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Le Bons Bay Domain Toilets

PRK-3596-BLDG-001

Detailed Engineering Evaluation

Quantitative Report

Version FINAL

Le Bons Bay Rd

INFRASTRUCTURE | MINING & INDUSTRY | DEFENCE | PROPERTY & BUILDINGS | ENVIRONMENT

Le Bons Bay Domain Toilets
PRK-3596-BLDG-001

Detailed Engineering Evaluation
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Le Bons Bay Rd

Christchurch City Council

Prepared By
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Reviewed By
Stephen Lee

Date
6 June 2013

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Quantitative Report Summary

Le Bons Bay Domain Toilets

PRK-3596-BLDG-001

Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

Le Bons Bay Rd

Banks Peninsula

Canterbury

Background

This is a summary of the Quantitative report for the above structure, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011 and visual inspections on 15 February 2013.

Brief Description

The building is rectangular in plan with nominal dimensions of 6.3 m by 2.3 m with a height of 2.2 m. It is approximated that the building was constructed in the 1960's.

The toilet block is a single storey, concrete masonry structure with a reinforced concrete roof slab. A concrete water tank is located on the roof. The floor and foundation are formed by a concrete raft foundation.

Key Damage Observed

Non seismic damage in the form of minor concrete spalling was observed in some areas of the concrete roof slab.

Indicative Building Capacity

Following a detailed assessment the building has been assessed as achieving 12% New Building Standard (NBS). Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the building is considered to be Earthquake Prone as it achieves less than 34% NBS.

Recommendations

The building has been assessed as an Earthquake Prone building. Although it is possible to strengthen the structure to 67% NBS as recommended by the NZSEE, due to the extent of works required it may be more prudent to demolish the existing structure and rebuild.

1. Background

GHD Limited has been engaged by Christchurch City council to undertake a detailed engineering evaluation of the Le Bons Bay Domain Toilet building located at Le Bons Bay Rd, Banks Peninsula.

This report is a Quantitative Assessment and is based on NZS 1170.5: 2004, the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance (06/2006) and the Assessment and Improvement of the Structural Performance of Buildings in earthquakes.

The quantitative assessment of the building comprises an investigation of the in plane and out of plane capacity of the reinforced concrete masonry walls. The investigation is based on the analysis of the seismic loads to which the structure is subjected, the analysis of the distribution of these loads throughout the structure and the analysis of the capacity of existing structural elements to resist the loads applied. The capacity of the existing structural elements is compared to the demand placed on the elements to give the percentage of New Building Standard (%NBS) of each of the structural elements.

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 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 capacity 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

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 any 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)) 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.

2.2.1 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.

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.

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 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;
- 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.

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.

After the February Earthquake, on 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

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

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 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 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	Unacceptable	Unacceptable

Figure 1 NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE

Table 1 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.

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

Table 1 %NBS compared to relative risk of failure

4. Building Description

4.1 General

The Le Bons Bay Domain toilet building is located at Le Bons Bay Road, Banks Peninsula. The site is relatively flat at approximately 2 m above sea level.

The building is rectangular in plan with approximate dimensions of 6.3 m by 2.3 m with a height of 2.2 m. It is estimated that the building was constructed in the 1960's.

The toilet block is a single storey, concrete masonry structure. The 70 mm thick concrete roof slab contains a mesh reinforcement of 6 mm diameter bars at 150 crs. There is a concrete water tank located on the roof. Walls are constructed with 190 mm thick concrete masonry except for two 90 mm thick 1.8 m partial-height partitions. The foundation and floor of the structure are formed by a concrete slab on grade.

