# Christchurch City Council
## PRK_3527_BLDG_001 EQ2
## Lyttelton Rec Ground - Pavilion
## Godley Quay, Lyttelton

QUANTITATIVE ASSESSMENT REPORT

FINAL

- Rev B
- 04 February 2013

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Sinclair Knight Merz
142 Sherborne Street
Saint Albans
PO Box 21011, Edgeware
Christchurch, New Zealand
Tel: +64 3 940 4900
Fax: +64 3 940 4901
Web: www.skmconsulting.com

# Contents

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# 1. Executive Summary

## 1.1. Background

A Qualitative Assessment was carried out on the Pavilion building located at the Lyttelton Recreation Ground on Godley Quay, Lyttelton. An aerial photograph illustrating these areas is shown below in Figure 1. Detailed descriptions outlining the buildings age and construction type is given in Section 5 of this report.


- **Figure 1 Aerial Photograph of Lyttelton Recreation Ground**

This Quantitative report for the building structure is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on 2/04/2012 and 23/04/2012 including a limited survey with a reinforcement ferro-scanner, intrusive investigations on 14/08/2012 and calculations dated 10/9/2012.

## 1.2. Key Damage Observed

Key damage observed includes:-

- Minor cracking to the bed joints of concrete block walls

## 1.3. Critical Structural Weaknesses

The following potential critical structural weaknesses have been identified.

- Unreinforced masonry walls may collapse in out of plane loading.

This critical structural weakness has been included in the quantitative assessment and hence can be ignored.

## 1.4. Indicative Building Strength

As described in the Engineering Advisory Group's "Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings" (from July 2011) we have assessed the percentage of new building standard seismic resistance using the quantitative method. Our assessment included consideration of geotechnical conditions, existing earthquake damage to the building and structural engineering calculations to assess both strength and ductility/resilience.

The assessments were based on the following:

- On-site investigation to assess the extent of existing earthquake damage including limited intrusive investigation.
- Qualitative assessment of critical structural weaknesses (CSWs) based on review of available structural drawings and inspection where drawings were not available.
- We have based this report on the geotech investigation report dated 17/10/2012, which includes interpretation of boreholes and standard penetration tests (SPT).
- Assessment of the strength of the existing structures taking account of the current condition.

Any building that is found to have a seismic capacity less than 33% of the new building standard is required to be strengthened up to a capacity of at least 67%NBS.

Based on the information available, and using the Quantitative Assessment Procedure, the buildings original capacity has been assessed to be in the order of 42%NBS and post earthquake capacity in the order of 42%NBS. This assessment has been made without structural drawings and is accordingly limited.

The building has been assessed to have a seismic capacity in the order of 42% NBS and is therefore not potentially earthquake prone.

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

## 1.5. Recommendations

It is recommended that:

a) The current placard status of the building should remain as green 1. Since there was no placard visible on the building it is assumed that the placard status was green

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

# 2. Introduction

Sinclair Knight Merz were engaged by Christchurch City Council to carry out a Quantitative Assessment of the seismic performance of Lyttelton Recreational Ground located at Godley Quay.

The scope of this quantitative analysis includes the following:

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

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

At the time of this report, an intrusive site investigation had been carried out. Construction drawings were not made available, and these have been considered in our evaluation of the building. The building description below is based on our intrusive and visual inspections.

[1] EAG 2011, *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury - Draft*, p 10
[2] http://www.dbh.govt.nz/seismicity-info

# 3. Compliance

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

## 3.1. Canterbury Earthquake Recovery Authority (CERA)

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

**Section 38 – Works**

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

**Section 51 – Requiring Structural Survey**

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

We understand that CERA will require a detailed engineering evaluation to be carried out for all buildings (other than those exempt from the Earthquake Prone Building definition in the Building Act). It is anticipated that CERA will adopt the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011. This document sets out a methodology for both qualitative and quantitative assessments.

The qualitative assessment is a desk-top and site inspection assessment. It is based on a thorough visual inspection of the building coupled with a review of available documentation such as drawings and specifications. The quantitative assessment involves analytical calculation of the buildings strength and may require non-destructive or destructive material testing, geotechnical testing and intrusive investigation.

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

- The importance level and occupancy of the building
- The placard status and amount of damage
- The age and structural type of the building

- Consideration of any critical structural weaknesses
- The extent of any earthquake damage

## 3.2. Building Act

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

### 3.2.1. Section 112 – Alterations

This section requires that an existing building complies with the relevant sections of the Building Code to at least the extent that it did prior to any alteration. This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

### 3.2.2. Section 115 – Change of Use

This section requires that the territorial authority (in this case Christchurch City Council (CCC)) be satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'. Regarding seismic capacity 'as near as reasonably practicable' has previously been interpreted by CCC as achieving a minimum of 67%NBS however where practical achieving 100%NBS is desirable. The New Zealand Society for Earthquake Engineering (NZSEE) recommend a minimum of 67%NBS.

### 3.2.3. Section 121 – Dangerous Buildings

The definition of dangerous building in the Act was extended by the Canterbury Earthquake (Building Act) Order 2010, and it now defines a building as dangerous if:

- in the ordinary course of events (excluding the occurrence of an earthquake), the building is likely to cause injury or death or damage to other property; or
- in the event of fire, injury or death to any persons in the building or on other property is likely because of fire hazard or the occupancy of the building; or
- there is a risk that the building could collapse or otherwise cause injury or death as a result of earthquake shaking that is less than a 'moderate earthquake' (refer to Section 122 below); or
- there is a risk that that other property could collapse or otherwise cause injury or death; or
- a territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

### 3.2.4. Section 122 – Earthquake Prone Buildings

This section defines a building as earthquake prone if its ultimate capacity would be exceeded in a 'moderate earthquake' and it would be likely to collapse causing injury or death, or damage to other property. A moderate earthquake is defined by the building regulations as one that would generate ground shaking 33% of the shaking used to design an equivalent new building.

### 3.2.5. Section 124 – Powers of Territorial Authorities

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

### 3.2.6. Section 131 – Earthquake Prone Building Policy

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

## 3.3. Christchurch City Council Policy

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

The 2010 amendment includes the following:

- A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
- A strengthening target level of 67% of a new building for buildings that are Earthquake Prone. Council recognises that it may not be practicable for some repairs to meet that target. The council will work closely with building owners to achieve sensible, safe outcomes;
- A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
- Repair works for buildings damaged by earthquakes will be required to comply with the above.

The council has stated their willingness to consider retrofit proposals on a case by case basis, considering the economic impact of such a retrofit.

We anticipate that any building with a capacity of less than 34%NBS (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67%NBS of new building standard as recommended by the Policy.

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

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

### 3.4. Building Code

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

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

a) Hazard Factor increased from 0.22 to 0.3 (36% increase in the basic seismic design load)
b) Serviceability Return Period Factor increased from 0.25 to 0.33 (80% increase in the serviceability design loads when combined with the Hazard Factor increase)

The increase in the above factors has resulted in a reduction in the level of compliance of an existing building relative to a new building despite the capacity of the existing building not changing.

# 4. Earthquake Resistance Standards

For this assessment, the building's earthquake resistance is compared with the current New Zealand Building Code requirements for a new building constructed on the site. This is expressed as a percentage of new building standard (%NBS). The new building standard load requirements have been determined in accordance with the current earthquake loading standard (NZS 1170.5:2004 Structural design actions - Earthquake actions - New Zealand).

