CHRISTCHURCH CITY COUNCIL
PRO 0822 B002 EQ2
Linwood Library Support Services
180 Smith St, Linwood

QUANTITATIVE ASSESSMENT REPORT
FINAL

- Rev C
- 13 January 2014

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1. Executive Summary

1.1. Background

A Quantitative Assessment was carried out on building PRO 0822 B002 EQ2 located at 180 Smith St, Linwood. Building 2 is a single storey building with mezzanine floor located at the northern end. The building is constructed from steel frames and precast concrete walls. An aerial photograph illustrating the buildings location is shown below in Figure 1. Detailed descriptions outlining the buildings age and construction type are given in Section 5 of this report.

- **Figure 1: Aerial Photograph of Building PRO 0822 B002 EQ2 Located at 180 Smith St**

This quantitative report for the building structure is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections carried out on 6th March 2012, survey drawings and calculations.

1.2. Key Damage Observed

Key damage observed includes:-

- Cracking to the concrete precast panels.
- Hairline cracking to internal wall linings.
- Damage to shelving units in the store room. Due to instability of these shelving units this part of the building has a yellow placard.

Further details describing the level of damage are given in section 5.5 of this report. A building consent is not likely to be required for repairing this damage.

1.3. Critical Structural Weaknesses

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

1.4. Indicative Building Strength

As described in the Engineering Advisory Group's "Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings" (from July 2011) we have assessed the percentage of 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.
- A geotechnical desktop investigation has been undertaken. We have based this report on our knowledge of the site and the absence of liquefaction ejecta on the site. This report can be found in Appendix 5.
- 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 is required to be strengthened up to a capacity of at least 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 39%NBS and post earthquake capacity in the order of 39%NBS.

The building has been assessed to have a seismic capacity in the order of 39% NBS and is therefore not potentially earthquake prone.

Please note that structural strengthening is required by law for buildings that are confirmed to have a seismic capacity of less than 33% NBS.

1.5. Recommendations

As this building is found to have a %NBS of above 33% there is no requirement to strengthen the building to above 67%. The client may wish to strengthen the building to this target; this will require a building consent.

It is recommended that:

a) The current placard status of yellow remain until the shelving units in the store room are repaired and securely restrained to resist the required earthquake loads Once this has been done then the placard status of the building can be changed to Green 1.

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

c) Consideration should be given to strengthening the building to at least 67% NBS.

2. Introduction

Sinclair Knight Merz was engaged by Christchurch City Council to carry out a Quantitative Assessment of the seismic performance of Linwood Library Support Services located at 180 Smith Street, Linwood.

The scope of this quantitative analysis includes the following:

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

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

This assessment identified that the seismic capacity of the building was likely to be less than 67% of the New Building Standard (NBS). A quantitative assessment was recommended to confirm the initial assessment findings and to determine a more accurate seismic rating of the building.

At the time of this report, intrusive site investigations had been carried out as recommended by the previous qualitative report. Construction drawings were not made available, and this has been considered in our evaluation of the building. The building description in section 5 is based on our visual inspections and intrusive investigations.

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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 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

Building PRO 0822 B002 EQ2 is a single storey building with a mezzanine floor located in the northern end. The building is primarily used as a library and offices by the local government. The building is constructed from steel portal frames and concrete precast panel walls. The mezzanine floor is constructed from timber framing and is accessed by one timber stair located on the south side of the floor. Garages are present along the eastern side of the building. They are constructed from concrete masonry block and have a steel roof trusses that support timber purlins and a light weight profiled steel cladding. The roof trusses are fixed to the concrete panels of the main building. The building is believed to be supported on concrete strip footings and pad foundations and has a concrete slab on grade at ground level.

5.2. Gravity Load Resisting system

Our evaluation was based on our site investigation conducted on the 6 March 2012 and the intrusive investigations in May 2012. These investigations allowed us to verify the structural system of the building. Drawings of the mezzanine floor dated February 1990 by Christchurch City Council – City Architects Division where provided however these drawings show no structural details.

Building PRO 0822 B002 EQ2 is essentially a single storey building with a mezzanine floor. The overall gravity system comprises of structural steel portal frames that support the roof structure and the concrete precast panel walls. The roof structure consists of light weight steel purlins and a light weight profiled steel cladding. The mezzanine floor is constructed from timber framing and is supported on a combination of timber framed walls and steel framing. The foundations are believed to be constructed from concrete pad foundations under the gravity columns and strip footings under the concrete walls. The ground floor consists of a concrete slab on grade. The drawings for the mezzanine floor are dated 1990 and so it is assumed that the building was constructed sometime in the mid 1980’s.

5.3. Seismic Load Resisting system

For the lateral analysis of this building the ‘across direction’ has been taken as North West-South East whereas the ‘along direction’ has been taken as North East-South West.

Lateral loads acting across the building will be resisted by the steel portal frames.

Lateral loads acting along the building from the roof will be taken by the roof braces which will link the forces back to the concrete walls resisting the force by shear.

For the mezzanine floor the loads acting in the along direction the steel frame supporting the mezzanine floor will resist the loads. For the loads acting in the across direction the concrete walls on the sides of the mezzanine floor will resist the loads

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.
- It is expected that the allowable bearing capacity of a shallow pad footing on this site will be in the region of 200 kPa. We estimate a conservative ultimate bearing capacity to be in the order of 400 kPa. However, these may be revised by a site specific investigation.
- Liquefaction risk is low at this site.

Unless a change of use is intended for the site we do not believe that any further geotechnical investigations are required. Specific ground investigation should be undertaken if significant alterations or new structures are proposed. If any excavations are required on the site further investigation of the potential for contamination should be undertaken. The full geotechnical desktop study can be found in Appendix 4.

5.5. Building 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.

External Damage

1) Western Wall – Cracking has occurred to the concrete panels along the construction joint near the centre of the wall. Concrete appears ready to spall off in some locations (PHOTO'S 4, 5 & 6).
2) North Wall – Cracking and spalling of the concrete panel wall has occurred along the edge of the panel on the west side of the timber infill (PHOTO'S 7 & 8).
3) East Wall (garage) - Hairline crack present along the joint between the timber rafter and the concrete column in the NE corner (PHOTO'S 9 & 10).
4) East Wall (garage) – Vertical hairline crack present in the masonry mortar joint on the northern side of the office door lintel (PHOTO 11).
5) South Wall – Gap present between the concrete wall and timber infill. Difficult to confirm if this is earthquake damage (PHOTO 12 & 13).

Internal Damage

- **Level 1 (L1)**

6) Hairline cracking present along the timber framed wall and concrete panel wall joint in the NW and NE corners (PHOTO'S 14 & 15).

7) Hairline cracks present along the joint between the northern window bulkhead and wall (PHOTO 16).
8) Mitre joints on the northern window architrave have opened up (PHOTO 16).
9) Cracking along ceiling joint on the western side where the ceiling changes slope. (PHOTO 17).
10) Hairline cracking present around the top of the partition wall at the entrance to the L1 office. This crack occurs along the wall and ceiling joint and continues the full length of the partition wall (PHOTO'S 18 & 19).

- **Ground Floor (L0)**

11) North office, east wall: - Diagonal hairline crack approximately 0.4mm wide present on the concrete wall radiating out from the top north corner of the opening (PHOTO'S 20, 21 & 22).
12) North office, south wall: - Diagonal hairline crack approximately 0.2-0.3mm wide present in concrete wall located on the eastern side of the personnel door (PHOTO 24).
13) Joints in the timber wall linings near the western entrance have opened up (PHOTO 25).
14) Wall in front of stairs: - Hairline cracking present in the timber wall linings around the wall opening (PHOTO'S 26, 27, 28 & 29).
15) Concrete wall dividing the east office and the kitchen: - Vertical hairline crack approximately 0.3mm wide above the opening in the concrete wall (PHOTO'S 30, 31 & 32).

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

6. Seismic Evaluation

6.1. 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
    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 2. 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

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

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 the Table 2 below contains the likely new recommendations.

- **Table 2: 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

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

6.2. Available Information

Following our inspections in May 2012, SKM carried out a seismic review on the structure. This review was undertaken using the limited drawings which were available. This included.

- Drawings of the mezzanine floor dated February 1990 by Christchurch City Council – City Architects Division. These drawings however provided no structural details.
- Structural information obtained during a number of site inspections by SKM which included site measurements, and intrusive investigations into the roof space and the mezzanine floor

6.3. Survey

There was no visible evidence of settlement and as such no survey has been carried out.

6.4. Design Criteria and 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 has been assessed as class D as described in AS/NZS1170.5:2004, Clause 3.1.3, Soft Soil. This has been assessed from borehole logs in the area. The ultimate bearing capacity on site is 400kPa. Liquefaction risk is low at this site.
- 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 period of 0.3 seconds in the along direction and 0.6 seconds in the across direction.
- 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 3: Assumed Building Ductility**

Direction	Ductility State
Along	1.00
Across	1.25

In the along direction the concrete shear walls will remain elastic and in the across direction the steel portal frames has been assumed to be nominally ductile.

- The following material properties were used in the analyses:
- **Table 4: Material Properties**

Material	Nominal Strength	Structural Performance
Structural Steel	f_y = 300MPa	S_p = 0.9
Roof bracing	f_y = 250MPa	S_p = 1.0
Concrete	f_c' = 30MPa	S_p = 1.0
Bolts	f_{uf} = 400MPa	S_p = 1.0

- Type of 460UB for the portal frame steel could not be confirmed on site, 460UB67 has been assumed as the section size
- Type of 200DHS for the purlins has been assumed to be DHS200/12, properties from the Dimond manufacturer have been used.
- Not all of the shear wall to steel column connections could be inspected. From the ones that were inspected it has been assumed all the connections on the south side consist of cleats welded to the columns with bolts connecting the concrete wall to the cleats. Bolts have been assumed to be M12 bolts epoxied with Ramset Chemset injection 101 plus.
- For the connections on the north side it has been assumed all the connections consist of cleats welded to the columns with the cleats welded to the concrete walls. 5mm welds have been assumed
- No panel joints were found on the south wall it has been assumed that the wall was cast in-situ.
- It is assumed the particle board and gib ceiling is adequately connected to the timber joists to provide diaphragm action in the mezzanine floor
- Details of how the concrete panels are connected to the foundation have not been provided. An assumption has been that the walls in the longitudinal direction do not have a connection at the

bottom so that when they act out of plane they will span between the steel columns, when acting in-plane they will resist loads by rocking.

- It has been assumed that the end walls and the concrete wall beneath the mezzanine floor will have a shear connection to resist loads in the out of plane direction.

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.

7. Results and Discussion

7.1. Critical Structural Weaknesses

This 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. A summary of the results of the analysis are reported in the following table as %NBS. The results below are calculated for the building in its initial and damaged state. The building results have been broken down into their seismic resisting elements

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

Table 5: DEE Results

Building	Seismic Resisting Element	Action	Seismic Rating %NBS
Linwood Library	Roof Tension Bracing	Tension	39%
	Concrete Side Walls	Out of plane bending	41%
	Steel Portal Frame	Bending	42%
	Roof Tension Bracing Connections	Compression	44%
	Bolt connection at concrete side walls to steel columns	Tension pull out	51%
	Mezzanine portal frame	Bending	70%

7.3. Recommendations

The quantitative assessment carried out on the Linwood Library indicates that the building has a seismic capacity above 34% of NBS and is therefore classed as being a 'Moderate Risk Building'. Strengthening of the building is not required as no elements were found to be less than 34% of NBS. The client may wish to strengthen the building to above a 67% target; this will require a building consent.

