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Toilets Marshland Reserve
PRK 0084 BLDG 001 EQ2

Detailed Engineering Evaluation

Quantitative Report

Version FINAL REV1

420 Prestons Road, Marshland

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

Toilets Marshland Reserve
PRK 0084 BLDG 001 EQ2

Detailed Engineering Evaluation
Quantitative Report
Version FINAL REV1

420 Prestons Road, Marshland

Christchurch City Council

Prepared By
Paul Clarke

Reviewed By
Stephen Lee

Date
01 August 2013

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Quantitative Report Summary

Toilets Marshland Reserve

PRK 0084 BLDG 001 EQ2

Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL REV1

420 Prestons Road, Marshland

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, visual inspections on 21 September 2012 and available drawings itemised in 5.2.

Building Description

The main structure consists of concrete masonry walls with a roof structure formed by a clad A-frame. The approximately 60 degree pitched roof is formed by corrugated steel sheets supported by timber purlins and Rectangular Hollow Section A-frames, clad internally by particle board. Partial fill 140mm concrete masonry form the internal, entrance and external walls. Foundations consist of strip footings with a floor formed by a reinforced concrete slab on grade. The pump house extension, on the west end of the original structure, matches the pitch of the original, but comprises solely a roof and floor. Construction details of the pump house match the original structure except the roof structure is formed completely from timber members.

Key Damage Observed

No damage was observed in the structure. The baseplate bolts of the A-frame have suffered substantial corrosion at some locations. These bolts have been assumed to provide adequate support to resist seismic demand.

Building Capacity Assessment

Based on the results of the quantitative assessment the building scored 52% NBS. Therefore the building is Earthquake Risk.

Recommendations

Bolts suffering from excessive corrosion should be replaced as best practice.

Currently the two concrete masonry gable walls are failing with a %NBS of 52%. Design concepts may be considered to strengthen these walls to 100% NBS.

1. Background

GHD has been engaged by Christchurch City Council (CCC) to undertake a detailed engineering evaluation of Toilets Marshland Reserve.

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.

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 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

Figure 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 2 %NBS compared to relative risk of failure

4. Building Description

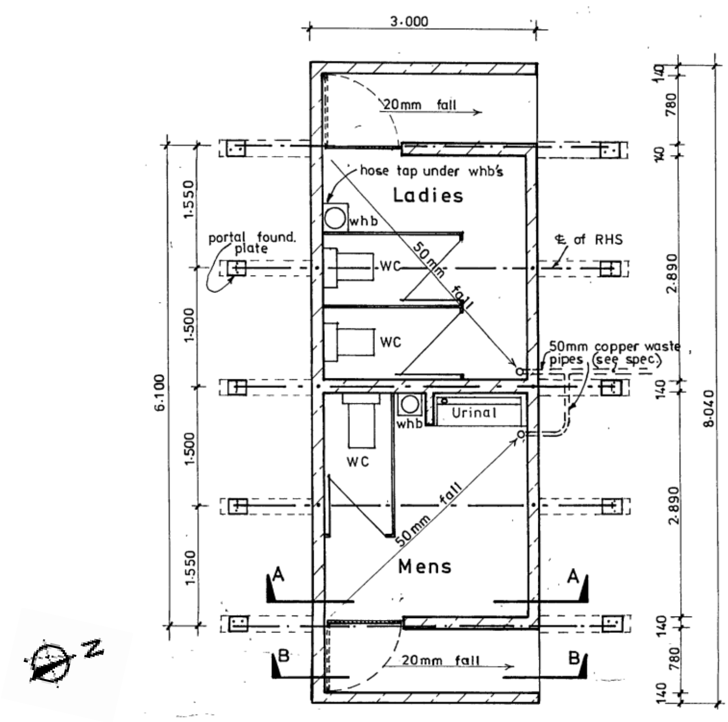
4.1 General

The building is located at 420 Preston's Road, Marshlands. The original building was constructed in 1976 with a pump house added in 1983. The sole purpose of the building is a public toilet.

The main structure consists of concrete masonry walls with a roof structure formed by a clad A-frame. The approximately 60 degree pitched roof is formed by corrugated steel sheets supported by timber purlins and Rectangular Hollow Section A-frames, clad internally by particle board. Partial fill 140mm concrete masonry form the internal, entrance and external walls. Foundations consist of strip footings with a floor formed by slab on grade. The pump house extension, on the west end of the original structure, matches the pitch of the original, but comprises solely a roof and floor. Construction details of the pump house match the original structure except the roof structure is formed completely from timber members.

The original building is approximately 8.0m in length by 3.0m in width and 4.3m in height. The pump house is 2.7m in length by 1.5m in width with a height of 2.0m. The overall footprint is approximately $29m^2$. The Marshland Scout Hall is approximately 2m from the structure. This building, while constructed of durable materials, has not been well maintained however, no obvious earthquake damage was observed. The predominantly flat site is located 1.5 km southeast of Styx River.