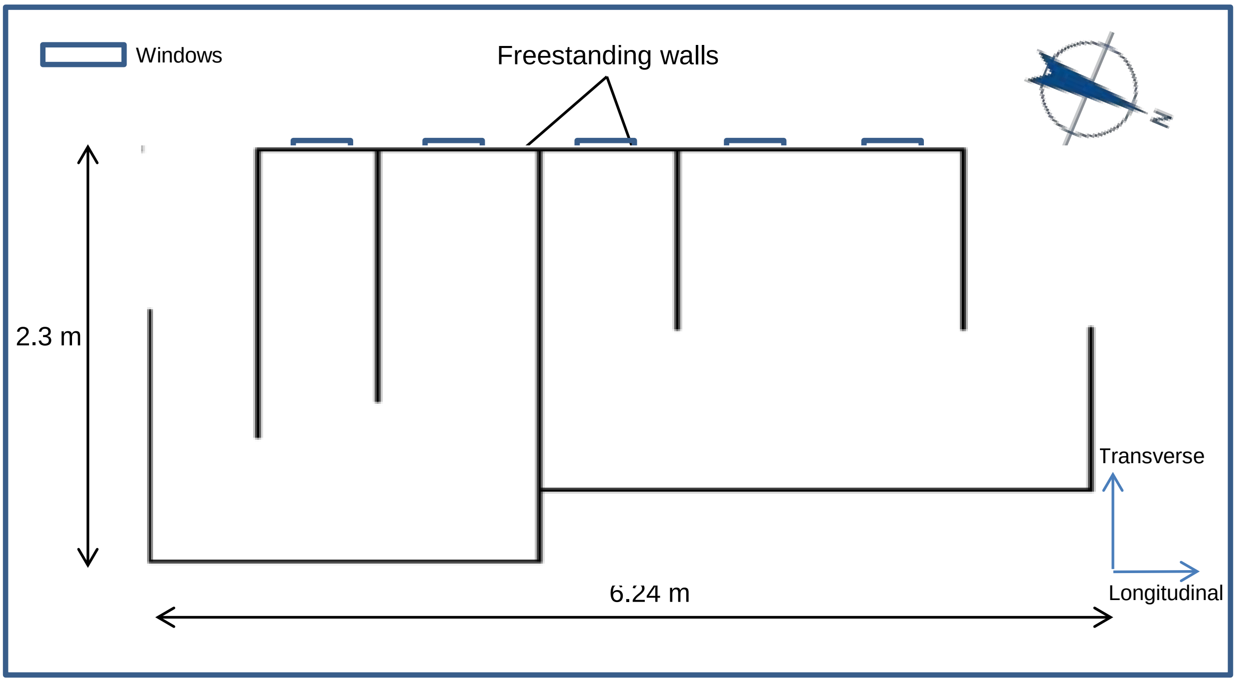


Figure 2 Le Bons Bay Domain Toilet Floor Plan

4.2 Gravity Load Resisting System

The gravity roof loads in the structure are transferred through the concrete roof slab into the concrete masonry walls by direct bearing. Gravity loads are then transferred axially down through the concrete masonry walls to the concrete foundations where it is distributed into the ground beneath.

4.3 Lateral Load Resisting System

The lateral load resisting system is similar for both the longitudinal and transverse directions. Lateral roof loads are transferred through the diaphragm action of the concrete roof slab to the concrete masonry walls which are in the plane of loading. These in-plane walls transfer the lateral loads down to the foundation by the panel action of the concrete masonry. The foundation distributes the lateral loads into the ground beneath. Walls subject to out of plane loading span vertically between the foundation and the restraints provided by the concrete roof diaphragm.

Partial height partitions are free-standing and cantilever from the foundation.

5. Inspection

A visual inspection of the building was undertaken on the 15th of February 2013. Both the interior and exterior of the building were inspected. The main structural components of the building were all able to be viewed due to the exposed nature of the structure.

The inspection consisted of observing the building to determine the structural systems and likely behaviour of the building during an earthquake. The site was assessed for damage, including examination of the ground conditions, checking for damage in areas where damage would be expected for the type of structure and noting general damage observed throughout the building in both structural and non-structural elements.

Concrete honeycombing of the roof slab during construction resulted in reduced cover to reinforcement and has allowed corrosion to occur in the reinforcing mesh. The corrosion has led to minor spalling in some areas of the roof. The spalling will not affect the lateral load capacity of the roof diaphram.

A Hilti PS 200 Ferroscan was used to confirm the position, depth and diameter of the reinforcement in the concrete masonry walls. The scan revealed the presence of reinforcement at a maximum of one end location in any wall panel. This results in effective lateral load resistance in only one direction of in-plane loading of the wall. The reversal of seismic loading therefore requires the reinforcement be ignored. As a result, the building has been assessed using techniques for unreinforced concrete masonry.

6. Damage Assessment

6.1.1 Surrounding Buildings

There are no buildings directly adjacent to the structure. Nearby buildings were enclosed by boundary hedges and unable to be visually inspected.

