CHRISTCHURCH CITY COUNCIL

PRK_1190_BLDG_011 EQ2
Hagley Park North—Toilet – Lake Albert
Hagley Park, Christchurch

QUANTITATIVE ASSESSMENT REPORT

FINAL

- Rev B
- 24 September 2013

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1. Executive Summary

1.1. Background

A quantitative Detailed Engineering Evaluation was carried out on the building PRK_1190_BLDG_011 EQ2 located in Hagley Park North, known as Toilet - Lake Albert. The building is single storey and is currently utilised as a toilet block and for storage. It is constructed from reinforced masonry walls and a timber-framed roof with lightweight roof cladding. An aerial photograph illustrating this area is shown below in Figure 1. Detailed descriptions outlining the building's age and construction type are given in Section 5 of this report.

Figure 1 Aerial Photograph of Hagley Park North—Toilet – Lake Albert

This Quantitative report for the building structure is based on the Engineering Advisory Group's "Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings" (from July 2011), visual inspection on 07 May 2012, building measure-up on 18-20 March 2013, intrusive investigation on 18 March 2013 and geotechnical desk study in July 2012. No drawings of the buildings were available for the evaluation.

1.2. Key Damage Observed

Key damage observed includes:-

1) Step cracking along mortar joints.
2) Cracks between internal and external masonry walls (approximately 1mm wide).
3) Gaps opening up between masonry wall, timber roof and roof cladding elements (gaps up to approximately 5mm wide).

Further details describing the level of damage and repair recommendations are given in section 5.4 of this report.

1.3. Critical Structural Weaknesses

No critical structural weaknesses have been discovered.

1.4. Building Capacity

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 capacity of the building structure and presented the results as a percentage of new building standard (%NBS). Our assessment considered geotechnical conditions, existing damage to the building and structural engineering calculations to assess both strength and ductility/resilience.

Due to the lack of geotechnical information, an estimate was made as to determine capacity of the foundations. Geotechnical input is recommended to validate the assessment although we don`t expect the difference in the overall building rating will be significant.

Any building that is found to have a seismic capacity less than 33% of the new building standard (NBS) is required to be strengthened up to a capacity of at least 67%NBS in order to comply with Christchurch City Council (CCC) policy - Earthquake-prone dangerous & insanitary buildings policy 2010.

Based on the information available, and using the Quantitative Assessment Procedure, the buildings original capacity has been assessed to be in the order of 51%NBS and post earthquake capacity in the order of 51%NBS. The buildings post earthquake capacity excluding critical structural weaknesses is in the order of 51%NBS.

The critical elements in the building with a low capacity are the unreinforced hollow block walls to Stage 1 building.

The building has been provisionally assessed to have a seismic capacity in the order of 51% NBS and is therefore not potentially earthquake prone.

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

1.5. Recommendations

Based on the findings of this assessment indicating the building is in the order of 51% NBS, no strengthening is required in order to comply with Christchurch City Council (CCC) policy – Earthquake-prone dangerous & insanitary buildings policy 2010.

However, an assumption was made for the verification of the foundation/ground bearing capacity and we recommend that the soil properties used for the calculations are reviewed and confirmed by a geotechnical engineer to fully validate the outcome of the quantitative assessment.

Our key findings and recommendations are:

a) There is no damage to the building that would cause it to be unsafe to occupy.

b) Barriers around the building are not necessary.

c) A geotechnical site investigation be undertaken to confirm subsoil properties and allowable bearing pressures – more precise information about current ground bearing capacity may lead to different overall rating of the building, however we don`t expect the result would significantly differ from the current rating.

2. Introduction

Sinclair Knight Merz was engaged by Christchurch City Council to carry out a Quantitative Detailed Engineering Evaluation of the toilet block located near Lake Albert in North Hagley Park in Christchurch. The scope of this quantitative analysis comprises:

- Analysis of the seismic load carrying capacity of the building compared with current seismic loading requirements expressed as a percentage of new building 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 capacity of the structure.
- Preparation of a summary report outlining the areas of concern in the 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].

In absence of structural drawings, building measure-up and intrusive investigation was carried out 18-20 March 2013 to assist with assessment of the building. Findings of the intrusive investigation were used to produce sketches of the building to indicate likely configuration of the structures – see Appendix B.

[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

The building description is based on the visual inspection on 07 May 2012 and the visual inspection, building measure-up and intrusive investigation on 18-20 March 2013. In absence of available drawings, findings of the intrusive investigation were used to produce sketches of the building to indicate likely configuration of the structures – see Appendix B.

5.1. Building description

The building is located near the north-east bank of Lake Albert in North Hagley Park. The building is one storey high structure currently utilised as a toilet block and a storage area (see PHOTOS 31-42 and sketches in Appendix B).

The building consists of various structural configuration which leads to conclusion that the building was built in number of stages over the time as shown in Figure 3

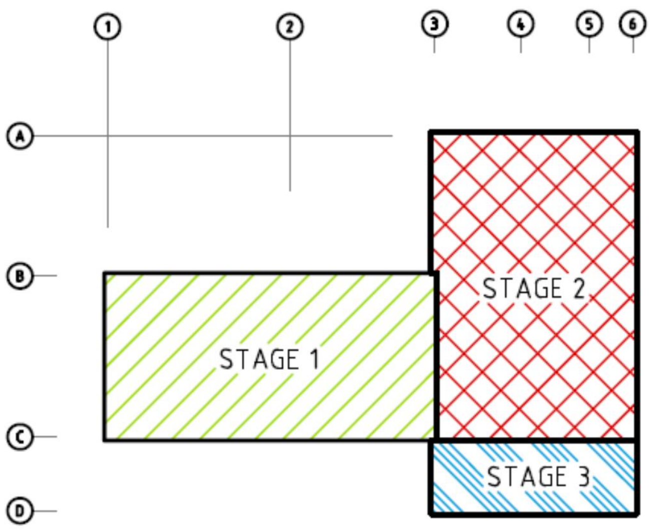

Figure 3: Sketch indicating likely construction sequence

While the Stage 1 building is constructed of 140mm thick unreinforced hollow bock masonry walls, the Stage 2 building is constructed of 190mm thick concrete infilled reinforced walls typically with D12 @ 600mm vertical and horizontal centres. The Stage 3 building is of similar construction to the Stage 2 building, to which it is evidently attached (with visible construction joints).