Figure 3 Plan of Original Structure



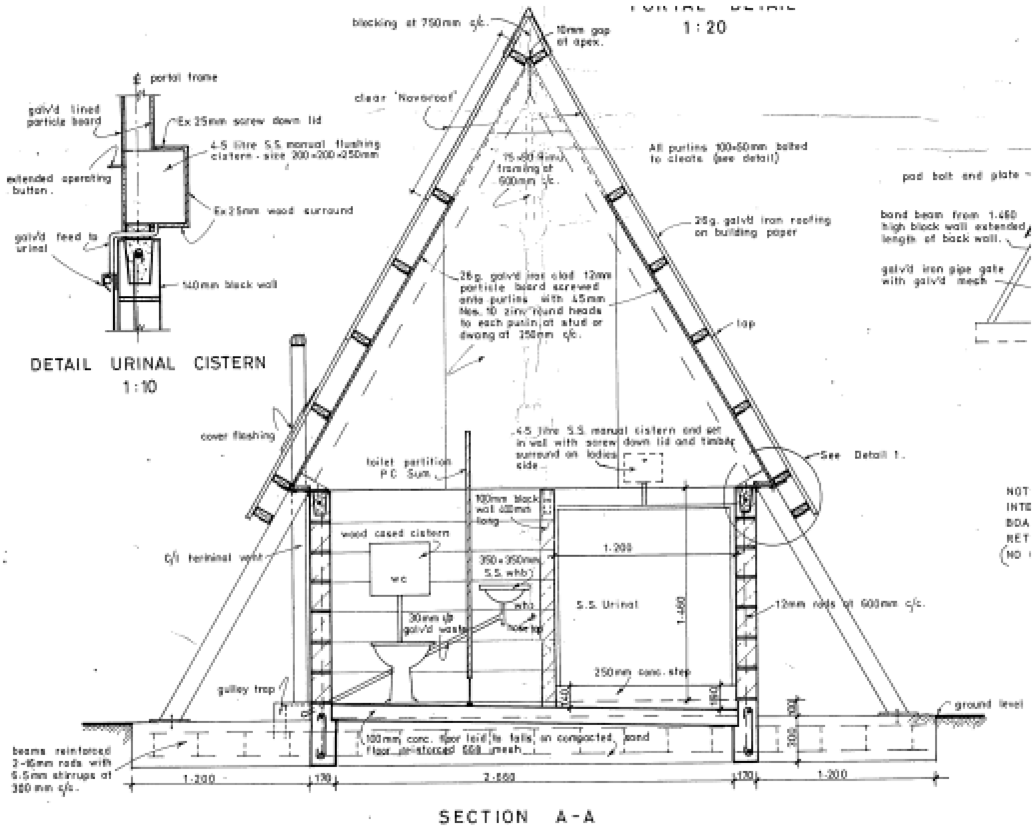


Figure 4 Section of Original Structure

4.2 Gravity Load Resisting System

Gravity roof loads are supported by timber purlins which transfer the load to the RHS sections of the A-frame, which in turn transfer the loads to the strip footings by a combination of axial compression of the RHS section and masonry walls. The masonry walls are supported on the strip footings, with the floor being a concrete slab on grade. The A-Frames are supported on a reinforced concrete ground beams.

4.3 Lateral Load Resisting System

In the transverse direction, lateral roof loads are resisted by the frame action of the roof A-frame. The lateral roof loads are transferred via timber purlins to the Rectangular Hollow Sections of the A-frames. The A-frame mechanism, whereby loads triangulate through the steelwork, transfer these loads to the foundations. Separately, the concrete masonry walls resist perpendicular lateral loads by spanning horizontally between walls in the plane of loading. These in-plane walls transfer the lateral loads to the foundations by the panel action of the concrete masonry.

In longitudinal direction, the lateral roof loads are transferred via timber purlins to the Rectangular Hollow Sections of the A-frames. These A-frames are restrained in the longitudinal direction at the foundations and the top edge of the concrete masonry walls, thus the lateral roof loads are transferred to both these elements. The concrete masonry walls resist perpendicular lateral loads by spanning horizontally between walls in the plane of loading. These in-plane walls transfer these, and lateral roof loads from the A-frame, to the foundations by the panel action of the concrete masonry. Those masonry panels lacking restraint from perpendicular walls, cantilever from the foundations where overturning restraint is provided.

5. Assessment

5.1 Site Inspection

An inspection of the building was undertaken on the 21st of September 2012. Both the interior and exterior of the building were inspected. The main structural components of the building were all able to be viewed. 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.

A Hilti PS 200 Ferroscan was used to confirm the position, depth and diameter of the reinforcement in the partial fill concrete masonry walls. This scanning equipment using electro-magnetic fields allowed for the determination of the capacity of the various walls in the building. In the case of conflicting results, the most conservative bar diameter was chosen for the capacity calculations. Reinforcement was found to be as detailed in drawings in all locations except one. This was not a significant reduction in quantity of reinforcement and was likely an oversight during construction. The reduced reinforcement was considered in the design check.

5.2 Available Drawings

The construction drawings of both the original and additional structure were made available.

All drawings are attached as Appendix B.

6. Damage Assessment

6.1 Surrounding Buildings

There was no observed earthquake damage to the surrounding buildings.

