Broad Park Toilets and Changing Rooms
PRK 0121 BLDG 001 EQ2

Detailed Engineering Evaluation

Quantitative Report

Version FINAL Rev 1

7a Broadpark Road, Waimairi Beach

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7a Broadpark Road, Waimairi Beach

Christchurch City Council

Prepared By

Peter O'Brien

Reviewed By

Hamish Mackinven

Date

17th February 2014

Contents

Table Index

Figure Index

Appendices

Quantitative Report Summary

Broad Park Toilets and Changing Rooms

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL Rev 1

7a Broadpark Road, Waimairi Beach

Background

This is a summary of the Quantitative report for the building 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 12 December 2012.

Building Description

The building is located in Broad Park, Waimairi Beach and is used as public toilets and changing rooms. The date of construction is estimated to be during the 1990s.

The building is approximately 15m in length by 4.7m in width with a height of 3.5m and has a footprint of approximately 70m^2. It is located 50m from the nearest structure. The predominantly flat site is located approximately 100m to the west of the Pegasus Bay coastline.

The building consists of 190mm thick concrete masonry units which form the shower wall, entrance wings and enclosed areas. These concrete masonry walls are reinforced with 12mm diameter bars at 600mm centres horizontally and vertically. The structure is partially covered by a pitched lightweight metal clad roof with a cement board lining, supported by steel tube stubs from the top of the concrete masonry walls. Plasterboard lined timber stud walls extend from internal masonry walls to the roof level. The foundations are most likely strip footings with the floor being a reinforced concrete slab on grade.

Key Damage Observed

No residual displacements of the structure were noticed during our inspection of the building.

The structure was observed to be in good condition during inspections. Minor cracking was noted in the concrete ground slab. It is unclear whether this damage has been caused by seismic activity.

Building Capacity Assessment

The building has been assessed to have a seismic capacity in the order of 80% NBS and is therefore not considered to be Earthquake Prone or Earthquake Risk.

Recommendations

As the building has been found to have a %NBS of greater than 67%, no further investigations are required.

As no critical structural weakness or immediate collapse hazards have been identified, general occupancy of the building can remain.

1. Background

GHD has been engaged by Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the public toilets and changing rooms in Broad Park.

This report is a Quantitative Assessment of the building structure, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011.

A quantitative assessment involves a full site measure of the building which is used to determine the building's bracing capacity in accordance with manufacturers' guidelines where available. When the manufacturers' guidelines are not available, values for material strengths are taken from the NZSEE guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes. The demand for the building is determined and the percentage of New Building Standard (%NBS) is assessed.

At the time of this report, no intrusive site investigation or modelling of the building structure had been carried out.

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 strength and may require non-destructive or destructive material testing, geotechnical testing and intrusive investigation.

It is anticipated that factors determining the extent of evaluation and strengthening level required will include:

- The importance level and occupancy of the building
- The placard status and amount of damage
- The age and structural type of the building
- Consideration of any critical structural weaknesses
- The extent of any earthquake damage

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 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 3.1 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 3.1 NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE

Figure 3.2 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

Figure 3.2 %NBS compared to relative risk of failure

4. Building Descriptions

4.1 General

The building is located in Broad Park, Waimairi Beach and is used as public toilets and changing rooms. The date of construction is estimated to be during the 1990s.

The building is approximately 15m in length by 4.7m in width with a height of 3.5m and has a footprint of approximately 70m^2. It is located 50m from the nearest structure. The predominantly flat site is located approximately 100m to the west of the Pegasus Bay coastline.

The structure consists of 190mm thick concrete masonry units which form the shower wall, entrance wings and enclosed areas. These concrete masonry walls are reinforced with 12mm diameter bars at 600mm centres horizontally and vertically. The structure is partially covered by a pitched lightweight metal clad roof with a cement board lining, supported by steel tube stubs from the top of the concrete masonry walls. Plasterboard lined timber stud walls extend from internal masonry walls to the roof level. The foundations are most likely strip footings tied into the reinforced concrete slab on grade floors. The floor slab is reinforced with 663 mesh at 150mm centres.

Figures 4.1 and 4.2 show the construction details. No drawings of the building were available.