Strengthening the building above the 67% target will involve:

Strengthening the side walls

Depending on what option is taken will affect how the rest of the structure is also strengthened. Strengthening of the side walls can be done a number of ways.

- Replace the existing concrete panels with new concrete panels with larger depth and increased reinforcing. Bolt connections would also be replaced with a stronger connection system. This will add additional seismic mass to the structure.
- Sprayed concrete on the exterior of the existing concrete panels. This solution will also add additional seismic mass to the structure. Additional steel cleats would also have to be welded to the steel columns and bolted to the existing concrete panel.
- Retrofit steel members on the interior of the existing concrete panels. This will encroach on the interior floor space. Discussions with the client will determine if this is an acceptable solution.
- Carbon fibre strengthening. Additional steel cleats would also have to be welded to the steel columns and bolted to the existing concrete panel.

It is difficult to determine which would be the most suitable solution. Further design work, cost analysis and discussions with the client will determine which solution would be most appropriate for the strengthening of the side walls.

Strengthening Steel Portal frame

This can be done by installing additional fly braces between the bottom flange of the UB section and the DHS purlins at the hip joint locations. This will be done for all the hip joint locations where fly braces are not already present.

Strengthening Roof Bracing

Strengthening of the roof can be done by adding extra bays of cross steel braces along the roof. The number of extra bays required will depend on how much additional seismic mass is added to the structure from the side wall solution.

We recommend that the following actions are taken:

a) The current placard status, yellow remains until the shelving units in the store room are repaired and securely restrained to resist the required earthquake loads. Once this has been done then the placard status of the building can be changed to green 1.
b) We consider that barriers around the building are not necessary.
c) Consideration should be given to strengthening the building to at least 67% NBS

8. Conclusion

SKM carried out a quantitative assessment on PRO 0822 B002 EQ2 located at 180 Smith St. This assessment concluded that the building is classified as not potentially earthquake prone.

▪ **Table 6: Quantitative assessment summary**

Description	Grade	Risk	%NBS	Structural performance
Linwood Library	C	Moderate	39	Acceptable legally. Improvement recommended.

The quantitative assessment carried out on the Linwood Library indicates that the building has a seismic capacity above 34% of NBS and is therefore classed as being in the category of 'Moderate Risk Building'. Strengthening of the building is not required as no elements were found to be less than 34% of NBS. The client may wish to strengthen the building to above a 67% target; this will require a building consent.

It is recommended that:

a) The current placard status of yellow remains until the shelving units in the store room are repaired and securely restrained to resist the required earthquake load. Once this has been done then the placard status of the building can be changed to green 1.
b) We consider that barriers around the building are not necessary.
c) Consideration should be given to strengthening the building to at least 67% NBS

9. Limitation Statement

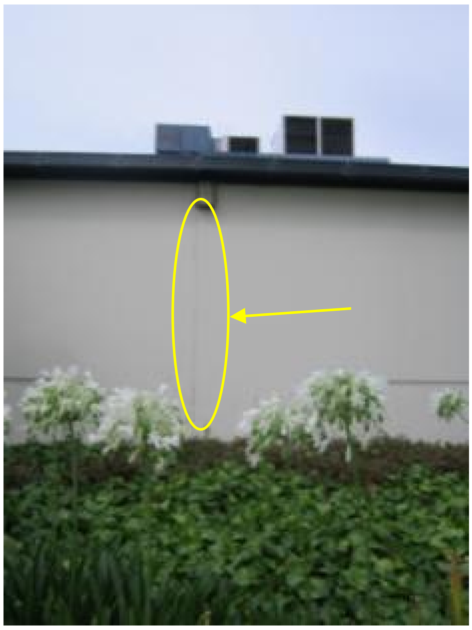
This report has been prepared on behalf of, and for the exclusive use of, SKM's client, and is subject to, and issued in accordance with, the provisions of the contract between SKM and the Client. It is not possible to make a proper assessment of this report without a clear understanding of the terms 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: South Elevation of Building 2	PHOTO 2: General Structural System of Building 2
	
PHOTO 3: Level 1 Office Looking East	PHOTO 4: Cracking along Construction Joint – West Wall

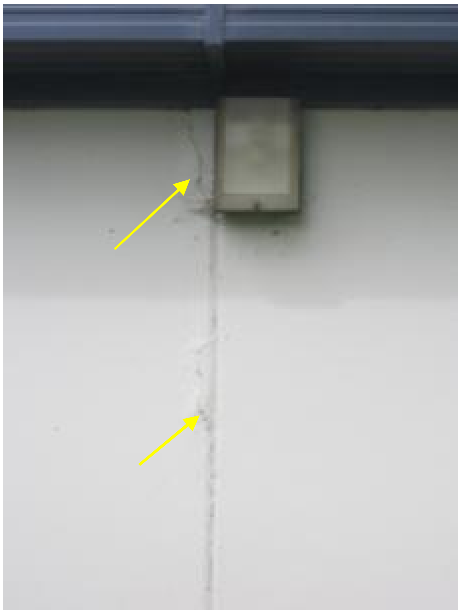
	
PHOTO 5: Close up of Photo 4 (bottom)	PHOTO 6: Close up of Photo 4 (top)
	
PHOTO 7: Cracking & Spalling to Concrete Panel - North Wall	PHOTO 8: Close up of Photo 7

	
PHOTO 9: Hairline Cracking along Timber Rafter & Concrete Column Joint – East Wall	PHOTO 10: Close up of Photo 9
	
PHOTO 11: Hairline Cracking to Masonry Lintel above Office Entrance – East Wall	PHOTO 12: Gap between Concrete Panel and Timber Infill – South Wall

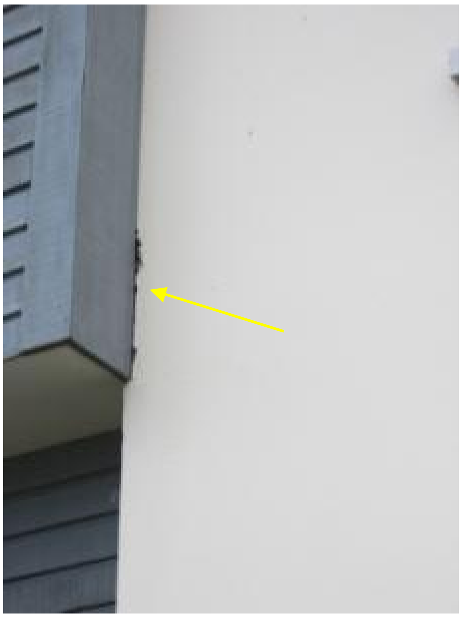


PHOTO 13: Close up of Photo 12	PHOTO 14: L1 - Hairline Cracking between Concrete Wall and Timber Wall in NW Corner

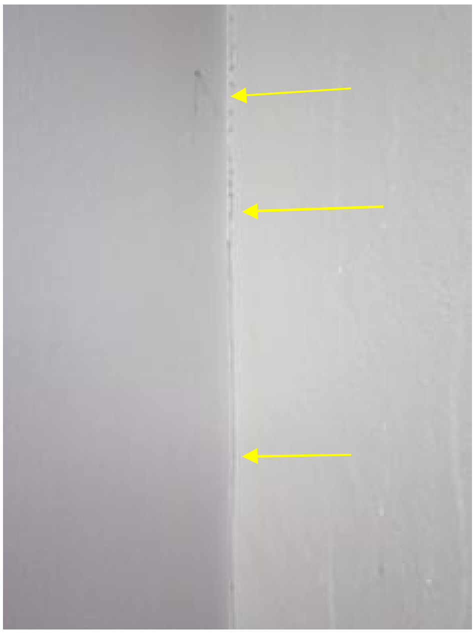



PHOTO 15: L1 - Close up of Photo 14	PHOTO 16: L1 - Hairline Cracking along the Northern Window Bulkhead and the Wall Joints. Mitre Joints to Window Architrave have also Opened up

PHOTO 17: L1- Cracking along Ceiling Lining Joint

PHOTO 18: L1- Hairline Cracking along the Timber Partition Wall and Ceiling Joint

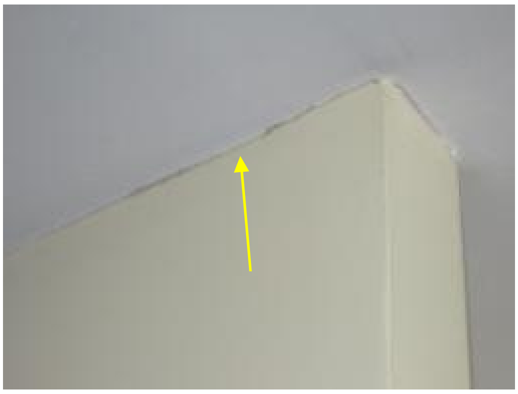


PHOTO 19: L1 – Close up of photo 18

PHOTO 20: L0 – Diagonal Hairline Crack in East Wall of North Office

PHOTO 21: L0 – Close up of Photo 20

PHOTO 22: L0 - Close up of Photo 21

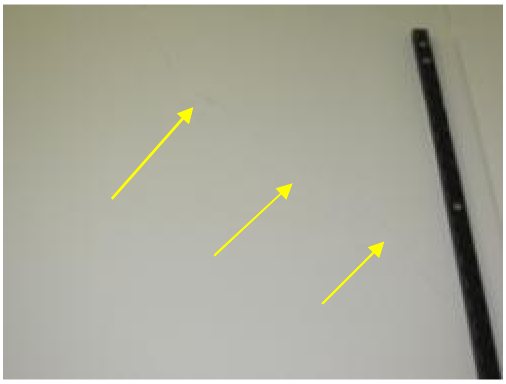
	
PHOTO 23: L0 – North Office, South Wall	PHOTO 24: L0 – Hairline Crack to South Wall of North Office
	
PHOTO 25: L0 – Joints in Wall Lining near Western Entrance have Opened up	PHOTO 26: L0 – Wall in front of Stair - Hairline Cracking in Wall Lining around Door Opening

PHOTO 27: L0 – Close up of Photo 26

PHOTO 28: L0 – Wall in front of Stair - Hairline Cracking in Wall Lining around Door Opening