6.1.2 Residual Displacements and General Observations

No residual displacements or damage in the structure were identified during the inspection.

6.1.3 Ground Damage

There was no evidence of ground damage in the vicinity immediately adjacent to the Le Bons Bay Toilets.

7. Geotechnical Considerations

7.1 Site Description

The site is situated in Le Bons Bay, in Banks Peninsula. It is relatively flat at approximately 2 m above mean sea level. It is approximately 12 km northeast of Akaroa, and is on the coast at Le Bons Bay.

7.2 Published Information on Ground Conditions

7.2.1 Published Geology

Brown & Weeber, 1992[1] describes the site geology as:

- Beach gravel and sand of post-glacial shorelines, including those of Lake Ellesmere (Q1b).

7.2.2 Environment Canterbury Logs

No nearby boreholes comprised lithographic logs. However, wells slightly further away (800 m South) indicate the area to be underlain by sand and clay to 39 m bgl, overlying volcanic bedrock.

Groundwater was recorded at 2.6 m bgl in the borehole log.

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	From Site	Log Summary	
N36/0052	61.0 m	2.6 m	800 m S	0.0 – 11.8 m	SAND
				11.8 – 23.2 m	CLAY
				23.2 – 26.8 m	SAND
				26.8 – 39.5 m	CLAY
				39.5 – 61.0 m	Volcanic rock

It should be noted that the logs have been written by the well driller and not a geotechnical professional or to a standard. In addition capacity data is not recorded.

7.2.3 EQC Geotechnical Investigations

The Earthquake Commission has not undertaken geotechnical testing in the area of the subject site.

7.2.4 CERA Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has indicated the site is situated within the Green Zone, indicating that repair and rebuild may take place.

Land in the CERA green zone has been divided into three technical categories. These categories describe how the land in expected to perform in future earthquakes.

[1] Brown, L. J. & Weeber, J.H. (1992): *Geology of the Christchurch Urban Area*. Institute of Geological and Nuclear Sciences 1:25,000 Geological Map 1. IGNS Limited: Lower Hutt.

The site has been categorised as “N/A – Port Hills and Banks Peninsula”. These areas have not been given a technical category as their geology differs significantly from the Canterbury Plains.

7.2.5 Post-Earthquake Land Observations

The site is not in coverage of the aerial photography following the major earthquakes of the Canterbury earthquake sequence.

7.2.6 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated to be sand and clay to 39 m bgl, overlying volcanic bedrock.

Groundwater is considered to be approximately 2.6 m bgl.

7.3 Seismicity

7.3.1 Nearby Faults

There are many faults in the Canterbury region, however only those considered most likely to have an adverse effect on the site are detailed below.

Table 3 Summary of Known Active Faults[2,3]

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	160 km	NW	~8.3	~300 years
Greendale Fault (2010)	60 km	W	7.1	~15,000 years
Hope Fault	140 km	N	7.2~7.5	120~200 years
Kelly Fault	145 km	NW	7.2	~150 years
Porters Pass Fault	105 km	NW	7.0	~1100 years
Port Hills Fault (2011)	*38 km*	NW	6.3	Not Estimated

The recent earthquake sequence since 4 September 2010 has identified the presence of a previously unmapped active fault system underneath the Canterbury Plains; this includes the Greendale Fault and Port Hills Fault listed in Table 3 above. Research and published information on this system is in development and the average recurrence interval is yet to be established for the Port Hills Fault.

7.3.2 Ground Shaking Hazard

New Zealand Standard NZS 1170.5:2004 quantifies the Seismic Hazard factor for Christchurch as 0.30, being in a moderate to high earthquake zone. This value has been provisionally upgraded recently (from 0.22) to reflect the seismicity hazard observed in the earthquakes since 4 September 2010.

The recent seismic activity has produced earthquakes of Magnitude 6.3 with significant peak ground accelerations (PGA) across large parts of the city.

[2] Stirling, M.W, McVerry, G.H, and Berryman K.R. (2002): “A New Seismic Hazard Model for New Zealand”, *Bulletin of the Seismological Society of America*, Vol. 92 No. 5, June 2002, pp. 1878-1903.

[3] GNS Active Faults Database, http://maps.gns.cri.nz/website/af/viewer

Conditional PGA's from the CGD are not available for Banks Peninsula.