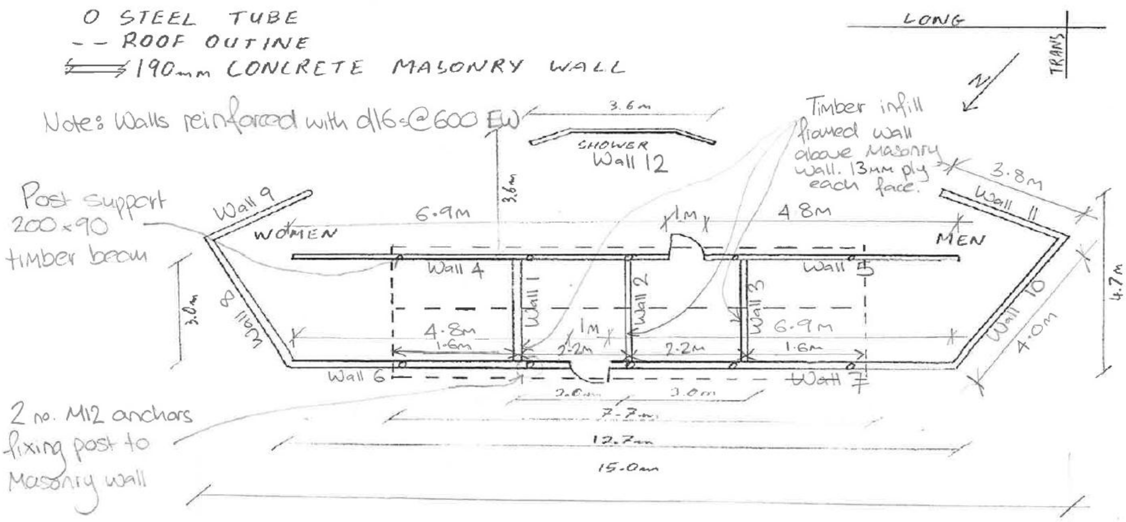


Figure 4.1 Plan of public toilets and changing rooms

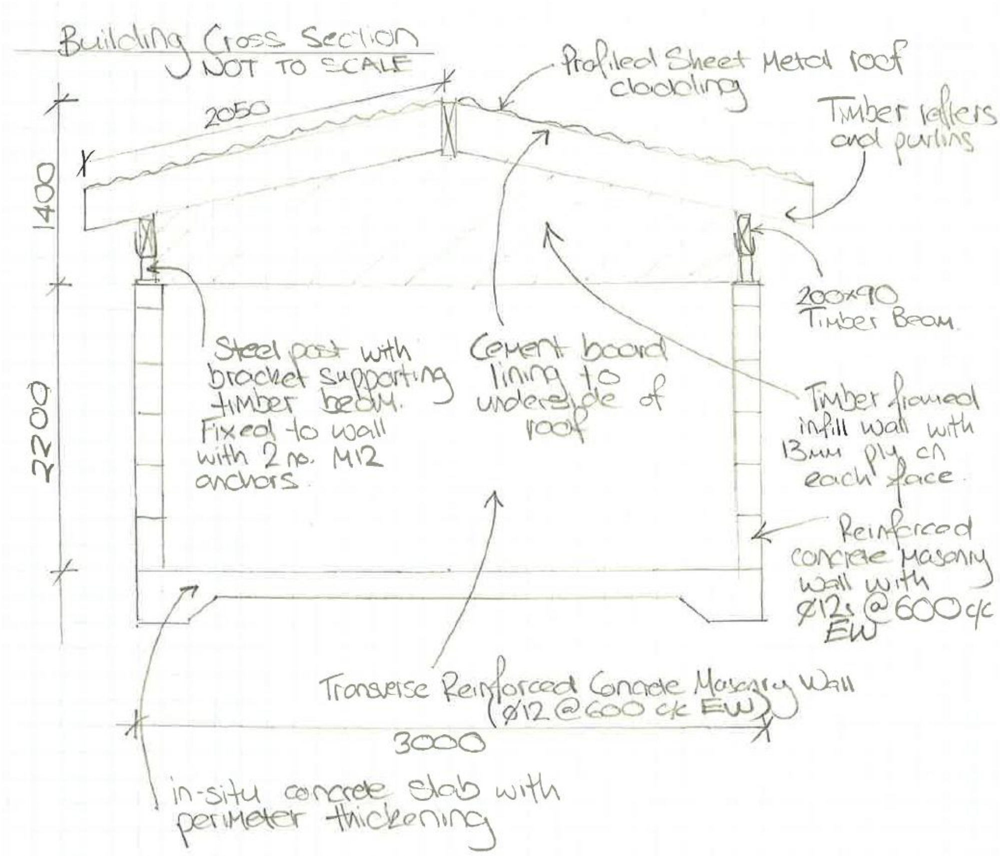


Figure 4.2 Typical section through building

4.2 Gravity Load Resisting Systems

Gravity loads acting on the building are resisted by load bearing concrete masonry walls. Gravity loads from the corrugated steel roof are transferred via the timber rafters and the timber ridge and eave beams to the concrete masonry walls. The gravity loads are transferred through the concrete masonry walls to the concrete strip footings where they are distributed into the ground. Floor gravity loads are transferred through the reinforced concrete slab to the underlying ground.

4.3 Lateral Load Resisting Systems

In the longitudinal direction, lateral seismic roof loads are transferred by composite panel action of the cement board lining and rafters to the eave beams. Cantilever action of the steel posts transfers the lateral loads from the eave beams to the concrete masonry walls. Panel action of the longitudinal concrete masonry walls resists the lateral seismic roof loads, as well as the loads from the transverse concrete masonry walls spanning onto the longitudinal concrete masonry walls. The longitudinal walls transfer the seismic loads to the foundations where they are distributed into the ground.

In the transverse direction, lateral seismic roof loads are transferred by the rafters to the ridge and eave beams. These beams transfer the lateral loads via shear and bending to the plywood lined internal timber stud walls and to the steel posts. The seismic forces are then transferred to the supporting concrete masonry walls. Panel action of the transverse concrete masonry walls resists the lateral seismic roof loads, as well as the loads from the longitudinal concrete masonry walls spanning onto the transverse concrete masonry walls. The transverse walls transfer the seismic loads to the foundations where they are distributed into the ground.