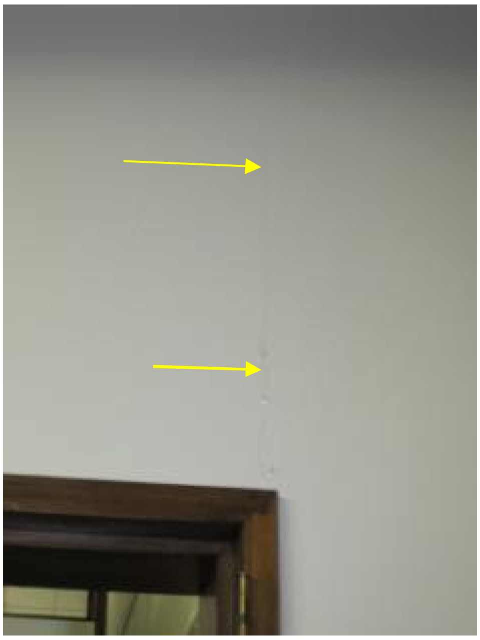


PHOTO 29: L0 – Close up of Photo 28

PHOTO 30: L0 – Vertical Hairline Crack in Concrete Wall near Kitchen

	
PHOTO 31: L0 - Close up of Photo 30	PHOTO 32: L0 – Close up of Photo 32

11. Appendix 2 – IEP Reports

Table IEP-1 **Initial Evaluation Procedure – Step 1**

(Refer Table IEP - 2 for Step 2; Table IEP - 3 for Step 3, Table IEP - 4 for Steps 4, 5 and 6)

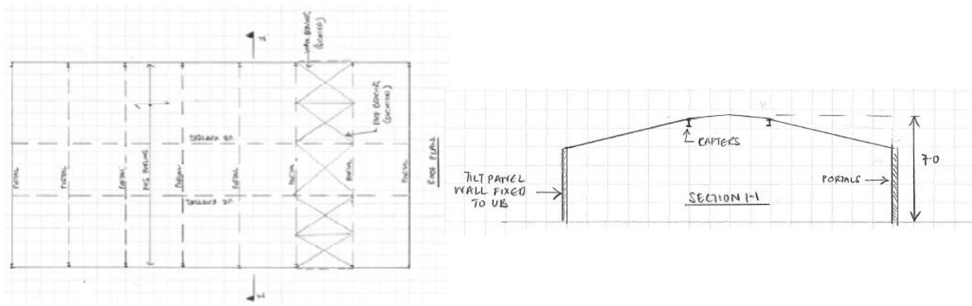
Page 1

Building Name:	Linwood Service Centre & Library Support - Building 2	Ref.	ZB01276.02
Location:	180 Smith Street, Woolston, Christchurch	By	KW
		Date	21/03/2012

Step 1 - General Information

1.1 Photos (attach sufficient to describe building)

1.2 Sketch of building plan



1.3 List relevant features

Building 2 is a single storey building with a mezzanine floor located in the northern end. The building is primarily used a library and offices. It is constructed from steel portal frames that span across the building. These portal frames are part of the overall gravity system and will also help to resist lateral loads acting in an east-west direction. The walls are predominately constructed from concrete tilt panels. There is steel rod braces present along the east and west walls of the building which will resist lateral loads acting in a north-south direction. The roof structure consists of light weight DHS purlins and light-weight profiled steel cladding. Steel tension braces are also present in the roof which will help transfer lateral loads into the wall braces. The mezzanine floor is constructed from timber framing and is accessed by one timber framed stair located at the south end of the floor. Garages are present along the eastern side of the building. These are constructed from concrete block and have a light-weight roof consisting of timber purlins which are supported on small steel trusses. These trusses are fixed to the main building. The building is believed to be supported on concrete pad foundations and strip footings under the concrete walls. The ground floor is a concrete slab on grade. Further investigation is required to confirm the foundation details.

1.4 Note information sources

	Tick as appropriate
Visual Inspection of Exterior	✓
Visual Inspection of Interior	✓
Drawings (note type)	
Specifications	
Geotechical Reports	
Other (list)	

Table IEP-2 Initial Evaluation Procedure – Step 2 Page 2

(Refer Table IEP - 1 for Step 1; Table IEP - 3 for Step 3, Table IEP - 4 for Steps 4, 5 and 6)

Building Name:	Linwood Service Centre & Library Support - Building 2	Ref.	ZB01276.02
Location:	180 Smith Street, Woolston, Christchurch	By	KW
Direction Considered:	**Longitudinal & Transverse**	Date	21/03/2012

(Choose worse case if clear at start. Complete IEP-2 and IEP-3 for each if in doubt)

Step 2 - Determination of (%NBS)b

2.1 Determine nominal (%NBS) = (%NBS)nom

Pre 1935			○	See also notes 1, 3
1935-1965			○	
1965-1976	Seismic Zone;	A	○	
		B	○	
		C	○	See also note 2
1976-1992	Seismic Zone;	A	○	
		B	◉	
		C	○	
1992-2004			○	

b) Soil Type

From NZS1170.5:2004, Cl 3.1.3

A or B Rock	○
C Shallow Soil	○
D Soft Soil	◉
E Very Soft Soil	○

From NZS4203:1992, Cl 4.6.2.2
(for 1992 to 2004 only and only if known)

a) Rigid	◉	N-A
b) Intermediate	○	

c) Estimate Period, T

building Ht = 7 meters

	Longitudinal	Transverse	
Ac =	35	20	m2

Can use following:

		Longitudinal	Transverse
$T = 0.09h_n^{0.75}$	for moment-resisting concrete frames	○ MRCF	○ MRCF
$T = 0.14h_n^{0.75}$	for moment-resisting steel frames	○ MRSF	◉ MRSF
$T = 0.08h_n^{0.75}$	for eccentrically braced steel frames	○ EBSF	○ EBSF
$T = 0.06h_n^{0.75}$	for all other frame structures	◉ Others	○ Others
$T = 0.09h_n^{0.75}/A_c^{0.5}$	for concrete shear walls	○ CSW	○ CSW
$T <= 0.4sec$	for masonry shear walls	○ MSW	○ MSW

Where hn = height in m from the base of the structure to the uppermost seismic weight or mass.
$Ac = \Sigma Ai(0.2 + Lwi/hn)2$
Ai = cross-sectional shear area of shear wall i in the first storey of the building, in m2
lwi = length of shear wall i in the first storey in the direction parallel to the applied forces, in m
with the restriction that lwi/hn shall not exceed 0.9

Longitudinal	Transverse	
0.3	0.6	Seconds

d) (%NBS)nom determined from Figure 3.3

Longitudinal	16.5	$(\%NBS)_{nom}$
Transverse	17.5	$(\%NBS)_{nom}$

		Factor
Note 1: For buildings designed prior to 1965 and known to be designed as public buildings in accordance with the code of the time, multiply (%NBS)nom by 1.25.	No	1
For buildings designed 1965 - 1976 and known to be designed as public buildings in accordance with the code of the time, multiply (%NBS)nom by 1.33 - Zone A or 1.2 - Zone B	No	1
Note 2: For reinforced concrete buildings designed between 1976 -1984 (%NBS)nom by 1.2	No	1
Note 3: For buildings designed prior to 1935 multiply (%NBS)nom by 0.8 except for Wellington where the factor may be taken as 1.	No	1

Longitudinal	16.5	$(\%NBS)_{nom}$
Transverse	17.5	$(\%NBS)_{nom}$

Continued over page

Table IEP-2 **Initial Evaluation Procedure – Step 2 continued**

Page 3

Building Name:	Linwood Service Centre & Library Support - Building 2	Ref.	ZB01276.02
Location:	180 Smith Street, Woolston, Christchurch	By	KW
Direction Considered:	**Longitudinal & Transverse**	Date	21/03/2012

(Choose worse case if clear at start. Complete IEP-2 and IEP-3 for each if in doubt)

2.2 Near Fault Scaling Factor, Factor A
If T < 1.5sec, Factor A = 1

a) **Near Fault Factor, N(T,D)** 1
(from NZS1170.5:2004, Cl 3.1.6)

b) **Near Fault Scaling Factor** = 1/N(T,D) **Factor A** 1.00

2.3 Hazard Scaling Factor, Factor B

Select Location: Christchurch

a) **Hazard Factor, Z, for site**
(from NZS1170.5:2004, Table 3.3) Z = 0.3
Z 1992 = 0.8

Auckland	0.6	Palm Nth	1.2
Wellington	1.2	Dunedin	0.6
Christchurch	0.8	Hamilton	0.67

b) **Hazard Scaling Factor**
For pre 1992 = 1/Z
\# For 1992 onwards = Z 1992/Z
(Where Z 1992 is the NZS4203:1992 Zone Factor from accompanying Figure 3.5(b))

Factor B 3.33

2.4 Return Period Scaling Factor, Factor C

a) **Building Importance Level** 2
(from NZS1170.0:2004, Table 3.1 and 3.2)

b) **Return Period Scaling Factor from accompanying Table 3.1** **Factor C** 1.00

2.5 Ductility Scaling Factor, D

a) **Assessed Ductility of Existing Structure, μ**
(shall be less than maximum given in accompanying Table 3.2)

Longitudinal	1	μ Maximum = 6
Transverse	2	μ Maximum = 6

b) **Ductility Scaling Factor**
For pre 1976 = k_μ
For 1976 onwards = 1
(where k_μ is NZS1170.5:2005 Ductility Factor, from accompanying Table 3.3)

Longitudinal	**Factor D**	1.00
Transverse	**Factor D**	1.00

2.6 Structural Performance Scaling Factor, Factor E

Select Material of Lateral Load Resisting System
Longitudinal Steel
Transverse Steel

a) **Structural Performance Factor, S_p**
from accompanying Figure 3.4

Longitudinal	Sp	0.90
Transverse	Sp	0.70

b) **Structural Performance Scaling Factor**

Longitudinal	$1/S_p$	**Factor E**	1.11
Transverse	$1/S_p$	**Factor E**	1.43

2.7 Baseline %NBS for Building, $(\%NBS)_b$
(equals $(\%NSB)_{nom}$ x A x B x C x D x E)

Longitudinal	61.1	(%NBS)b
Transverse	83.3	(%NBS)b

Table IEP-3 **Initial Evaluation Procedure – Step 3**
(Refer Table IEP - 1 for Step 1; Table IEP - 2 for Step 2, Table IEP - 4 for Steps 4, 5 and 6)

Page 4

Building Name:	Linwood Service Centre & Library Support - Building 2	Ref.	ZB01276.02
Location:	180 Smith Street, Woolston, Christchurch	By	KW
Direction Considered:	**a) Longitudinal**	Date	21/03/2012

(Choose worse case if clear at start. Complete IEP-2 and IEP-3 for each if in doubt)

Step 3 - Assessment of Performance Achievement Ratio (PAR)
(Refer Appendix B - Section B3.2)

Critical Structural Weakness	**Effect on Structural Performance** (Choose a value - Do not interpolate)				**Building Score**
	Severe	Significant	Insignificant		
3.1 Plan Irregularity Effect on Structural Performance	○	○	◉	**Factor A**	1
Comment					
3.2 Vertical Irregularity Effect on Structural Performance	○	○	◉	**Factor B**	1
Comment					
3.3 Short Columns Effect on Structural Performance	○	○	◉	**Factor C**	1
Comment					

3.4 Pounding Potential
(Estimate D1 and D2 and set D = the lower of the two, or =1.0 if no potential for pounding)

a) Factor D1: - Pounding Effect
Select appropriate value from Table

Note:
Values given assume the building has a frame structure. For stiff buildings (eg with shear walls), the effect of pounding may be reduced by taking the co-efficient to the right of the value applicable to frame buildings.