7.4 Slope Failure and/or Rockfall Potential

The topography surrounding the site suggests that rockfall is not a potential hazard. In addition, any retaining structures or embankments nearby should be further investigated to determine the site-specific local slope instability potential.

7.5 Liquefaction Potential

The site is considered to have a low to moderate susceptibility to liquefaction, due to the following reason:

- Presence of saturated sands beneath the site.

7.6 Summary & Recommendations

This assessment is based on a review of the geology and existing ground investigation information, and observations from the Christchurch earthquakes since 4 September 2010.

The site appears to be situated on sand and clay to 39 m bgl, overlying volcanic bedrock. Associated with this the site also has a low to moderate liquefaction potential, in particular where sands are present.

A soil class of **D** (in accordance with NZS 1170.5:2004) should be adopted for the site.

Should a more comprehensive liquefaction and/or ground condition assessment be required, it is recommended that intrusive investigation be conducted.

8. Quantitative Assessment

8.1 Quantitative Assessment Procedure

The New Zealand Society for Earthquake Engineering (NZSEE) publication "Assessment & Improvement of Structural Performance of Buildings (2006)" and the relevant New Zealand material standards were used to provide a framework and method for the analysis. The analysis applied gravity loads, imposed loads and seismic loads to the structure. The elements were then assessed against their respective load capacities.

8.2 Seismic Parameters

The elastic site hazard spectrum for horizontal loading, C(T), for the building was derived from Equation 3.1(1) of NZS 1170:2004

$$C(T) = C_h ZRN(T,D)$$

Where

$C_h(T)$ = the spectral shape factor determined from Cl 3.1.2

Z = the hazard factor from Cl 3.1.4 and the subsequent amendments which increased the hazard factor to 0.3 for Akaroa

R = 1, the return period factor from Table 3.5 for an annual probability of exceedance of 1/500 for an Importance Level 2 building.

N(T,D) = the near-fault scaling factor from Cl 3.1.6

The structural performance factor, S_p, was calculated in accordance with Cl 4.4.2

$$S_p = 1.3 - 0.3\mu$$

Where μ is the structural ductility factor. A structural ductility factor of 1.25 has been taken for lateral loading across and along the building.

For T1 < 0.7s and soil class D, the seismic weight coefficient was determined in accordance with Cl 5.2.1.1 of NZS 1170.5: 2011. For the purposes of calculating the seismic weight coefficient a period, T1, of 0.4 was assumed for the in-plane masonry walls. The coefficient was then calculated using Equation 5.2(1);

$$C_d(T_1) = \frac{C(T_1)S_p}{k_\mu}$$

Where

$$k_\mu = \frac{(\mu - 1)T_1}{0.7} + 1$$

8.3 Equivalent Static Method

Equivalent static forces were calculated in accordance with NZS 1170:2004. The lateral seismic forces have been distributed to the concrete masonry walls assuming that the concrete roof slab behaves as a rigid diaphragm and that the lateral load resisted by each wall is proportional to the stiffness of each wall. An accidental eccentricity of 10% has been assumed in each direction.

The structure has been considered brittle. As a result, 30% loading from the other orthogonal direction has been included when determining the loading on the masonry walls for an earthquake in a particular direction as per NZS 1170.5:2004 requirements.

8.4 Capacity of Un-reinforced Masonry Walls

8.4.1 In-Plane Capacity of the Unreinforced Walls

The in-plane shear capacity of the unreinforced concrete masonry walls was determined using section 8.4 of the NZSEE guidelines "Assessment & Improvement of Unreinforced Masonry Building for Earthquake Performance (2011)". The strength reduction factor, Ø, for shear and shear friction was taken as 0.85 in accordance with NZSEE guidelines. The overall shear capacity of each wall was evaluated considering four shear failure modes. These are diagonal tension failure, rocking failure, bed-joint sliding failure and toe crushing failure. The in-plane shear capacity of each wall is,

$$\mathrm{V_n} = \min(\mathrm{V_{dt}}, \mathrm{V_s}, \mathrm{V_r}, \mathrm{V_{tc}})$$