The likely capacity of this building has been derived in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) guidelines 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes' (AISPBE), 2006. These guidelines provide an Initial Evaluation Procedure that assesses a buildings capacity based on a comparison of loading codes from when the building was designed and currently. It is a quick high-level procedure that can be used when undertaking a Qualitative analysis of a building. The guidelines also provide guidance on calculating a modified Ultimate Limit State capacity of the building which is much more accurate and can be used when undertaking a Quantitative analysis.

The New Zealand Society for Earthquake Engineering has proposed a way for classifying earthquake risk for existing buildings in terms of %NBS and this is shown in Figure 2 below.

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

- **Figure 2: NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE Guidelines**

Table 1 below provides an indication of the risk of failure for an existing building with a given percentage NBS, relative to the risk of failure for a new building that has been designed to meet current Building Code criteria (the annual probability of exceedance specified by current earthquake design standards for a building of 'normal' importance is 1/500, or 0.2% in the next year, which is equivalent to 10% probability of exceedance in the next 50 years).

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- **Table 1: %NBS compared to relative risk of failure**

| Percentage of New Building Standard (%NBS) | Relative Risk (Approximate) |
|---|---|
| >100 | <1 time |
| 80-100 | 1-2 times |
| 67-80 | 2-5 times |
| 33-67 | 5-10 times |
| 20-33 | 10-25 times |
| <20 | >25 times |

# 5. Building Details

## 5.1. Building description

The pavilion building is located at Godley Quay, Lyttelton. There are no buildings near to the pavilion building. The building is a single story structures which is primarily used as public changing facilities for the adjacent sports ground. The building is constructed from concrete block shear walls with a light weight timber roof. The building is supported on concrete strip edge foundations and concrete piles internally. Since no drawings were made available we have assumed that the building was designed and constructed between 1965 and 1976.

## 5.2. Gravity Load Resisting system

The gravity load resisting structure is comprised of timber roof trusses spanning onto concrete block walls which bear on concrete strip foundations. The internal floor structure is supported on concrete piles on concrete pads. The concrete block walls were scanned with a reinforcement ferro-scanner which indicated that the walls were generally unreinforced although the lintels indicated the presence of reinforcement.

## 5.3. Seismic Load Resisting system

The lateral load resisting system is comprised of unreinforced concrete block shear walls on concrete strip foundations. The timber roof structure and its associated ceiling linings are assumed to act as a diaphragm to transfer horizontal forces to shear walls parallel with the direction of loading. There appeared to be sufficient internal walls to reduce the spacing of shear walls in the transverse direction of the building.

For the purposes of this assessment the transverse direction of the building is taken to be the east-west direction while the longitudinal direction is taken to be the north-south direction.

## 5.4. Building Damage

SKM undertook inspections on the following dates 2/04/2012 and 23/04/2012. The following areas of damage were observed during the time of inspection:

1) There are cracking in the bed joints of the concrete block shear walls all around the building. These vary in thickness with approximate maximum thickness of around 0.5mm. Refer Photo 4 to Photo 12.

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

# 6. Available Information and Assumptions

## 6.1. Available Information

Following our inspections on the 2/04/2012 and 23/04/2012 and 14/08/2012 SKM carried out a seismic review on the structure. This review was undertaken using the available information which was as follows:

- SKM site measurements and inspection findings of the building.

## 6.2. Survey

The building was not surveyed.

## 6.3. Design Criteria and Assumptions

The following design criteria and assumptions made in undertaking the assessment include:

- The soil on site is class E as described in AS/NZS1170.5:2004, Clause 3.1.3, Soft Soil. The ultimate bearing capacity on site is 105kPa. Liquefaction does not need to be accounted for in the foundation design, due to the performance of the site suggesting that minor liquefaction effects were observed. Angle of internal friction of surface granular fill (0-0.7mbgl) is 30 degrees.
- Standard design criteria for typical office and factory buildings as described in AS/NZS1170.0:2002:
  - 50 year design life, which is the default NZ Building Code design life.
  - Structure Importance Level 2. This level of importance is described as ‘normal’ with medium or considerable consequence for loss of human life, or considerable economic, social or environmental consequence of failure.
- The building has a short period less than 0.4 seconds.
- Site hazard factor, Z = 0.3, NZBC, Clause B1 Structure, Amendment 11 effective from 1 August 2011
- A ductility of, $\mu$=1 was used in the building for both directions. This is appropriate due to the masonry walls being unreinforced.
- The perimeter foundation beam has been assumed to be 400d x 250w. We have assumed that this foundation is also beneath the internal masonry walls.
- Based on our knowledge of historical pile construction methods, we have assumed that the 200x200 concrete piles are shallow founded 250mm below ground, and set into a 400x400x100 thick site concrete footing. Lateral resistance is assumed to be provided by a combination of passive pressure on the buried footing, and the mass of the pile and floor

above. There is significant degree of uncertainty associated with our evaluation of the lateral load capacity of the piles.

- It has been assumed the masonry walls are filled, this is from visual evidence.

- The following material properties were used in the analyses:
- **Table 2: Material Properties**

| Material | Material Property |
|---|---|
| Masonry in shear | $f'_{ms} = 0.25$ Mpa |
| Tensile strength of mortar joint | $f'_{mt} = 0.19$ Mpa |
| Lateral modulus of rupture of masonry | $f'_{ut} = 0.8$ Mpa |
| Concrete | $f_c' = 25$MPa |
| Tongue and Groove floor diaphragm | Diaphragm capacity = 4.2 kN/m |
| Friction angle of Soil | $\varphi = 30^o$ |
| Unit weight of soil | $\gamma = 17$kN/m3 |

The detailed engineering analysis is a post construction evaluation therefore it has the following limitations:

- It is not likely to pick up on any concealed construction errors (if they exist)
- Other possible issues that could affect the performance of the building such as corrosion and modifications to the structure will not be identified unless they are visible and have been specifically mentioned in this report.
- The detailed engineering evaluation deals only with the structural aspects of the structure. Other aspects such as building services are not covered.
- It has been assumed that a building consent will be required to repair the damage to the building. The likely requirements for a building consent would be repairs costing in excess of $ 50,000 or structural alteration.

## 6.4. The Detailed Engineering Evaluation (DEE) process

The DEE is a procedure written by the Department of Building and Housing's Engineering Advisory Group and grades buildings according to their likely performance in a seismic event. The procedure is not yet recognised by the NZ Building Code but is widely used and recognised by the

Christchurch City Council as the preferred method for preliminary seismic investigations of buildings[3].

The procedure of the DEE is as follows:

1) Qualitative assessment procedure
    a. Determine the building's status following any rapid assessment that have been done
    b. Review any existing documentation that is available. This will give the engineer an understanding of how the building is expected to behave. If no documentation is available, site measurements may be required
    c. Review the foundations and any geotechnical information available. This will include determining the zoning of the land and the likely soil behaviour, a site investigation may be required
    d. Investigate possible Critical Structural Weaknesses (CSW) or collapse hazards
    e. Assess the original and post earthquake strength of the building (this assessment is subsequently superseded by the quantitative assessment)
2) Quantitative procedure
    a. Carry out a geotechnical investigation if required by the qualitative assessment
    b. Analyse the building according to current building codes and standards. Analysis accounts for damage to the building.

