Christchurch City Council
PRK_2004_BLDG_018 EQ2
Rawhiti Domain - Silver Band Shed
35-37 Bowhill Road, New Brighton

QUANTITATIVE REPORT
FINAL

- Rev B
- 19 November 2013

Christchurch City Council
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Rawhiti Domain - Silver Band Shed
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QUANTITATIVE ASSESSMENT REPORT

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Contents

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1. Executive Summary

1.1. Background

A Quantitative Assessment was carried out on the Rawhiti Domain Silver Band Hall located at 35-37 Bowhill Road, New Brighton. The building located on this site comprises of a main band hall, a storage room to the east and a kitchen and amenities block to the south. From consent approval held by the Silver band hall the building was designed in 1976 and approved 1978. The building is constructed of partially filled concrete masonry blockwork walls with light timber framed roofing. An aerial photograph illustrating the area is shown below in Figure 1 Aerial Photograph of 35-37 Bowhill Road, New Brighton. Detailed descriptions outlining the buildings age and construction type are given in Section 5 Building Details of this report.

Figure 1 Aerial Photograph of 35-37 Bowhill Road, New Brighton

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 22nd of March 2013, and building mark ups and calculations.

1.2. Key Damage Observed

Key damage observed includes:-

1) The building has sustained minor damage to the south west where cracking, approximately 1.5 mm wide, has occurred through the masonry block units below the library window
2) Cracking in the masonry blockwork mortar joints on the south east of the building which has since been repointed.

A more detailed account of the damage can be found in section 5.

1.3. Critical Structural Weaknesses

The building has no critical structural weaknesses

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 building inspection as drawings were not available.
- A geotechnical desktop study was carried out for the surrounding area as part of the assessment of PRK_2004_BLDG_021 and is applicable to this site. Findings of that report have been used for the quantitative assessment of BLDG_018
- 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 target capacity of 67%NBS.

Based on the information available, and using the Quantitative Assessment Procedure, the buildings original capacity has been assessed to be in the order of 40%NBS and post-earthquake capacity in the order of 40%NBS. The buildings post-earthquake capacity excluding critical structural weaknesses is in the order of 40%NBS.

The building has been assessed to have a seismic capacity in the order of 40% 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 40%NBS, strengthening is not required in order to comply with Christchurch City Council (CCC) policy – Earthquake-prone dangerous & insanitary buildings policy 2010.

It is recommended that:

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

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

2. Introduction

Sinclair Knight Merz were engaged by Christchurch City Council to carry out a Quantitative Assessment of the seismic performance of Rawhiti Domain Silver Band Hall located at 35-37 Bowhill Road, New Brighton. An aerial of the Building is shown in Figure 1 Aerial Photograph of 35-37 Bowhill Road, New Brighton

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.

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 quantitative assessment was completed and this assessment has identified that the seismic capacity of the building is likely to be more than 33% of the new building standard (NBS).

At the time of this report, limited intrusive investigation of the amenities roof space was completed to determine structural member sizes as construction drawings were not available. The building description below is based on our visual 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 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 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 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: NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE Guidelines 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: %NBS compared to relative risk of failure 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 Rawhiti Domain Silver Band Hall building is located at 35-37 Bowhill Road, New Brighton. The building was designed in approximately 1976 to include the main hall and amenities block, the storage area to the east of the main hall has been added at a later date. The roof structure differs between the amenities area and main hall. In the amenities block the roof is light timber framed with built up trusses and profiled metal roofing on tongue and groove boarding. In the main hall the roof is light timber framed on timber trusses spanning 11 m. The perimeter walls are reinforced partially filled concrete masonry blockwork. Internal walls in the amenities block are light timber framed with hardboard and plasterboard linings. The floors throughout the original building are timber with timber joists and bearers founded on internal concrete piles and a concrete perimeter strip footing. The storage room is founded on a concrete slab on grade.

Drawings for the building were not available.

The building was designed in 1976 and has been assumed to be built at least 2 years later from consent approval documents held at the hall.

5.2. Gravity Load Resisting system

The gravity load resisting structure of the building is made up of light timber framed trusses which transfer roof loads to the load bearing concrete masonry walls, the walls transfer load to the ground through bearing of the concrete perimeter strip footings. Floor loads are transferred from the timber flooring into the joists and bearers and into the ground through concrete piles.

5.3. Seismic Load Resisting system

For the purposes of this report the longitudinal direction of the building is defined as being the north-south direction and the transverse direction is defined as being in the east-west direction.

Lateral loads on the building's roof in the longitudinal direction are carried along the purlins into the trusses and transferred out to the concrete masonry walls through shear of the trusses. In the transverse direction roof loads are transferred along the purlins in shear and into the masonry walls by axial load on the trusses. In both loading directions loads from the roof level are carried by the masonry walls in in-plane shear and out of plane bending. These loads are transferred into the ground via the perimeter strip foundations. Floor loads are transferred into the ground through shear of the internal concrete piles.