A number of concrete masonry panels with minimal lateral restraint at the top were noted in the structure. The shower area wall is one such panel and is shown in Photograph 6. These walls are restrained against lateral loading by their connection to the reinforced concrete floor slab.

5. Assessment

5.1 Site Inspection

An inspection of the buildings was undertaken on the 12th of December 2012. Both the interior and exterior of the building was inspected. It should be noted that inspection of the foundations of the structure was limited to the top of the external strips exposed above ground level.

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

5.2 Available Drawings

Drawings of the structure were not available.

Sketches of the key structural features of the building are attached as Appendix B.

5.3 Damage Assessment

5.3.1 Surrounding Buildings

No damage to nearby buildings was noted during inspections.

5.3.2 General Observations

No residual displacements of the structure were noticed during our inspection of the building.

The structure was observed to be in good condition during inspections. Minor cracking was noted in the concrete ground slab. It is unclear whether this damage has been caused by seismic activity.

5.3.3 Ground Damage

The adjacent car park has storm water ponding in a subsidence, with further investigation being necessary to identify the cause of the subsidence.

A ‘pump station’ adjacent to the site displays a concentrated subsidence, most likely due to a localised collapse of services.

6. Geotechnical Consideration

6.1 Site Description

The site is situated within a recreational reserve, within the suburb of North New Brighton in eastern Christchurch. It is relatively flat at approximately 3m above mean sea level. It is approximately 100m west of the coast (Pegasus Bay) at North New Brighton.

6.2 Published Information on Ground Conditions

6.2.1 Published Geology

The geological map of the area[1] indicates that the site is on or near the boundary of the following units:

- Holocene soils of the Christchurch Formation, comprising dominantly of sand of fixed and semi-fixed dunes and beaches
- Sand of active dunes and present day beaches

6.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that five boreholes are located within a 200m radius of the site (see Table 6.1). Of these boreholes, three of them had a lithographic logs, those logs indicates the area is typically sand with some clay and shingle layers. Varying amounts of gravel and silt are also indicated to be present.

Table 6.1 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35/12755	~1.90m	N/A	65m WNW
M35/1511	~85.60m	N/A	~200m S
M35/16536	~1.80m	~1.3m bgl	~160m SW

It should be noted that the boreholes were sunk for groundwater extraction and not for geotechnical purposes. Therefore, the amount of material recovered and available for interpretation and recording will have been variable at best and may not be representative. The logs have been written by the well driller and not a geotechnical professional or to a standard. In addition strength data is not recorded.

6.2.3 EQC Geotechnical Investigations

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

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

6.2.4 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.

The technical categories – TC1 (grey), TC2 (yellow) and TC3 (blue) describe the foundation systems most likely to be required in the corresponding areas on the maps

For TC3 areas site specific geotechnical work will be required to determine the actual foundations required for each house. In some cases this will mean TC2 level foundations will be enough in TC3 areas based on actual ground tests.

The technical category term N/A can mean either the site is" Rural & Unmapped" or "Urban Non-residential"

The site is classified as green zone TC2 (yellow).

6.2.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows no signs of liquefaction outside the building footprint or adjacent to the site, as shown in Figure 6.1. Due to the sites location near to the sea, the sand shown in the aerial photos is likely to be beach/dune sand and not liquefied material.

Figure 6.1 Post February 2011 Earthquake Aerial Photography [2]

6.2.6 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated to comprise multiple strata of sand, with varying amounts of shingle, clay, silt and gravel.

6.3 Seismicity

6.3.1 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.

[2] Aerial Photography Supplied by Koordinates sourced from http://koordinates.com/layer/3185-christchurch-post-earthquake-aerial-photos-24-feb-2011/

Table 6.2 Summary of Known Active Faults[34]

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	130 km	NW	~8.3	~300 years
Greendale (2010) Fault	30 km	W	7.1	~15,000 years
Hope Fault	100 km	N	7.2~7.5	120~200 years
Kelly Fault	110 km	NW	7.2	~150 years
Porters Pass Fault	65 km	NW	7.0	~1100 years

Recent earthquakes since 22 February 2011 have identified the presence of a previously unmapped active fault system underneath Christchurch City and the Port Hills. Research and published information on this system is in development and not generally available. Average recurrence intervals are yet to be estimated.

6.3.2 Ground Shaking Hazard

This seismic activity has produced earthquakes of Magnitude-6.3 with peak ground accelerations (PGA) up to twice the acceleration due to gravity (2g) in some parts of the city. This has resulted in widespread liquefaction throughout Christchurch.

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.

In addition, anticipation of sands overlying bedrock in excess of 500m, a 475-year PGA (peak ground acceleration) of ~0.4 (Stirling et al, 2002[4]), ground shaking is likely to be moderate to high.

6.4 Slope Failure and/or Rockfall Potential

Given the site's location in North New Brighton, a flat suburb in eastern Christchurch, global slope instability is considered negligible. However, any localised retaining structures or embankments should be further investigated to determine the site-specific slope instability potential.

6.5 Liquefaction Potential

Due to the presence of sand and evidence from the post-earthquake aerial photography it is considered possible and likely that liquefaction will occur where sands and silts are present.

