Bishopdale Community Crèche

Farrington Avenue

Bishopdale

Christchurch

Detailed Engineering Evaluation

Quantitative Assessment Report

Bishopdale Community Crèche
Farrington Avenue
Bishopdale,
Christchurch

Detailed Engineering Evaluation

Quantitative Assessment Report

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Detailed Engineering Evaluation
Quantitative Report - SUMMARY
Version 1

Bishopdale Community Crèche, Christchurch

Background
This is a summary of the quantitative report for the building structure, and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections, selective field investigations, and available drawings.

Key Damage Observed
Key damage observed includes:-

1. In the art storage room there is a horizontal crack on the external (south east) wall at about 2.0m high. This is marking out the position of an old door opening which has been blocked up in the past.

2. On one blockwork column supporting the canopy to the northwest side there is a crack through the block at low level. However, observing the crack closely shows that there is paint inside the crack from when the column was last repainted. We were informed that the paint was "several years ago" and maybe even up to 10 years. The crack therefore pre-dates any of the recent seismic events.

Critical Structural Weaknesses
The main structural weakness is the lack of adequate a complete roof diaphragm and load path to the concrete block shear walls. Factors limiting the %NBS of the building are summarized below:

1. The seismic performance of the primary components (those that are required parts of the lateral resisting system) are governed by the lack of complete roof diaphragm and shear transfer to concrete block walls below. The computed strength of the timber walls and mansard roof is less than 33% NBS (around 20% NBS).

2. The seismic performance of the secondary components (those that are not required parts of the lateral load resisting system but which must be able to maintain their gravity load capacity while the building undergoes deformation due to earthquake loading) are governed by:

    a. At the northwest elevation, the steel truss connections are not adequate to resist the out-of-plane loading from the block wall. The failure mode is flexure of the steel connector plates. Although the failure is a ductile type failure, given the consequence of failure is that roof trusses may lose their support, this condition should be addressed.

    b. Unreinforced block columns provide gravity support for the veranda canopy. The columns do not have adequate capacity to resist calculated lateral load.

    c. At the southwest and portion of the southeast elevation, the cavity walls do not have adequate strength to resist out-of-plane forces resulting in approximately 13% NBS. Although failure of this wall will not lead to collapse, the wall is approximately 2.4m high and failure could lead to injury to occupants or pedestrians near the wall.

Indicative Building Strength (from quantitative assessment)
Based on the information available, and from undertaking a quantitative assessment, the building's original capacity has been assessed to be less than 33%NBS and post-earthquake capacity is less than 33%NBS. The building is therefore classified as an earthquake prone building.

Recommendations
We recommend that further work is undertaken in order to develop the scope of the strengthening and repair options. This work should involve:

1. Developing a strengthening works scheme to increase the seismic capacity of the building to as near as practicable to 100%NBS, and at least 67%NBS. This will need to consider compliance with accessibility and fire requirements. It may be beneficial to engage a quantity surveyor to consider costs for strengthening options.

2. It is recommended that the building not be occupied, given its earthquake prone building status and the elevated level of seismic risk in Christchurch.

Contents

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council (CCC) to undertake a detailed seismic assessment of the Bishopdale Community Crèche, located at Bishopdale Mall, 129 Farrington Avenue, Christchurch following the M6.3 Christchurch earthquake on 22 February 2011.

The purpose of the assessment is to determine if the building is classed as being earthquake prone in accordance with the Building Act 2004.

The seismic assessment and reporting have been undertaken based on the qualitative and quantitative procedures detailed in the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011.

2 Compliance

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

2.1 Canterbury Earthquake Recovery Authority (CERA)

CERA was established on 28 March 2011 to take control of the recovery of Christchurch using powers established by the Canterbury Earthquake Recovery Act enacted on 18 April 2011. This act gives the Chief Executive Officer of CERA wide powers in relation to building safety, demolition and repair. Two relevant sections are:

Section 38 – Works

This section outlines a process in which the chief executive can give notice that a building is to be demolished and if the owner does not carry out the demolition, the chief executive can commission the demolition and recover the costs from the owner or by placing a charge on the owners' land.

Section 51 – Requiring Structural Survey

This section enables the chief executive to require a building owner, insurer or mortgagee to carry out a full structural survey before the building is re-occupied.

We understand that CERA 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). CERA have adopted the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011. This document sets out a methodology for both initial qualitative and detailed quantitative assessments.

It is anticipated that a number of factors, including the following, will determine the extent of evaluation and strengthening level required:

1. The importance level and occupancy of the building.
2. The placard status and amount of damage.
3. The age and structural type of the building.
4. Consideration of any critical structural weaknesses.

Any building with a capacity of less than 33% of new building standard (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67% as required by the CCC Earthquake Prone Building Policy.

2.2 Building Act

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

Section 112 - Alterations

This section requires that an existing building complies with the relevant sections of the Building Code to at least the extent that it did prior to the alteration.

This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

Section 115 – Change of Use

This section requires that the territorial authority (in this case Christchurch City Council (CCC)) is satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'.

This is typically interpreted by CCC as being 67% of the strength of an equivalent new building. This is also the minimum level recommended by the New Zealand Society for Earthquake Engineering (NZSEE).

Section 121 – Dangerous Buildings

This section was extended by the Canterbury Earthquake (Building Act) Order 2010, and defines a building as dangerous if:

1. 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
2. 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
3. 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
4. There is a risk that other property could collapse or otherwise cause injury or death; or
5. A territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

Section 122 – Earthquake Prone Buildings

This section defines a building as earthquake prone (EPB) 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 loads 33% of those used to design an equivalent new building.

Section 124 – Powers of Territorial Authorities

This section gives the territorial authority the power to require strengthening work within specified timeframes or to close and prevent occupancy to any building defined as dangerous or earthquake prone.

Section 131 – Earthquake Prone Building Policy

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

2.3 Christchurch City Council Policy

Christchurch City Council adopted their Earthquake Prone, Dangerous and Insanitary Building Policy in 2006. This policy was amended immediately following the Darfield Earthquake on 4 September 2010.

The 2010 amendment includes the following:

1. A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
2. A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;
3. A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
4. 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.

If strengthening works are undertaken, a building consent will be required. A requirement of the consent will require upgrade of the building to comply 'as near as is reasonably practicable' with:

- The accessibility requirements of the Building Code.
- The fire requirements of the Building Code. This is likely to require a fire report to be submitted with the building consent application.

2.4 Building Code

The Building Code outlines performance standards for buildings and the Building Act requires that all new buildings comply with this code. Compliance Documents published by The Department of Building and Housing can be used to demonstrate compliance with the Building Code.

On 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

- 36% increase in the basic seismic design load for Christchurch (Z factor increased from 0.22 to 0.3);
- Increased serviceability requirements.

2.5 Institution of Professional Engineers New Zealand (IPENZ) Code of Ethics

One of the core ethical values of professional engineers in New Zealand is the protection of life and safeguarding of people. The IPENZ Code of Ethics requires that:

Members shall recognise the need to protect life and to safeguard people, and in their engineering activities shall act to address this need.

1.1 Giving Priority to the safety and well-being of the community and having regard to this principle in assessing obligations to clients, employers and colleagues.

1.2 Ensuring that responsible steps are taken to minimise the risk of loss of life, injury or suffering which may result from your engineering activities, either directly or indirectly.

All recommendations on building occupancy and access must be made with these fundamental obligations in mind.

3 Earthquake Resistance Standards

For this assessment, the building's earthquake resistance is compared with the current New Zealand Building Code requirements for a new building constructed on the site. This is expressed as a percentage of new building standard (%NBS). The loadings are in accordance with the current earthquake loading standard NZS1170.5 [1].