5.4. Geotechnical Conditions

A geotechnical desktop study was carried out for the adjacent Rawhiti Domain Community Gardens site PRK_2004_BLDG_021. The main conclusions from this report have been used due to the proximity of the buildings and are as follows:

- The site has been assessed as NZS1170.5 Class D (deep or soft soil) from an adjacent borehole log.
- It is expected that the allowable bearing capacity of a shallow pad footing on this site will be in the region of 200 kPa. However, this may be revised by a site specific investigation.
- Liquefaction risk is low at this site.

It is noted these parameters should not be used for consent or design purposes and ground conditions should be confirmed by a geotechnical investigation. It is possible that further investigations would result in an increase to the recommended ultimate bearing capacity. The recommended bearing capacity was estimated for loose to medium dense sand as indicated by the Scala penetrometer test. If any excavations are required on the site further investigation of the potential for contamination should be undertaken. The full geotechnical desktop study used can be found in Appendix 3.

5.5. Building Damage

SKM undertook inspections on the 22nd of March 2013. The following areas of damage were observed during the time of inspection:

1) The building has sustained minor damage to the south west where cracking, approximately 1.5 mm wide, has occurred through the masonry block units below the library window. See Photo 21: Cracking beneath south west window
2) Cracking in the masonry blockwork mortar joints on the south east of the building which has since been repointed. See Photo 1: Repointed mortar joints on the Southeast amenities

Photos of the above damage can be found in Appendix 1 – Photos.

6. Available Information and Assumptions

6.1. Available Information

Following our inspection on the 22 March 2013, SKM carried out a seismic review on the structures. This review was undertaken using the available information which was as follows:

- SKM site measurements and inspection findings for the building.
- Limited intrusive investigation of the amenities roof space to determine the roof structure

6.2. Survey

There was no visible settlement of the structure, nor were there any significant ground movement issues around the building. The building is adjacent to land which is zoned TC2 under the CERA Residential Technical Categories Map. The combination of these factors means that we do not recommend that any survey be undertaken at this point.

6.3. Assumptions

The assumptions made in undertaking the assessment include:

- The building was built according to good practice at the time.
- 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
Rawhiti Silver Band Hall	1.25[3]	1.25

The ductility of the building has been limited to 1.25, for the purpose of assessing the building, based on the concrete block masonry walls in both the transverse and longitudinal directions.

- The following material properties were used in the analyses:
- **Table 3: Material Properties**

Material	Nominal Strength	Structural Performance
Structural Steel	f_y = 250MPa	S_p = 0.9
Masonry (reinforced)	f_m = 12MPa	S_p = 1.0
Concrete	f_c' = 25MPa	S_p = 1.0
Timber - Assumed No.1 framing	f_b = 10MPa & f_c = 15MPa	S_p = 1.0

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[4].

[3] Ductility for the reinforced and partially filled concrete block walls was taken as 1.0
[4] http://resources.ccc.govt.nz/files/EarthquakeProneDangerousAndInsanitaryBuildingsPolicy2010.pdf

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
    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 4. 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[5]. 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[6].

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

Table 4: 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

The building has no critical structural weaknesses

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 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. While not legally required it is our recommendation that the building be strengthened so that the overall building capacity is greater than 67%NBS.

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

- **Table 5: DEE Results**

Seismic Resisting Element	Action	Seismic Rating %NBS
Transverse masonry walls out-of-plane	Bending	40%
Transverse masonry walls in-plane	Shear	49%
Longitudinal masonry walls in-plane	Shear	56%
Longitudinal masonry walls out-of-plane	Bending	85%
Internal concrete foundations piles	Shear	>100%

7.3. Recommendations

The quantitative assessment carried out on Rawhiti Silver Band hall indicates that the building has a seismic capacity greater than 34% but less than 67% of NBS and is therefore classed as being in the category of 'Moderate Risk Buildings'. Strengthening of the building is recommended to bring it up to a target of 67% of NBS.

If it is determined that the building should be repaired there are a number of issues which will need to be investigated and associated documents prepared in order to submit a building consent application. These issues will need to be considered during the initial phase of strengthening works. Listed below are the likely items the council may require to be explored:

- A fire report will be required and all necessary upgrades to egress routes, emergency lighting and specified systems will need to be undertaken.
- An emergency lighting design will be required to meet the provisions noted in the fire report.
- A disabled access summary will be required including provision for disabled facilities.
- The site amenities (toilets and the like) will need to be reviewed to ensure that there are sufficient facilities for the expected number of people on site.
- Landscaping will need to be considered although we do not anticipate that any modifications will be required since you will not be adjusting the footprint area of buildings on site and will likely only be required for the new build option.

8. Conclusion

SKM carried out a quantitative assessment on PRK_2004_BLDG_018 EQ2 located at 35-37 Bowhill Road, New Brighton. This assessment concluded that the building is not classified as being an Earthquake Prone Building.

Table 6: Quantitative assessment summary

Description	Grade	Risk	%NBS	Structural performance
Rawhiti Domain Silver band Hall	C	High	40	Acceptable legally. Improvement recommended.

Strengthening is not legally required on the building. It is our recommendation that strengthening be carried out to bring the seismic capacity up to a target of 67% of NBS.

It is recommended that:

a) The building does not contain any damage that would cause it to be 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: Repointed mortar joints on the Southeast amenities block	Photo 2: Repointed mortar joints on the Southeast amenities block
	
Photo 3: Stepped mortar joint cracking below the window from photo 1	Photo 4: Mortar cracking around the building transition from amenities to main hall.