Factor D1 1

Table for Selection of Factor D1	Severe	Significant	Insignificant
Separation	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Alignment of Floors within 20% of Storey Height	○ 0.7	○ 0.8	◉ 1
Alignment of Floors not within 20% of Storey Height	○ 0.4	○ 0.7	○ 0.8

b) Factor D2: - Height Difference Effect
Select appropriate value from Table

Factor D2 1

Table for Selection of Factor D2	Severe	Significant	Insignificant
Separation	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Height Difference > 4 Storeys	○ 0.4	○ 0.7	○ 1
Height Difference 2 to 4 Storeys	○ 0.7	○ 0.9	○ 1
Height Difference < 2 Storeys	○ 1	○ 1	◉ 1

Factor D 1
(Set D = lesser of D1 and D2 or..
set D = 1.0 if no prospect of pounding)

3.5 Site Characteristics - (Stability, landslide threat, liquefaction etc)

Effect on Structural Performance	Severe	Significant	Insignificant		
	○ 0.5	○ 0.7	◉ 1	**Factor E**	1

3.6 Other Factors

For < 3 storeys - Maximum value 2.5,
otherwise - Maximum value 1.5. No minimum. **Factor F** 1

Record rationale for choice of Factor F:

3.7 Performance Achievement Ratio (PAR)
(equals A x B x C x D x E x F)

PAR 1

Table IEP-3 **Initial Evaluation Procedure – Step 3**

(Refer Table IEP - 1 for Step 1; Table IEP - 2 for Step 2, Table IEP - 4 for Steps 4, 5 and 6)

Page 5

Building Name:	Building Name:	Ref.	ZB01276.02
Location:	Location:	By	KW
Direction Considered:	**b) Transverse**	Date	21/03/2012

(Choose worse case if clear at start. Complete IEP-2 and IEP-3 for each if in doubt)

Step 3 - Assessment of Performance Achievement Ratio (PAR)
(Refer Appendix B - Section B3.2)

Critical Structural Weakness	**Effect on Structural Performance** (Choose a value - Do not interpolate)				**Building Score**
	Severe	Significant	Insignificant		
3.1 Plan Irregularity Effect on Structural Performance	○	○	◉	**Factor A**	1
Comment					
3.2 Vertical Irregularity Effect on Structural Performance	○	○	◉	**Factor B**	1
Comment					
3.3 Short Columns Effect on Structural Performance	○	○	◉	**Factor C**	1
Comment					

3.4 Pounding Potential

(Estimate D1 and D2 and set D = the lower of the two, or =1.0 if no potential for pounding)

a) Factor D1: - Pounding Effect
Select appropriate value from Table

Note:
Values given assume the building has a frame structure. For stiff buildings (eg with shear walls), the effect of pounding may be reduced by taking the co-efficient to the right of the value applicable to frame buildings.

Factor D1 1

Table for Selection of Factor D1	Severe	Significant	Insignificant
Separation	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Alignment of Floors within 20% of Storey Height	○ 0.7	○ 0.8	◉ 1
Alignment of Floors not within 20% of Storey Height	○ 0.4	○ 0.7	○ 0.8

b) Factor D2: - Height Difference Effect
Select appropriate value from Table

Factor D2 1

Table for Selection of Factor D2	Severe	Significant	Insignificant
Separation	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Height Difference > 4 Storeys	○ 0.4	○ 0.7	○ 1
Height Difference 2 to 4 Storeys	○ 0.7	○ 0.9	○ 1
Height Difference < 2 Storeys	○ 1	○ 1	◉ 1

Factor D 1

(Set D = lesser of D1 and D2 or..
set D = 1.0 if no prospect of pounding)

3.5 Site Characteristics - (Stability, landslide threat, liquefaction etc)

	Severe	Significant	Insignificant		
Effect on Structural Performance	○ 0.5	○ 0.7	◉ 1	**Factor E**	1

3.6 Other Factors

For < 3 storeys - Maximum value 2.5,

otherwise - Maximum value 1.5. No minimum. **Factor F** 1

Record rationale for choice of Factor F:

3.7 Performance Achievement Ratio (PAR)
(equals A x B x C x D x E x F)

PAR 1

Table IEP-4 **Initial Evaluation Procedure – Steps 4, 5 and 6**
(Refer Table IEP - 1 for Step 1; Table IEP - 2 for Step 2, Table IEP - 3 for Step 3)

Page 6

Building Name:	Linwood Service Centre & Library Support - Building 2	Ref.	ZB01276.02
Location:	180 Smith Street, Woolston, Christchurch	By	KW
Direction Considered:	**Longitudinal & Transverse**	Date	21/03/2012

(Choose worse case if clear at start. Complete IEP-2 and IEP-3 for each if in doubt)

Step 4 - Percentage of New Building Standard (%NBS)

	Longitudinal	Transverse
4.1 Assessed Baseline $(\%NBS)_b$ (from Table IEP - 1)	61	83
4.2 Performance Achievement Ratio (PAR) (from Table IEP - 2)	1.00	1.00
4.3 PAR x Baseline $(\%NBS)_b$	61	83
4.4 Percentage New Building Standard (%NBS) (Use lower of two values from Step 4.3)		61

Step 5 - Potentially Earthquake Prone?
(Mark as appropriate)

%NBS ≤ 33 — NO

Step 6 - Potentially Earthquake Risk?

%NBS < 67 — YES

Step 7 - Provisional Grading for Seismic Risk based on IEP

Seismic Grade — C

Evaluation Confirmed by

_______________ Signature

Trevor Robertson — Name

28892 — CPEng. No

Relationship between Seismic Grade and % NBS :

Grade:	A+	A	B	C	D	E
%NBS:	> 100	100 to 80	80 to 67	67 to 33	33 to 20	< 20

12. Appendix 3 – CERA Standardised Report Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Linwood Library Support Services	Reviewer:	Trevor Robertson
	Unit / No: / Street	CPEng No:	28892
Building Address:	180 Smith Street	Company:	SKM
Legal Description:		Company project number:	ZB01276.003
		Company phone number:	09 940 4900
	Degrees / Min / Sec		
GPS south:		Date of submission:	
GPS east:		Inspection Date:	6/03/2012 & 05/12
		Revision:	B
Building Unique Identifier (CCC):	PRO 0822 B002	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	mixed	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):	

Building

No. of storeys above ground:	2	single storey = 1	
		Ground floor elevation (Absolute) (m):	0.00
Ground floor split?	no	Ground floor elevation above ground (m):	0.00
Storeys below ground	0		
Foundation type:	other (describe)	if Foundation type is other, describe:	Pad foundations have been assumed under the portal legs, however it is likely that strip footings will be present under the concrete walls.
Building height (m):	7.00	height from ground to level of uppermost seismic mass (for IEP only) (m):	7.00
Floor footprint area (approx):			
Age of Building (years):	30	Date of design:	1976-1992
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	commercial	Brief strengthening description:	
Use (upper floors):	commercial		
Use notes (if required):			
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	frame system		
Roof:	steel framed	rafter type, purlin type and cladding	Steel 460 UB portal frames supporting 200 DHS purlins and a light weight profiled steel cladding
Floors:	timber	joist depth and spacing (mm)	290x45 at 500crs
Beams:	steel non-composite	beam and connector type	310 UB 40
Columns:	structural steel	typical dimensions (mm x mm)	460 DP x 190 W
Walls:	non-load bearing		0

Lateral load resisting structure

Lateral system along:	single level tilt panel	Note: Define along and across in detailed report!	note total length of wall at ground (m):	5.7
Ductility assumed, μ:	1.00		wall thickness (m):	0.130m and 0.170m
Period along:	0.30	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):	25		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):	0		estimate or calculation?	estimated
Lateral system across:	welded and bolted steel moment frame		note typical bay length (m)	27
Ductility assumed, μ:	1.25			
Period across:	0.60	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):	115		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):	0		estimate or calculation?	estimated

Separations:

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

Non-structural elements

Stairs:	timber	describe supports	
Wall cladding:	precast panels	thickness and fixing type	130mm and 170mm - welded and bolted to UB cleats
Roof Cladding:	Metal	describe	Light weight profiled steel cladding
Glazing:	aluminium frames		
Ceilings:	light tiles		
Services(list):	Air conditioning, lighting, sprinklers.		

Available documentation

Architectural	partial	original designer name/date	For mezzanine floor only. No structural details shown
Structural	none	original designer name/date	
Mechanical	none	original designer name/date	
Electrical	none	original designer name/date	
Geotech report	partial	original designer name/date	Desktop study by SKM dated 26 March 2012

Damage

Site:
(refer DEE Table 4-2)

Site performance:	1	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:	yellow		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	damage observed does not deminish the capacity of the structure
	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:	

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

Recommendations

	Level of repair/strengthening required:	minor structural	Describe:	
	Building Consent required:	no	Describe:	
	Interim occupancy recommendations:	partial occupancy	Describe:	limited occupancy to store room where shelving is damaged
Along	Assessed %NBS before:	39%	%NBS from IEP	
	Assessed %NBS after:	39%	If IEP not used, please detail assessment methodology:	Quantitative Assessment carried out (refer to SKM report)
Across	Assessed %NBS before:	41%	%NBS from IEP	
	Assessed %NBS after:	41%		

Official Use only:

Accepted By	
Date:	

13. Appendix 4 – Geotechnical Desk Study

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Christchurch City Council - Structural Engineering Service

Geotechnical Desk Study

SKM project number	ZB01276
SKM project site number	002 to 005 inclusive
Address	Linwood Resource Centre and Library, 180 Smith Street and 332 Linwood Ave
Report date	26 March 2012
Author	Ross Roberts / Ananth Balachandra
Reviewer	Leah Bateman
Approved for issue	Yes

1. Introduction

This letter 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 assessment 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.