A generally accepted classification of earthquake risk for existing buildings in terms of %NBS that has been proposed by the NZSEE 2006 [2] is presented in Figure 1 below.

Description	Grade	Risk	%NBS	Existing Building Structural Performance	Improvement of Structural Performance	
					Legal Requirement	NZSEE Recommendation
Low Risk Building	A or B	Low	Above 67	Acceptable (improvement may be desirable)	The Building Act sets no required level of structural improvement (unless change in use) This is for each TA to decide. Improvement is not limited to 34%NBS.	100%NBS desirable. Improvement should achieve at least 67%NBS
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally. Improvement recommended		Not recommended. Acceptable only in exceptional circumstances
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement required under Act)	Unacceptable	Unacceptable

Figure 1: 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

3.1 Minimum and Recommended Standards

Based on governing policy and recent observations, Opus makes the following general recommendations:

3.1.1 Occupancy

The Canterbury Earthquake Order in Council 16 September 2010, modified the meaning of “dangerous building” to include buildings that were identified as being EPB’s. As a result of this, we would expect such a building would be issued with a Section 124 notice, by the Territorial Authority, or CERA acting on their behalf, once they are made aware of our assessment. Based on information received from CERA to date, this notice is likely to prohibit occupancy of the building (or parts thereof), until its seismic capacity is improved to the point that it is no longer considered an EPB.

3.1.2 Cordoning

Where there is an overhead falling hazard, or potential collapse hazard of the building, the areas of concern should be cordoned off in accordance with current CERA/Christchurch City Council guidelines.

3.1.3 Strengthening

Industry guidelines (NZSEE 2006 [2]) strongly recommend that every effort be made to achieve improvement to at least 67%NBS. A strengthening solution to anything less than 67%NBS would not provide an adequate reduction to the level of risk.

It should be noted that full compliance with the current building code requires building strength of 100%NBS.

3.1.4 Our Ethical Obligation

In accordance with the IPENZ code of ethics, we have a duty of care to the public. This obligation requires us to identify and inform CERA of potentially dangerous buildings; this would include earthquake prone buildings.

4 Background Information

4.1 Building Description

The Bishopdale Community Crèche, located in Bishopdale Mall on Farrington Ave, is a single storey structure designed in 1974. The building was extended towards the northeast side sometime prior to 1991 and a veranda canopy was added to the northwest elevation. For the purpose of this report, the longitudinal direction runs northwest to southeast. Refer to site plan in Figure 2.

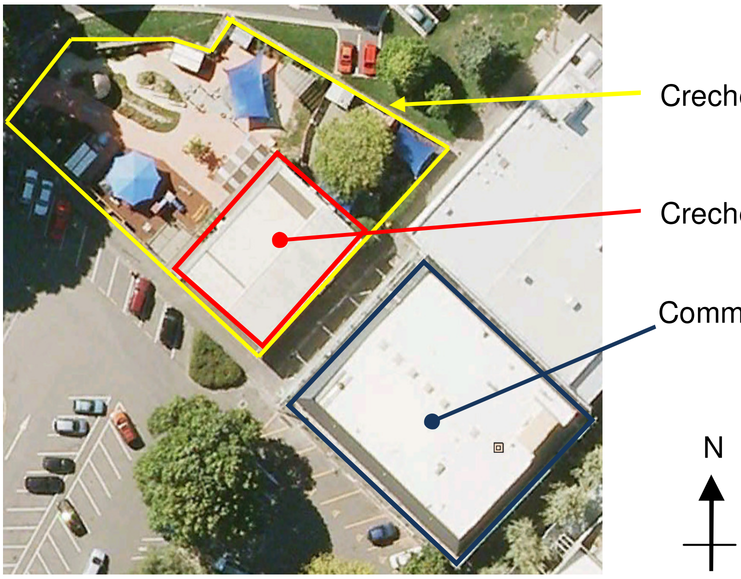

Figure 2: Site Plan (Source: Google Maps)

The overall building dimensions are approximately 14.6m by 19m in plan. The roof framing consists of timber purlins supported on steel trusses spanning the transverse direction. Steel trusses are supported on 305mm square concrete columns along the southeast elevation and concrete encased steel columns along the northwest elevation. 305mm by 457mm concrete ring beams occur around the perimeter as well as the wall between the original crèche and the addition.

Along the northwest elevation, the 200mm concrete blockwork is built-in between the columns and extends to the roof level. Along the northeast wall and the now internal wall between the original crèche and the addition, a timber wall that supports the roof joists exists above the concrete beam. A 200mm block wall exists below the concrete beam.

Externally to the northwest side there is a canopy supported on timber beams which are themselves supported by a series of blockwork columns. Refer to Figure 3 below:

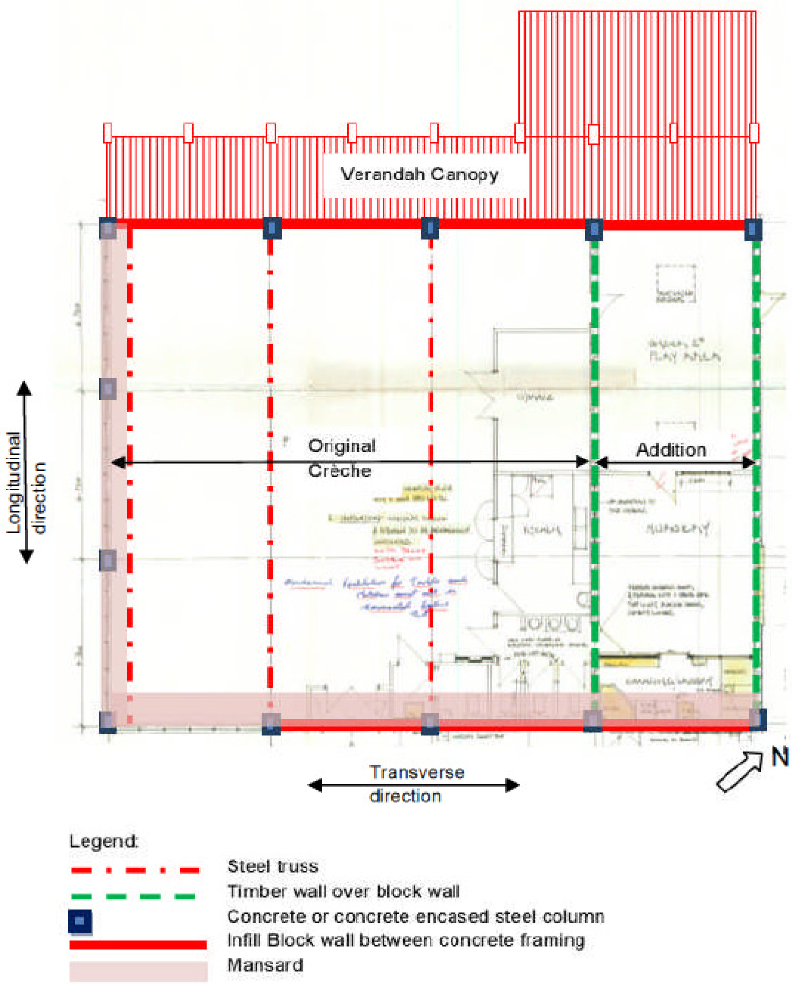

Figure 3: Floor Plan

At the southwest and southeast elevations, a steep mansard constructed of timber framing occurs between the concrete beam and the roof. Cavity walls constructed with an outer leaf of "La Strada" stone, a 25mm cavity and a 100mm thick block inner leaf occurs between the concrete framing on these elevations. A series of window occurs above the cavity wall along the southeast elevation (See Figure 4).