	
Photo 5: Repointed masonry, still to be painted	Photo 6: East storage room/ hall entry off the main hall
	
Photo 7: South end of the building with ventilation to the subfloor	Photo 8: Open ventilation point

	
Photo 9: Subfloor timber particle board flooring on timber joists and bearers. Joist spacing approximately 600mm bearer and pile spacing approximately 2000mm	Photo 10: Particle board flooring
	
Photo 11: Bearers tied to concrete piles by wire	Photo 12: Photo up through the masonry block cell, above the ventilation point. Masonry walls partially filled

	
Photo 13:Southwest corner of the building with cracking below the window. See photos 14 and 21 to 23	Photo 14: Block separation between masonry units
	
Photo 15: North west corner of the building	Photo 16: Spalling to the concrete infill along the gable end wall between blockwork and roof

Photo 17: Amenities have masonry perimeter walls with light timber framed internal walls and plasterboard ceiling linings.

Photo 18: View west through amenities block showing internal cladding

Photo 19: Plasterboard around internal door penetration

Photo 20: Plasterboard ceiling lining showing nailing to battens

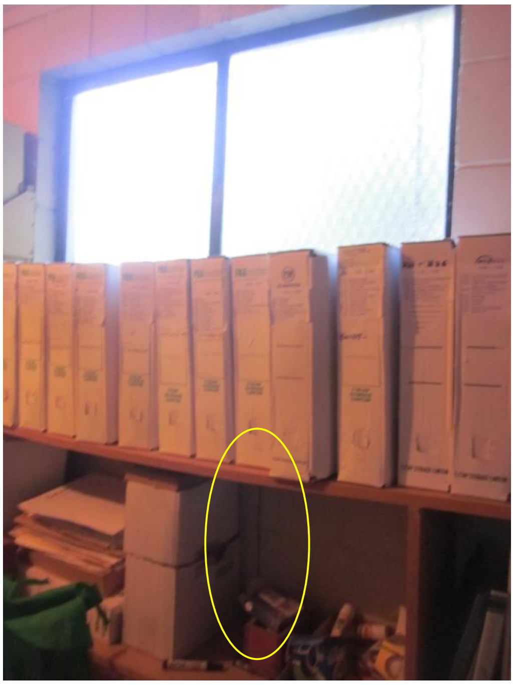


Photo 21: Cracking beneath south west window	Photo 22: Cracking extends between masonry units and through blocks
	
Photo 23: Cracking approximately 1.5 to 2mm in width	Photo 24: Plasterboard roofing in amenities area library

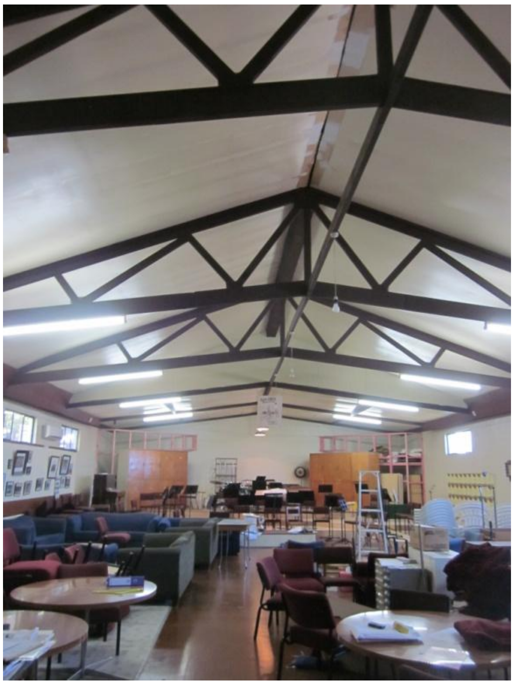
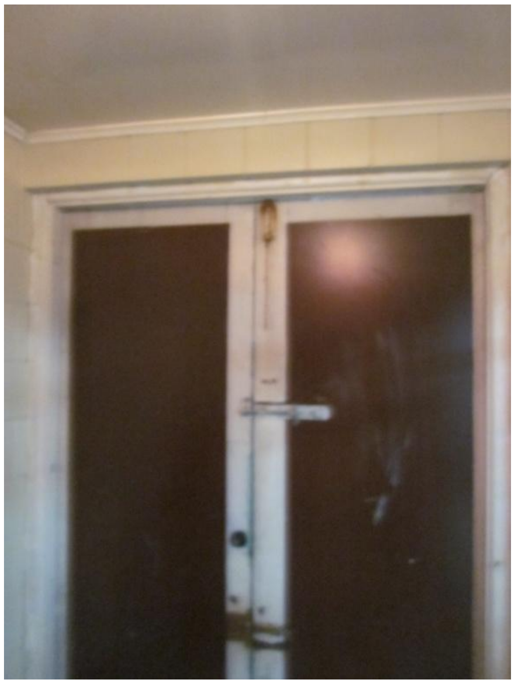
	
Photo 25: Timber roof trusses spaced at 3.6m centres through main band hall	Photo 26: Bottom chord of trusses restrained by perpindicular members
	