The DEE assessment ranks buildings according to how well they are likely to perform relative to a new building designed to current earthquake standards, as shown in Table 4. The building rank is indicated by the percent of the required New Building Standard (%NBS) strength that the building is considered to have. Earthquake prone buildings are defined as having less than 34 %NBS strength which correlates to an increased risk of approximately 20 times that of 100% NBS[4]. Buildings that are identified to be earthquake prone are required by law to be strengthened within 30 years of the owner being notified that the building is potentially earthquake prone[5]. This timeframe is likely to be adjusted by CERA and Table 6 below contains the likely new recommendations.

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

- **Table 3: DEE Risk classifications**

| Description | Grade | Risk | %NBS | Structural performance |
|---|---|---|---|---|
| Low risk building | A+ | Low | > 100 | Acceptable. Improvement may be desirable. |
| | A | | 100 to 80 | |
| | B | | 80 to 67 | |
| Moderate risk building | C | Moderate | 67 to 33 | Acceptable legally. Improvement recommended. |
| High risk building | D | High | 33 to 20 | Unacceptable. Improvement required. |
| | E | | < 20 | |

The DEE method rates buildings based on the plans (if available) and other information known about the building and some more subjective parameters associated with how the building is detailed and so it is possible that %NBS derived from different engineers may differ.

This assessment describes only the likely seismic Ultimate Limit State (ULS) performance of the building. The ULS is the level of earthquake that can be resisted by the building without catastrophic failure. The DEE does also consider Serviceability Limit State (SLS) performance of the building and or the level of earthquake that would start to cause damage to the building but this result is secondary to the ULS performance.

The NZ Building Code describes that the relevant codes for determining %NBS are primarily:

- AS/NZS 1170 Structural Design Actions
- NZS 3101:2006 Concrete Structures Standard
- NZS 3404:1997 Steel Structures Standard
- NZS4230:2004 Design of Reinforced Concrete Masonry Structures
- NZS 3603:1993 Timber Structures Standard
- NZS 3604:2011 Timber Framed Buildings

# 7. Results and Discussions

## 7.1. Critical Structural Weaknesses

The building has the following critical structural weaknesses:

- Unreinforced masonry

These critical structural weaknesses have been incorporated into the quantitative results below and hence can be ignored. The effect of these will be a lower quantitative assessment result when compared to a building containing no critical structural weaknesses.

## 7.2. Analysis Results

The equivalent static force method was used to analyse the seismic capacity of the building. The results of the analysis are reported in the following table as %NBS. The results below are calculated for the building in its damaged state. The building results have been broken down into their seismic resisting elements.

(%NBS = probable strength / new building standards)

- **Table 4: DEE Results**

| Seismic Resisting Element | Action | Seismic Rating %NBS |
|---|---|---|
| Masonry walls – out of plane | Bending | 46% |
| Subfloor – transverse direction | Shear | 70% |
| Masonry walls – shear in transverse direction | Shear | 81% |
| Subfloor - diaphragm | Shear | 100% |
| Masonry walls – shear in longitudinal direction | Shear | 100% |
| Sub Floor - longitudinal | Shear | 100% |

## 7.3. Recommendations

The quantitative assessment carried out on the Lyttelton Recreational Ground Pavilion indicates that the building has a seismic capacity between 33% and 67% of NBS and is therefore classed as being in the category of 'Moderate Risk Building'.

It is recommended the buildings masonry walls be strengthened to a target of at least 67%.

# 8. Conclusion

SKM carried out a quantitative assessment on Lyttelton Recreational Ground Pavilion located at Godley Quay, Lyttelton. This assessment concluded that the building is classified as not Earthquake Prone.

- **Table 5: Quantitative assessment summary**

| Grade | Risk | %NBS | Structural Performance |
|---|---|---|---|
| C | Moderate | 46 | Acceptable legally, Improvement recommended |

It is recommended that:

a) No placard was displayed on the building however we recommend that the current placard status of the building be Green 2

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

# 9. Limitation Statement

This report has been prepared on behalf of, and for the exclusive use of, SKM's client, and is subject to, and issued in accordance with, the provisions of the contract between SKM and the Client. It is not possible to make a proper assessment of this report without a clear understanding of the terms of engagement under which it has been prepared, including the scope of the instructions and directions given to, and the assumptions made by, SKM. The report may not address issues which would need to be considered for another party if that party's particular circumstances, requirements and experience were known and, further, may make assumptions about matters of which a third party is not aware. No responsibility or liability to any third party is accepted for any loss or damage whatsoever arising out of the use of or reliance on this report by any third party.

Without limiting any of the above, in the event of any liability, SKM's liability, whether under the law of contract, tort, statute, equity or otherwise, is limited in as set out in the terms of the engagement with the Client.

It is not within SKM's scope or responsibility to identify the presence of asbestos, nor the responsibility of SKM to identify possible sources of asbestos. Therefore for any property pre-dating 1989, the presence of asbestos materials should be considered when costing remedial measures or possible demolition.

Should there be any further significant earthquake event, of a magnitude 5 or greater, it will be necessary to conduct a follow-up investigation, as the observations, conclusions and recommendations of this report may no longer apply Earthquake of a lower magnitude may also cause damage, and SKM should be advised immediately if further damage is visible or suspected.

# 10. Appendix 1 – Photos

| | |
|---|---|
|  |  |
| Photo 1: Elevation looking North East | Photo 2: Elevation looking South East |
|  |  |
| Photo 3: Elevation looking South West | Photo 4: Blockwork cracking on the eastern facade. |

| | |
|---|---|
|  |  |
| Photo 5: Blockwork cracking on the eastern facade. | Photo 6: Blockwork cracking on the eastern facade. |
|  |  |
| Photo 7: Blockwork cracking on the eastern facade. | Photo 8: Blockwork cracking on the eastern facade. |

Photo 9: Blockwork cracking on the western facade.

Photo 10: Blockwork cracking on the western facade.

Photo 11: Blockwork cracking on the western facade.

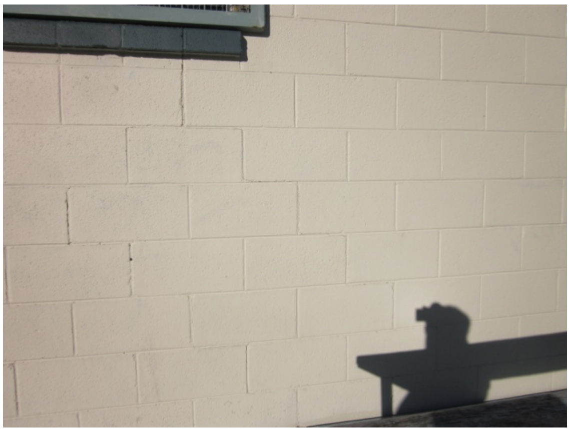

Photo 12: Blockwork cracking on the western facade.

| | |
|---|---|
|   |   |
| Photo 13: View inside subfloor | Photo 14: View of inside roofspace |

# 11. Appendix 2 – Quantitative Calculations

SKM

Job No. ZB01276.37

Calc. Series

Client CCC

Page 0

Job Name Lyttelton Rec ground Pavillion

By Nigel Chan

Calcs Title

Date

## Scope

Carry out a quantitative assessment on Lyttelton Rec ground pavillion.

Building is a single storey masonry building. Roof is timber framed clad with corrugated iron. The building sits on a concrete perimeter foundation beam, with internal concrete piles.