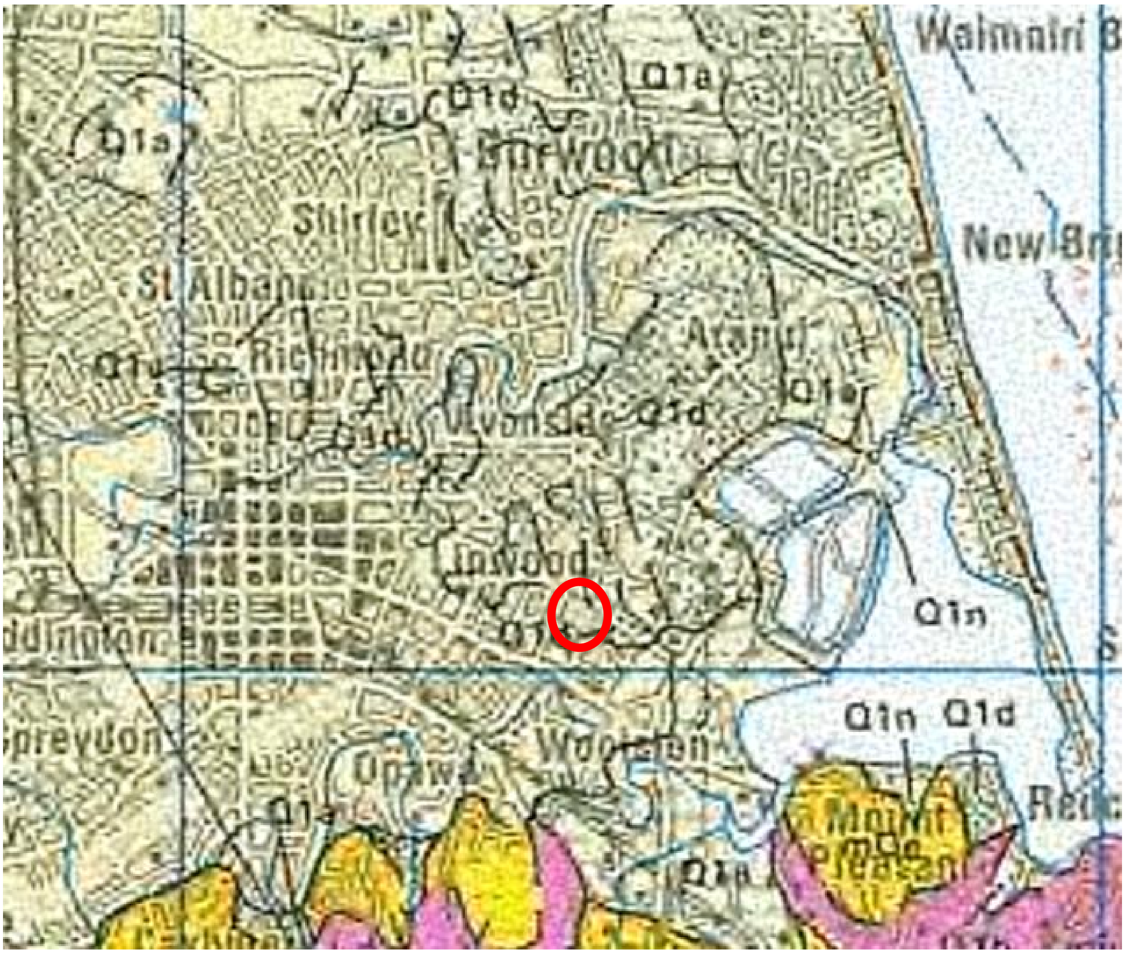
4. Site location

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

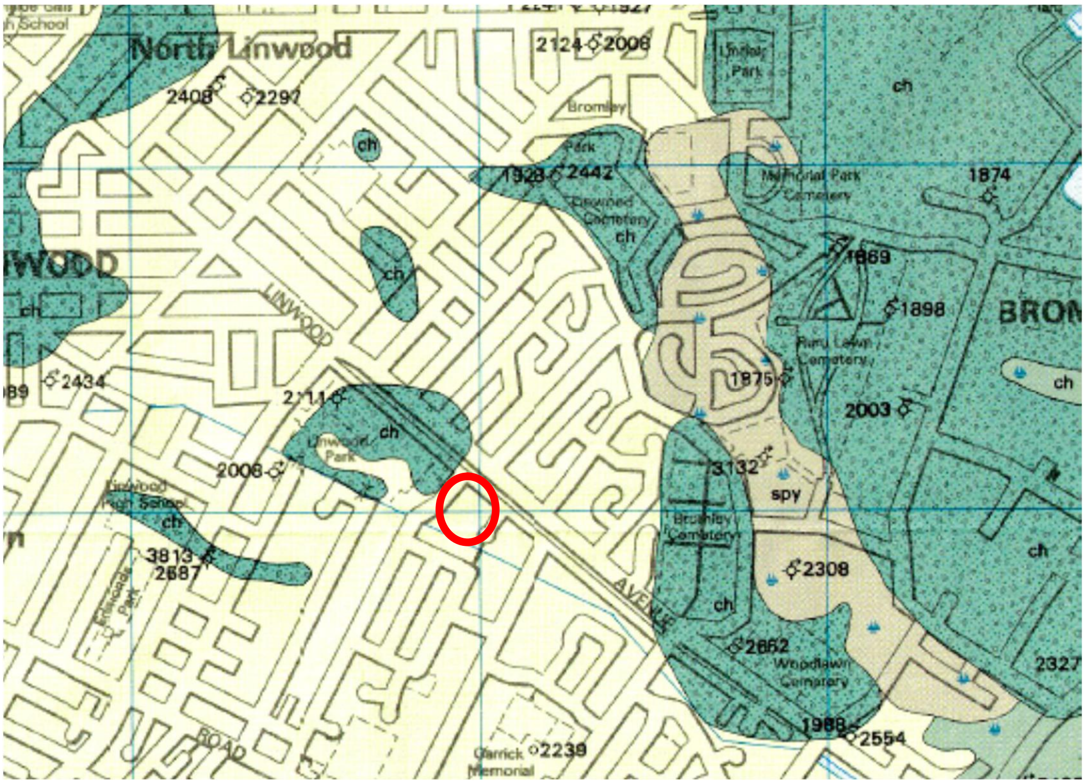
These structures are located on the corner of Linwood Avenue and Smith Street at grid reference 1573957 E, 5179440 N (NZTM).

5. Review of available information

5.1 Geological maps



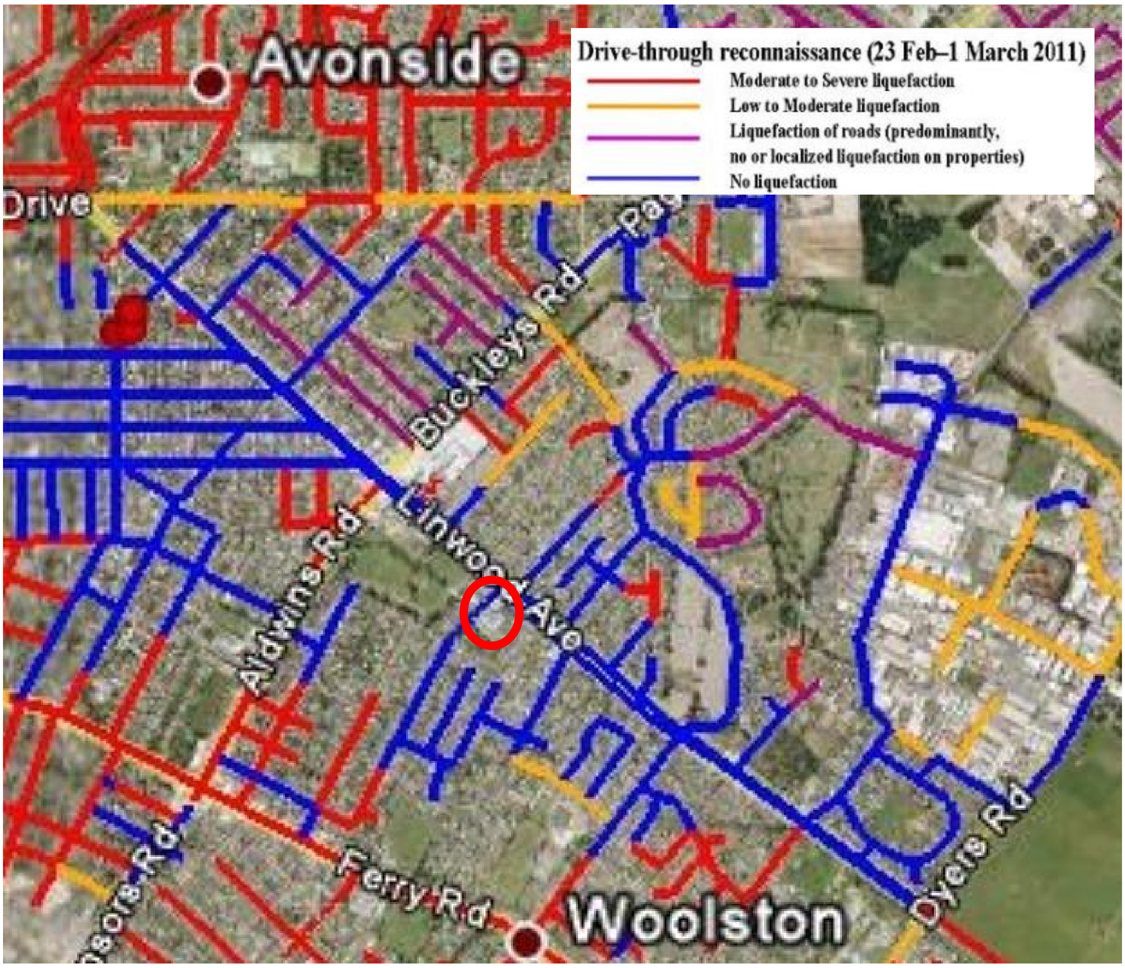
- **Figure 2 – Regional geological map (Forsyth et al, 2008). Site marked in red.**



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

The site is shown to be underlain by Holocene deposits comprising predominantly alluvial sand and silt overbank deposits of the Springston Formation. Immediately to the north west lies an area of Christchurch Formation sand of fixed dunes.

5.2 Liquefaction map



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

Following the 22 February 2011 event drive through reconnaissance was undertaken from 23 Feb until 1 Mar 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/)**

Aerial photography shows relatively little damage after the 22 Feb 2011 event. There appears to be a burst water main on Linwood Avenue, and what may be a single source of liquefied material in the tennis courts to the north west of the property. This coincides with a change of geology as identified on the geological maps.

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) – adjacent properties are TC2

5.5 Historical land use

Reference to historical documents (eg Appendix A) shows that the site lies immediately south and east of land that was recorded as marshland or swamp in 1856. It is therefore likely that soft or liquefiable ground would be present near the site. Given the relatively low accuracy of these historical documents, it should be considered possible that old swamp deposits are present on the site.

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

The available council records were limited to building consents applied for the demolition of existing garage and shed and reconstruction of a double door garage and other documents relating to the above construction. The Council foundation record identifies top soil or sandy clay to be present to a depth of 0.5 m and medium sand from a depth of 0.5 m to 3.8 m for the site. No ground investigation for depth greater than 3.8 m was found in the council property files. The ground water table was estimated to be between 1.6 m to 3 m. The council record identifies an allowable bearing pressure of 200 kPa for the sand layer with comments stating that the identified allowable bearing pressure is adequate for the proposed buildings.

Drawings for the utility shed showed 500 mm square pad footings at a depth of 1 m. Drawings for the new garage at 180 Smith Street show a 100 mm thick reinforced raft foundation with edge thickening to 150 mm. Piles are shown inconsistently in the record. One drawing identifies 250 mm diameter piles are shown at each corner and at 2 m centres along the edge of the slab (depth not recorded). Another shows 150 mm 'piles' to 400 mm depth at 1.2 m spacing.

The council property files identify possible contamination under the "old work shop area" due to the presence of two tanks containing flammable liquid, which have since been removed.

5.8 Site walkover

An engineer from SKM undertook a site walkover in the week commencing 12 March 2012.

The Linwood Resource Centre and Library were mostly constructed using masonry block with an iron roof, Figure 7 shows the overview of the site. The buildings were both in good condition, with no external evidence of structural damage. The majority of the land on the site was asphalt, which showed no signs of land damage. There was no evidence that liquefaction occurred on the site.