8.4.2 Unreinforced Masonry In-Plane Moment Capacity

The in-plane flexural capacity of the unreinforced concrete masonry walls was calculated as,

$$M_n = N_b \left[Z - \frac{1}{2}\right] x \, \frac{N_b}{0.85 f_m{}' t_w}$$

$$\mathrm{Z} = \frac{\mathrm{L_w}}{2}$$

Where

N_b = normal force acting at wall base

f'_m = compressive strength of masonry

t_w = wall thickness

L_w = wall length

8.4.3 Unreinforced Masonry Out-of-Plane Capacity

The out-of-plane flexural capacity of the concrete masonry walls was determined using Section 10.3 of the NZSEE guidelines "Assessment & Improvement of the Structural Performance of Buildings in Earthquakes (2006)." The overall out-of-plane capacity of each wall was evaluated by comparing the likely displacement of the wall during an earthquake and the displacement that would cause instability of the wall. The out-of-plane capacity of each wall is,

$$\%NBS = 0.72\ \frac{\Delta_i}{D_{ph}}$$

Where

Δ_i = out-of-plane deflection that would cause instability

D_{ph} = out-of-plane displacement response demand for a wall panel

8.4.4 %NBS

The shear and bending moment capacities of the concrete masonry walls were compared to their respective demands to determine the overall %NBS.

$$\%NBS = \frac{V_n}{V^*}x100$$

$$\%NBS = \frac{M_n}{M^*}x100$$

8.5 Results

Calculations show that the structure achieves **12% NBS** and is therefore **Earthquake Prone**.

The in-plane capacities of the structure's walls were checked against lateral seismic loading. It was found that the shear capacities of the walls were the controlling in-plane values. The in-plane shear capacities of the walls are shown in Table 4 below. Walls are identified in Fig 3 below. The critical in-plane shear capacity was found to be 13.8% for Wall 7.

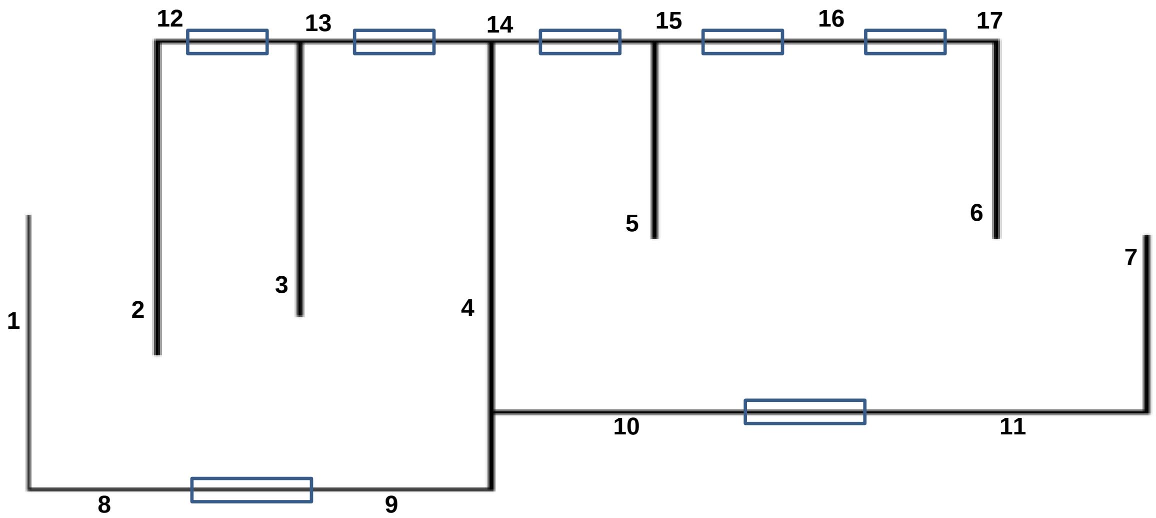


Figure 3 Wall Numbering

Table 4 In-plane shear and moment capacities of full height walls

Wall	Shear Demand V* (kN)	Moment Demand M* (kNm)	In Plane Shear Capacity ØVn (kN)	In-Plane Moment Capacity ØMn (kNm)	Shear %NBS	Moment %NBS
1	13.7	12.8	2.0	5.4	14.8%	42.1%
2	16.1	14.1	2.9	7.8	18.2%	55.3%
4	23.3	15.4	3.4	15.7	14.5%	101.9%
6	7.2	6.8	1.1	3.1	15.6%	45.6%
7	6.5	6.4	0.9	2.5	13.8%	39.1%
8	8.0	17.6	1.1	3.1	14.1%	17.6%
9	5.1	11.2	0.8	2.0	14.7%	17.9%
10	15.8	34.8	2.3	6.2	14.7%	17.9%
11	15.8	34.8	2.3	6.2	14.7%	17.9%
12-17	10.0	22	1.8	26.8	18.0%	121.8%