6.2 Residual Displacements and General Observations

There were no settlement or damage issues identified during the inspection of the Marshland reserve toilets. The baseplate bolts of the A-frame have suffered substantial corrosion at some locations. These bolts have been assumed to provide adequate support to resist seismic demand.

6.3 Ground Damage

No ground damage was observed during our inspection of the site.

7. Structural Analysis

7.1 Seismic Parameters

Earthquake loads shall be calculated using New Zealand Code.

◗ 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)	1/100 (ULS) Importance Level 1
◗ Return Period Factor (Ru)	
(Table 3.5, NZS 1170.5:2004)	0.5 (ULS)
◗ Ductility Factor (μ)	1.25
◗ Ductility Scaling Factor (k_μ)	1.14
◗ Performance Factor (Sp), based on NZS 3.1.0.1	0.925
◗ Gravitational Constant (g)	9.81 m/s^2

An increased Z factor of 0.3 for Christchurch has been used in line with recommendations from the Department of Building and Housing recommendations resulting in a reduced % NBS score.

7.2 Equivalent Static Method

Equivalent Static forces were calculated in accordance with NZS 1170.5:2004. A ductility factor of 1.25 has been assumed given the age and partially filled construction used. The structure is expected to have nominally ductile behavior given the lightly reinforced partially filled concrete masonry construction.

The elastic site hazard spectrum for horizontal loading:

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

$C_h=3.0$ – Value from 3.1 table for the period (T=0.4s)

Z=0.3 – Hazard factor determined from the table 3.3 (NZS 1170.5:2004)

R=0.5 – Return period factor determined from the table 3.5 (NZS 1170.5:2004)

N (T,D) = 1.0 – Near fault factor- clause 3.1.6. (NZS 1170.5:2004)

$C(T_1)= 3.0 \cdot 0.3 \cdot 0.5 \cdot 1.0 = 0.45$

The horizontal design action coefficient:

$$C_d(T_1)=\frac{C(T_1)\cdot S_p}{k_\mu}=\frac{0.45\cdot 0.925}{1.14}=0.37$$

The structure is relatively simple, with direct load paths and no opportunity for redistribution of loads through the structure. Thus elements were considered individually, and subject to loads from seismic self-weight or those directly applied.

7.3 Dependable Capacity

7.3.1 Reinforced Masonry-Shear Capacity

The shear capacity of the reinforced concrete masonry shear walls was calculated using Sections 10.3 of NZS 4230:2004, and 11.3 of NZS 3101:2006.

Shear capacity comprises two components; that from the masonry, and that from the steel reinforcement. These are calculated separately, and added together.

This first involved calculating the shear capacity of the masonry, V_m, based on the following equations:

For reinforced 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 = 1.0;

v_m = shear strength of masonry;

b_w = t wall thickness when fully filled;

d = 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;

d = depth from compression end of wall to centroid of tension force.

7.3.2 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.

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

$$a = {A_s f_{yt}} / {\emptyset A_m f'_m}$$

Where

t = wall thickness

A_s = area steel

A_m = area of masonry

f'_m = masonry strength

7.3.3 Rectangular Hollow Section Moment Capacity

The following formula was used to calculate the moment capacity of the rectangular hollow sections.

$$\emptyset M_s = \emptyset f_y Z_e$$

Where

fy = yield stress used in design

Ze = effective section modulus

8. Geotechnical Consideration

8.1 Site Description

The site is situated within the rural residential area of Marshland, north of Christchurch. It is relatively flat at approximately 30m above mean sea level. It is approximately 1.5km southeast of the Styx River, and 4.2km west of the coast (Pegasus Bay).

8.2 Public Information on Ground Conditions

8.2.1 Published Geology

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

- Peat swamps, now drained, being Holocene soils of the Yaldhurst Member, sub-group of the Springston Formation; and,
- Dominantly sand of fixed and semi-fixed dunes and beaches, being Holocene marine soils of the Christchurch Formation.

8.2.2 Environmental Canterbury Logs

Information from Environment Canterbury (ECan) indicates that five boreholes containing lithographic logs are located within 200m of the site (see **Table 1**). The site geology described in these logs indicates the area is predominantly underlain by shallow sand and silt with interbedded sand and gravel at depth. Organic peat material is recorded at depth in one borehole.

Table 1 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35/1564	69m	4.3m	180m E
M35/6362	31m	0.2m	110m NW
M35/8443	30m	0.1m	130m W
M35/9993	32m	0.4m	170m NW
M35/11720	10m	-	170m SE

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.

[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. Institute of Geological and Nuclear Sciences Limited: Lower Hutt.

8.2.3 EQC Geotechnical Investigation

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

8.2.4 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 site is categorised technical category "N/A – Rural & Unmapped".[2] The nearest zoned residential area (600m due east of the site) is zoned TC2 - minor to moderate land damage from liquefaction is possible in future significant earthquakes. Land to the west is also zoned TC2.

8.2.5 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 5. The grey areas southeast of the tennis courts are the accessway.

[2] CERA Land check website, http://cera.govt.nz/my-property/

Figure 5 Post February 2011 Earthquake Aerial Photography3

8.2.6 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated to comprise multiple strata of silty sand with gravel. Layers containing peat are also indicated to be present at depth.