A qualitative report was completed on 2nd May 2012, with visual inspections carried out on 2/4/12 and 23/4/12. It indicated the %NBS was below 10%.
An intrusive investigation was carried out on 14/8/12.

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| Calcs Title: Quantitative Calcs | Page: 101 |
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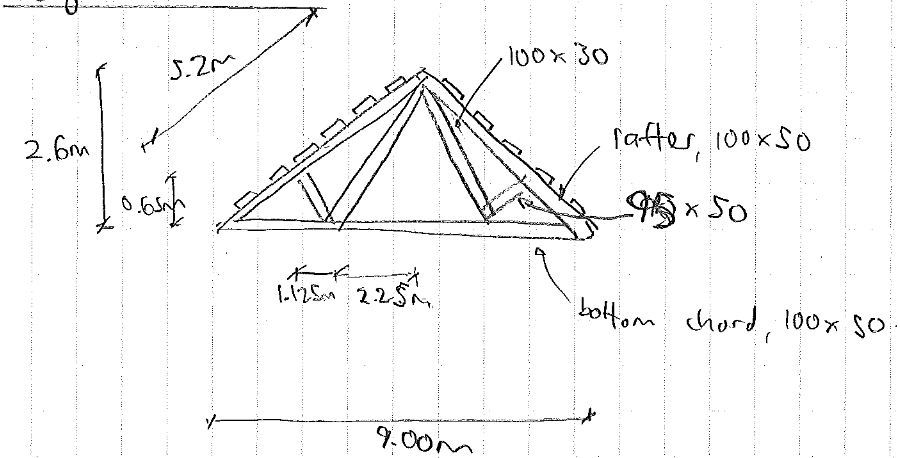
| | Date: 14-8-12 |

## Loadings

Weight of roof




roof truss @ 600

joists @600
2 of 75×35

roof battens @400
70×30

truss unit $= [\text{rafter}] + [\text{bottom chord}] + [\text{truss members}]$

$= [2 \times \sqrt{2.6^2 + 4.5^2} \times 0.100 \times 0.050] +$
$[9 \times 0.100 \times 0.050] + [2\sqrt{0.65^2 + 1.125^2} \times 0.095 \times 0.050 + 2\sqrt{2.25^2 + 2.6^2} \times 0.100 \times 0.070]$

$= [0.052] + [0.045] + [0.033]$

$= 0.13\ m^3$

joists @600, 600 length $= 2 \times (5.2m \div 0.6m) \times 2 \times 0.075m \times 0.035m \times 0.6m$

$= 0.051\ m^3$

roof battens $= 9.00m \div 0.400m \times 0.070 \times 0.030 \times 0.6$

$= 0.03 m^3$

total timber $= (0.13 m^3 + 0.05 m^3 + 0.03 m^3) \times 4.6\ kN/m^3$

$= 0.966$ kN per truss @ 0.6m

corrugated iron assume 0.8mm $= 0.10\ kN/m^2 \times 10.4m \times 0.6m$

$= 0.624$ kN per truss @ 0.6m

ceiling $= 0.13\ kN/m^2 \times 9m \times 0.6m$

$= 0.702$ kN per truss @ 0.6m

SKM

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total = 0.966 + 0.624 + 0.702
= 2.3 kN per 0.6m width

take as unit area
= 2.3 kN ÷ (9m × 0.6m)
≈ 0.43 kN/m²

**SKM**

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Calc. Series A
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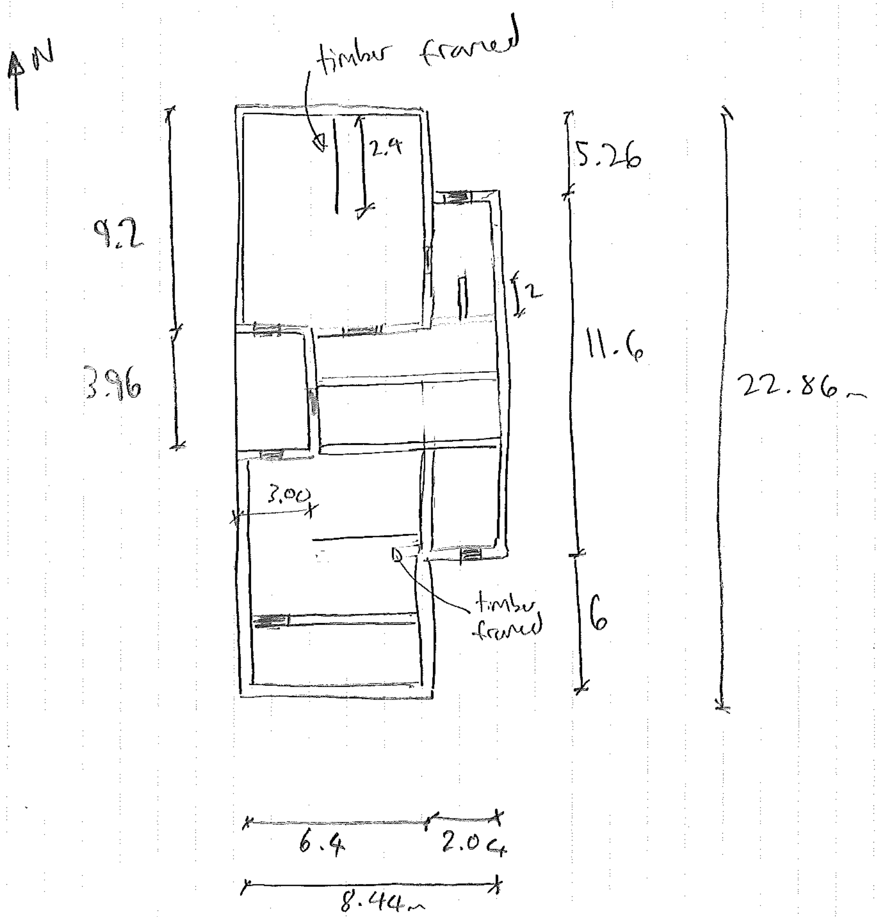
By Nigel Chan
Calcs Title Quantitative Calcs
Date 10-9-12

1st floor




Along
Across

- Walls have been taken as filled

Masonry walls : $20\ kN/m^3 \times 0.190m \times 2.4m \times L$

$L = [\text{Perimeter}] + [\text{internal}]$

$= [22.86 \times 2 + 8.44m \times 2] + [\text{internal along}] + [\text{internal across}]$

$= [62.6m] + [11.6m + 2m] + [8.44 \times 2 + 5.44 + 6.4]$

$= 62.6m + 42.32m$

$= 104.9m$

$= 20\ kN/m \times 0.190 \times 2.4 \times 104.9$

$= 957\ kN$

**SKM**

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Timber framed wall = $(6.4 - 3.0 + 2.4) \times 2m \times 0.25\,kPa$
= 2.9 kN