Wall 3 and Wall 5 as identified in figure 3 are partial height walls. The out of plane capacities of full and partial height walls are shown in Table 5 below. The critical out-of-plane capacity was found to be 12% NBS for partial height walls.

Table 5 Out-of-plane capacities of walls

Wall	%NBS
Full Height Walls	61%
Partial Height Walls	12%

The overall 12% NBS rating of the structure is governed by the out-of-plane capacity of the partial height walls.

8.6 Discussion of Results

The results obtained are consistent with a building this age that is constructed of concrete masonry founded on class D soils. The building achieves 12% NBS which is less than 34% NBS and it is therefore deemed an Earthquake Prone Building. This building would have been designed to the loading standards current in the 1960's, namely NZS1900:1965. The design loads used in accordance with this standard are likely to have been less than those required by the current loading standard. When combined with the increased hazard factor for Akaroa to 0.3, and the known poor performance of unreinforced masonry in seismic events, it is reasonable to expect the building to be classified as Earthquake Prone.

9. Conclusions and Recommendations

The building has been assessed as an Earthquake Prone building. Although NZSEE guidelines recommend strengthening to 67% NBS, due to the extent of the works required to achieve this, it may be prudent to demolish the existing structure and rebuild.

10. Limitations

10.1 General

This report has been prepared subject to the following limitations:

- No intrusive structural investigations have been undertaken.
- No intrusive geotechnical investigations have been undertaken.
- No level or verticality survey has been carried out
- No material testing has been undertaken.
- No calculations, other than the capacity of the masonry walls, have been carried out on the structure

It is noted that this report has been prepared at the request of Christchurch City Council, and is intended to be used for their purposes only. GHD accepts no responsibility for any other party or person who relies on the information contained in this reportrite a specific limitations section.

10.2 Geotechnical Limitations

This report presents the results of a geotechnical appraisal prepared for the purpose of this commission, and for prepared solely for the use of Christchurch City Council, and their advisors. The data and advice provided herein relate only to the project and structures described herein and must be reviewed by a competent geotechnical engineer before being used for any other purpose. GHD Limited (GHD) accepts no responsibility for other use of the data.

The advice tendered in this report is based on a visual geotechnical appraisal. No subsurface investigations have been conducted. An assessment of the topographical land features have been made based on this information. It is emphasised that Geotechnical conditions may vary substantially across the site from where observations have been made. Subsurface conditions, including groundwater levels can change in a limited distance or time. In evaluation of this report cognisance should be taken of the limitations of this type of investigation.

An understanding of the geotechnical site conditions depends on the integration of many pieces of information, some regional, some site specific, some structure specific and some experienced based. Hence this report should not be altered, amended or abbreviated, issued in part and issued incomplete in any way without prior checking and approval by GHD. GHD accepts no responsibility for any circumstances, which arise from the issue of the report, which have been modified in any way as outlined above.