8.3 Seismicity

8.3.1 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.

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

Table 2 Summary of Known Active Faults[4,5]

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	120 km	NW	~8.3	~300 years
Greendale (2010) Fault	26 km	W	7.1	~15,000 years
Hope Fault	100 km	N	7.2~7.5	120~200 years
Kelly Fault	100 km	NW	7.2	~150 years
Porters Pass Fault	60 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.

8.3.2 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.

8.3.3 Slope Failure and/or Rockfall Potential

Given the site's location in Marshland, a flat area north of 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.

8.3.4 Liquefaction Potential

The liquefaction potential of the site is considered to be minor to moderate based on:

- the adjacent land zoned by CERA as TC2; and,
- the presence of liquefaction-prone soils observed in nearby boreholes comprising shallow silt sands.

However, liquefaction was not observed in the aerial photos following the 22 February 2011 earthquake.

[4] 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.

[5] GNS Active Faults Database

8.3.5 Conclusions & 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 stratified alluvial deposits, comprising silty sand overlying gravels and sand. Associated with this the site also has a minor to moderate liquefaction potential where sands and silts 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 an intrusive investigation comprising of at least one piezocone CPT be conducted.

9. Results of Analysis

The structure was considered as separate elements such as Rectangular Hollow Section lengths or wall panels. Each element was checked for internal section capacity and restraint/support capacity from other elements such as foundations.

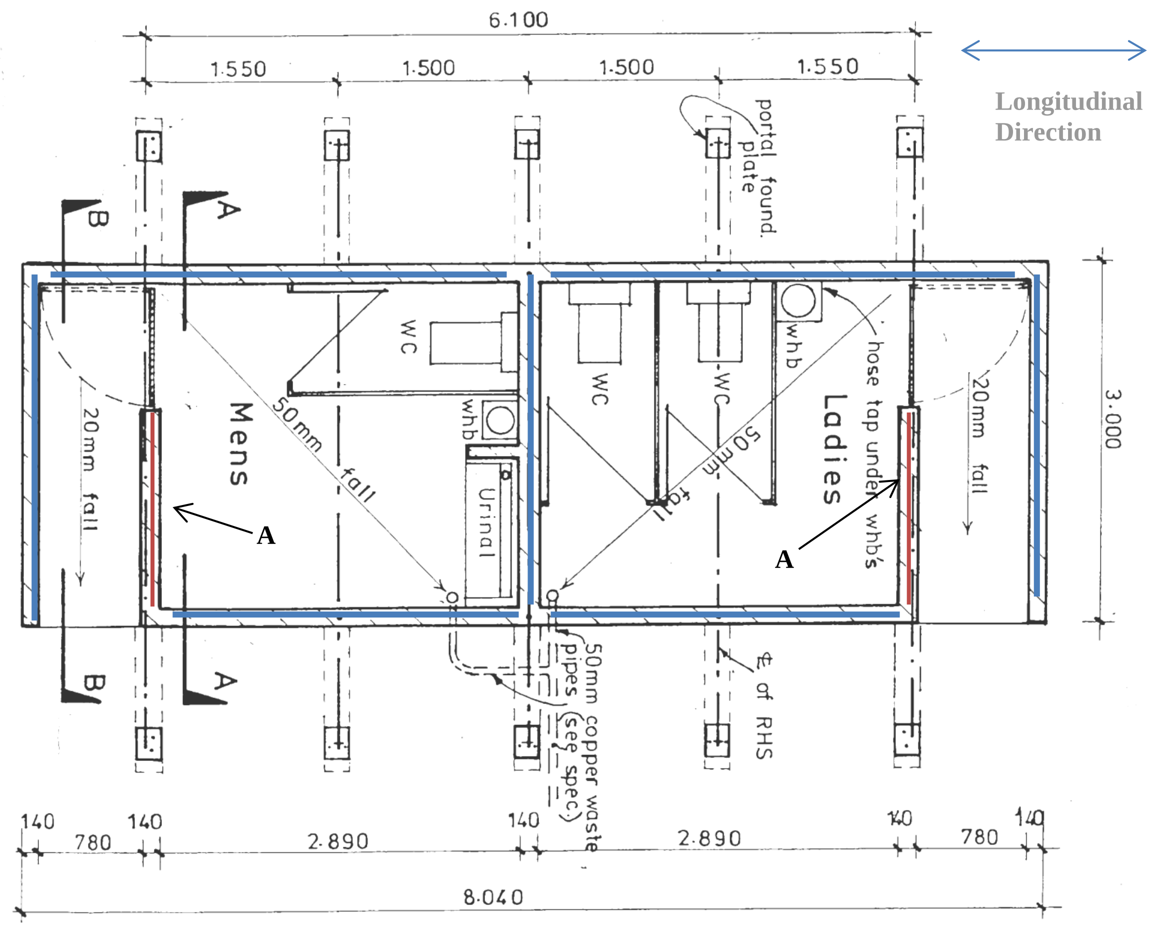


Figure 6 Plan showing wall panels as considered.