Photo 27: Perpindicular member connected to a timber end plate to the end masonry wall.	Photo 28: Area above the stage has a ceiling structure within the trusses

	
Photo 29: Structure of ceiling support unknown	Photo 30: Entry/ storage room off the main hall. East wall
	
Photo 31: Storage room/ hall entry off main hall	Photo 32: Storage room/ hall entry off main hall

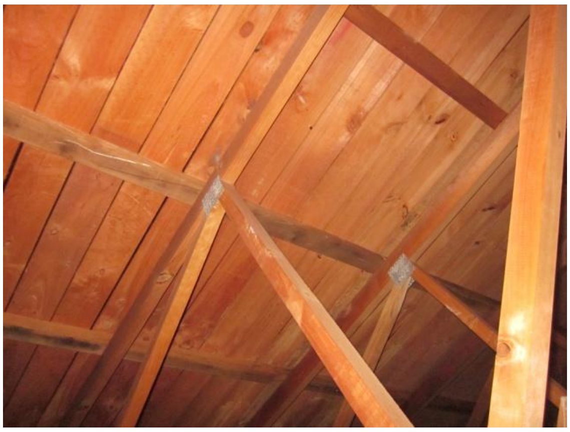


Photo 33: Roof structure above the amenities block	Photo 34: Timber tongue and groove roofing beneath corrugated metal. Supported by light timber trusses

11. Appendix 2 – CERA Standardised Report Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Rawhiti Domain - Silver Band Hall	Reviewer:	Nick Calvert
	Unit / No: / Street	CPEng No:	242062
Building Address:	35-37 Bowhill road	Company:	Sinclair Knight Merz
Legal Description:		Company project number:	ZB01276.231
		Company phone number:	39404923
	Degrees / Min / Sec		
GPS south:		Date of submission:	19/11/2013
GPS east:		Inspection Date:	22/03/2013
		Revision:	B
Building Unique Identifier (CCC):	PRK_2004_BLDG_018	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:		Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	2.00

Building

No. of storeys above ground:	1 (single storey = 1)	Ground floor elevation (Absolute) (m):	2.00
Ground floor split?	no	Ground floor elevation above ground (m):	0.30
Storeys below ground	0		
Foundation type:	bored cast-insitu concrete piles	if Foundation type is other, describe:	
Building height (m):	6.10	height from ground to level of uppermost seismic mass (for IEP only) (m):	6.1
Floor footprint area (approx):	326		
Age of Building (years):	35	Date of design:	1976-1992
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	public	Brief strengthening description:	
Use (upper floors):			
Use notes (if required):			
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber truss	truss depth, purlin type and cladding	1.2m deep truss, with timber purlins and profiled metal roofing
Floors:	timber	joist depth and spacing (mm)	200mm at 0.6m spacings, bearers at 2.0m crs
Beams:			
Columns:			
Walls:	partially filled concrete masonry	thickness (mm)	190

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	partially filled CMU		
Ductility assumed, μ:	1.25	note total length of wall at ground (m):	
Period along:	0.20	estimate or calculation?	estimated
Total deflection (ULS) (mm):	10	estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):	10	estimate or calculation?	estimated
Lateral system across:	partially filled CMU		
Ductility assumed, μ:	1.25	note total length of wall at ground (m):	
Period across:	0.20	estimate or calculation?	estimated
Total deflection (ULS) (mm):	10	estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):	10	estimate or calculation?	estimated

Separations:

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

Non-structural elements

Stairs:			
Wall cladding:	other heavy	describe	Partially filled CMU
Roof Cladding:	Metal	describe	Corrugated metal
Glazing:	steel frames		
Ceilings:	fibrous plaster, fixed		
Services(list):			

Available documentation

Architectural	none	original designer name/date	
Structural	none	original designer name/date	
Mechanical	none	original designer name/date	
Electrical	none	original designer name/date	
Geotech report	none	original designer name/date	

Damage

Site: (refer DEE Table 4-2)

Site performance:		Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	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:	slight	notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	Damaged blocks not likely to effect resistance of the longitudinal system as a whole given the length of the wall
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

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

Recommendations

	Level of repair/strengthening required:	Minor structural	Describe:	Repair of cracked masonry units on the south west of the building through the library wall
	Building Consent required:	no	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	
Along	Assessed %NBS before e'quakes:	40%	If IEP not used, please detail assessment methodology:	Quantitative assessment
	Assessed %NBS after e'quakes:	40%		
Across	Assessed %NBS before e'quakes:	49%		
	Assessed %NBS after e'quakes:	49%		

SINCLAIR KNIGHT MERZ

12. Appendix 3 – Desktop Geotechnical Study of Rawhiti Domain Community Garden

Sinclair Knight Merz
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Web: www.globalskm.com

Christchurch City Council - Structural Engineering Service

Geotechnical Desk Study

SKM project number	ZB01276
SKM project site number	052
Address	Community Building, 136 Shaw Avenue
Report date	2 April 2012
Author	Ross Roberts / Ananth Balachandra / David Bae
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 Detailed Engineering Evaluation (DEE) of whether the building can be economically repaired, 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