Appendix A
Photographs

Photograph 1 Eastern elevation of structure

Photograph 2 Western elevation of structure

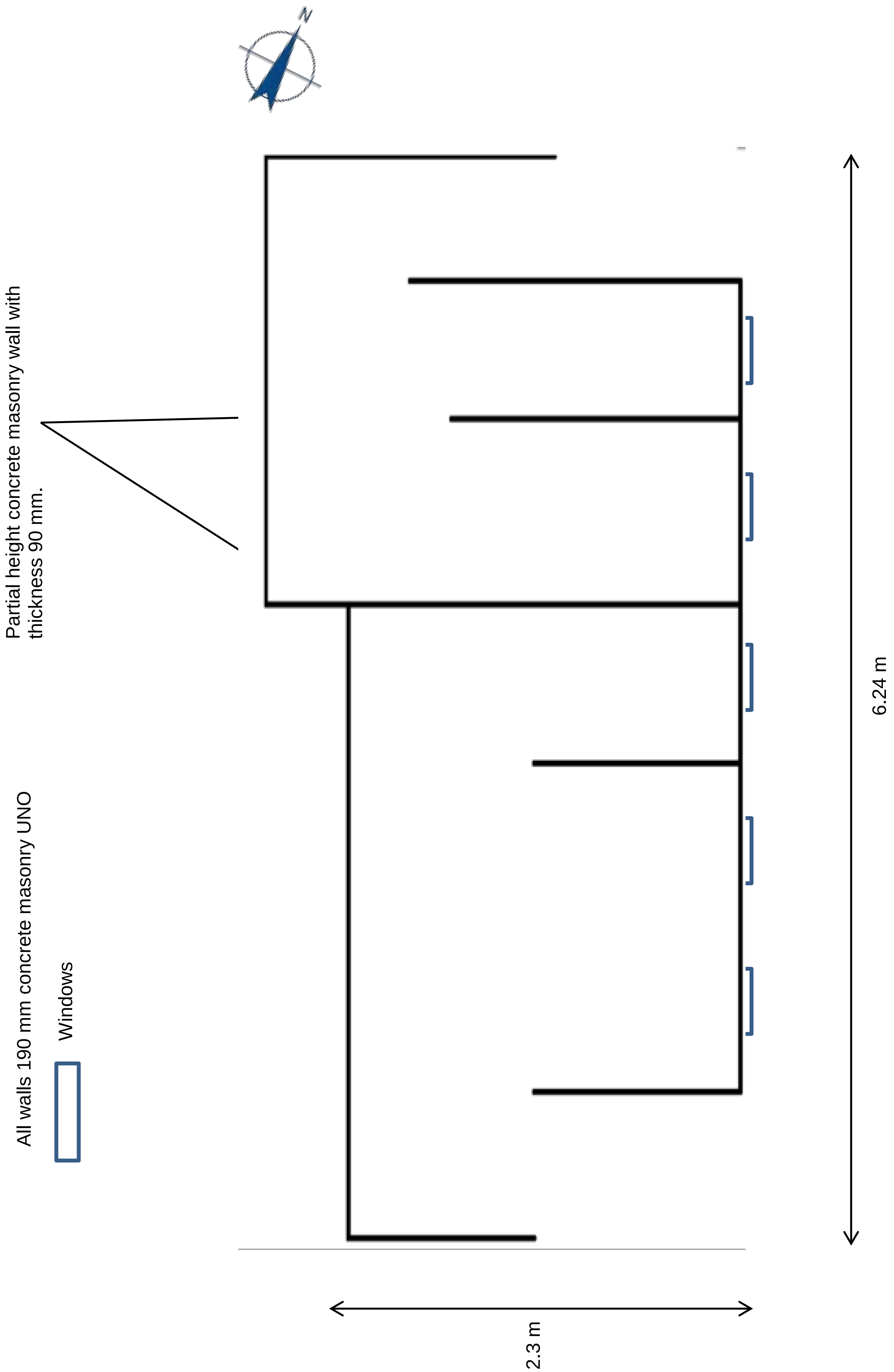
Photograph 3 Partial height wall on left and full height wall on right

Photograph 4 Water storage tank on roof of structure

Photograph 5 Example of minor spalling in concrete roof slab

Appendix B

Sketch


Partial height concrete masonry wall with
thickness 90 mm.
All walls 190 mm concrete masonry UNO
Windows
6.24 m
2.3 m

Appendix C

CERA Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Le Bons Bay Toilets	Reviewer:	Stephen Lee
	Unit / No: / Street	CPEng No:	1006840
Building Address:	Le Bons Bay Rd	Company:	GHD
Legal Description:		Company project number:	513090283
		Company phone number:	04 472 0799
	Degrees / Min / Sec		
GPS south:		Date of submission:	15/04/2013
GPS east:		Inspection Date:	15/02/2013
		Revision:	final
Building Unique Identifier (CCC):	PRK-3596-BLDG-001	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:	1	single storey = 1	Ground floor elevation (Absolute) (m):	
Ground floor split?	no		Ground floor elevation above ground (m):	
Storeys below ground	0			
Foundation type:	mat slab		if Foundation type is other, describe:	
Building height (m):	2.20		height from ground to level of uppermost seismic mass (for IEP only) (m):	2.2
Floor footprint area (approx):	15			
Age of Building (years):	48		Date of design:	1965-1976
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	public		Brief strengthening description:	
Use (upper floors):				
Use notes (if required):	Toilet Building			
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	concrete	slab thickness (mm)	70
Floors:	concrete flat slab	slab thickness (mm)	
Beams:			
Columns:			
Walls:			