The critical loading condition for concrete masonry panels in this structure is lateral loading perpendicular to the wall panels. The critical load condition for Wall A is lateral loads in the longitudinal direction. Both loading directions are considered critical for RHS (Rectangular Hollow Sections).

Table 3 %NBS of Structural Elements

Element	% NBS
Wall A	52%
Remaining Walls	>100%
RHS (Longitudinal)	>100%
RHS (Transverse)	>100%

9.1 Discussion of Results

The results obtained from the analysis are generally consistent with those expected for a building of this size, age and construction type, founded on Class D soils.

The Toilets Marshland Reserve was designed in 1976 and was likely designed in accordance with the previous loading standard, NZS 1900:1965, superseded that year. The design loads used are likely to have been less than those required by the current loading standard. However, given the low seismic demand on elements due to the structure's limited height, a good distribution of load resisting elements and a robust basic reinforcement detailing, it is likely that this structure would perform well against current standards. While is the case for most elements, some do not have suitable load paths for seismic loads.

All structural elements except Wall A has been found to have a %NBS greater than 100%. Wall A, the two gable walls, were found to have a %NBS of 52%. The wall is unrestrained along the top and one vertical edge, requiring the wall to cantilever from the foundations beneath. The out-of-plane moment and shear capacity of the wall was found to be greater than 100% in this condition, however the reinforced concrete strip footing could not provide adequate restraint. The strip footing was found to fail in torsion with a %NBS of 52%.

10. Conclusions and Recommendations

The building overall has been assessed as having a seismic capacity of 52% NBS and is therefore classified as being ‘Earthquake Risk’.

Bolts suffering from excessive corrosion should be replaced as best practice.

Currently the two concrete masonry gable walls are failing with a %NBS of 52%. Design concepts may be considered to strengthen these walls to 100% NBS.

11. Limitations

11.1 General

This report has been prepared subject to the following limitations:

- Available drawings itemised in 5.2 was used in the assessment.
- The foundations of the building were unable to be inspected beyond those exposed above ground level externally.
- No level or verticality surveys have been undertaken.
- 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.

11.2 Geotechnical Limitations

The data and advice provided herein relate only to the project and structures described herein and must be reviewed by a competent geotechnical professional before being used for any other purpose. GHD Limited (GHD) accepts no responsibility for other use of the data by third parties.

Where drill hole or test pit logs, cone tests, laboratory tests, geophysical tests and similar work have been performed and recorded by others under a separate commission, the data is included and used in the form provided by others. The responsibility for the accuracy of such data remains with the issuing authority, not with GHD.

The advice tendered in this report is based on information obtained from the desk study investigation location test points and sample points. It is not warranted in respect to the conditions that may be encountered across the site other than at these locations. It is emphasised that the actual characteristics of the subsurface materials may vary significantly between adjacent test points, sample intervals and at locations other than where observations, explorations and investigations have been made. Subsurface conditions, including groundwater levels and contaminant concentrations can change in a limited time. This should be borne in mind when assessing the data.

It should be noted that because of the inherent uncertainties in subsurface evaluations, changed or unanticipated subsurface conditions may occur that could affect total project cost and/or execution. GHD does not accept responsibility for the consequences of significant variances in the conditions and the requirements for execution of the work.

The subsurface and surface earthworks, excavations and foundations should be examined by a suitably qualified and experienced Engineer who shall judge whether the revealed conditions accord with both the assumptions in this report and/or the design of the works. If they do not accord, the Engineer shall modify advice in this report and/or design of the works to accord with the circumstances that are revealed.

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

Photo 1. View of toilets from the east.

Photo 2. Gable and entrance wing walls.

Photo 3. Broken block corner exposing concrete of bond beam within.

Photo 4. A-frame connection to bond beam.

Photo 5.   A-frame RHS supported on tip of ground beam.