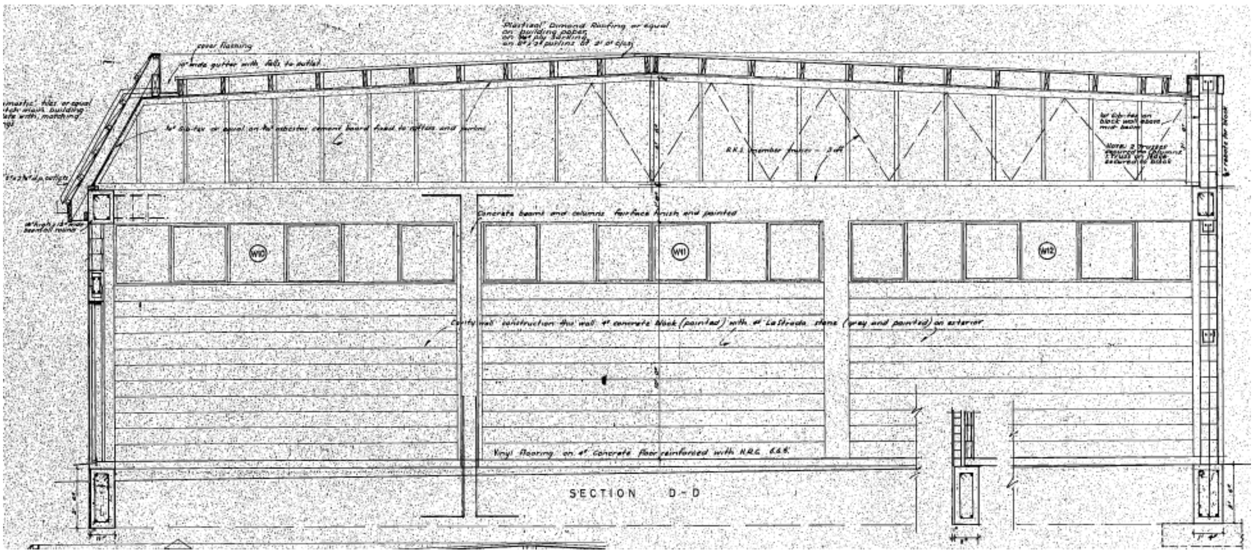

Figure 4: Building Section

There is no explicit roof diaphragm present. The 10mm thick plywood sarking provides some diaphragm action. However, a continuous gutter occurs along the two sides of the roof, limiting the diaphragm capacity in the transverse direction.

The lateral load resisting elements in the longitudinal direction consist of the concrete frame partially infilled with cavity wall along the southeast and timber walls above concrete block infill walls.

The lateral load resisting elements in the transverse direction consist of concrete block infill walls along the southeast and northwest elevations.

4.2 Building Damage Assessments

4.2.1 Post 22 February 2011 Rapid Assessment

Structural (Level 2) assessments of the structure were undertaken on 8 March 2011 by Opus International Consultants Limited. This inspection included external and internal visual inspections of all structural elements, without the benefit of opening up works.

The site was posted with a green placard.

4.2.2 Further Inspections

A further inspection was undertaken by Opus International Consultants Limited on 28 October 2011.

4.3 Original Documentation

Copies of the following construction drawings were provided by CCC:

- Construction drawings number 2565/Sheets 1 to 22.

The drawings have been used to confirm the structural systems, investigate potential critical structural weaknesses (CSW) and identify details which required particular attention.

Some specification information is available but no structural calculations for the building have been located.

Structural drawings of the addition is not available for our review.

4.4 Field Investigation

Field investigation was performed by City Care to verify existing construction and to obtain information not shown on documents reviewed. The following is a summary of the findings:

- The northwest wall is grouted. The block wall above the concrete beam has D12 reinforcement bars at 400mm centres. Presumably the same reinforcement occurs below the concrete beam as well.
- The wall between the original crèche and the addition is not grouted and assumed to not be reinforced.
- The southeast wall is not grouted and assumed not to be reinforced.
- Typical columns have four D20 longitudinal bars with R10 stirrups at 100 centres.
- Typical beams have four D20 longitudinal bars and R10 stirrups at 300 centres.

4.5 Qualitative Assessment

A qualitative assessment [1] for the building was completed in November 2011 following the 22 February 2011 earthquake. The findings of this report were that the building had some critical weaknesses which affected the likely seismic capacity of the building. The evaluated capacity of the building was determined to be 40%NBS by qualitative assessment. The damage sustained to the building was minor, but there was some evidence of the perceived structural weaknesses resisting seismic loads. A quantitative assessment was recommended following the completion of the qualitative assessment report.

5 Structural Damage

A damage assessment survey was carried out by Opus International Consultants Limited on 28 October 2011.

5.1 Surrounding Buildings

The nearest building is the Bishopdale Community Centre and Library to the southeast. There is a 6.3m wide service corridor between the two buildings thus pounding is not a concern.

5.2 Residual Displacements and Damage

The damage noted has been reported in the qualitative report for this property issued by Opus International Consultants Limited on 14 November 2011.

The report identified that there were two minor areas of damage and likely not related to earthquake actions.

- In the art storage room there is a horizontal crack on the external (south east) wall at about 2.0m high. This is marking out the position of an old door opening which has been blocked up in the past.
- On one blockwork column supporting the canopy to the northwest side there is a crack through the block at low level. However, observing the crack closely shows that there is paint inside the crack from when the column was last repainted. We were informed that the paint was "several years ago" and maybe even up to 10 years. The crack therefore pre-dates any of the recent seismic events.

Note: Photographs showing the structural damage noted above are included within the Opus Qualitative Assessment report dated 14 November 2011.

5.3 Foundations

No evidence of ground damage or foundation settlement has been noted at the site.

5.4 Primary Gravity Structure

As noted above and in the qualitative report some cracking damage has been noted to the exterior block wall and blockwork column supporting the canopy to the northwest.

5.6 Non Structural Elements

No damage has been noted to non-structural elements.

6 Detailed Seismic Assessment

The detailed seismic assessment has been based on the NZSEE 2006 [2] guidelines for the "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" together with the Detailed Engineering Evaluation Procedure [3] (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011.

6.1 Critical Structural Weaknesses

The term Critical Structural Weakness (CSW) refers to a component of a building that could contribute to increased levels of damage or cause premature collapse of a building.

The following critical structural weaknesses have been identified for this building:

a) Short Columns: The presence of "short columns" formed by the partial height blockwork infill panels in between the concrete columns at the southwest elevation.

b) Lack of roof diaphragm: The roof level has little structural bracing in the transverse direction. The diaphragm capacity is also likely to be low for the longitudinal direction.

c) Plan Stiffness Irregularity: Some plan stiffness irregularity is present in both directions.

6.2 Quantitative Assessment Methodology

The assessment assumptions and methodology have been included in Appendix 2 of the report due to the technical nature of the content. A brief summary follows:

1. A 3D model of the building was created in ETABS, which is a finite element structural analysis programme.
2. A linear dynamic modal response spectrum analysis was carried out using the spectral values established from NZS1170.5, with an updated Z factor of 0.3 (B1/VM1). This analysis was used to establish the actions on the structural elements.
3. Based on the actions determined from the analysis, demand to capacity ratios (DCR's) were determined for each component in question. The highest DCR was then converted to a %NBS for the structure.

6.3 Limitations and Assumptions in Results

Our analysis and assessment is based on an assessment of the building in its undamaged state. Therefore the current capacity of the building may be lower than that stated.

The results have been reported as a %NBS and the stated value is that obtained from our analysis and assessment. Despite the use of best national and international practice in this analysis and assessment, this value contains uncertainty due to the many assumptions and simplifications which are made during the assessment. These include:

- Simplifications made in the analysis, including boundary conditions such as foundation fixity.
- Assessments of material strengths based on drawings and site inspections
- The normal variation in material properties which change from batch to batch.
- Approximations made in the assessment of the capacity of each element.