Bond Beam = $24\,kN/m^3 \times 0.2m \times 0.3m \times 104.9\,m$
= 151.1 kN

$\Sigma$ = 957 kN + 2.9 kN + 151.1 kN + 73.5 kN
= 1185 kN

roof = $0.43\,kN/m^2 \times [(23 \times 6.4) + (11.6 \times 2.04)]$
= 73.5 kN

SKM

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earthquake load factor

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

$C_h(T) = 3$ Class E soil

$Z = 0.3$ Christchurch

$R = 1.0$ 1/500 year earthquake

$N(T,D) = 1.0$

$C(T) = 0.9$

$C_d(T_1) = \frac{C(T) S_p}{k_\mu}$ $\mu = 1$

$= \frac{0.9 \times 1}{1}$

$= 0.9$

$V = C_d(T_1) W_t$

$= 0.9 \times 1185$ kN

$= 1066$ kN on ground floor

SKM

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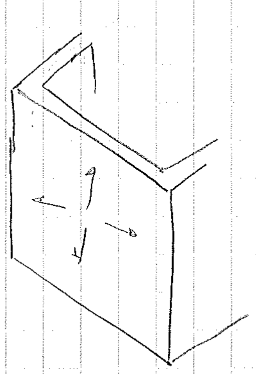
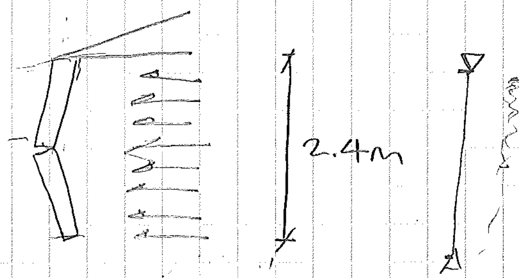
## Design South Wall - out of plane - vertical






horizontal force $= 20\,kN/m^3 \times 0.190m \times 0.9$
$= 3.42\ kN/m^2$

$M_{dv} \leq M_{cv}$

$M_{dv} = wl^2/8$
$= 3.42 \times 2.4^2/8$
$= 2.46\ kNm\ /m$

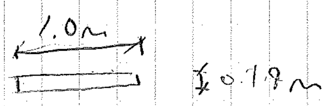
$M_{cv} =$ lesser of $\begin{cases} \phi f'_{mt} Z_d + f_d Z_d \\ 3.0\ \phi f'_{mt} Z_d \end{cases}$

$\phi f'_{mt} Z_d + f_d Z_d$

$\phi = 0.6$

(NZSEE, EQ 2.5)

$f'_{mt} = 0.19\ MPa = 190\ kPa$




$Z_d = bd^2/6$
$= 6.02 \times 10^{-3}\ m^3$

$f_d$ = [roof load + bond beam + half height of wall] ÷ [1m × 0.19m]
= [(0.43 kN/m² × 3.2m) + (24 kN/m³ × 0.3m × 0.2m) + (20 kN/m³ × 1.2m × 0.19m)] ÷ [0.19m]
= 38.8 kPa

$\phi f'_{mt} Z_d + f_d Z_d$ = $0.6 \times 190$ kPa $\times 6.02 \times 10^{-3} + 38.8 \times 6.02 \times 10^{-3}$
= 0.92 kNm

$3.0 \phi f'_{mt} Z_d$ = $3.0 \times 0.6 \times 190 \times 6.02 \times 10^{-3}$
= 2.06 kNm

so $M_{cv}$ = 0.92 kNm

$M_{dv} \leq M_{cv}$
2.46 kNm ≤ 0.92 kNm

37% NBS

SKM

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Date 10-9-12

Design North wall - out of plane - horizontal

$M_{oh} \leq M_{ch}$

$M_{oh} = wl^2/8$  $l = 6.2m$

$= 3.42 kN/m^2 \times 6.2^2/8$

$= 16.4$ kNm

$M_{ch} \leq 2.0 \phi k_p (\sqrt{f'_{mt}})(1 + f_d/f'_{mt}) Z_d$
$\leq 4.0 \phi k_p (\sqrt{f'_{mt}}) Z_d$
$\leq \phi (0.44 f_{ut} Z_u + 0.56 f'_{mt} Z_p)$

$\phi = 0.6$
$k_p = 1$
$f'_{mt} = 0.190$ MPa
$f_d = 38.8$ kPa $= 0.0388$ MPa
$f'_{ut} = 0.8$ MPa
$Z_d = 6.02 \times 10^{-3} m^3 = 6020 \times 10^3 mm^3$
$Z_u = 6.02 \times 10^{-3} m^3 = 6020 \times 10^3 mm^3$
$Z_p = 6.02 \times 10^{-3} m^3 = 6020 \times 10^3 mm^3$

$\leq 2.0 \times 0.6 \times 1 \times (\sqrt{0.19})(1 + 0.0388/0.19) \times 6020 \times 10^3$
$\leq 4.0 \times 0.6 \times 1 \times (\sqrt{0.19}) \times 6020 \times 10^3$
$\leq 0.6 \times (0.44 \times 0.8 \times 6020 \times 10^3 + 0.56 \times 0.190 \times 6020 \times 10^3)$

$\leq 3.79$ kNm/m
$\leq 6.30$ kNm/m
$\leq 1.66$ kNm/m

$M_{ch} = 1.66$ kNm/m

Check

$M_{oh} \leq M_{ch}$
$16.4 \leq 1.66$

10% NBS

SKM

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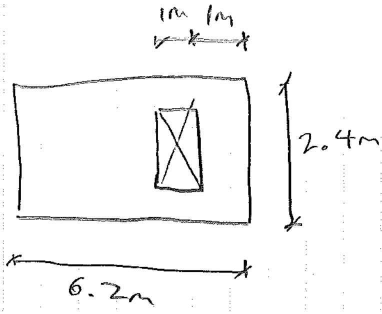
Design South Wall - out of plane - 2 way bending

$W_d \leq W$




$$W = \frac{2a_f}{L_d^2}(k_1 M_{ch} + k_2 M_{cd})$$

$$a_f = f(\alpha)$$

$$\alpha = \frac{G L_d}{H_d}$$

$$G = \frac{2(h_u + t_j)}{l_u + t_j}$$

$h_u = 0.190m$
$t_j = 0.010m$
$l_u = 0.390m$

$$= 1.$$

$$L_d = 6.2 - 2$$
$$= 4.2m$$ opening present

$$H_d = 2.4/2$$
$$= 1.2m$$ top edge is laterally supported

$$= (1 \times 3.1m)/1.2m$$
$$= 2.58$$

$$a_f = \frac{\alpha}{\left(1 - \frac{1}{3\alpha} + \frac{L_o}{2L_d}\right)}$$

$L_o = 1m$

$$= \frac{2.58}{\left(\left(1 - \frac{1}{3 \times 2.58}\right) + \frac{1}{2 \times 4.2}\right)}$$

$$= 2.61$$

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$k_1 = R_{f1} = 1$

$k_2 = 1 + \frac{1}{G^2}$

$= 1 + \frac{1}{1^2}$

$= 2$

$M_{ch} = 1.66$ kNm/m

$M_{cd} = \phi f'_t Z_t$

$\phi = 0.6$

$f'_t = 2.25\sqrt{f'_{mt}} + 0.15 f_d$

$f'_{mt} = 0.19$ MPa

$f_d = 0.0388$ MPa

$= 2.25 \times \sqrt{0.19} + 0.15 \times 0.0388$

$= 0.99$ MPa

$Z_t =$

$B = \frac{h_u + t_j}{\sqrt{1+G^2}}$

$h_u = 0.190$m

$t_j = 0.010$m

$G = 1$

$= 0.14$

$t_u = 0.190$m

$l_u = 0.390$m

$t_u = 0.190$m

$B < t_u$

$0.14 < 0.190$

$$Z_t = \frac{\frac{2B^2 t_u^2}{3t_u + 1.8B}}{\left[(l_u + t_j)\sqrt{1+G^2}\right]}$$

$$= \frac{\frac{2 \times 0.14^2 \times 0.190^2}{3 \times 0.190 + 1.8 \times 0.14}}{(0.390 + 0.010) \times \sqrt{1+1^2}}$$