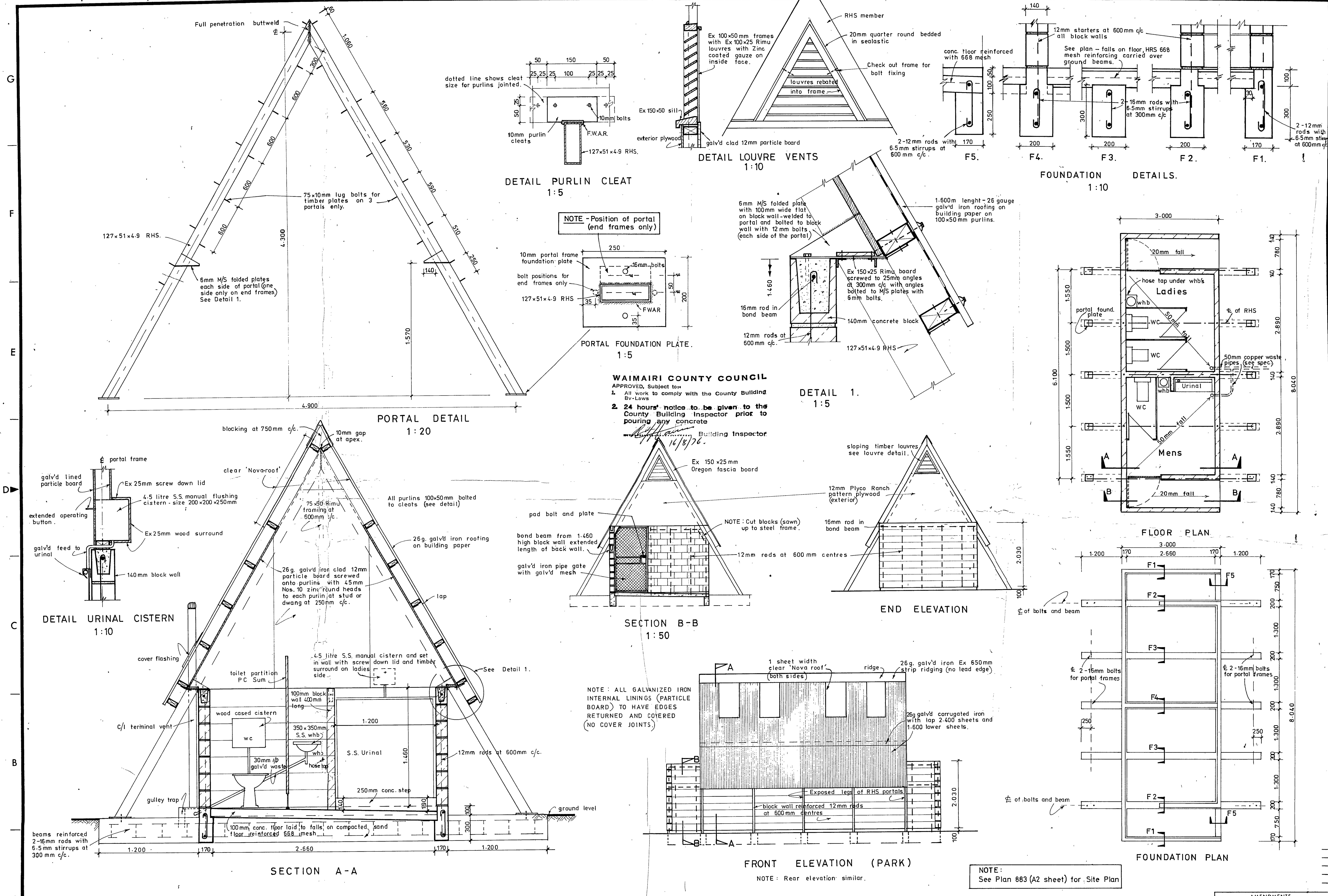
Photo 6.   Excessive corrosion of base-plate bolts.

Photo 7.   RHS framing gable wall at corner of building.

Photo 8.   Internal view of ceiling lining, window and A-frame.

Appendix B

Existing Drawings



APPROVED		NAME	DATE
COUNTY ENGINEER	SURVEYED		
DATE 7/76	DRAWN	L. Wooding	7/76
	TRACED	P. Wills	7/76
	CHECKED		
	DESIGNED	L. Wooding	5/76
	DES. CHCK.		

DATUM ....
BENCH MK. ....
LEVEL BK No ....P....
FIELD BK No ....P....