6.4 Quantitative Assessment

A summary of the structural performance of the building is shown in the Table 2. Note that the values given represent the critical elements in the building, as these effectively define the building's capacity. Other elements within the building will have significantly greater capacity when compared with the governing elements.

As noted in Appendix A2.2 Analysis Parameters, the building was analysed using a ductility factor (μ) equal to 1.25 due to the presence of unfilled concrete blockwall and potential short column condition along the southwest elevation.

Modes of failure that do not govern the building's performance are not included in the table except as noted for cases where higher ductility factors have led to the component being classified as non-critical.

Table 2: Summary of Seismic Performance

Structural Element/System	**Failure mode or description of limiting criteria based on elastic capacity of critical element**	**Critical Structural Weakness and Collapse Hazard**	**% NBS based on calculated capacity**
Primary Components (those that are required parts of the lateral resisting system)			
Grouted masonry block wall along northwest (NW) elevation	Filled concrete block wall exists along the NW elevation. Various openings have been made.	No	100% NBS
Mansard Roofs	Lateral load from the roof level relies on mansard roof to deliver load to the walls below. Plywood sarking provides some shear resistance but the capacity is not adequate to transfer this load.	Yes	< 33% NBS
Unfilled masonry block wall in NW to SE direction.	Based on the field investigation, the wall between the original crèche and the addition is unfilled. The failure mode of the unreinforced masonry is shear failure.	No	48% NBS
Concrete columns along southwest (SW) elevation	Window openings between the concrete beam and cavity block wall below create a short column condition. However, column failure mode is in flexure. Failure is ductile and thus not considered a CSW.	No	68% NBS
Timber walls above masonry wall at northeast elevation and wall between original crèche and addition.	Masonry walls do not extend to the roof level thus the lateral load from the roof level relies on the timber walls to transfer load to the block walls below. Shear strength of timber walls is dependent on the type of sheathing and spacing of fasteners. The detail of shear transfer is unknown.	Yes	< 33% NBS

Structural Element/System	Failure mode or description of limiting criteria based on elastic capacity of critical element	Critical Structural Weakness and Collapse Hazard	% NBS based on calculated capacity
Secondary Components (those that are not required parts of the lateral load resisting system but which must be able to maintain their gravity load capacity while the building under goes deformation due to earthquake loading)			
Out-of-plane loading at northwest wall	The steel truss connection is not adequate to resist out of plane loading from the wall. Failure mode is in bending of connection plate, thus some ductility is present in this connection. Note: the thickness of connection plate is not legible from the drawings. Calculations assume a minimum thickness of 6mm for lower bound %NBS and 10mm for upper bound %NBS.	Yes, possible local collapse	25 - 50% NBS
Out-of-plane loading at northeast wall and wall between original crèche and addition	Based on the field investigation, the block wall is not filled or reinforced. The wall fails at 50 to 75% due to out-of-plane forces. Additionally the concrete beam above the block wall spans horizontally (approximately 14m) to perpendicular walls. The beam also fails in bending at 50 to 75% NBS.	No	50 - 75% NBS
Out-of-plane loading at southwest wall	Cavity wall construction consists of exterior 100mm “la strada” stone and 100mm unreinforced block wall. Out-of-plane forces are resisted either by the wall cantilevering from the ground or spanning horizontally between columns.	Yes	13% NBS
Out-of-plane loading at southeast wall	Based on field investigation, the block wall is not filled or reinforced. It fails at 50 to 75% NBS due to out-of-plane forces.	No	50 - 75% NBS
Unreinforced masonry block work column at veranda canopy	No reinforcement was found in the masonry block work columns. Failure is brittle and local collapse is possible.	Yes	< 33% NBS

7 Summary of Geotechnical Appraisal

A copy of the desktop geotechnical report is attached to this report as Appendix 2. A summary of the report is as follows:

a) The site has been identified by the ECan liquefaction study as having low to moderate liquefaction risk however there has been no evidence of surface deformation on the site following the recent earthquakes and no liquefaction in the immediate vicinity of the building.

b) ECan well logs and Soils & Foundation Ltd map of Christchurch indicate the building is likely to be founded on layers of silty clay, gravel and firm clay overlying the Riccarton gravel formation.

c) Based on site observations by structural engineers, the existing foundations appear to have performed well and are considered appropriate for the building. It is understood that seismic strengthening including new bracing of the Community Centre & Library building is proposed. Further site investigations will be required to assist with tension pile design.

d) GNS Science indicates an elevated risk of seismic activity is expected in the Canterbury region as a result of the earthquake sequence following the 4 September 2010 earthquake. Recent advice (Geonet) indicates there is a 16% probability of another Magnitude 6 or greater earthquake occurring in the next 12 months in the Canterbury region. It is expected that the probability of occurrence is likely to decrease with time, following periods of reduced seismic activity.

e) Based on the past performance in recent earthquakes, the existing foundations should be acceptable in terms of future ULS and SLS loadings, although further site specific test data will be required for the design of new foundations to carry compression and tension loads from structural retrofitting.

8 Remedial Options

The building requires repair and strengthening, with a target of increasing the seismic performance to as near as practicable to 100%NBS, and at least 67%NBS. We have recommended possible options for how this may be achieved:-

1. Install a plywood diaphragm on the underside of roof purlins and mansard purlins.

2. Provide plywood sheathing on timber walls along northeast elevation and wall between original crèche and addition. Review the existing connection between the timber wall to roof sheathing to determine if a load path exists. If no load path does exist, add connections between the top of timber wall and roof sheathing to transfer shear.

3. Remove and replace unreinforced block walls with reinforced block wall. Alternatively, shotcrete the interior of the block walls.

4. Tie the exterior leaf of “la strada” stone to the inner leaf using through rods and expansion anchors or proprietary “Helifix” type anchors. Provide out-of-plane support to the inner leaf (see item 3 above). Fill the cavity with grout.

5. Provide horizontal rod bracing at concrete beam level to provide lateral support to the concrete beams.

6. Provide supplementary steel column supports under the northwest end of the steel roof trusses.

7. Provide supplementary column supports on either side of the unreinforced block columns at the veranda canopy. Alternatively, replace the block columns with steel columns.

9 Conclusions

Based on our quantitative assessment, the building is considered to be earthquake prone. This is primarily due to the shear walls and diaphragms having capacities less than 33%NBS. Factors limiting the %NBS of the building are summarized below:

1. The seismic performance of the primary components (those that are required parts of the lateral resisting system) are governed by the lack of a complete roof diaphragm and shear transfer to the concrete block walls below. The calculated strength of the timber walls and mansard roof is less than 33% NBS (around 20% NBS).

2. The seismic performance of the secondary components (those that are not required parts of the lateral load resisting system but which must be able to maintain their gravity load capacity while the building undergoes deformation due to earthquake loading) are governed by:

   a. At the northwest elevation, the steel truss connections are not adequate to resist the out-of-plane loading from the block wall. The failure mode is in flexure of the steel connector plates. Although the failure is a ductile type failure, given the consequence of failure that the roof trusses may lose their support, this condition should be addressed.

   b. Unreinforced block columns provide gravity support for the veranda canopy. The columns to not have adequate capacity to resist the calculated lateral load.

   c. At the southwest and portion of the southeast elevation, the cavity walls do not have adequate strength to resist out-of-plane forces resulting in approximately 13% NBS. Although failure of this wall will not lead to collapse, the wall is approximately 2.4m high and failure could lead to injury to occupants or pedestrians near the wall.