Residents report that the only damage occurred to the footpaths, with paving slabs being displaced,(Figure 8) and that no liquefaction occurred on the site.


ORARY
VOOD
RARY
OPEN

- **Figure 7 Overview of Linwood Services Centre and Library**

- **Figure 8 Damaged paving slabs at Linwood Resource Centre**

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 (mBLG)	Soil type
0 - 1	Sensitive fine grained soils (clay or silt)
1 - 8	Very stiff clays and loose to dense clayey sand
8 – 19	Dense sand
19 – 21	Interbedded clay and silt
21 - 23	Dense sand
23 +	Soft to firm clay or silt

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 second preferred method has been used to make the assessment utilising records from sites at least 50 m from the site. It is therefore possible that site specific investigation could revise the site class.

6.3 Building performance

Although detailed records of the existing foundations are not available, the performance to date suggests that they are adequate for their current purpose.

6.4 Ground performance and properties

It is expected that the allowable bearing capacity of a shallow pad footing on this site will be in the region of 200 kPa, as stated in the council records and supported by the findings of the nearby ground investigations. We estimate a conservative ultimate bearing capacity to be in the order of 400 kPa. However, these may be revised by a site specific investigation.

For the purposes of shallow foundation design, the following parameters are recommended for the near surface clayey sand:

- Effective angle of friction = 35 degrees
- Apparent cohesion = 1 kPa
- Unit weight = 18 kPa

Liquefaction risk is low at this site.

6.5 Further investigations

Unless a change of use is intended for the site we do not believe that any further geotechnical investigations are required. Specific ground investigation should be undertaken if significant alterations or new structures are proposed. If any excavations are required on the site further investigation of the potential for contamination should be undertaken.

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.

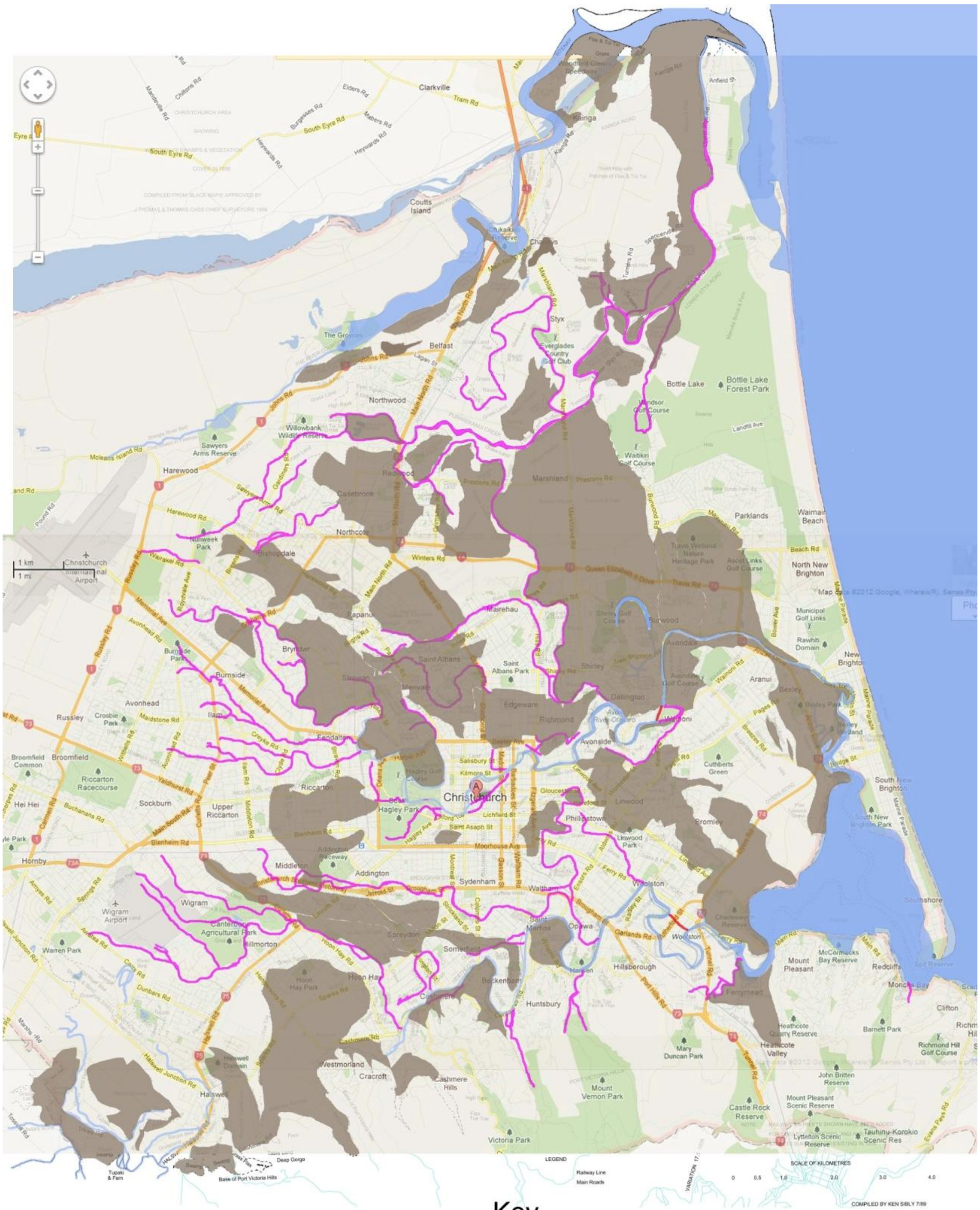
Cubrinovski & Taylor, 2011. Liquefaction map summarising preliminary assessment of liquefaction in urban areas following the 2010 Darfield Earthquake.

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/)