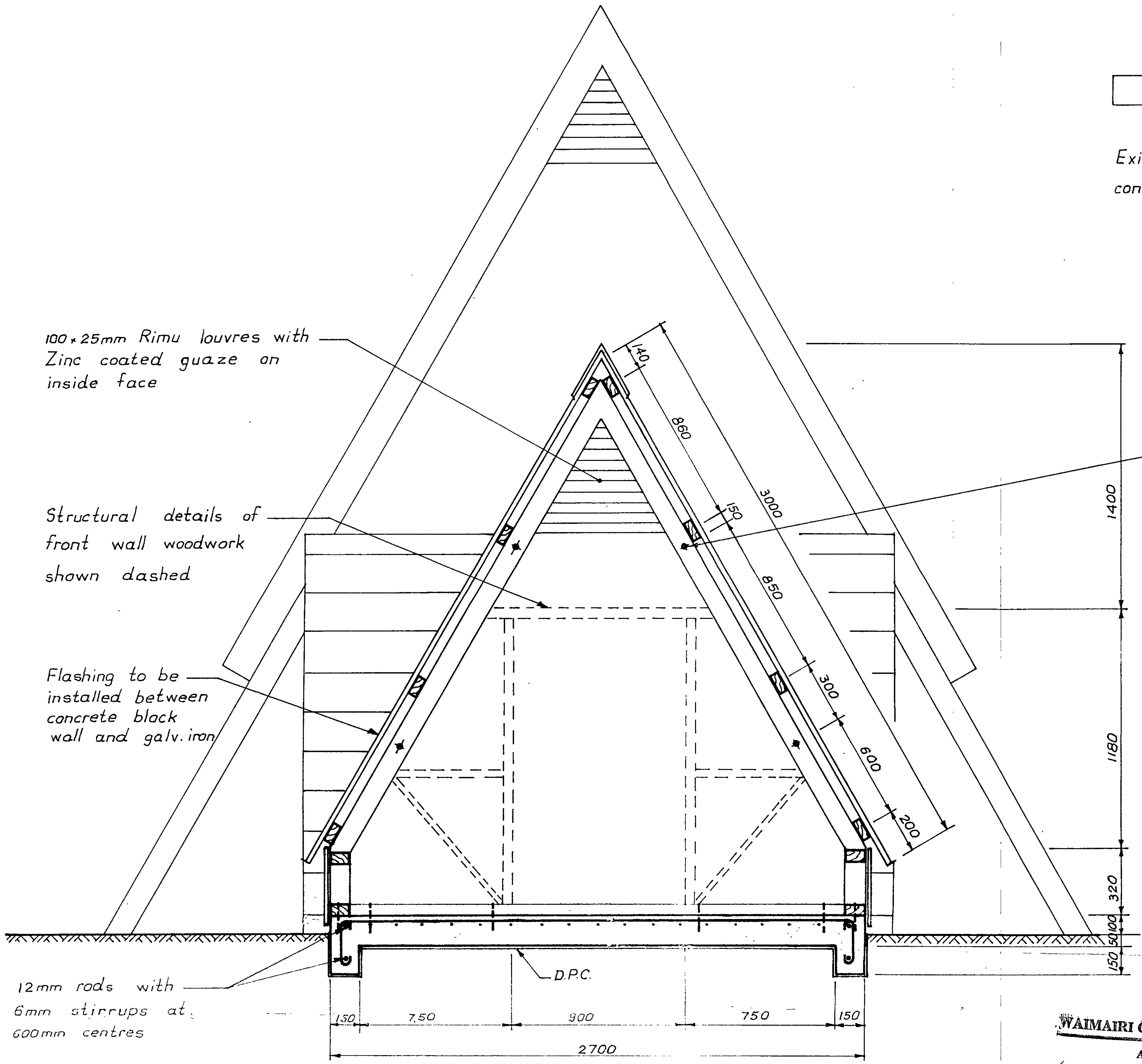
PROPOSED TOILET BLOCK
MARSHLANDS DOMAIN

BCP32885

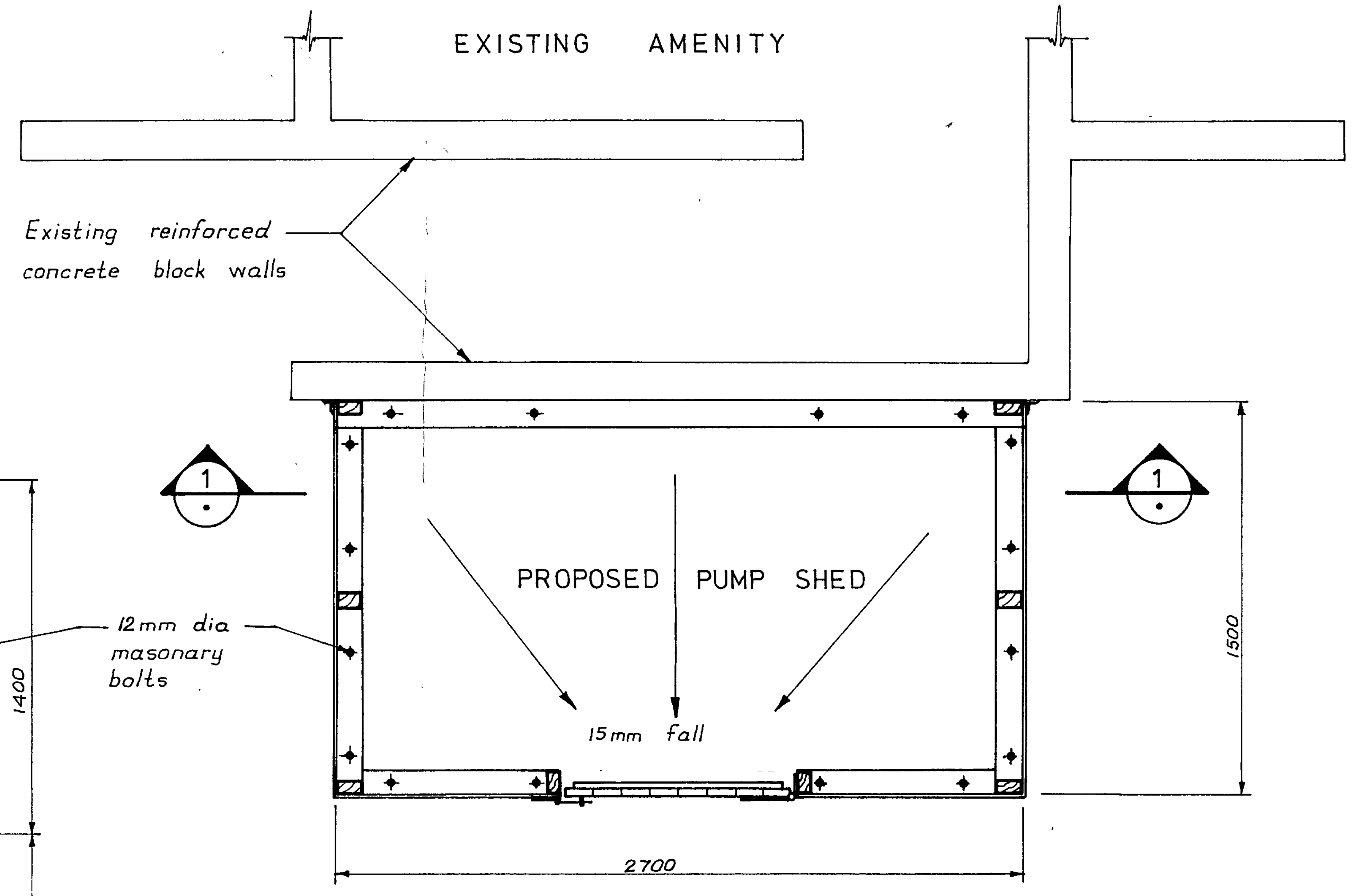
WAIMAIRI COUNTY COUNCIL

SCALE
Plan and Elevations 1:50
Section 1:20
Details as shown

AMENDMENTS
SHEET SIZE A1
PLAN No.
2601
SHEET OF SHEETS



SECTION 1



PLAN

NOTES:

1. All structural woodwork to be 100 x 50mm Pinus Radiata.
2. Roofing to be 3.0m long run 26gauge galvanised iron on building paper.
3. Cladding to be 26g galvanised iron.
4. 150mm 17.5 mpa concrete floor to be laid on compacted sand to fall, reinforced with 668 mesh.
5. Door to be T. & G. Rimu secured by hasp and staple.

WAIMAIRI COUNTY COUNCIL
APPROVED
SUBJECT TO BY-LAWS
BUILDING INSPECTOR
17-3-83.
DATE

BCP H6645

	NAME	DATE
SURVEYED		
DRAWN		
TRACED		
CHECKED		
DESIGNED	B.Boddy	
DES. CHCK.		

DATUM ....
BENCH MK ....
LEVEL BK No. .... P. ....
FIELD BK No. .... P. ....
APPROVED
.... / .... / .... COUNTY ENGINEER

PROPOSED PUMP SHED
MARSHLANDS DOMAIN

WAIMAIRI COUNTY COUNCIL

SCALE 1: 20

AMENDMENTS
SHEET SIZE A2
PLAN No. 1021
SHEET 2 OF 2 SHEETS

Appendix C

CERA Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Toilets Marshland Reserve	Reviewer:	Stephen Lee
	Unit / No: 420 / Street: Prestons Road	CPEng No:	1006840
Building Address:	420 Prestons Road	Company:	GHD
Legal Description:		Company project number:	513090222
		Company phone number:	04 472 0799
	Degrees / Min / Sec		
GPS south:		Date of submission:	
GPS east:		Inspection Date:	21-Sep-12
		Revision:	Final REV1
Building Unique Identifier (CCC):	PRK_0084_BLDG_001 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	0.75
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):	30.00

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	30.00
Ground floor split?	no		Ground floor elevation above ground (m):	30.00
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	
Building height (m):	4.40		height from ground to level of uppermost seismic mass (for IEP only) (m):	4.4
Floor footprint area (approx):	18			
Age of Building (years):	36		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):	Public Toilet			
Importance level (to NZS1170.5):	IL1			

Gravity Structure

Gravity System:	frame system		
Roof:	steel framed	rafter type, purlin type and cladding	RHS A-frame roof
Floors:	other (note)	describe sytem	Slab on grade