Further investigation is recommended to better determine subsoil conditions. From this, a more comprehensive liquefaction assessment could be undertaken.

[3]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, pp 1878-1903, June 2002.

[4]GNS Active Faults Database

6.6 Recommendations

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

Given the anticipated ground conditions, we recommend that further investigation is undertaken. Specifically, at least one CPT investigation should be conducted to a target depth of 20m bgl.

6.7 Conclusions & Summary

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 to 30m, underlain by gravel, silt, clay and shingle deposits. Associated with this the site also has a moderate liquefaction potential, in particular where sands and/or silts are present.

It is recommended that intrusive investigation comprising at least one piezocone CPT be conducted. This will allow a more comprehensive liquefaction and ground condition assessment to be made.

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

7. Structural Analysis

7.1 Seismic Parameters

Seismic loading on the structure has been determined using New Zealand Standard 1170.5:2004.

- Site Classification — D
- Seismic Zone factor (Z)
  (Table 3.3, NZS 1170.5:2004 and NZBC Clause B1 Structure) — 0.30 (Christchurch)
- Annual Probability of Exceedance
  (Table 3.3, NZS 1170.0:2002) — 1/500 (ULS) Importance Level 2
- Return Period Factor (R_u)
  (Table 3.5, NZS 1170.5:2004) — 1.0 (ULS)

Longitudinal Direction

- Ductility Factor (μ) — 1.5
- Ductility Scaling Factor (k_μ) — 1.29
- Performance Factor (S_p) — 0.85

Transverse Direction

- Ductility Factor (μ) — 1.5
- Ductility Scaling Factor (k_μ) — 1.29
- Performance Factor (S_p) — 0.85

An increased Z factor of 0.3 for Christchurch has been used in line with recommendations from the Department of Building and Housing recommendations.

The structural performance factor, S_P, was calculated in accordance with Clause 4.4.2 NZS 1170.5.

$$S_P = 1.3 - 0.3\mu$$

The seismic weight coefficient was then calculated in accordance with Clause 5.2.1.1 of NZS 1170.5:2011. For the purposes of calculating the seismic weight coefficient a period, T_1, of 0.4 was assumed for both directions of the building. The coefficient was then calculated using Equation 5.2(1);

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

Where

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

7.2 Equivalent Static Method

Equivalent Static forces were calculated in accordance with NZS 1170.5:2004. The lateral seismic forces have been distributed to the concrete masonry walls based on the tributary areas of each wall. This is because the roof structure is unlikely to provide any diaphragm action as it is fixed intermittently to the supporting concrete masonry walls.

A ductility factor of 1.5 has been assumed in both the longitudinal and transverse direction based on the reinforced concrete masonry wall system that resists lateral seismic loading. The structure is expected to have nominally ductile behavior given the relatively lightly reinforced concrete masonry construction.

The elastic site hazard spectrum for horizontal loading:

$C(T_1)\ \ C_h\ Z\ R\ N(T,D)$

$C_h\ \ 3.0$ – Value from Table 3.1 (T ≤ 0.4s)

$Z = 0.3$ – Hazard factor determined from Table 3.3 (NZS 1170.5:2004)

$R = 1.0$ – Return period factor determined from Table 3.5 (NZS 1170.5:2004)

$N\ (T,D) = 1.0$ – Near fault factor from Clause 3.1.6 (NZS 1170.5:2004)

$C(T_1) = 3.0{\cdot}0.3{\cdot}1.0{\cdot}1.0 = 0.9$

The horizontal design action coefficient:

$$C_d(T_1)=\frac{C(T_1){\cdot}S_p}{k_\mu}=\frac{0.9\times0.85}{1.29}=0.593$$

7.3 Capacity of Structural Elements

7.3.1 Reinforced Masonry Shear Capacity

The shear capacity of the reinforced concrete masonry walls was determined using NZS 4230:2004. As there are no details as to the level of supervision during the construction stage, an Observation Type of B was used in accordance with Table 3.1. The strength reduction factor, ϕ, for shear and shear friction was taken as 0.85 in accordance with NZSEE guidelines. The overall shear capacity of the wall was calculated from Clause 10.3.2.1, Equation 10-4.

For reinforced concrete masonry;

$$V_m = 0.8 d b_w v_m$$

$$v_m = (C_1 + C_2) v_{bm}$$

$$C_2 = 33 p_w \frac{f_y}{300}$$

$$p_w = A_s/b_w d$$

Where

C_1 = wall proportion factor

v_m = shear strength of masonry

b_w = t wall thickness when fully filled

d = 0.8 x length of wall

A_s = area of reinforcement

The shear capacity component from the reinforcing steel, V_S, was calculated using equation below;

$$V_S = A_V\, f_{yt} \frac{d}{s}$$

Where

A_V = area of transverse (horizontal) reinforcing at spacing s

f_{yt} = characteristic yield strength of the transverse steel

7.3.2 Reinforced Masonry In-Plane Moment Capacity

The following method was used to calculate the in-plane moment capacity of the reinforced masonry walls.

$$\emptyset M_n = \emptyset \left[\sum F_{si}(x_i - c) + C_m\left(c - \frac{a}{2}\right) + N\left(\frac{L_w}{2} - c\right)\right]$$