10 Recommendations

a) Scheme designs for repair and basic strengthening options to increase the seismic capacity of the building to at least 67% NBS should be prepared.

b) A quantity surveyor be engaged to determine the costs for either strengthening the building or demolishing and rebuilding.

c) If a scheme for strengthening proves to be economically viable a full design should be carried out to produce this scheme. This will need to take into account all the structural weaknesses identified within the property.

d) It is recommended that the building not be occupied, given its earthquake prone building status and the elevated level of seismic risk in Christchurch.

11 Limitations

a) This report is based on an inspection of the structure of the buildings and focuses on the structural damage resulting from the 22 February 2011 Canterbury Earthquake and aftershocks only. Some non-structural damage is described but this is not intended to be a complete list of damage to non-structural items.

b) Our inspections have been visual and non-intrusive, no linings or finishes were removed to expose structural elements. Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practicing in this field at this time.

c) This report is prepared for CCC to assist with assessing the remedial works required for council buildings and facilities. It is not intended for any other party or purpose.

12 References

[1] Detailed Engineering Evaluation, Stage 1 Qualitative Report; November 2011, Opus International consultants

[2] NZS 1170.5: 2004, Structural design actions, Part 5 Earthquake actions. Standards New Zealand.

[3] NZSEE: 2006, Assessment and improvement of the structural performance of buildings in earthquakes, New Zealand Society for Earthquake Engineering.

Appendix 1 - Photographs

Bishopdale Community Crèche		
No.	**Item description**	**Photo**
1.	General view of the crèche from the west	
2.	South west facade	
3.	SE Elevation and service corridor	

4.	North West wall	
5.	Horizontal crack on wall in art store	
6.	Crack on column at south west corner (paint filled)	

7.	Field investigation of southeast wall. Face shell removed and block is not filled.	
8.	Field investigation of southeast wall. Face shell removed and block is not filled.	

9.	Field investigation of concrete column at southwest elevation.	
10.	Field investigation of concrete beam at northwest elevation.	
11.	Field investigation of concrete beam at northwest elevation.	

12.	Timber wall at northeast elevation above concrete beam. Interior side is not sheathed.	
13.	Timber wall between original crèche and addition above concrete beam.	
14.	Steel truss.	

Appendix 2 - Quantitative Assessment Methodology and Assumptions

A2.1. Referenced Documents

- AS/NZS 1170.0:2002, *Structural design actions, Part 0: General principles,* Standards New Zealand.

- AS/NZS 1170.1:2002, *Structural design actions, Part 1: Permanent, imposed and other actions,* Standards New Zealand.

- NZS 1170.5:2004, *Structural design actions, Part 5: Earthquake actions – New Zealand,* Standards New Zealand.

- NZS 3101: Part 1: 2006, *Concrete Structures Standard, The Design of Concrete Structures,* Standards New Zealand.

- NZS 3101: Part 2: 2006, *Concrete Structures Standard, Commentary on the Design of Concrete Structures,* Standards New Zealand.

- NZBC, *Clause B1 Structure, Verification Method B1/VM1,* Department of Building and Housing.

- NZSEE: 2006, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes,* New Zealand Society for Earthquake Engineering.

- Engineering Advisory Group, *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure*, Draft Prepared by the Engineering Advisory Group, Revision 5, 19 July 2011.

- ASCE/SEI 41-06, *Seismic Rehabilitation of Existing Buildings,* Structural Engineering Institute of the American Society of Civil Engineers, 2007.

A2.2. Analysis Parameters

The following parameters are used for the seismic analysis:

- Site soil category — Cl. 3.1.3, NZS1170.5
  D (deep or soft soil)

- Seismic hazard factor — Cl. $2.2.14_B$, B1/VM1
  $Z = 0.30$

- Return period factor — Table 3.5, NZS1170.5
  $R_u = 1.0$ (*Importance* Level 2 structure, 50 year design life)

- Ductility factor — Cl. 2.6.1.2, NZS3101:2006
  $\mu = 1.25$

- Structural performance factor — Cl. 2.6.2.2, NZS3101:2006
  $S_p = 0.925$

- Material properties

Table A1: Analysis Material Properties

Concrete nominal compressive strength, f'_c (MPa) (1)	25
Mild reinforcing nominal yield strength, f_y (MPa) (2)	275

Notes:
1. Based on guidance from *NZSEE 2006,* probable concrete compressive strength is based on a value of 1.5 times the nominal compressive strength (Cl. 7.1.1)
2. Based on guidance from *NZSEE 2006,* probable reinforcement yield strength is based on a value of 1.08 times the nominal yield strength (Cl. 7.1.1)

- Effective section properties

Table A2: Effective section properties from NZS 3101

Table C6.6 – Effective section properties, I_e

Type of member	Ultimate limit state		Serviceability limit state		
	f_y = 300 MPa	f_y = 500 MPa	μ = 1.25	μ = 3	μ = 6
1 Beams					
(a) Rectangular ¶	0.40 I_g (use with E_{40}) §	0.32 I_g (use with E_{40}) §	I_g	0.7 I_g	0.40 I_g (use with E_{40})§
(b) T and L beams ¶	0.35 I_g (use with E_{40}) §	0.27 I_g (use with E_{40}) §	I_g	0.6 I_g	0.35 I_g (use with E_{40})§
2 Columns					
(a) $N^*/A_g f'_c > 0.5$	0.80 I_g (1.0 I_g)‡	0.80 I_g (1.0 I_g)‡	I_g	1.0 I_g	As for the ultimate limit state values in brackets
(b) $N^*/A_g f'_c = 0.2$	0.55 I_g (0.66 I_g)‡	0.50 I_g (0.66 I_g)‡	I_g	0.8 I_g	
(c) $N^*/A_g f'_c = 0.0$	0.40 I_g (0.45 I_g)‡	0.30 I_g (0.35 I_g)‡	I_g	0.7 I_g	
3 Walls ¶					
(a) $N^*/A_g f'_c = 0.2$	0.48 I_g	0.42 I_g	I_g	0.7 I_g	As for the ultimate limit state values
(b) $N^*/A_g f'_c = 0.1$	0.40 I_g	0.33 I_g	I_g	0.6 I_g	
(c) $N^*/A_g f'_c = 0.0$	0.32 I_g	0.25 I_g	I_g	0.5 I_g	
4 Diagonally reinforced coupling beams	0.6I_g for flexure Shear area, A_{shear}, as in text		I_g 1.5 A_{shear} for ULS	0.75 I_g 1.25 A_{shear} for ULS	As for ultimate limit state

NOTES –
(§) With these values the E value should be the elastic modulus for concrete with a strength of 40 MPa regardless of the actual concrete strength.
(‡) The values in brackets apply to columns which have a high level of protection against plastic hinge formation in the ultimate limit state.
(¶) For additional flexibility, within joint zones and for conventionally reinforced coupling beams refer to the text.

- Earthquake load combination — Cl. 4.2.2, AS/NZS1170.0

  $G + E_u + \Psi_E Q$

- Building seismic weight — Cl. 4.2, NZS1170.5

  W_t = 1175 kN

A2.3. Assessment Methodology

<u>Static & Modal Response Spectrum Analysis</u>

The seismic assessment was undertaken by completing static and modal response spectrum (MRS) analyses for the building in accordance with NZS 1170.5:2004.

A 3D model was set up using the structural analysis program ETABS, and effective section properties for structural members were taken from Table A2 above. The diaphragms were modelled as flexible diaphragms.