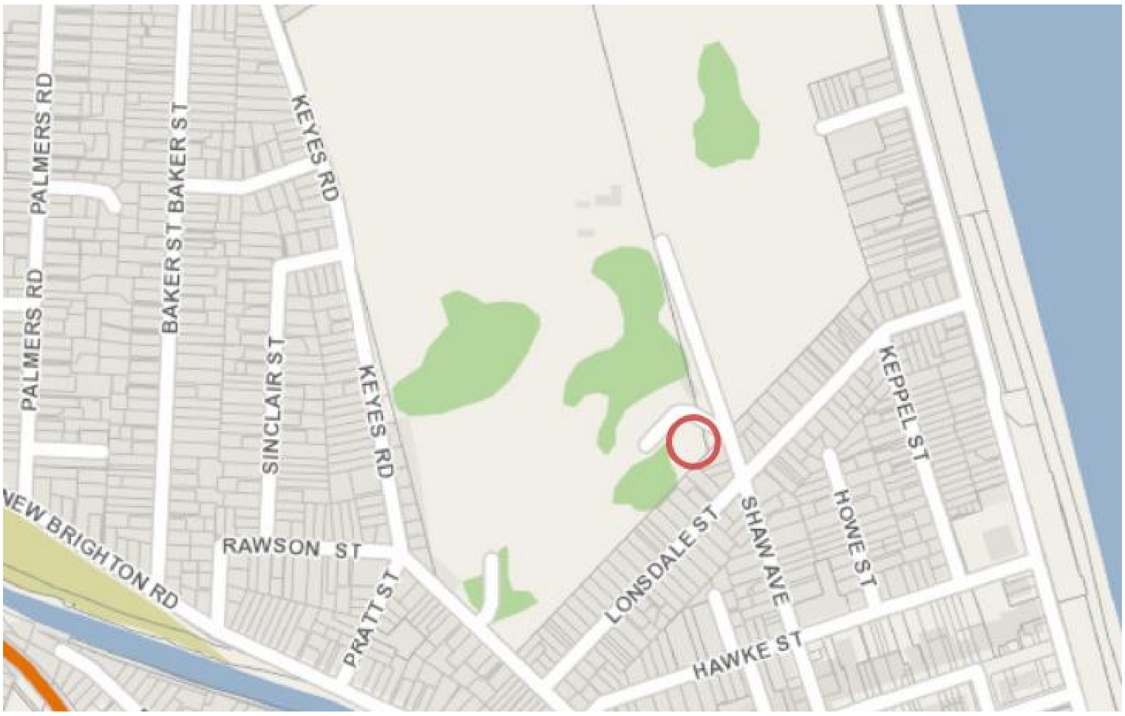
3. Limitations

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



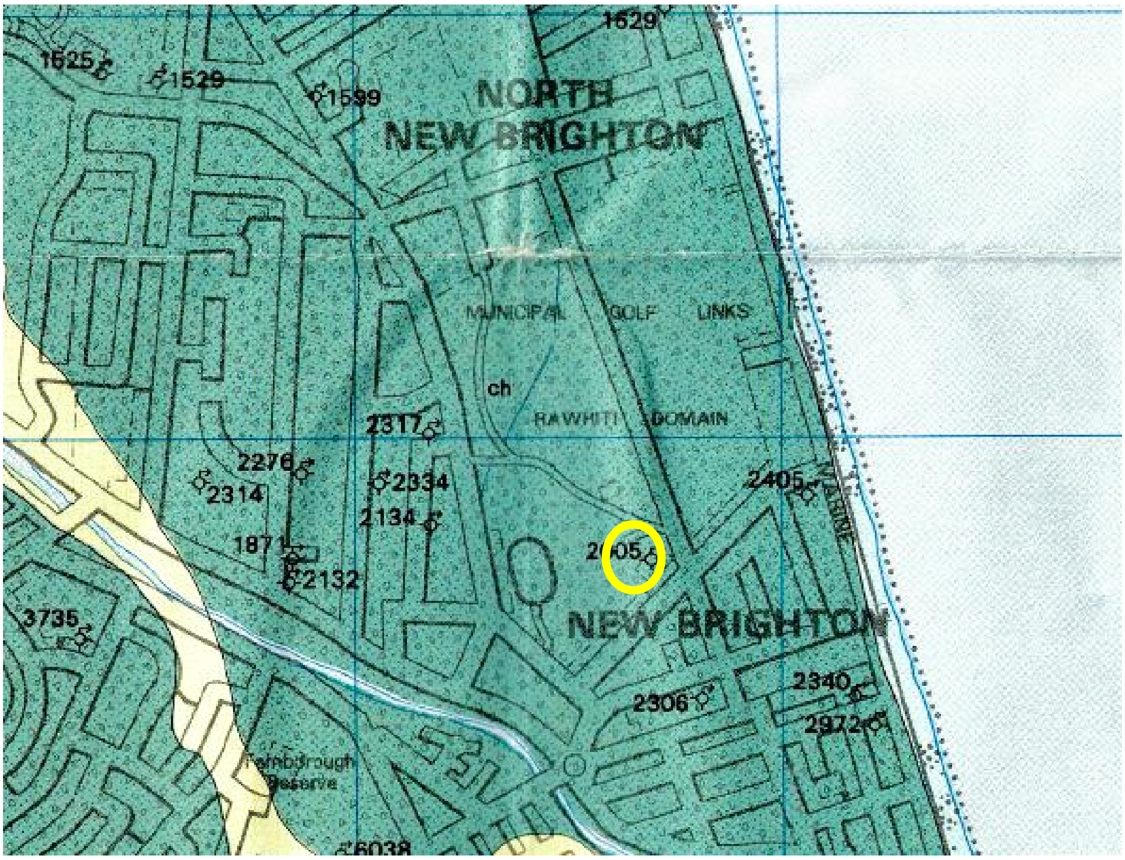
- **Figure 1 – Site location (courtesy of LINZ http://viewers.geospatial.govt.nz)**