SKM

| | |
|---|---|
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| Job Name: Lyttelton Rec Pavillion | Calc. Series: A |
| Calcs Title: Quantitative Calcs | Page: 206 |
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$$= 3.04 \times 10^{-3} \, m^3$$

$$M_{cd} = \phi f'_t Z_t$$
$$= 0.6 \times 0.99 \, MPa \times 3.04 \times 10^{-3} \, m^3$$
$$= 1.81 \, kNm$$

$$w = \frac{2af}{Ld^2} \times (k_1 M_{ch} + k_2 M_{cd})$$

$$= \frac{2 \times 2.61}{4.2^2} \times (1 \times 1.66 + 2 \times 1.81)$$

$$= 1.56 \, kPa$$

$$w_d = 3.42 \, kPa$$

$$\frac{w}{w_d} = \frac{1.56}{3.42} = 46\%$$

$$\boxed{\%NBS = 46\%}$$

SKM

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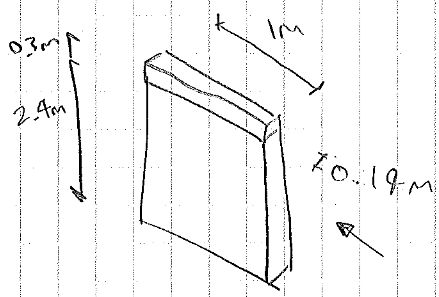
## In plane - Shear

$V_d \leq V_0 + V_1$




$V_0 = \phi f'_{ms} A_d$

$\phi = 0.6$

$f'_{ms} = 1.25 f'_{mt}$ $\quad 0.15_{MPa} \leq f'_{ms} \leq 0.35 MPa$

$= 1.25 \times 0.15 MPa$

$= 0.188 MPa = 188 kPa$

$A_d = 1.00_m \times 0.190 m$

$0.19 m^2$

$= 0.6 \times 188 kPa \times 0.19 m^2$

$= 21.4 kN$

$V_1 = k_v f_d A_d$

$k_v = 0.3$

roof load

$f_d = 2.4 \times 20 kN/m^3 + 0.3 \times 24 kN/m^3 + 0.43 kN/m^2 \times 3.2 m$

$= 56.6 kPa$

$A_d =$

$= 0.3 \times 56.6 \times 0.19$

$= 3.22 kN$

$V_d \leq V_0 + V_1$

$V_d \leq 21.4 + 3.2$

$\leq 24.6 kN$ (per metre length)

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| Date | 10-9-12 |

Capacity in North-South direction

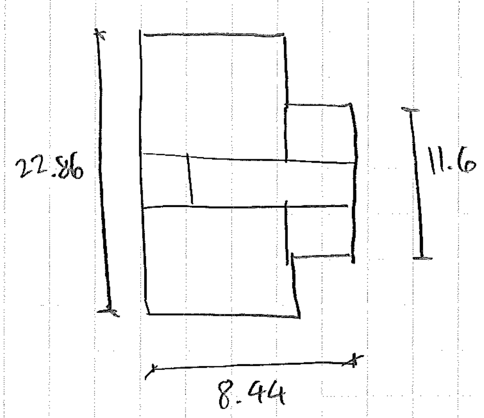
$= (22.86 \times 2 + 11.6) \times 24.6 \text{ kN/m}$

$= 1410 \text{ kN}$

Capacity in East - West

$= (8.44 \times 4) \times 24.6$

$= 830 \text{ kN}$




Demand $= 1022 \text{ kN}$

100% NBS North-South Direction, longitudinal
81% NBS east-west Direction, transverse

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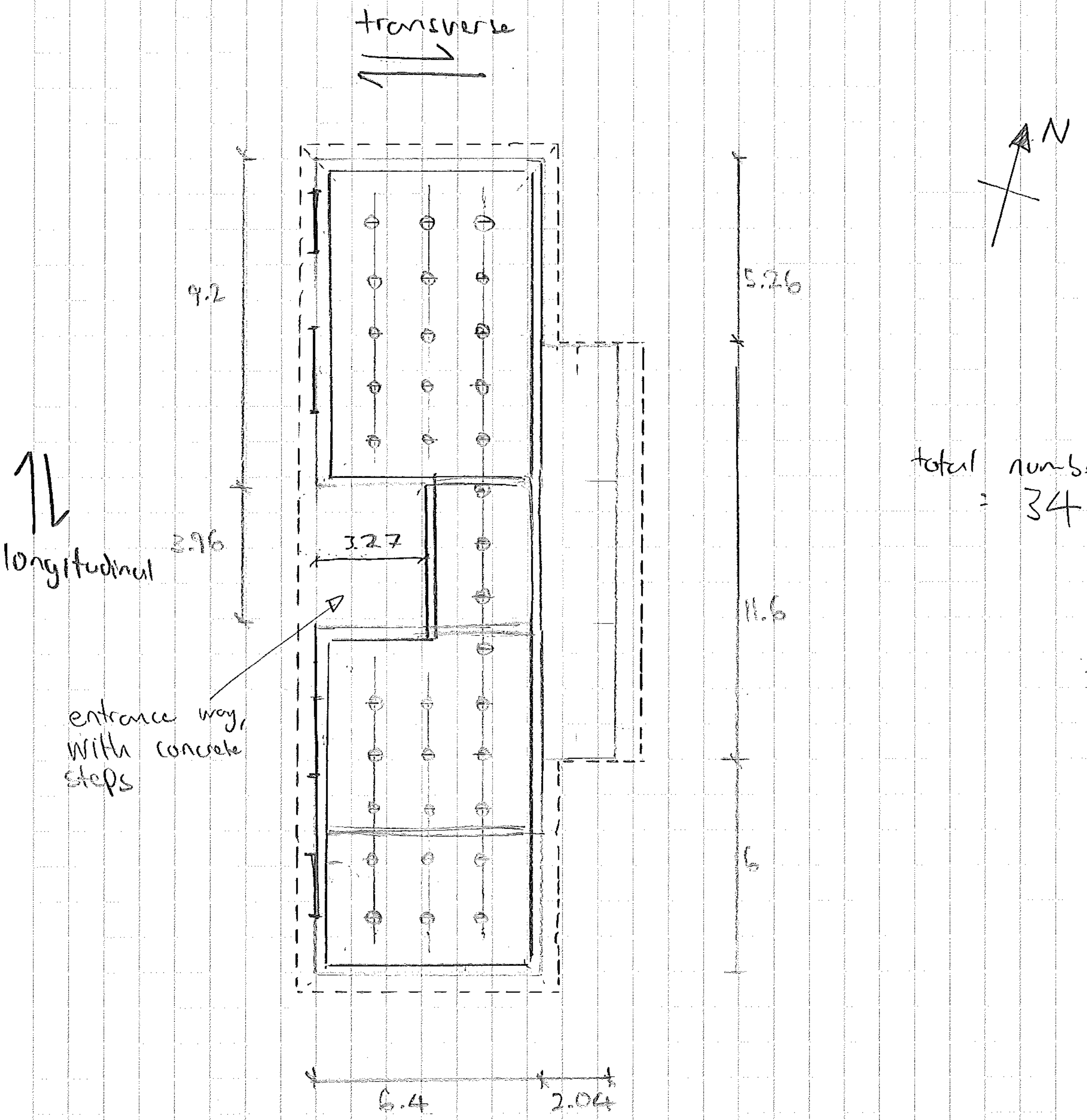
## Subfloor




total number of piles = 34

- * 200 x 200 sq concrete piles
- * piles 1540 apart
- * 100 x 75 Bearers
- * 125 x 50 joists @ 470 crs
- * T&G floor 10mm thick