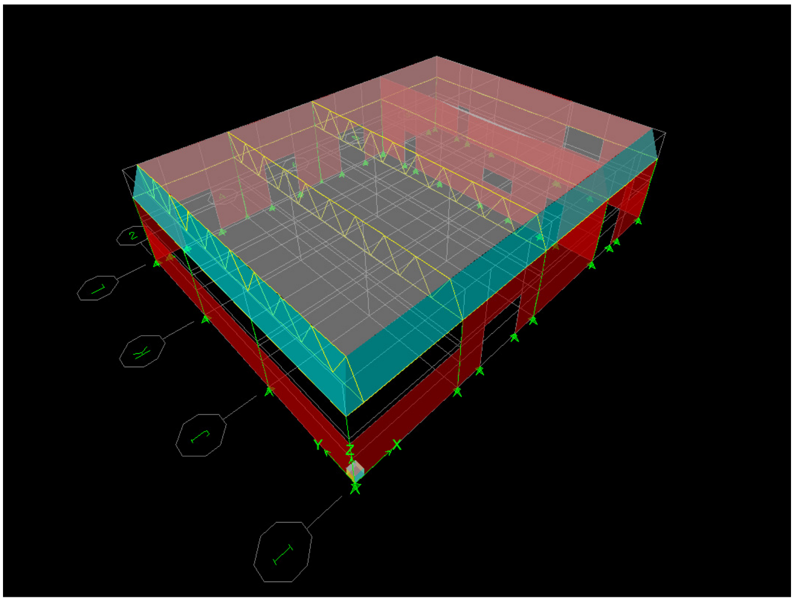

Figure A1: ETABS model of the Bishopdale Creche

The fundamental building periods output from ETABS are:

T (SW to NE direction) = 0.04 sec
T (NW to SE direction) = 0.11 sec

A total of 30 modes are used in the MRS analysis resulting in 96% and 96% effective participating mass in the longitudinal and transverse directions, respectively.

An equivalent static analysis was also carried out as a consistency check of the MRS analysis output. The base shear from the MRS analysis is scaled to 100% of the equivalent static method base shear, as required by NZS1170.5 for an irregular structure. The base shears resulting from the equivalent static method are:

V_{ELF} = 856 kN (both directions)

The base shears resulting from the MRS are:

V_{MRS} = 345 kN (*SW to NE direction*)
V_{MRS} = 470 kN (*NW to SE direction*)

The forces from the MRS analysis were scaled up by 2.5 and 1.8 in the *SW to NE direction* and *NW to SE direction* directions, respectively.

The building was analysed as having ductility (μ = 1.25) and the design actions were applied separately in each perpendicular direction.

Element Demand to Capacity

Element force demands were extracted from the MRS analysis and compared to calculated capacities based on the material properties assumed in Table A1. The results of these demand to capacity checks are summarized in further detail in the report and reported as %NBS.

Appendix 3 – Geotechnical Report

12 March 2012

Christchurch City Council
C/O:- Michael Sheffield

Dear Michael

6-QUCCC.47/48

Geotechnical Desktop Study – Bishopdale Community Centre & Library and Bishopdale Community Crèche

1. Introduction

Christchurch City Council (CCC) has commissioned Opus International Consultants (Opus) to undertake a brief geotechnical desktop study of the Bishopdale Community Centre & Library and the Bishopdale Community Crèche, Bishopdale, Christchurch. The purpose of this study is to collate existing subsoil information and undertake an appraisal of the potential geotechnical hazards at this site and to determine whether further investigations are required.

This Geotechnical Desk Study has been prepared in accordance with the Engineering Advisory Group's Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Revision 5, 19 July 2011.

The Geotechnical Desk Study forms part of a Detailed Engineering Evaluation prepared by Opus. Based on a site inspection by Opus Engineers, no ground damage or visual evidence of differential settlement has been observed at the site. A level survey has not been undertaken. The Geotechnical Desk Study has been undertaken without the benefit of any site specific investigations and is therefore preliminary in its nature.

2. Desktop Study

2.1 Site Description

The Bishopdale Community Centre & Library and Bishopdale Community Crèche are located off Farrington Ave on the south west side of the Bishopdale Mall. The buildings are surrounded by a large sealed carpark to the south and commercial buildings to the north, east and west; refer to the location plan in Appendix A.

The Community Centres foundations appear to be reinforced concrete pads of various sizes supporting the columns with tie beams running between the pads. The Crèche's foundations consist of reinforced concrete perimeter strip footings and isolated pad foundations for the columns. Refer to the Opus Qualitative Structural Assessment Report for more detailed description of the building.

No Geotechnical Reports or site specific investigations were available from the CCC Property file.

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Christchurch Office

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Christchurch 8140, New Zealand

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Website: www.opus.co.nz

The ground profile is relatively flat and level with the adjacent carpark and surrounding areas.

2.2 Regional Geology

The published geological map of the area, (Geology of the Christchurch Urban Area 1:25,000, Brown and Weeber, 1992) indicates the site is the Yaldhurst member of the Springston Formation with dominantly alluvial sand and silt overbank deposits.

2.3 Expected Ground Conditions

A review of the Environmental Canterbury (ECan) wells database showed five wells located within approximately 160m of the property (refer to Site Location Plan in Appendix A). The locations of Boreholes and CPT's undertaken by the Earthquake Commission have also been reviewed. The nearest CPT is located 560m south east of the site. Due to the large distance from the site, the CPT log has been excluded from this study. Material logs available from the four closest ECan wells have been used to infer the ground conditions at the site as shown in Table 1 below.

Table 1: Inferred Ground Conditions

Stratigraphy	Thickness (m)	Depth Encountered From (m)
Topsoil	0.3-1.2m	Surface
Silty CLAY	1.5-2.3m	0.3-3.5m
Grey GRAVEL and SAND	6.7-11m	1.8-5.8m
Firm blue CLAY with peat lenses	3-4.4m	10.2-12.8m
Sandy GRAVEL (Riccarton Formation)	-	12.5-17.2m

A groundwater depth of approximately 3m to 4m below ground level has been estimated from groundwater depth contour maps (Brown and Weeber (1992)).

2.4 Liquefaction Hazard

A liquefaction hazard study was conducted by the Canterbury Regional Council (ECan) in 2004 to identify areas of Christchurch susceptible to liquefaction during an earthquake. The Bishopdale site is located on the border of areas identified as 'moderate liquefaction ground damage potential' and 'no liquefaction ground damage potential' for a low groundwater scenario. According to this study, the liquefaction ground damage potential is low, indicating the ground may be affected by up to 100mm of subsidence in a seismic event.

Tonkin and Taylor Ltd (T&T Ltd) have been engaged as the Earthquake Commission's (EQC) geotechnical consultants and have prepared maps showing areas of liquefaction interpreted from high resolution aerial photos for the 4th September earthquake, and the aftershocks of February 2011 and June 2011. There is no surface evidence of liquefaction around the Bishopdale Community Centre & Library and Bishopdale Community Crèche or in the vicinity. The nearest surface rupture of liquefaction was observed 450m south east of the buildings.

DBH Residential Foundation Technical Categories map last updated 16 November 2011 has classified the surrounding residential properties located 110m south east of the buildings as Technical Category 2. This indicates that 'minor to moderate damage from liquefaction is possible in future large earthquakes.

3. Discussion

The site has been identified by the ECan liquefaction study as having low to moderate liquefaction risk however there has been no evidence of surface deformation on the site following the recent earthquakes and no liquefaction in the immediate vicinity of the building.

ECan well logs and Soils & Foundation Ltd map of Christchurch indicate the building is likely to be founded on layers of silty clay, gravel and firm clay overlying the Riccarton gravel Formation.

No level survey or site investigations have been undertaken as part of this Desk Study.

Based on site observations by Opus Engineers, the existing foundations appear to have performed well and are considered appropriate for the building. It is understood that seismic strengthening including new bracing of the Community Centre & Library building is proposed. Further site specific investigations will be required to assist with tension pile design.