Where

$$\sum F_{si} - C_m + N = 0$$

F_{si} = tension or compression force in the vertical wall reinforcement

x_i = vertical reinforcing bar position

c = neutral axis depth

C_m = masonry compressive force

a = βc = masonry compression block parameter

N = axial load

7.3.3 Reinforced Masonry Out-of-Plane Moment Capacity

The following method was used to calculate the out-of-plane moment capacity of the reinforced masonry walls.

$$\emptyset M_n = \emptyset\left(\frac{t}{2} - \frac{a}{2}\right)\left(f_{yt}A_s\right)$$

$$a = {A_s f_{yt}}\Big/{0.85f'_m b}$$

Where

t = thickness of the masonry wall

b = unit width of wall

A_s = area of steel reinforcement

A_m = area of masonry

f'_m = specified compressive strength of masonry

f_y = the strength of steel as specified by the NZSEE guidelines

7.3.4 Overturning of Concrete Masonry Walls

The unrestrained concrete masonry end walls and shower wall were checked for their capacity to resist overturning.

$$\%NBS = \frac{Resisting\ Moment\ (kNm)}{Overturning\ Moment\ (kNm)} \times 100$$

7.3.5 Steel Section Moment Capacity

The following formula was used to calculate the moment capacity of the circular hollow steel post and the steel end plate.

$$\emptyset M_n = \emptyset f_y Z_e$$

Where

f_y = yield stress used

Z_e = effective section modulus

7.3.6 Shear Capacity of Anchors

The shear capacity of the M12 anchors fixing the steel posts to the concrete masonry walls has been approximated by using the TruBolt design guides and making conservative assumptions where applicable.

$$\emptyset V_{ur} \geq V^*$$

$$\emptyset V_{ur} = min\{\emptyset V_{urc}, \emptyset V_{us}\}$$

7.3.1 %NBS

The shear and moment capacities of the structural elements were compared to their respective demands to determine the overall %NBS for each element.

$$\%\mathrm{NBS} = \frac{\emptyset \mathrm{V_n}}{\mathrm{V^*}} \text{ x } 100$$

$$\%\mathrm{NBS} = \frac{\emptyset \mathrm{M_n}}{\mathrm{M^*}} \text{ x } 100$$

8. Results

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. Our analysis applied live loads, imposed dead loads and seismic loads to the structure. The elements were then assessed against their respective load capacities.

Our calculations show that the structure achieves **100% NBS** and is therefore **Earthquake Prone**.

The structural analysis results are discussed in the following sections.

8.1.1 Reinforced Concrete Masonry Walls

In-Plane Shear

The reinforced concrete masonry walls achieve **100% NBS** under in-plane shear seismic loading.

In-Plane Moment

The reinforced concrete masonry walls achieve **100% NBS** when considering in-plane bending of the walls.

Out-of-Plane Moment

The reinforced concrete masonry walls achieve **80% NBS** when considering out-of-plane bending of the walls.

Several 190mm thick reinforced concrete masonry walls are restrained out-of-plane at eaves level by reinforced bond beams spanning between return walls. The remaining walls are effectively unrestrained along the top edge as the roof structure is supported by steel posts at intermittent points along the concrete masonry walls.

Overturning

The unrestrained reinforced concrete masonry end walls and shower wall achieve **100% NBS** when considering out-of-plane overturning of the walls.

8.2 Steel Elements

Steel Post

The 80mm diameter steel posts achieve **100% NBS** when considering cantilever bending of the posts.

The steel posts that support the timber framed roof structure are required to cantilever in order to transfer lateral seismic loads from the roof structure to the supporting concrete masonry walls.

Steel End-Plate

The 10mm thick steel end plate achieves **100% NBS** when considering bending.

The plate is required to transfer forces from the steel posts to the M12 anchors through bending in order to distribute forces to the supporting concrete masonry walls.

M12 Anchors

The 2 no. M12 anchors achieve **100% NBS** when considering shear loading.

The critical consideration for the performance of the anchors in the concrete masonry walls is the failure of the concrete masonry under shear. The worst case occurs when the loading on the anchors is perpendicular to the direction the wall spans. The concrete masonry walls are 190mm thick and the anchors are centrally placed. As a result, the edge distances are within the required limits.

8.3 Summary

Element	Seismic Action	%NBS
Longitudinal Direction		
Reinforced Concrete Masonry Walls	In-Plane Shear	100
	In-Plane Bending	100
	Out-of-Plane Bending	80
	Overturning	100
Transverse Direction		
Reinforced Concrete Masonry Walls	In-Plane Shear	100
	In-Plane Bending	100
	Out-of-Plane Bending	80
	Overturning	100
Steel Elements		
Steel Posts	Bending	100
End Plate	Bending	100
M12 Anchors	Shear	100

Table 8.1 Summary of %NBS scores

8.4 Discussion of Results

The results obtained from the analysis are reasonably consistent with those expected for a building of this age and construction type.

The building is assumed to have been designed in the early 1990s and was likely designed in accordance with the earlier loading standard, NZS 4203:1984. The design loads used are likely to have been less than those required by the current loading standard.