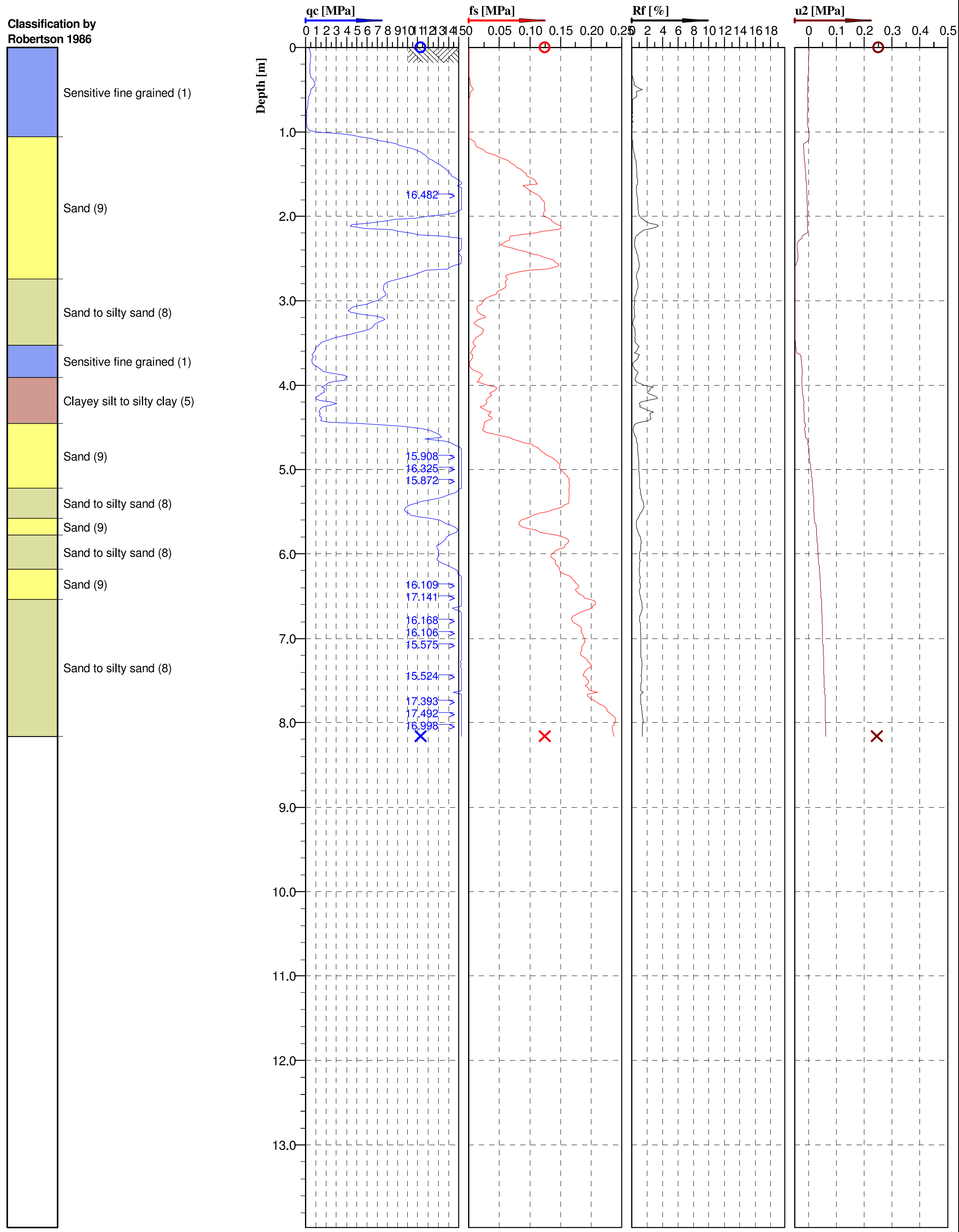
Appendix A – Christchurch 1856 land use



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

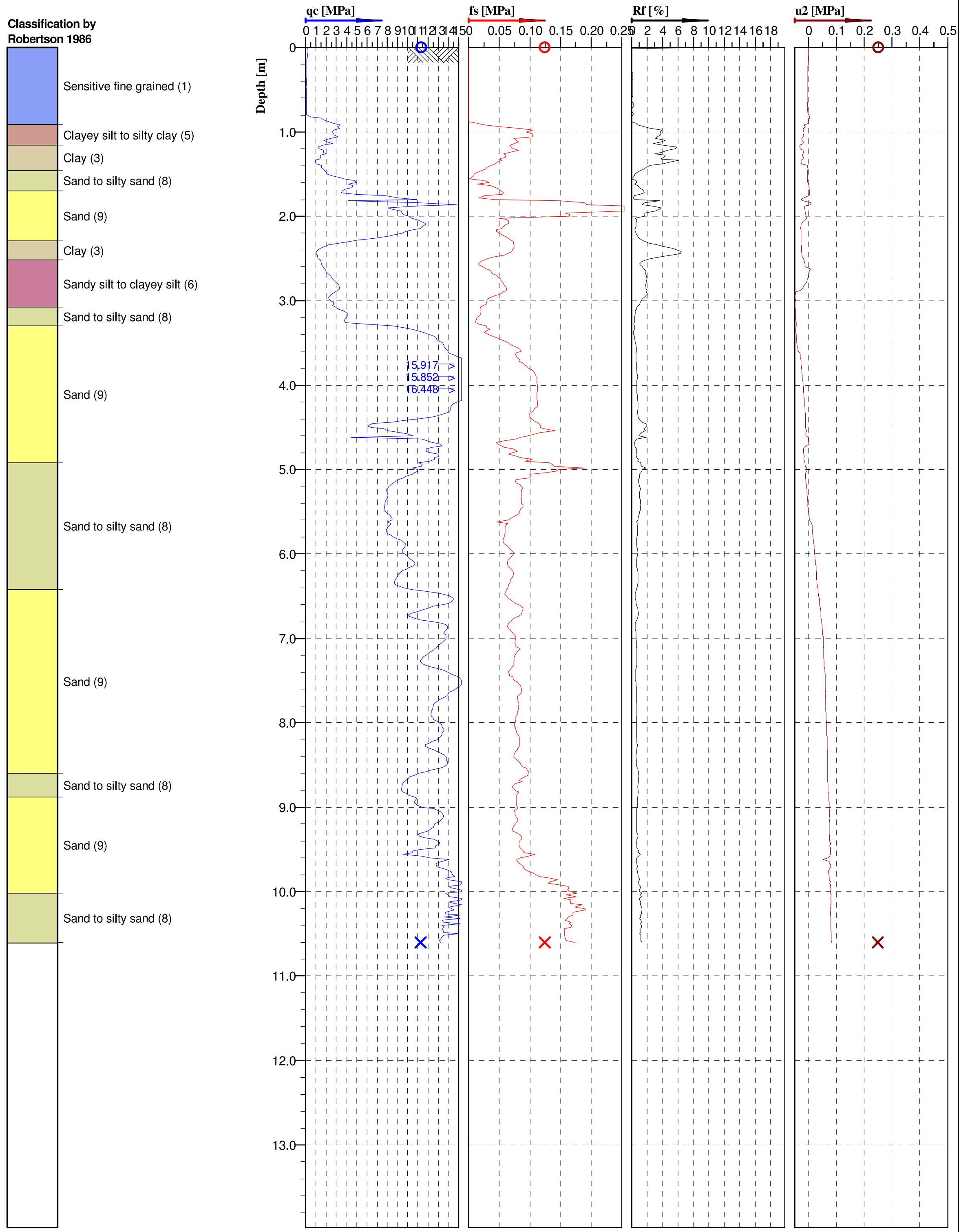
Key
- Previous creeks/rivers
- Existing creeks/rivers
- New creeks/rivers
- Swamp/Marshland

Appendix B – Existing ground investigation logs



Cone No: 100KN 4341
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: LINWOOD	Position: X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no: CPT-LWD-35
Project ID:	Client: TONKIN & TAYLOR LTD	Date: 20/05/2011	Scale: 1 : 60
Project: EQC SITES		Page: 1/1	Fig:
		File: CPT-LWD-35.CPT	

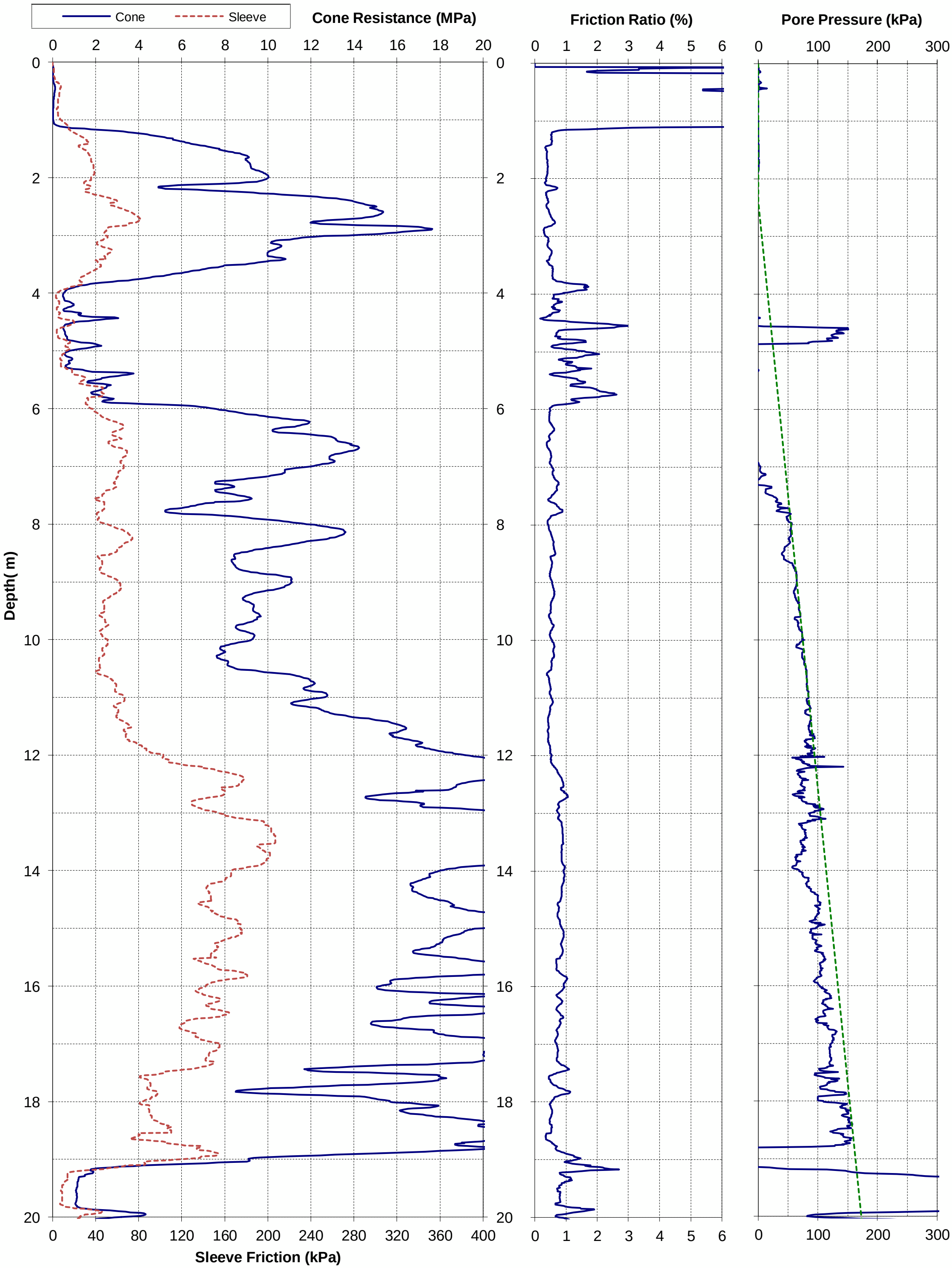


U2

Cone No: 100KN 4341
Tip area [cm2]: 10
Sleeve area [cm2]: 150

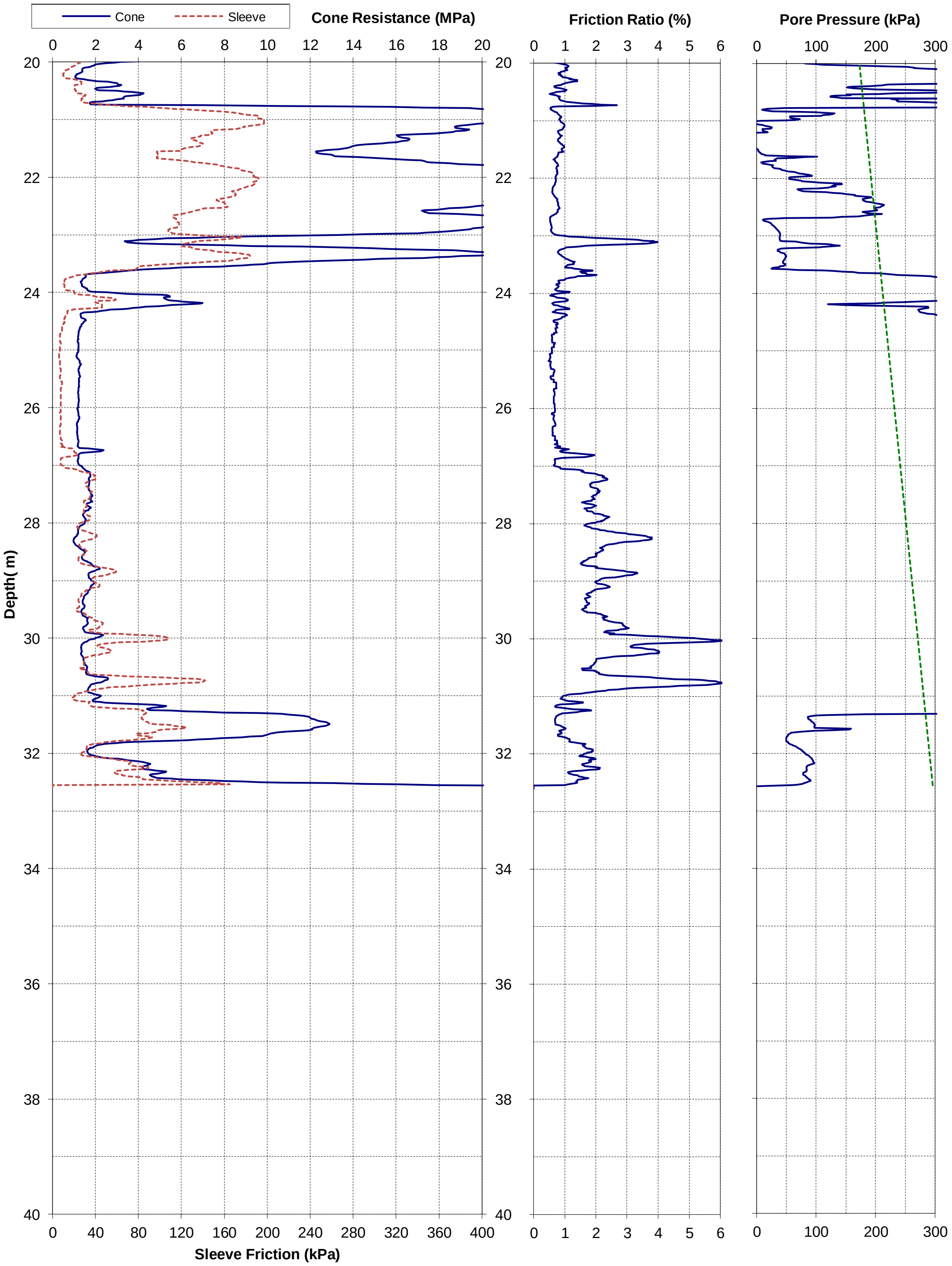
Location: LINWOOD	Position: X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no: CPT-LWD-34
Project ID:	Client: TONKIN & TAYLOR LTD	Date: 20/05/2011	Scale: 1 : 60
Project: EQC SITES		Page: 1/1	Fig:
		File: CPT-LWD-34.CPT	