The structure is located on 136 Shaw Avenue at grid reference 1577661 E, 5183161 N (NZTM).

5. Review of available information

5.1 Geological maps

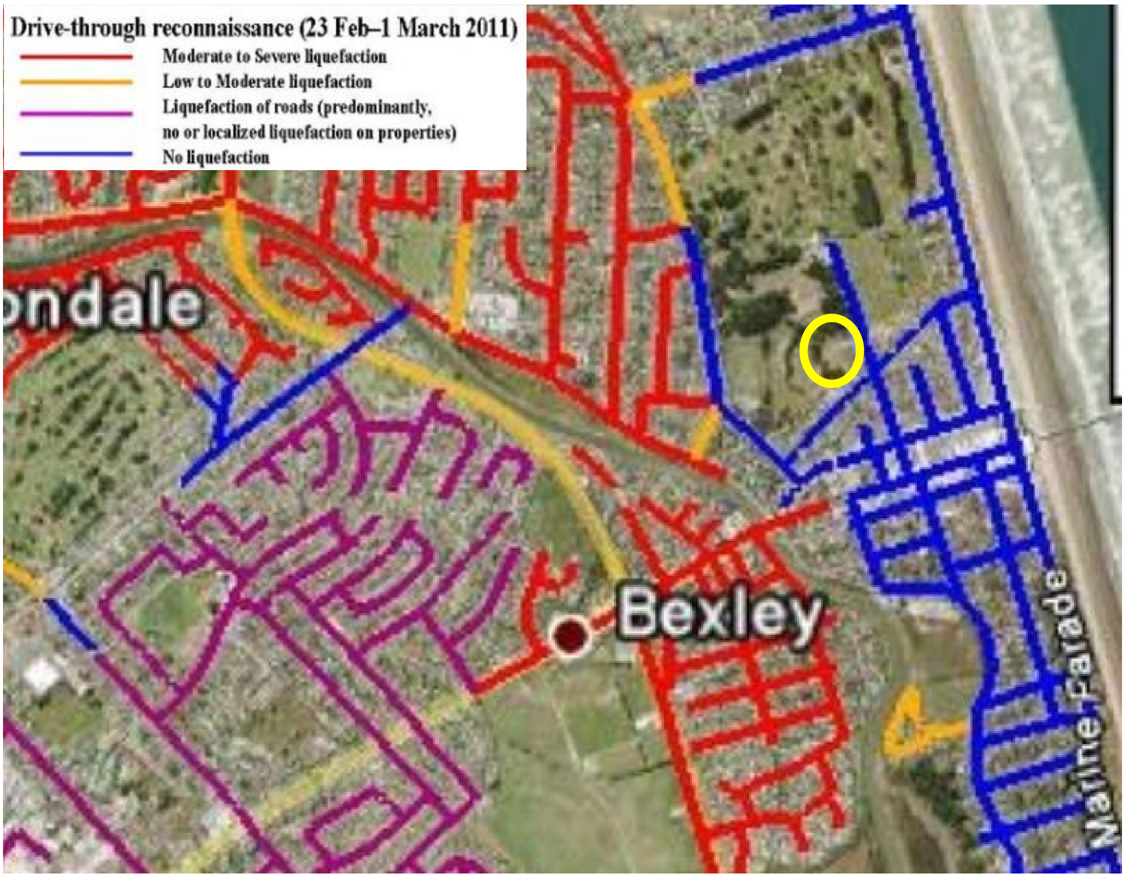
- **Figure 2 – Regional geological map (Forsyth et al, 2008). Site marked in red.**



- **Figure 3 – Local geological map (Brown et al, 1992). Site marked in yellow.**

The site is shown to be underlain by Holocene deposits comprising sand of fixed and semi-fixed dunes and beaches of the Christchurch formation.

5.2 Liquefaction map



- **Figure 4 – Liquefaction map (Cubrinovski & Taylor, 2011). Site marked in yellow.**

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. Their findings show no liquefaction at this site.

5.3 Aerial photography

- **Figure 5 – Aerial photography from 24 Feb 2011 (http://viewers.geospatial.govt.nz/)**

Some lighter patches are visible west of the site there is a slight possibility that this may be dried silt ejecta. This is thought unlikely as there was no other significant evidence of liquefaction or land damage is visible from the aerial photographs. The site is very close to the beach and the patches are likely to be windblown sand or dune sand exposed by lack of vegetation.