The building performs well when considering in-plane forces, achieving 100% NBS in both the transverse and longitudinal directions.

The concrete masonry end walls and shower wall perform relatively poorly out-of-plane as they are effectively unrestrained along the top and lateral edges and rely on cantilever action to resist lateral seismic loads. These concrete masonry walls are insufficiently reinforced to resist 100% of the out-of-plane lateral seismic loads.

9. Conclusions and Recommendations

The building has been assessed to have a seismic capacity in the order of 80% NBS and is therefore not considered to be Earthquake Prone or Earthquake Risk.

As no immediate collapse hazards or critical structural weaknesses have been identified for the structure, general occupancy of the building can remain.

10. Limitations

10.1 General

This report has been prepared subject to the following limitations:

- The foundations of the building were unable to be inspected beyond those exposed above ground level externally.
- No material testing has been undertaken.

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 report.

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 Ministry of Education 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.

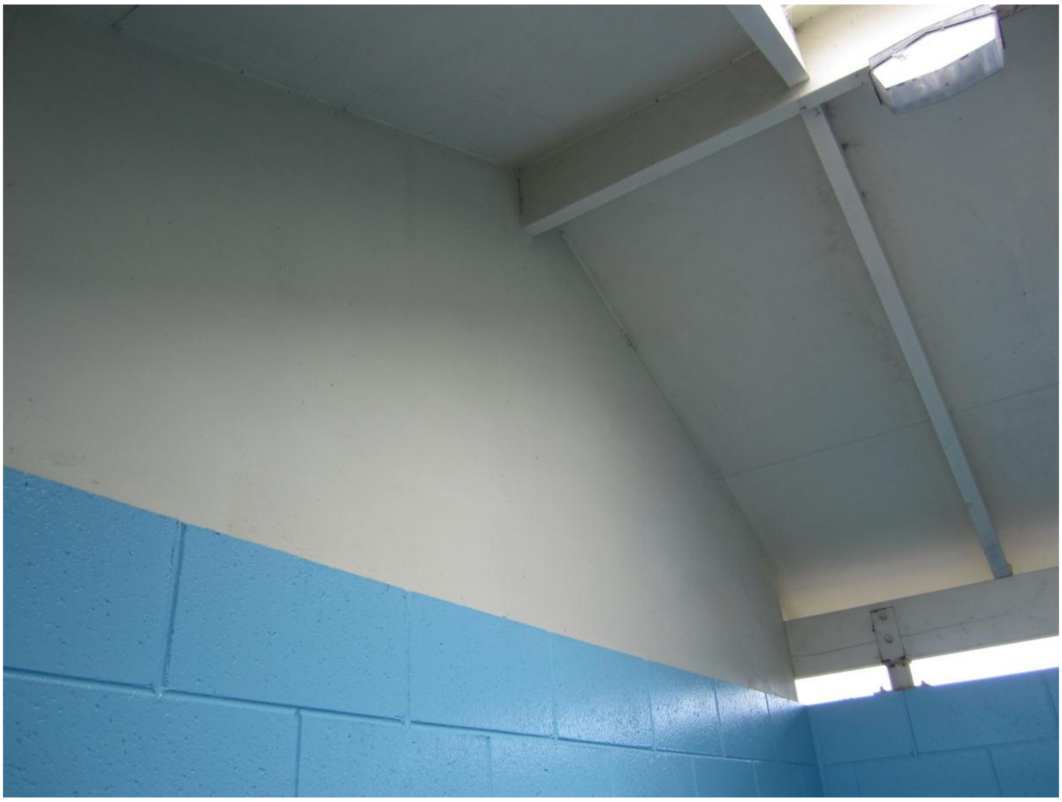
Appendix A

Photographs

Photograph 1 View of the building from the north

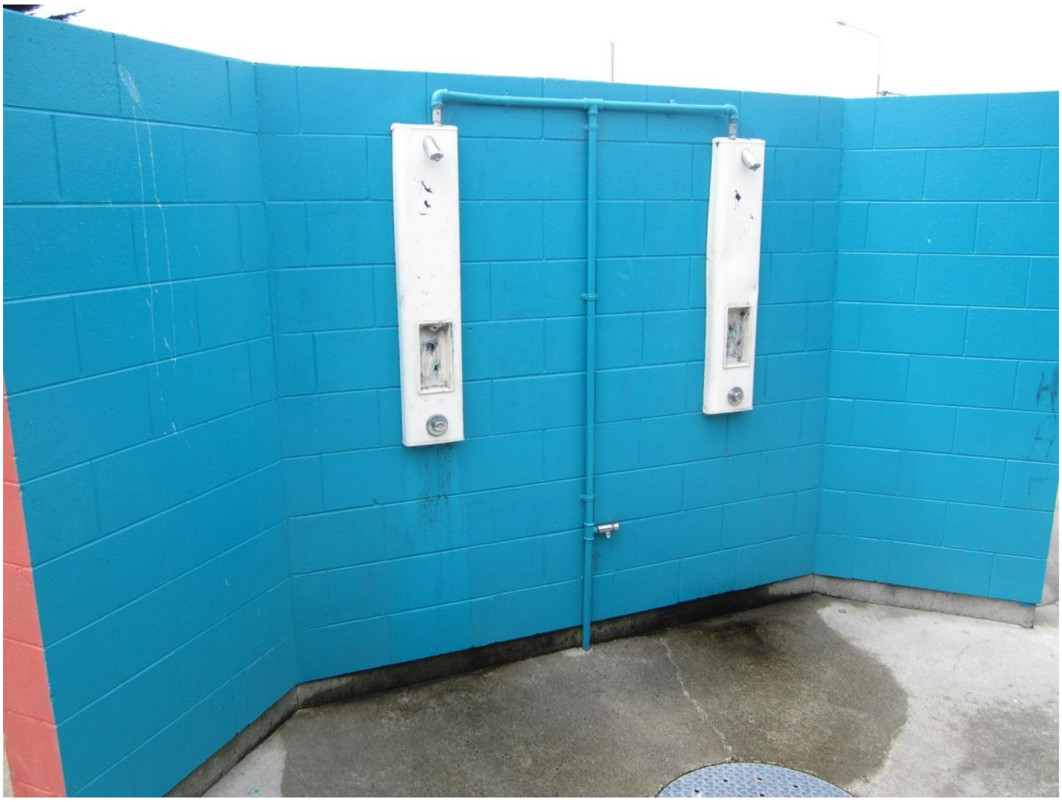
Photograph 2 View of the rear wall

Photograph 3 **View of concrete masonry wing walls and shower wall**



Photograph 4 **View of internal ply lined timber framed gable walls**

Photograph 5 View of timber roof structure



Photograph 6 Unrestrained wall forming public showers

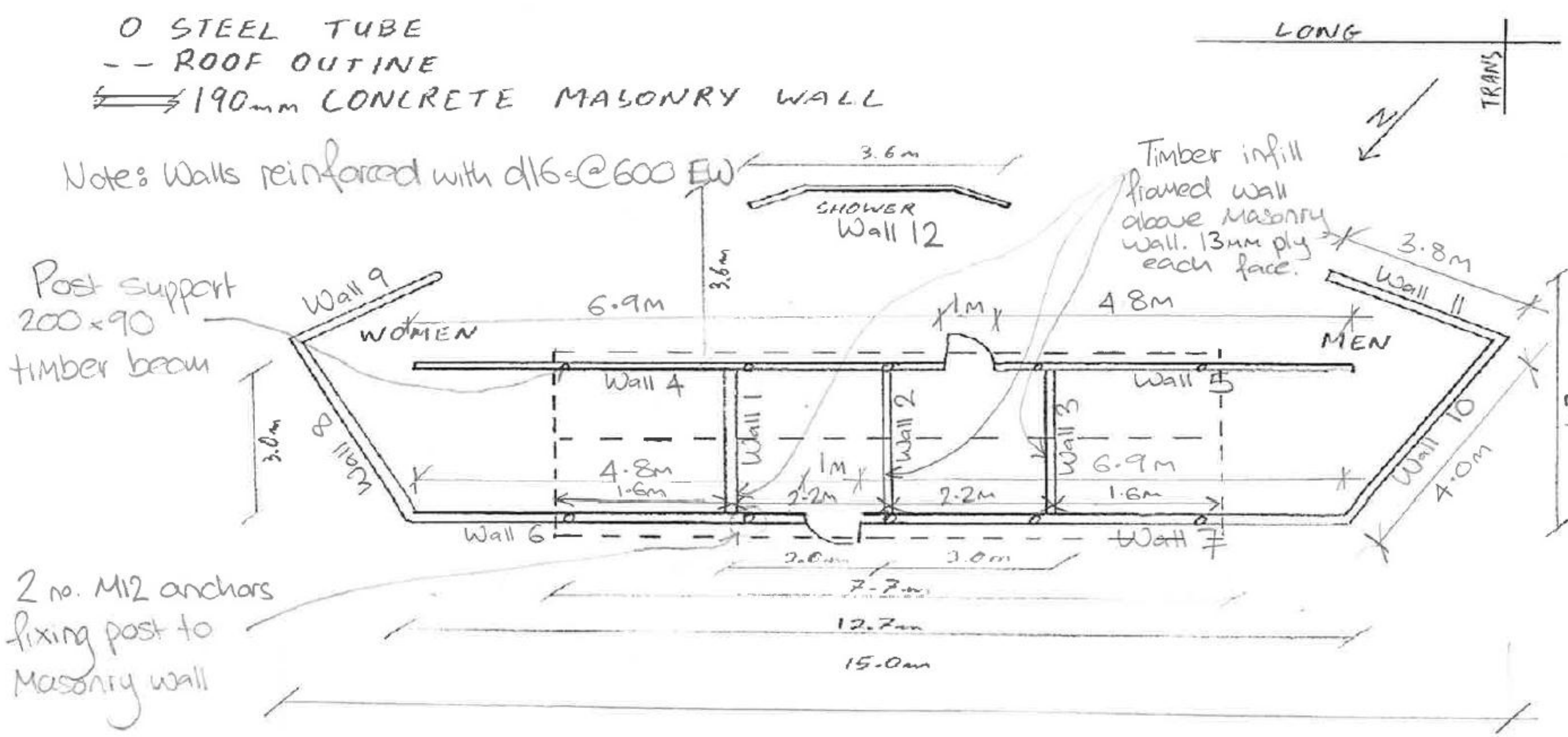
Photograph 7 **Bracket fixing steel posts to the 200x90 timber beam**