GNS Science indicates an elevated risk of seismic activity is expected in the Canterbury region as a result of the earthquake sequence following the 4 September 2010 earthquake. Recent advice (Geonet) indicates there is a 16% probability of another Magnitude 6 or greater earthquake occurring in the next 12 months in the Canterbury region. It is expected that the probability of occurrence is likely to decrease with time, following periods of reduced seismic activity.

RecommendationsIt is recommended that:

- Based on the past performance in recent earthquakes, the existing foundations should be acceptable in terms of future ULS and SLS loadings.
- Further site specific test data will be required to confirm the conclusions of this study and to design new foundations to carry compression and tension loads from the structural retrofitting.

4. Limitations

This report has been prepared solely for the benefit of CCC as our client with respect to the particular brief given to us. Data or opinions in this desk study may not be used in other contexts, by any other party or for any other purpose.

It is recognised that the passage of time affects the information and assessment provided in this Document. Opus's opinions are based upon information that existed at the time of the production of this Desk Study. It is understood that the Services provided allowed Opus to form no more than an opinion on the actual conditions of the site at the time the site was visited and cannot be used to assess the effect of any subsequent changes in the quality of the site, or its surroundings or any laws or regulations.

5. References:

Brown, LJ; Webber, JH 1992: Geology of the Christchurch Urban Area. Scale 1:25,000. Institute of Geological and Nuclear Sciences geological map, 1 sheet + 104p.

Environment Canterbury, Canterbury Regional Council (ECan) website:

ECan Well Card
http://ecan.govt.nz/services/online-services/tools-calculators/Pages/well-card.aspx

ECan 2004: The Soild Facts on Christchurch Liquefaction. Canterbury Regional Council, Christchurch, 1 sheet.

Project Orbit, 2011: interagency/organisation collaboration portal for Christchurch recovery effort. https://canterburyrecovery.projectorbit.com/SitePages/Home.aspx

GNS Science reporting on Geonet Website: http://www.geonet.org.nz/canterbury-quakes/aftershocks/ updated on 24 February 2012.

Appendices:
Appendix A: Site Plan
Appendix B: Environment Canterbury Borehole Logs

APPENDIX A:
Site Plan

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ECan Borehole Location

BH	ECan Ref
1	M35/1366
2	M35/2627
3	M35/2628
4	M35/9440
5	M35/9409

0 100m

Approximate Scale 1:1250 at A3

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Project: Bishopdale Community Centre and Creche
Geotechnical Desk Study

Project No.: 6-QUCCC.47/48

Client: Christchurch City Council

Site Location Plan

Drawn:

Date: 9-Mar-12

APPENDIX B:
Environment Canterbury Borehole Logs

Borelog for well M35/9409

Gridref: M35:7658-4656 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 17.8 +MSD
Driller : CW Drilling and Investigation
Drill Method : Rotary Rig
Drill Depth : -12m Drill Date : 11/09/2002

Water Level: -2.0CalcMin

Scale(m): -5, -10

Depth(m)	Full Drillers Description	Formation Code
-0.60m	Top soil garden gravel layer compact	sp
-1.00m	Firm red into brown silt	sp
-1.90m	Brown silt coarse sand	sp
-2.00m	Medium gravels brown silt dry	sp
-3.00m	Brownish silts medium gravels some fine sands wet. Water layer at 2.7m	sp
-4.00m	Brownish silts medium gravels fine to coarse sands wet	sp
-5.00m	Brownish silts medium gravels fine sands	sp
-6.00m	Brownish silts medium gravels round fine sands	sp
-7.50m	Brownish silts medium large gravels fine sands wet firm drilling	sp
-9.00m	Medium large gravels rounded brownish silts fine sand layers of sand	sp
- 10.5m	Greenish into reddish silt brown stained gravels some quartz chips loose	sp
- 12.0m	Rusty red gravels into brownish silts firm medium gravels round	sp

Borelog for well M35/2628

Gridref: M35:7668-4626 Accuracy : 2 (1=best, 4=worst)
Ground Level Altitude : 17.2 +MSD
Driller : A M Bisley & Co
Drill Method : Cable Tool
Drill Depth : -32.4m Drill Date : 25/08/1982

Water Level: -2.2CalcMin

Depth(m)	Full Drillers Description	Formation Code
-0.30m	Surface soil	sp?
-1.80m	Silt	sp?
-6.90m	Grey gravel & sand	sp?
-7.00m	Yellow clay	sp?
- 10.2m	Grey/Brown gravel & sand	sp?
- 10.3m	Yellow clay	sp?
- 10.5m	Grey sand, some gravel	sp?
- 12.8m	Soft Blue clay, wood, lenses of Yellow clay, some gravel	sp?
- 15.8m	Firm Blue clay, peat	ch
- 32.4m	Grey/Brown gravel, sand	ri

Scale(m): -5, -10, -15, -20, -25, -30

Borelog for well M35/2627

Gridref: M35:76673-46238 Accuracy : 2 (1=best, 4=worst)
Ground Level Altitude : 17.3 +MSD
Driller : A M Bisley & Co
Drill Method : Cable Tool
Drill Depth : -28.1m Drill Date : 6/09/1982

Water Level: -2.2CalcMin

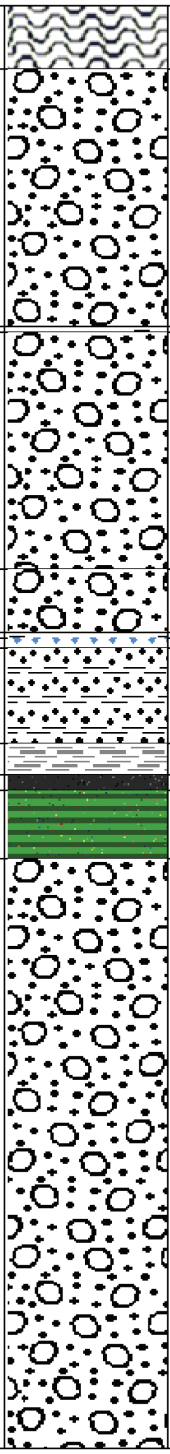
Depth(m)	Full Drillers Description	Formation Code
-0.30m	Surface soil	sp?
-1.80m	Silt	sp?
-9.50m	Grey gravel & sand	sp?
- 10.2m	Blue gravel & sand	sp?
- 11.0m	Blue clay	ch
- 14.6m	Soft Blue clay, lenses of Yellow clay	ch
- 17.2m	Firm Grey clay & wood	ch
- 28.1m	Sandy Grey/Brown gravel	ri

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

Borelog for well M35/1366

Gridref: M35:76639-46251 Accuracy : 2 (1=high, 5=low)
Ground Level Altitude : 17 +MSD
Driller : A M Bisley & Co
Drill Method : Cable Tool
Drill Depth : -27.4m Drill Date : 1/07/1962

Water Level: -2.2CalcMin



Depth(m)	Full Drillers Description	Formation Code
-1.20m	Topsoil	sp?
-6.09m	Coarse Grey gravel & sand	sp?
-6.19m	Yellow/Brown soft clay	sp?
- 10.7m	Fine to coarse Grey gravel & sand	sp?
- 11.9m	Coarse Blue gravel & sand	sp?
- 12.0m	Sand Blue clay & wood	ch
- 12.2m	Blue sand	ch
- 14.0m	Blue sand, lenses of Brown/ Yellow clay	ch
- 14.6m	Firm Grey clay	ch
- 14.9m	Peat & wood	ch
- 16.2m	Firm Grey clay & peat	ch
- 27.4m	Coarse to fine Grey gravel & medium sand	ri