- * Perimeter foundation beam assume 400 x 250

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loading

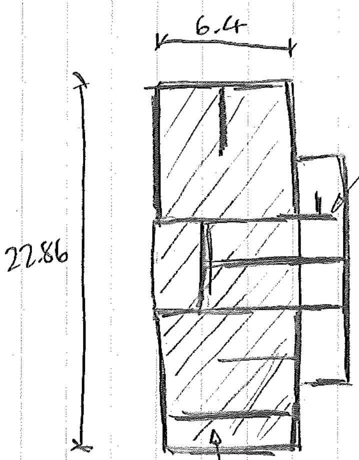
weight of perimeter foundation = $(23\times2 + 6.4\times2 + 3.27\times2) * 0.4 \times 0.25 \times 24\ kN/m^3$
= 157 kN

Total self weight (G) = 1185 kN + 157 kN = 1342 kN

Live load (Q) = 3 kPa × [(23×6.4) + (11.6×2.04)]
= 512 kN

$G + \psi_E Q$ = 1342 kN + 0.3 × 513 kPa
= 1496 kN

Total V = 0.9 × 1496 kN
= 1346 kN




this appears to be an addition with its own slab on ground foundation will not include this portion of the building in the subfloor calculations

look at this part of the building

length of masonry walls in addition
L = 11.6 m + 5 × 2.04 + 2
= 23.8 m

weight of masonry walls excluding addition
= (104.9 - 23.8) × 20 kN/m × 0.190 × 2.4
= 739.6 kN

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Timber framed wall = 2.9 kN

Bond Beam = $24\ kN/m^3 \times 0.2m \times 0.3m \times (104.9 - 23.8)$
= 116.8 kN

roof = $0.43\ kN/m^2 \times 23 \times 6.4$
= 63.3 kN

perimeter foundation = 157 kN

$\Sigma G$ = 739.6 kN + 2.9 kN + 116.8 kN + 63.3 kN + 157 kN
= 1080 kN

Q = 3 kPa × 23 × 6.4
= 441.6 kN

$G + \psi_E Q$ = 1080 kN + 0.3 × 441.6
= 1212 kN

Total V = 0.9 × 1212 kN
= 1091 kN

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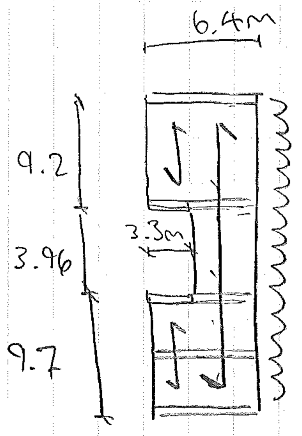
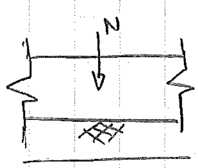
Capacity

Capacity of perimeter foundation in transverse direction




$\Leftarrow V^* = 1091$ kN




sliding capacity on perimeter foundation

$\phi V = \phi N \tan \phi$

N = roof + masonry wall + floor + perimeter foundation + bond beam

take 3.2m as trib width

roof = 3.2m × 0.43 kN/m²
= 1.38 kN/m

masonry wall = 0.2m × 2.4m × 20 kN/m³
= 9.6 kN/m

floor =

unit weight
= bearers + joists + T&G floor

= [0.100 × 0.075 × 1000/1500] + [0.125 × 0.050 × 1000/470] + [0.010] × 5 kN/m³

SKM

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= 0.14 kN/m²

floor = 0.14 kN/m² × 3.2m
= 0.45 kN/m

perimeter foundation = 0.4 × 0.25 × 24 kN/m³
= 2.4 kN/m

bond beam = 0.2 × 0.3 × 24 kN/m³
= 1.44 kN/m

N = 1.38 + 9.6 + 0.45 + 2.4 + 1.44
= 15.3 kN/m

$\phi V = \phi N \tan\phi$
= 0.9 × 15.3 kN/m × tan 30
= 23.5 kN/m

Length of perimeter foundation resisting load - trasverse
= 6.4 × 5
= 32m

Capacity of perimeter foundation - transverse
= 32 m × 23.5 kN/m
= 752 kN

Length of perimeter foundation resisting load - longitudinal
= 22.9 m × 2 = 45.8m

Capacity of perimeter foundation - longitudinal
= 45.8m × 23.5 kN/m
= 1076.3 kN

SKM

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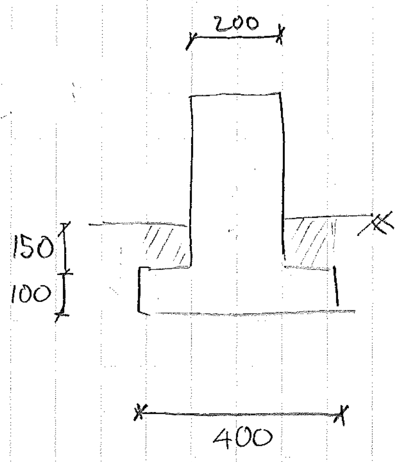
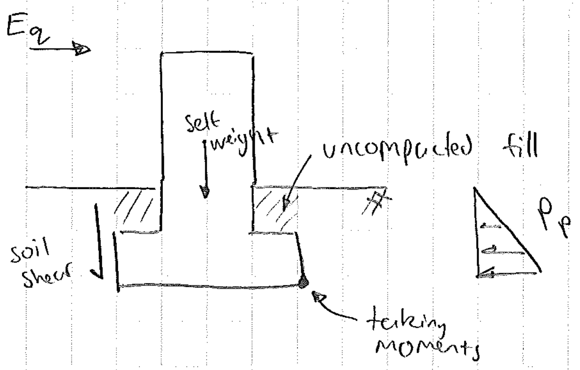
## Pile footings




- Dimensions below ground have been assumed, as they could not be measured on site
- Have not included weight of internal floors in calculation of self weight




## Self weight

Concrete pile + footing $= [0.200 \times 0.200 \times 0.650 + 0.400 \times 0.400 \times 0.100] \times 24\ kN/m^3$
$= 1.01\ kN$

uncompacted fill $= [(0.400 \times 0.400) - (0.200 \times 0.200)] \times 0.150m \times 15\ kN/m^3$
$= 0.27\ kN$

flooring $= 1.54m \times 1.54m \times 0.14\ kN/m^2$
$= 0.33\ kN$

$\Sigma = 1.61\ kN$

| | |
|---|---|
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| | Calc. Series |
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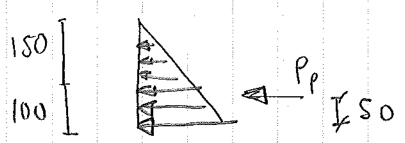
| Calcs Title Quantitative Assessment | Date 12-9-12 |

Passive Pressure ($P_p$)