Beams:			
Columns:			
Walls:	partially filled concrete masonry	thickness (mm)	140

Lateral load resisting structure

Lateral system along:	partially filled CMU	**Note: Define along and across in detailed report!**		Partial fill concrete masonry
Ductility assumed, μ:	1.25		note total length of wall at ground (m):	
Period along:	0.40	#### enter height above at H31	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	other (note)			Partial fill concrete masonry
Ductility assumed, μ:	1.25		describe system	
Period across:	0.40	0.00	estimate or calculation?	
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:	Metal	describe	Corrugate sheet
Glazing:			
Ceilings:			
Services(list):			

Available documentation

Architectural		original designer name/date	
Structural	partial	original designer name/date	
Mechanical		original designer name/date	
Electrical		original designer name/date	
Geotech report		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 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:	52%	#### %NBS from IEP below	If IEP not used, please detail assessment methodology: Quantitative Analysis
	Assessed %NBS after e'quakes:	52%		
Across	Assessed %NBS before e'quakes:	100%	#### %NBS from IEP below	
	Assessed %NBS after e'quakes:	100%		

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: 4.4m

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): 1
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 =kμ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	0.00	0.00

2.6 Structural Performance Scaling Factor:

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 **$\%NBS_b$:** **#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 1.0
Height Difference effect D2, from Table to right 1.0

Therefore, Factor D: 1

3.5. Site Characteristics insignificant 1

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

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum	1.8	1.0
Rationale for choice of F factor, if not 1	(.7x2.5) Unrestrained panels x benificial features	

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) **1.75** **1.00**

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

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

Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
Final REV1	Paul Clarke	Stephen Lee		Nick Waddington		1/8/13