Lateral load resisting structure

Lateral system along:	partially filled CMU	**Note: Define along and across in**		
Ductility assumed, μ:	1.25	**detailed report!**	note total length of wall at ground (m):	
Period along:	0.40	#### enter height above at H31	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	partially filled CMU			
Ductility assumed, μ:	1.25		note total length of wall at ground (m):	
Period across:	0.40	#### enter height above at H31	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

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

Non-structural elements

Stairs:	
Wall cladding:	
Roof Cladding:	
Glazing:	
Ceilings:	
Services(list):	

Available documentation

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

Damage

Site:
(refer DEE Table 4-2)

Site performance:	Good	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:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:			
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

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

Recommendations

	Level of repair/strengthening required:		Describe:	
	Building Consent required:		Describe:	
	Interim occupancy recommendations:		Describe:	
Along	Assessed %NBS before e'quakes:	12%	#### %NBS from IEP below	If IEP not used, please detail assessment methodology: Detailed Calculations
	Assessed %NBS after e'quakes:	12%		
Across	Assessed %NBS before e'quakes:	14%	#### %NBS from IEP below	
	Assessed %NBS after e'quakes:	14%		

IEP **Use of this method is not mandatory - more detailed analysis may give a different answer, which would take precedence. Do not fill in fields if not using IEP.**

Period of design of building (from above): 1965-1976

h_n from above: 2.2m

Seismic Zone, if designed between 1965 and 1992:

not required for this age of building
not required for this age of building

	along	across
Period (from above):	0.4	0.4
(%NBS)nom from Fig 3.3:		

Note:1 for specifically design public buildings, to the code of the day: pre-1965 = 1.25; 1965-1976, Zone A =1.33; 1965-1976, Zone B = 1.2; all else 1.0
Note 2: for RC buildings designed between 1976-1984, use 1.2
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)

	along	across
Final $(\%NBS)_{nom}$:	**0%**	**0%**

2.2 Near Fault Scaling Factor

Near Fault scaling factor, from NZS1170.5, cl 3.1.6:

	along	across
Near Fault scaling factor (1/N(T,D), **Factor A**:	#DIV/0!	#DIV/0!

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:
Z_{1992}, from NZS4203:1992
Hazard scaling factor, **Factor B:** #DIV/0!

2.4 Return Period Scaling Factor

Building Importance level (from above): 2
Return Period Scaling factor from Table 3.1, **Factor C**:

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)		
Ductility scaling factor: =1 from 1976 onwards; or $=k_\mu$, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	0.00	0.00

2.6 Structural Performance Scaling Factor:

	along	across
Sp:		
Structural Performance Scaling Factor **Factor E**:	#DIV/0!	#DIV/0!

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

	along	across
$\%NBS_b$:	**#DIV/0!**	**#DIV/0!**

Global Critical Structural Weaknesses: (refer to NZSEE IEP Table 3.4)

3.1. Plan Irregularity, factor A:	insignificant	1
3.2. Vertical irregularity, Factor B:	insignificant	1
3.3. Short columns, Factor C:	insignificant	1

3.4. Pounding potential

Pounding effect D1, from Table to right
Height Difference effect D2, from Table to right

Therefore, Factor D: 0

Table for selection of D1	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Alignment of floors within 20% of H	**0.7**	**0.8**	**1**
Alignment of floors not within 20% of H	**0.4**	**0.7**	**0.8**

Table for Selection of D2	Severe	Significant	Insignificant/none
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

3.5. Site Characteristics	insignificant	1

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum		
Rationale for choice of F factor, if not 1		

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)
List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

3.7. Overall Performance Achievement ratio (PAR)	**0.00**	**0.00**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	**#DIV/0!**	**#DIV/0!**
4.4 Percentage New Building Standard (%NBS), (before)		**#DIV/0!**

GHD

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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
Final	Kimberly Rodgers	Stephen Lee		Nick Waddington		