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

Borelog for well M35/9440 page 1 of 5

Gridref: M35:7666-4626 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 17.1 +MSD
Driller : Clemence Drilling Contractors
Drill Method : Rotary/Percussion
Drill Depth : -197.8m Drill Date : 20/08/2003

Depth(m)	Full Drillers Description	Formation Code
-0.40m	Top soil	sp
-3.50m	Yellow clay	sp
-5.80m	Silty clay	sp
- 12.5m	Timber and blue gravel	sp
- 39.6m	Sandy water-bearing gravel	ri

Scale(m): -5, -10, -15, -20, -25, -30, -35

Water Level

Borelog for well M35/9440 page 2 of 5

Gridref: M35:7666-4626 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 17.1 +MSD
Driller : Clemence Drilling Contractors
Drill Method : Rotary/Percussion
Drill Depth : -197.8m Drill Date : 20/08/2003

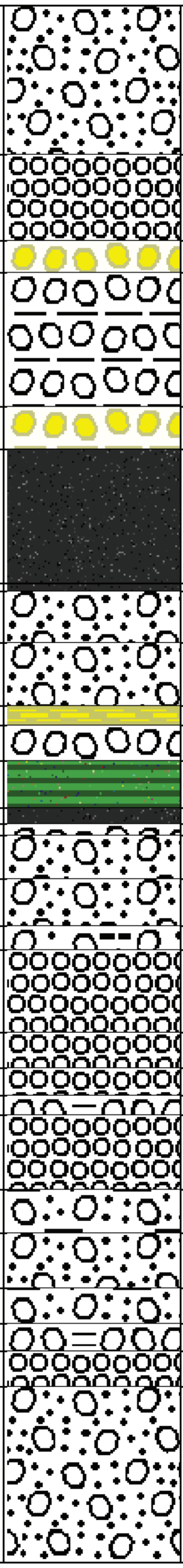
Scale(m)	Water Level	Depth(m)	Full Drillers Description	Formation Code
-40			Sandy water-bearing gravel	
-45		- 47.0m		ri
		- 48.0m	Yellow claybound gravel	br
-50		- 51.0m	Grey clay	br
-55		- 56.5m	Yellow claybound gravel	li
		- 58.0m	Very sandy - well sorted water-bearing gravel	li
-60			Very sandy - loose stained water-bearing gravel	
-65				
-70				
-75		- 75.0m		li
		- 75.5m	Solid yellow clay	li
		- 76.6m	Solid yellow clay - traces of gravel	li
		- 39.6m	Heavy stained sandy water-bearing gravel	li

Borelog for well M35/9440 page 3 of 5

Gridref: M35:7666-4626 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 17.1 +MSD
Driller : Clemence Drilling Contractors
Drill Method : Rotary/Percussion
Drill Depth : -197.8m Drill Date : 20/08/2003

Environment Canterbury

Scale(m): -80, -85, -90, -95, -100, -105, -110, -115



Water Level	Depth(m)	Full Drillers Description	Formation Code
	- 82.9m	Heavy stained sandy water-bearing gravel	li
	- 85.1m	Tight water-bearing gravel	li
	- 85.9m	Yellow claybound gravel	li
	- 89.3m	Loose claybound gravel	li
	- 90.4m	Solid yellow clay with seams of gravel	li
	- 93.8m	Peat and timber	he
	- 94.0m	Poor water-bearing gravel	he
	- 95.3m	Heavy water-bearing sand and gravel	he
	- 96.9m	Loose sandy water-bearing gravel	he
	- 97.4m	Hard yellow clay	he
	- 98.3m	Soft claybound gravel	he
	- 99.5m	Hard grey pug and peat	he
	- 99.9m	Hard black peat	he
	- 100.2m	Claybound gravel	bu
	- 101.3m	Very loose lightly stained sandy gravel	bu
	- 102.5m	Loose sandy water-bearing gravel	bu
	- 103.1m	Loose sandy water-bearing gravel (Silty yellow clay lumps)	bu
	- 105.2m	Tight water-bearing gravel	bu
	- 106.1m	Loose water-bearing gravel	bu
	- 106.8m	Tight water-bearing gravel	bu
	- 107.3m	Very loose claywashed gravel	bu
	- 109.2m	Very loose clean water-bearing gravel	bu
	- 110.3m	Tight sandy gravels (Traces of clay)	bu
	- 111.7m	Loose sandy gravel	bu
	- 112.6m	Tight sandy gravels (Traces of clay)	bu
	- 113.3m	Tight black claywashed gravel	bu
	- 114.2m	Loose water-bearing gravel	bu
	- 39.6m	Tight sandy water-bound gravel	sh?

Borelog for well M35/9440

Gridref: M35:7666-4626 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 17.1 +MSD
Driller : Clemence Drilling Contractors
Drill Method : Rotary/Percussion
Drill Depth : -197.8m Drill Date : 20/08/2003

Scale(m)	Water Level	Depth(m)	Full Drillers Description	Formation Code
-120		- 123.9m	Tight sandy water-bound gravel	sh?
-125		- 125.2m	Loose heavily stained gravel	sh
		- 126.0m	Hard yellow clay	sh
		- 126.9m	Claybound gravel	wa
		- 127.8m	Tight heavily stained sandy gravel	wa
		- 129.4m	Tight heavily stained sandy water-bearing gravel (Clay traces)	wa
-130		- 131.0m	Loose lightly stained gravel	wa
		- 132.0m	Tight water-bearing gravel	wa
		- 133.9m	Loose water-bearing gravel	wa
-135		- 137.8m	Tight sandy gravel	wa
		- 138.2m	Tight heavily stained gravel	wa
		- 138.5m	Loose stained gravel (Rusty coloured water)	wa
		- 139.5m	Hard sticky yellow clay	
-140		- 142.8m	Hard silty blue pug	
		- 144.8m	Hard silty grey pug	
-145		- 145.8m	Pug and peat	
		- 146.8m	Hard sticky grey pug	
		- 148.3m	Hard sticky yellow clay	
		- 148.9m	Claybound gravel	
		- 149.6m	Loose heavily stained sandy gravels	
-150		- 153.2m	Loose lightly stained sandy gravels	
		- 154.1m	Very loose stained sandy gravels	
-155		- 155.6m	Tight sandy water-bearing gravel	
		- 156.0m	Loose black stained gravel	
		- 39.6m	Tight sandy gravel	

Borelog for well M35/9440 page 5 of 5

Gridref: M35:7666-4626 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 17.1 +MSD
Driller : Clemence Drilling Contractors
Drill Method : Rotary/Percussion
Drill Depth : -197.8m Drill Date : 20/08/2003

Depth(m)	Full Drillers Description	Formation Code
- 159.4m	Tight sandy gravel	
- 159.9m	Heavily stained gravel - Traces of clay	
- 165.5m	Loose stained gravel - traces of clay	
- 165.9m	Hard yellow clay	
- 168.9m	Loose stained gravel - very sandy rusty water	
- 170.8m	Tight sandy gravels	
- 171.6m	Very loose small heavily stained gravel	
- 171.7m	Hard pale yellow clay	
- 171.9m	Blue gravel	
- 172.1m	Hard sticky grey pug	
- 173.8m	Hard and sticky peat	
- 176.2m	Hard and sticky grey pug	
- 177.3m	Hard sticky yellow clay	
- 178.2m	Hard cemented claybound gravel	
- 178.7m	Tight water-bearing sand	
- 180.1m	Tight sandy water-bearing large gravel	
- 180.3m	Yellow clay	
- 180.6m	Tight gravel	
- 190.5m	Loose water-bearing gravel	
- 192.3m	Loose sandy gravel	
- 192.6m	Hard silty yellow clay	
- 194.8m	Loose lightly stained very sandy gravel	
- 39.6m	Tight stained sandy gravel	

OPUS