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 (Urban Non-residential)

5.5 Historical land use

Reference to historical documents (eg Appendix A) shows that no specific historic land use for the site was recorded in 1856. Taking this into account it is likely that the site was a general grass area or used as farmland.

5.6 Existing ground investigation data

- **Figure 6 – 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

A review of available council files found one relevant geotechnical investigation in the vicinity of the site which included three shallow test pits approximately 75 m, 175 m, and 265 m to the east of the site respectively. The ground investigation shows a geological profile consisting of 100-200 mm of topsoil beneath the ground surface underlain by loose sand to the base of the excavations at approximately 1.5 m. Scala Penetrometer test values ranged from 0 to 10+ blows /100 mm, with typical values of 5+ blows/ 100 mm. Groundwater was encountered typically at 1.5 m depth.

5.8 Site walkover

A walkover inspection of the site was undertaken by a Geotechnical Engineer from SKM on 24 April 2012.

The building was noted to be a single storey masonry block building with a sheet metal roof. The foundation for the building was most likely a reinforced concrete slab on grade foundation. During the external site walkover o evidence of liquefaction or other land damage was observed. Any sand present on the surface was likely to be windblown sand from the nearby beach. There was undulating ground observed close to the site and through the car park but this could be due to tree roots or past sand dunes rather than due to liquefaction occurring on site.

- **Figure 7 Overview of community building**

- **Figure 8 Observed undulating ground**

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 - 15	Sand and clay layers
15 - 25	Sand and gravelly sand
25+	Predominantly sand and gravel

The ground water table was inferred to be approximately 1.5m below ground level as indicated in the test data available in the council records.

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. The available investigations however present a considerable distance away from the site. Therefore, site specific investigation could revise the assessed site class.

6.3 Building Performance

The performance to date suggests that the existing foundations are adequate for their purpose.

6.4 Ground performance and properties

Liquefaction risk appears to be low for this site. The reconnaissance performed following the 22nd February earthquake and the conclusions from the site walkover conducted by a SKM engineer suggests that no significant liquefaction occurred on site.

The only available investigation with measurements of geotechnical parameters was the one scala penetrometer test available in the council records. However, as a borehole log is available within 50m of the site and no significant liquefaction or damage to the structure was observed during the external site walkover, soil parameters for the shallow soil layer that could be used for the purposes of quantitative DEE are provided below:

- Effective angle of friction = 32 degrees
- Apparent cohesion = 0 kPa
- Unit weight = 18 kPa
- Ultimate bearing capacity = 200 kPa

It is noted these parameters should not be used for consent or design purposes and ground conditions should be confirmed by a geotechnical investigation. It is possible that further investigations would result in an increase to the recommended ultimate bearing capacity. The recommended bearing capacity was estimated for loose to medium dense sand as indicated by the Scala penetrometer test.

6.5 Further investigations

If consent is required additional investigations will be needed to confirm the recommended ground properties and to perform a full liquefaction assessment. Recommended investigations are:

- Two cone penetration tests to refusal

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/)

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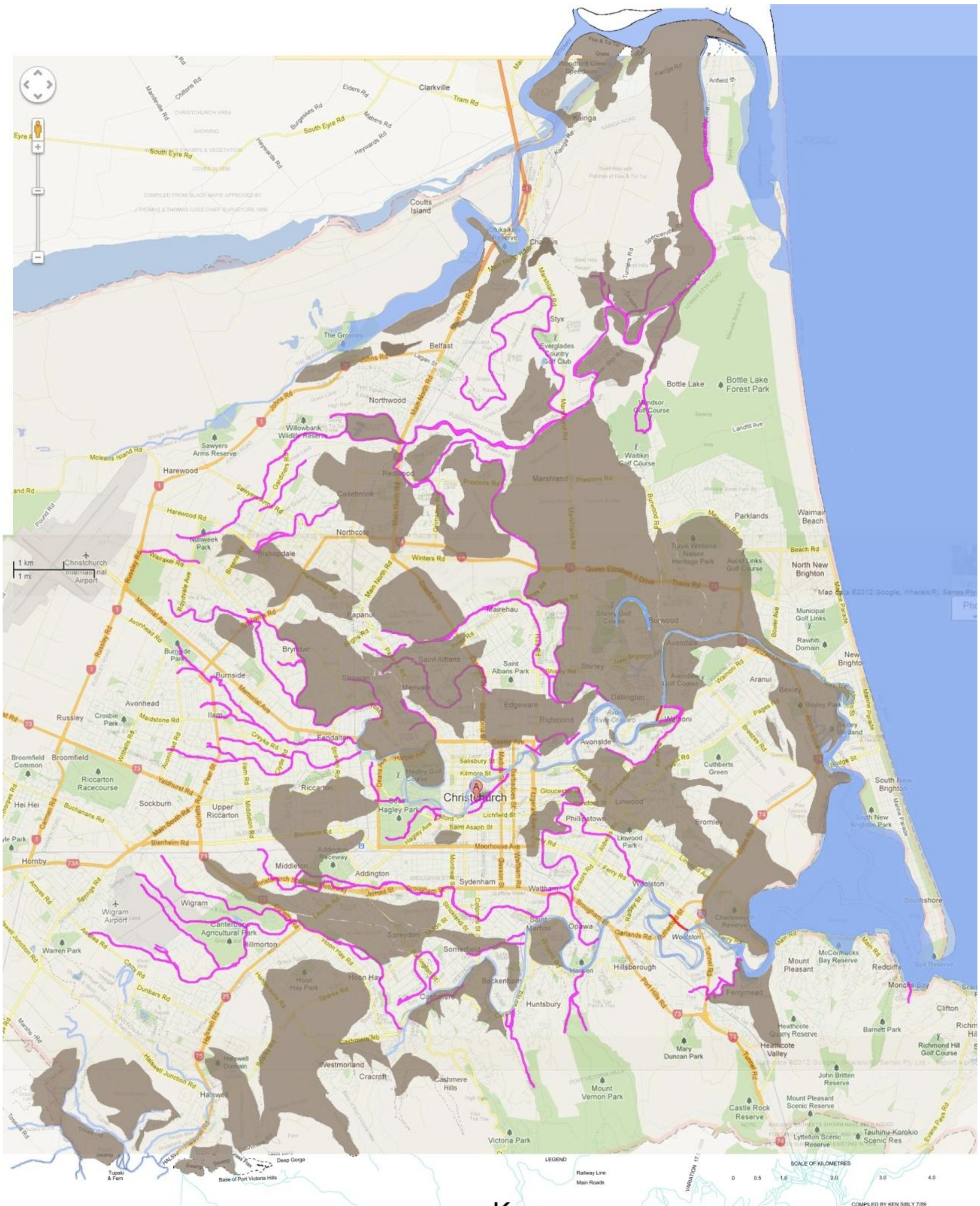
Appendix A – Christchurch 1856 land use