Project:	Christchurch 2011 Earthquake - EQC Ground Investigations			Page: 1 of 2		CPT-BRY-18
Test Date:	17-Jun-2011	Location:	Bromley	Operator:	Opus	
Pre-Drill:	1.2m	Assumed GWL:	2.4mBGL	Located By:	Survey GPS	
Position:	2484276.9mE	5741214.3mN	2.39mRL	Coord. System:	NZMG & MSL	
Other Tests:				Comments:		



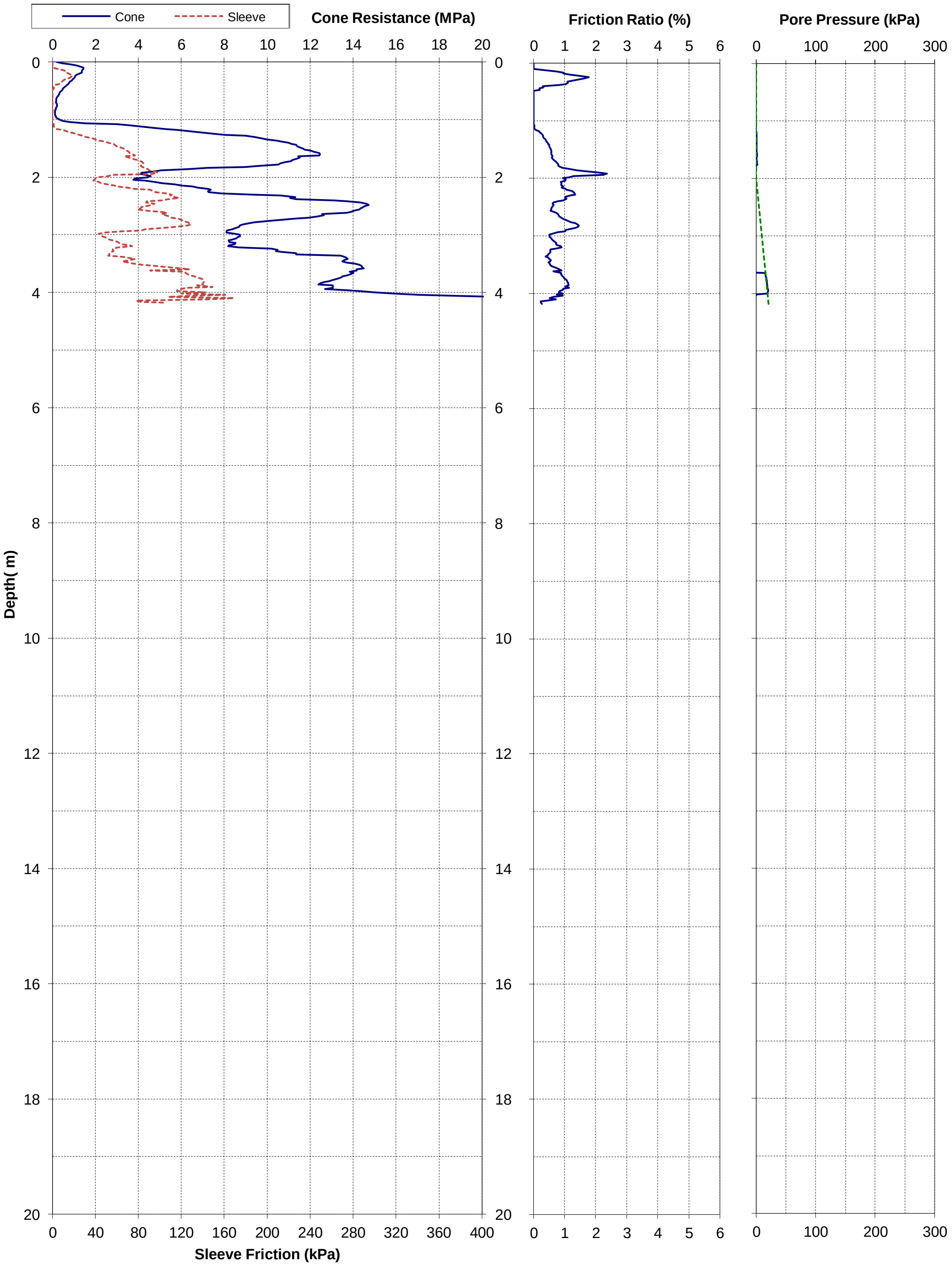
T+T Ref: 52000.3000 Printed: 7/10/2011 3:15 p.m. Template: CPT Graph Template v0.41.xls

Project:	Christchurch 2011 Earthquake - EQC Ground Investigations			Page: 2 of 2		CPT-BRY-18
Test Date:	17-Jun-2011	Location:	Bromley	Operator:	Opus	
Pre-Drill:	1.2m	Assumed GWL:	2.4mBGL	Located By:	Survey GPS	
Position:	2484276.9mE	5741214.3mN	2.39mRL	Coord. System:	NZMG & MSL	
Other Tests:				Comments:		



T+T Ref: 52000.3000
Printed: 7/10/2011 3:15 p.m.
Template: CPT Graph Template v0.41.xls

Project:	Christchurch 2011 Earthquake - EQC Ground Investigations			**Page:** 1 of 1		CPT-LWD-28
Test Date:	19-May-2011	**Location:**	Linwood	**Operator:**	Geotech	
Pre-Drill:	1.2m	**Assumed GWL:**	2mBGL	**Located By:**	Survey GPS	
Position:	2483547.1mE	5741248.6mN	2.73mRL	**Coord. System:**	NZMG & MSL	
Other Tests:				**Comments:**		



T+T Ref: 52000.3000 Printed: 7/10/2011 10:50 a.m. Template: CPT Graph Template v0.41.xls

Borelog for well M35/2111

Gridref: M35:836-414 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 3.1 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -66.4m Drill Date : 8/12/1944

Water Level: Artesian

Depth(m)	Full Drillers Description	Formation Code
-9.10m	Sand & clay	sp?
- 19.8m	Sand & gravel	sp?
- 26.8m	Blue sand & clay	ch
- 32.3m	Blue clay & peat	ch
- 40.5m	Blue shingle	ri
- 53.6m	Brown shingle	ri
- 53.9m	Blue sand	br
- 58.8m	Yellow clay	br
- 63.0m	Brown sand	br
- 66.4m	Brown shingle	li-1

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

Appendix C – Geotechnical investigation summary

ID	1	2	3	4	5
Type *	CPT	CPT	CPT	CPT	BH
Ref	LWD-35	LWD-34	BRY-18	LWD-28	M35-2111
Depth (m)	8	11	32	32	66
Distance from site (m)	100	200	375	450	500
Ground water level (mBGL)	4	3	2.5	2	Artesian
Simplified recorded geological profile (depth below ground level to top of stratum, m) 0	N/A	N/A		N/A	
1				MD	
2				MD	
3				MD	
4				So	
5				So	
6				MD	
7				MD	
8				MD	
9				MD	
10				MD	
11				D	
12				D	
13				D	
14				D	
15				D	
16				D	
17				D	
18				D	
19				F	
20				F	
21				D	
22				D	
23				St	
24				St	
25				St	
Greater depths			Clay to 31 m	Clay to 32 m	

*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

14. Appendix 5 – Calculations Philosophy

No existing drawings were available. Calculations were based on Survey Plans drawn by SKM and site photos.

14.1. Project description

Building is a single storey portal frame building with concrete panels. The building has a mezzanine floor on the north eastern side and an attachment on the south eastern side. The building is used as a library and an office

14.2. Site location, street, city, country

180 Smith St, Linwood, Christchurch, New Zealand

14.3. Type of project

Quantitative Assessment on existing building

14.4. Environmental conditions – durability requirements

Is outside of the scope

14.5. Site constraints

No buildings in close vicinity

14.6. Movement Joints

Were not found in the building

14.7. Construction materials

Portal Frame Steel = 460UB.

Type of 460UB could not be confirmed on site. 460UB67 has been assumed. f_y = 300Mpa

Purlins = 200DHS

Type of 200DHS has been assumed to be DHS200/12, properties from Dimond have been assumed.

Roof Bracing = 20mm diameter, Assume f_y = 250Mpa

Bolts in apex = Assume 4.6/S f_{uf}=400 MPa

Bolts in shearwall = Assume M12 bolt epoxied into concrete wall with Ramset Chemset Injection 101 Plus

Concrete = 30 Mpa, γ= 24kN/m^3

14.8. Bracing strategy

14.8.1. Along

Lateral loads from the roof are taken by the purlins to roof bracing to steel columns then to the concrete panel walls. No foundation detail has been provided for the concrete walls so have assumed the walls resist the load by gravity only.

14.8.2. Across

Lateral loads from the roof are taken by the purlins to the steel portal frames. No foundation details have been provided for the column

14.9. Ground conditions

A class D type soil has been used in the analysis.

14.10. Loading assumptions

50 year design life, soil type D, z=0.3.

14.10.1. Along direction

T=0.3seconds, μ=1, this has been used as the concrete shear walls will provide elastic ductility

14.10.2. Across direction

T=0.6seconds, μ=1.25, steel portal frame has been assumed to be nominally ductile. A higher ductility may be possible but there are limited details available to confirm this.

14.11. Fire resistance

Is outside of the scope

14.12. Codes used

Loadings NZS1170, Steel Structures NZS3404, Timber Framed Buildings NZ3604, NZSEE "Assessment and Improvement of the structural performance of Buildings in Earthquakes", Concrete Structures NZS 3101

14.13. Deflection limits

Has been assumed to be within acceptable limits based on the low ductility of the structure

14.14. Other Assumptions

Shear wall connections

Not all of the shear wall to steel column connections could be inspected. From the ones that were inspected it has been assumed all the connections on the on the south side consist of cleats welded to the columns with bolts connecting the concrete wall to the cleats.

For the connections on the north side it has been assumed all the connections consist of cleats welded to the columns with the cleats welded to the concrete walls.

5mm welds have been assumed.

Shear wall

No panel joints were found on the south wall it has been assumed that the wall was cast in-situ.

Floor Diaphragm

It is assumed the particle board and gib ceiling is adequately connected to the timber joists to provide diaphragm action.

Concrete Panels

Details of how the concrete panels are connected to the foundation have not been provided. An assumption has been that the walls in the longitudinal direction do not have a connection at the bottom so that when they act out of plane they will span between the steel columns, when acting in-plane they will resist loads by rocking.

It has been assumed that the end walls have a shear connection to resist loads in the out of plane direction.

14.15. Loadings

14.15.1. Dead loads

Dead loads have been evaluated using NZS1170.1

14.15.2. Live Loads

Roof live load has been taken as $0.25kN/m^3$ and live load on the mezzanine floor has been taken as $3kN/m^3$ for office.

14.15.3. Wind Loads

Wind loads have not been considered as it is outside of the scope of a quantitative assessment.

14.15.4. Snow and Ice loads

Snow and ice loads have not been considered as it is outside the scope of a quantitative assessment.

14.15.5. Earthquake Loads

Earthquake load case $G+E_u+\psi_E Q$ has been considered in these calculations. The horizontal design action coefficient has been evaluated in the along direction $C_d(T) = 0.9$, across direction $C_d(T) = 0.63$.

14.15.6. Temperature Loads

Temperature loading was not considered critical for the quantitative assessment.