$\gamma = 17\ kN/m^3 \quad K_p = \frac{1 + \sin 30}{1 - \sin 30}$

$= 3.0$

$\sigma_{250} = K_p H \gamma$

$= 3 \times 0.250 \times 17$

$= 12.8\ kN/m^2$

$\sigma_{150} = 3 \times 0.150 \times 17$

$= 7.7\ kN/m^2$

Average $= (12.8 + 7.7) \div 2$

$= 10.3\ kN/m^2$

$P_p = 10.3\ kN/m^2 \times 0.4 \times 0.1$

$= 0.41\ kN$

Moment capacity

Self weight $= 1.61\ kN \times 0.2m$

$= 0.32\ kNm$

Passive Pressure $= 0.41\ kN \times 0.050\ m$

$= 0.02\ kNm$

Eq load to be capacity by one pile

$= (0.32 + 0.02) \div 0.7m$

$= 0.49\ kN$

---

Total Demand $= 1113\ kN$

Total capacity transverse $= 752\ kN + 34 \times 0.49\ kN = 769\ kN$

total capacity longitudinal $= 1076\ kN + 34 \times 0.49\ kN = 1093\ kN$

% NBS transverse $= 70\ \%$

% NBS longitudinal $= 100\ \%$

**SKM**

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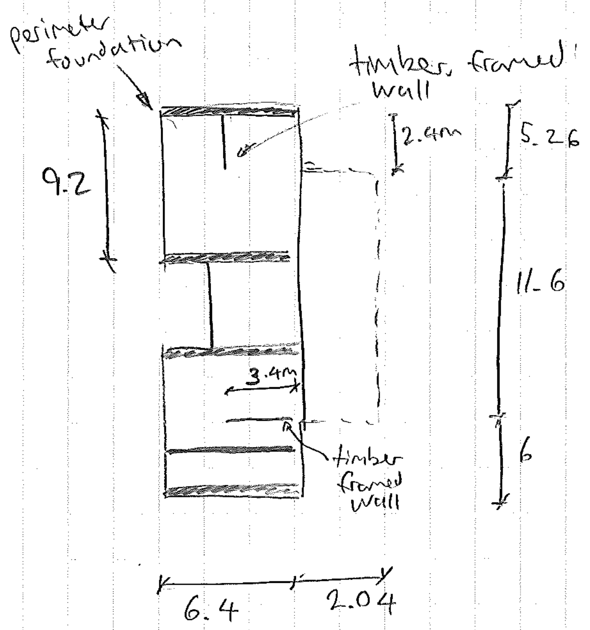
## Diaphragm

Diaphragm capacity is not likely to be a concern in the longitudinal direction due to the diaphragm being 23m deep.

In the transverse direction will need to transfer seismic load from floor to concrete perimeter foundation




**Demand**

= [ floor ] + [ internal timber frame wall ] × Cd(T)

= [ (9.2m/2 × 6.4m) × 0.14 kN/m² ] + [ 3.4m × 2m × 0.25 kPa ]

× 0.9

= 5.2 kN

**Capacity** - diaphragm

= 6.4m × [ 6 kN/m × 0.7 ] ← from NZSEE Table 11.1 for T & G

= 26.9 kN

=

% NBS = 100 %

# 12. Appendix 3 – CERA Standardised Report Form

**Detailed Engineering Evaluation Summary Data** V1.11

## Location

| | | | |
|---|---|---|---|
| Building Name: | PRK_3527_BLDG_001 EQ2 | Reviewer: | N Calvert |
| | Unit / No: / Street | CPEng No: | 242062 |
| Building Address: | Lyttelton Recreation Ground - Pavilion / Godley Quay, Lyttelton | Company: | Sinclair Knight Merz |
| Legal Description: | Lot 3, DP11243 | Company project number: | ZB01276.039 |
| | | Company phone number: | 03 940 4900 |
| | Degrees / Min / Sec | | |
| GPS south: | | Date of submission: | 4-Feb |
| GPS east: | | Inspection Date: | 14/08/2012 |
| | | Revision: | C |
| Building Unique Identifier (CCC): | | Is there a full report with this summary? | yes |

## Site

| | | | |
|---|---|---|---|
| Site slope: | flat | Max retaining height (m): | 0 |
| Soil type: | silt | Soil Profile (if available): | Granular fill (0-0.7mbgl), man made hydraulic fill (0.7-20mbgl) |
| Site Class (to NZS1170.5): | E | | |
| Proximity to waterway (m, if <100m): | | If Ground improvement on site, describe: | None |
| Proximity to clifftop (m, if < 100m): | | | |
| Proximity to cliff base (m,if <100m): | | Approx site elevation (m): | 1.00 |

## Building

| | | | |
|---|---|---|---|
| No. of storeys above ground: | 1 | single storey = 1 | |
| | | Ground floor elevation (Absolute) (m): | 1.00 |
| Ground floor split? | no | Ground floor elevation above ground (m): | 0.30 |
| Storeys below ground | 0 | | |
| Foundation type: | strip footings | if Foundation type is other, describe: | |
| Building height (m): | 3.00 | height from ground to level of uppermost seismic mass (for IEP only) (m): | 3.5 |
| Floor footprint area (approx): | 146 | | |
| Age of Building (years): | 40 | 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): | | | |
| Importance level (to NZS1170.5): | IL2 | | |

### Gravity Structure

| | | | |
|---|---|---|---|
| Gravity System: | load bearing walls | | |
| Roof: | timber truss | truss depth, purlin type and cladding | |
| Floors: | timber | joist depth and spacing (mm) | |
| Beams: | | | |
| Columns: | | | |
| Walls: | fully filled concrete masonry | thickness (mm) | 190 |

### Lateral load resisting structure

**Note: Define along and across in detailed report!**

| | | | | |
|---|---|---|---|---|
| Lateral system along: | fully filled CMU | | note total length of wall at ground (m): | 45 |
| Ductility assumed, μ: | 1.00 | | wall thickness (m): | 0.19 |
| Period along: | 0.40 | 0.01 from parameters in sheet | estimate or calculation? | estimated |
| Total deflection (ULS) (mm): | 5 | | estimate or calculation? | estimated |
| maximum interstorey deflection (ULS) (mm): | 5 | | estimate or calculation? | estimated |
| Lateral system across: | fully filled CMU | | note total length of wall at ground (m): | 30 |
| Ductility assumed, μ: | 1.00 | | wall thickness (m): | 0.19 |
| Period across: | 0.40 | 0.01 from parameters in sheet | estimate or calculation? | estimated |
| Total deflection (ULS) (mm): | 5 | | estimate or calculation? | estimated |
| maximum interstorey deflection (ULS) (mm): | 5 | | estimate or calculation? | estimated |

### 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: | | 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: | green | | |
| Along | Damage ratio: | 46% | Describe how damage ratio arrived at: | Engineering judgement of level of damage |
| | Describe (summary): | Damage insignificant in building capacity | | |
| Across | Damage ratio: | 46% | | |
| | Describe (summary): | Damage insignificant in building capacity | | |
| 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: | minor structural | Describe: | Cracking repairs to blockwork |
| Building Consent required: | no | Describe: | Like for like repair |
| Interim occupancy recommendations: | full occupancy | Describe: | No restrictions required |

| | | | | | |
|---|---|---|---|---|---|
| Along | Assessed %NBS before: | 46% | %NBS from IEP below | If IEP not used, please detail assessment methodology: | Quantitative Assessment |
| | Assessed %NBS after: | 46% | | | |
| Across | Assessed %NBS before: | 46% | %NBS from IEP below | | |
| | Assessed %NBS after: | 46% | | | |