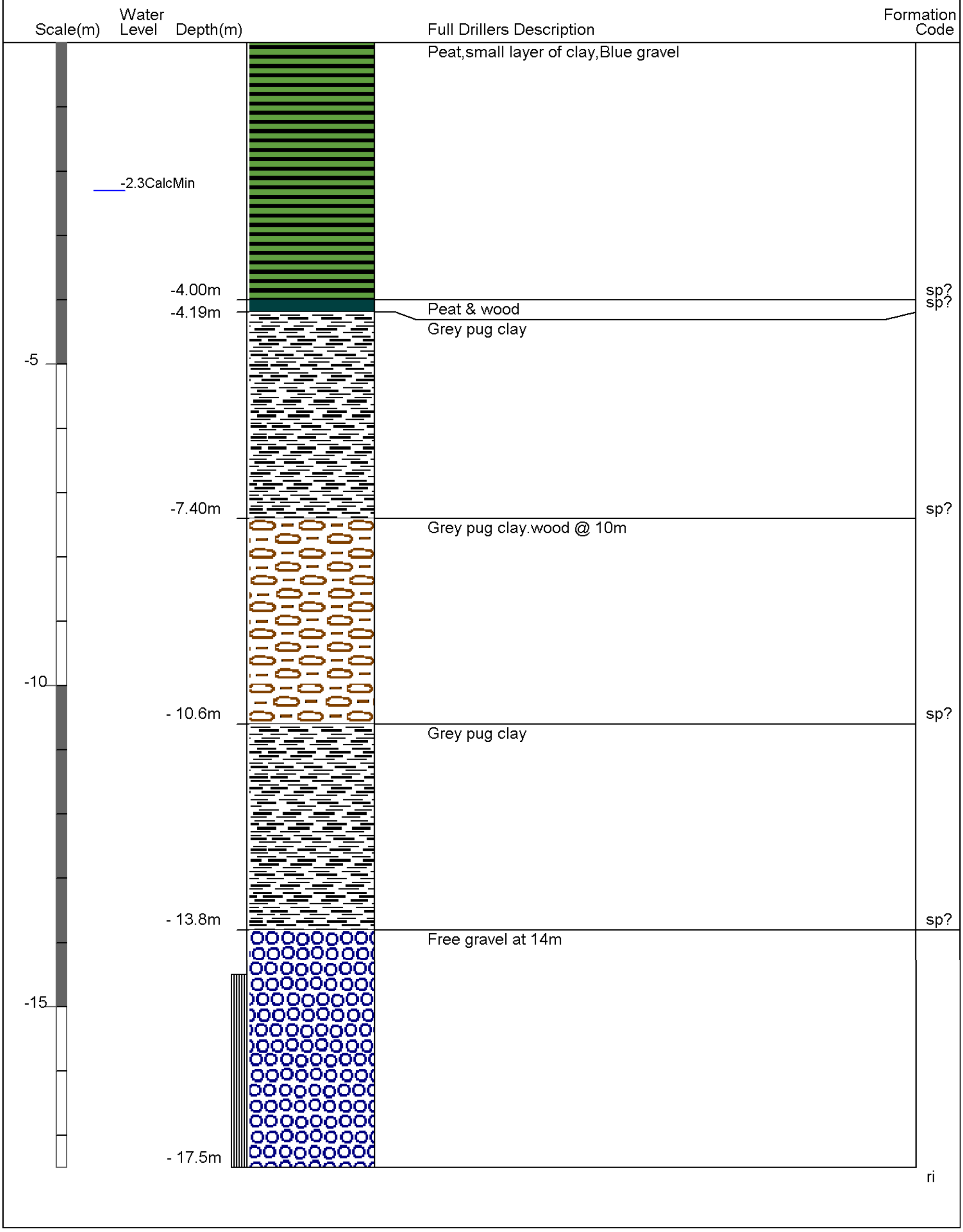
Water Level: -2.3CalcMin



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

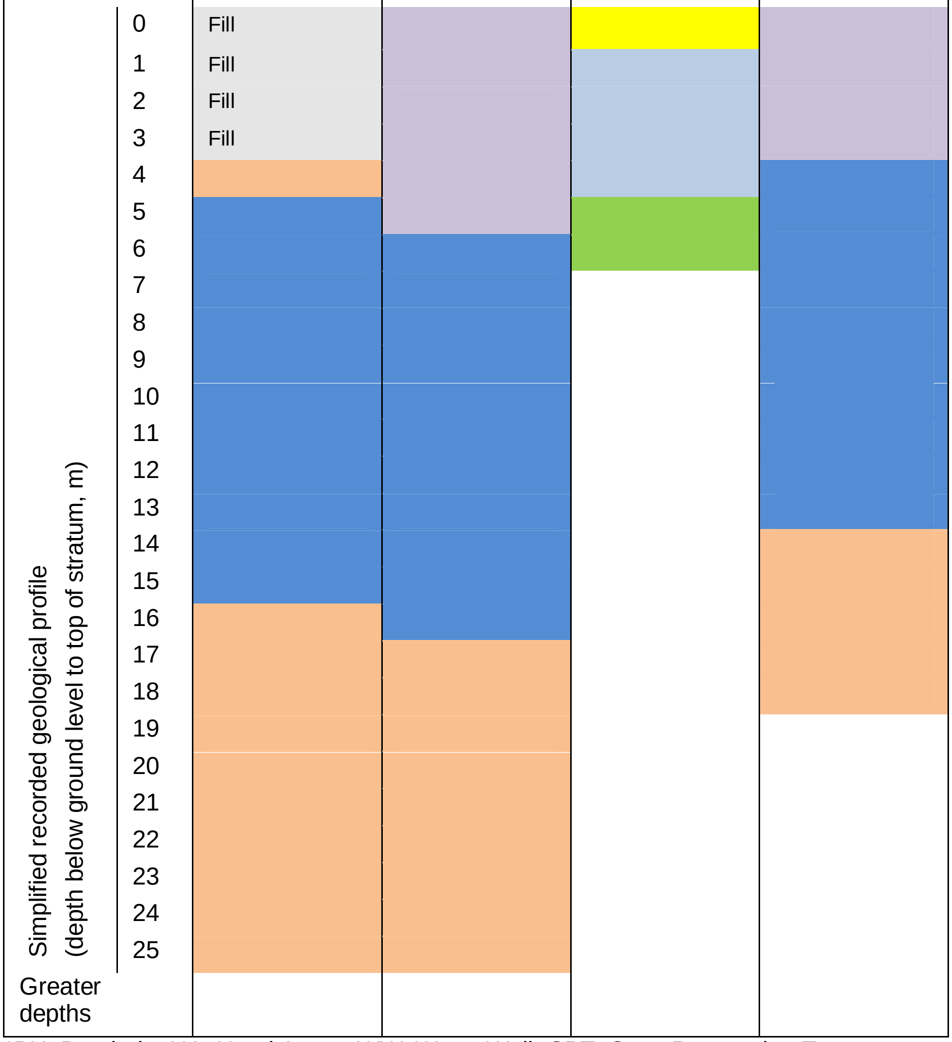
Gridref: M35:785-505 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 13.1 +MSD
Driller : Smith, J R & I G
Drill Method : Cable Tool
Drill Depth : -17.5m Drill Date : 29/11/1983



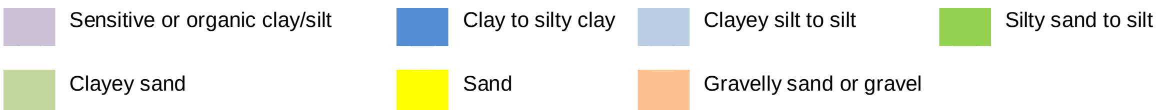
Appendix C – Geotechnical Investigation Summary

- **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



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



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?)