Photograph 8 **2 No. M12 anchors fixing the steel end plate to the concrete masonry walls**

Photograph 9 Fixing between timber rafters and the supporting timber beam

Appendix B

Existing Drawings / Sketches


O STEEL TUBE
-- ROOF OUTINE
190mm CONCRETE MASONRY WALL
Note: Walls reinforced with d16s@600 EW
Post support 200×90 timber beam
2 no. M12 anchors fixing post to Masonry wall
Timber infill framed wall above Masonry wall. 13mm ply each face.
LONG
TRANS
N
SHOWER
Wall 12
Wall 9
Wall 8
Wall 4
Wall 1
Wall 2
Wall 3
Wall 5
Wall 6
Wall 7
Wall 10
Wall 11
WOMEN
MEN
3.6m
3.6m
6.9m
4.8m
1m
1m
4.8m
1.6m
2.2m
2.2m
6.9m
1.6m
3.0m
3.8m
4.0m
4.7m
2.0m
3.0m
7.7m
12.7m
15.0m

Appendix C

CERA IEP Spreadsheet

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name: Broad Park Toilet & Changing Rooms
Building Address: Unit: — No: 7a Street: Broad Park Road, Waimairi Beach
Legal Description:
GPS south: Degrees / Min / Sec
GPS east:
Building Unique Identifier (CCC): PRK_0121_BLDG_001 EQ2

Reviewer: Hamish Mackinven
CPEng No: 1003941
Company: GHD
Company project number: 513090211
Company phone number: 04 472 0799
Date of submission: 17/02/2014
Inspection Date: 12/12/2012
Revision: FINAL Rev 1
Is there a full report with this summary? yes

Site

Site slope: flat
Soil type: mixed
Site Class (to NZS1170.5): D
Proximity to waterway (m, if <100m):
Proximity to clifftop (m, if < 100m):
Proximity to cliff base (m,if <100m):
Max retaining height (m): 0
Soil Profile (if available):
If Ground improvement on site, describe:
Approx site elevation (m): 3.00

Building

No. of storeys above ground: 1 (single storey = 1)
Ground floor split? no
Storeys below ground: 0
Foundation type: strip footings
Building height (m): 3.60
Floor footprint area (approx): 70
Age of Building (years): 20
Ground floor elevation (Absolute) (m):
Ground floor elevation above ground (m):
if Foundation type is other, describe:
height from ground to level of uppermost seismic mass (for IEP only) (m):
Date of design: 1992-2004

Strengthening present? no
If so, when (year)?
And what load level (%g)?
Brief strengthening description:
Use (ground floor): other (specify)
Use (upper floors):
Use notes (if required): Public Toilet & Changing Room
Importance level (to NZS1170.5): IL2

Gravity Structure
Gravity System: load bearing walls
Roof: timber framed — rafter type, purlin type and cladding:
Floors: other (note) — describe sytem: slab on grade
Beams: timber — type: 200 x 90
Columns:
Walls:

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

Lateral system across: fully filled CMU
Ductility assumed, μ: 1.25
Period across: 0.40 #### enter height above at H31
Total deflection (ULS) (mm):
maximum interstorey deflection (ULS) (mm):
note total length of wall at ground (m):
estimate or calculation? estimated
estimate or calculation?
estimate or calculation?

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

Non-structural elements
Stairs:
Wall cladding:
Roof Cladding: Metal
Glazing:
Ceilings:
Services(list):
describe

Available documentation

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

Damage

Site: (refer DEE Table 4-2)
Site performance:
Describe damage:
Settlement: none observed — notes (if applicable):
Differential settlement: none observed — notes (if applicable):
Liquefaction: none apparent — notes (if applicable):
Lateral Spread: none apparent — notes (if applicable):
Differential lateral spread: none apparent — notes (if applicable):
Ground cracks: none apparent — notes (if applicable):
Damage to area: none apparent — notes (if applicable):

Building:
Current Placard Status:

Along — Damage ratio: 0%
Describe (summary): No damage to the structure observed.
Describe how damage ratio arrived at: No damage to the structure observed.

Across — Damage ratio: 0%
Describe (summary): No damage to the structure observed.

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

Diaphragms — Damage?: no — Describe:
CSWs: — Damage?: no — Describe:
Pounding: — Damage?: no — Describe:
Non-structural: — Damage?: no — Describe:

Recommendations

Level of repair/strengthening required: — Describe:
Building Consent required: — Describe:
Interim occupancy recommendations: — Describe:

Along
Assessed %NBS before e'quakes: 80% #### %NBS from IEP below
Assessed %NBS after e'quakes: 80%
If IEP not used, please detail assessment methodology: Detailed Assessment

Across
Assessed %NBS before e'quakes: 80% #### %NBS from IEP below
Assessed %NBS after e'quakes: 80%

GHD

Level 11, Guardian Trust House
15 Willeston street, Wellington 6011
T: 64 4 472 0799   F: 64 4 472 0833   E: wgtnmail@ghd.com

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