Key

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

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

Appendix B – Existing ground investigation logs

Borelog for well M35/2005

Gridref: M35:877-447 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 2.2 +MSD
Driller : J W Horne (& Co)
Drill Method : Unknown
Drill Depth : -87.09m Drill Date : 2/10/1936

Water Level: Artesian

Depth(m)	Full Drillers Description	Formation Code
- 21.3m	Black sand	ch
- 24.3m	Black sand	ch
- 27.4m	Black sand	ch
- 39.0m	Black sand	ch
- 49.3m	Black shingle, flow at 42.6 - 45.7m & best flow at 48.7m, rose 0.9m	ri
- 50.2m	Brown sand	br
- 51.8m	Black sand	br
- 69.4m	Black sand	br
- 77.7m	Brown shingle, flow at 73.1m rises 2.1m	li-1
- 81.6m	Black clay & sand	li-2
- 83.5m	Yellow clay	li-2
- 87.1m	Brown shingle, flow at 84.1m to 85.9m & 87.1m	li-2

Borelog for well M35/2388

Gridref: M35:876-450 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 3.8 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -84.69m Drill Date : 19/03/1930

Water Level	Depth(m)	Full Drillers Description	Formation Code
Artesian	- 15.2m	Sand & clay	ch
	- 30.4m	Sand & gravel	ch
	- 39.3m	Clay & peat	ch
	- 44.5m	Blue shingle	ri
	- 49.9m	Brown shingle (Water rises +0.9m)	ri
	- 51.8m	Brown sand & gravel	ri
	- 55.7m	Yellow clay	br
	- 67.0m	Yellow sand & gravel	br
	- 69.2m	Yellow clay	br
	- 73.2m	Brown shingle (Water rises 3.05m)	li-1
	- 80.8m	Clay	li-2
	- 84.7m	Brown shingle	li-2

Scale(m): -10, -20, -30, -40, -50, -60, -70, -80

Borelog for well M35/2045

Gridref: M35:873-451 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 2.5 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -86.5m Drill Date : 21/11/1923

Environment Canterbury Regional Council

Scale(m): -10, -20, -30, -40, -50, -60, -70, -80

Water Level: Artesian

Depth(m)	Full Drillers Description	Formation Code
- 37.7m	Sand & clay	ch
- 40.5m	Blue clay & peat	ch
- 43.5m	Blue shingle	ri
- 44.5m	Brown shingle	ri
- 49.3m	Blue shingle	ri
- 50.5m	Brown sand & clay	br
- 51.8m	Blue sand & clay	br
- 67.0m	Brown sand	br
- 69.2m	Blue sand & clay	br li-1
- 69.5m	Blue shingle	
- 79.6m	Brown shingle	li-1
- 84.1m	Blue sand & clay	li-2
- 86.5m	Brown shingle	li-2

Appendix C – Summary of existing investigations

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

ID		1	2	3
Type *		BH	BH	BH
Ref		M35-2005	M35-2388	M35-2045
Depth (m)		87.1	84.7	86.5
Distance from site (m)		50	280	550
Ground water level (mBGL)		Artesian	Artesian	Artesian
Simplified recorded geological profile (depth below ground level to top of stratum, m)	0			
	1			
	2			
	3			
	4			
	5			
	6			
	7			
	8			
	9			
	10			
	11			
	12			
	13			
	14			
	15			
	16			
	17			
	18			
	19			
	20			
	21			
	22			
	23			
	24			
	25			
Greater depths		Sand and gravel to drill depth		

*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