The tops of the walls are tied with reinforced concrete bond beams that provide support to the roof structure, which is attached to the bond beams via timber wall plates fixed to concrete beams using M10 bolts @ 600-700mm centres.

The roof is constructed of timber rafters sitting on bond beams at the base and propped by rafters coming from the other side of the roof at the ridgeline. In absence of horizontal ties between base supports, the horizontal reactions are taken by lateral flexure of the supporting bond beams. The roof to Stage 1 building is covered by timber sarking (85x10 timber planks) and Stage 2&3 by ply sarking (18mm thick ply wood). Corrugated metal sheeting provides water tightness.

The ground floor is concrete slab on grade:

- Stage 1 – 70mm thick with no reinforcement
- Stage 2&3 – 120mm thick with mesh reinforcement (wires at 150mm centres)

The walls are typically found on 200-220mm wide reinforced concrete strip footing of variable depth ranging from 380 to 600mm.

In absence of drawings, the construction date of the building is estimated to be 1965-1976, based on the era of the construction materials, although this could be conservative.

5.2. Gravity load resisting system

At the roof level the gravitational loads are transferred into supporting walls through the timber roof structure with timber rafters, typically spanning in the transverse direction. The rafters are supported off concrete bond beams built on top of masonry walls.

Weight of masonry walls and applied loads are transferred into concrete strip footing/ground slab thickening and resisted by sub-soil.

The ground floor consists of a concrete slab on grade.

5.3. Seismic Load Resisting system

Lateral loads at the roof level are distributed to the supporting walls by action of the roof diaphragms.

18mm thick ply wood sarking is provided to the roof planes above Stage 2&3 buildings, which effectively act in both directions.

Timber planks attached to the Stage 1 roof frame form a diaphragm element which acts in transverse direction by nail couple. In the longitudinal direction the diaphragm loads are transmitted, by tension and compression in these planks, into hip rafters located at the interface with Stage 2&3 roof.

Lateral loads at ground level have been omitted from consideration of seismic assessment. It is assumed that horizontal forces will be resisted by friction between the ground bearing slab and the ground below.

Horizontal forces are transferred to foundation level by means of concrete walls acting as shear walls.

Horizontal forces at foundation level are resisted by friction and ground pressures between the surrounding soil and foundations.

5.4. Building Damage

SKM undertook an inspection on 7 May 2012. The following areas of damage were observed during the time of inspection:

General

Hairline cracking in the block masonry walls is a possible indication of very minor settlement or a result of wall movements during shaking. The geotechnical report indicates a moderate to severe liquefaction risk at the site. However, there was no other visual evidence of settlement noted at this site.

Building Defects

The defects observed at this property were minor. Hairline stepping cracks in external block wall joints appear to be consistent with minor ground settlement, which may or may not be earthquake related. Gaps between internal walls, wall linings, timber elements and joinery appear to be consistent with long term shrinkage and thermal movement, and some may also date from the time of original construction (or a combination of these factors). However, in most cases earthquake movement is a plausible alternative explanation for the observed defects, or may have worsened the pre-existing defects. Here is a list of the specific defects observed:

1) Step cracking along mortar joints on the north and east walls (approximately 0.5mm). (PHOTOS 5 and 14)
2) Vertical separation at joint between internal and external masonry walls in the storage rooms (approximately 3mm). (PHOTO 6)
3) Gap between masonry wall and soffit lining (approximately 1mm). (PHOTO 21)
4) Gap between masonry wall and timber elements in the soffit on the east wall (approximately 4mm). (PHOTO 15)
5) Gap between masonry wall and internal cladding in the toilet entrances (approximately 2mm). (PHOTO 17)
6) Gap between internal wall and roof cladding in the toilets (approximately 3mm). (PHOTO 18)
7) Gap at the connection of the timber members in the centre of the north wall at roof level (approximately 10mm). (PHOTO 26)
8) Cracks in soffit lining on west side (approximately 0.5mm). (PHOTO 27)

9) Crack in external timber rafter on the south side of the building at the apex. This is not believed to be earthquake related, but appears to be the result of weathering and thermal or shrinkage movement. (PHOTO 29)
10) Impact damage to soffit lining on west side noted, but this is not believed to be earthquake related. (PHOTO 26)
11) Damage to end of external timber rafter on the south side of the building was noted, but this is not believed to be earthquake related. (PHOTO 30)
12) Existing low quality pointing noted in the northeast top corner of masonry. This is not earthquake related. (PHOTO 21)
13) Spalling of 110mm high concrete strip under masonry wall was noted, but is not as a result of earthquake damage. (PHOTO 20)
14) Removal of internal gable linings (lightweight corrugated material) above interior masonry wall was noted, but this is not a result of earthquake damage. (PHOTO 8)

During the visual inspections on 18-20 March 2013, the further damage was identified, as follows:

15) Vertical rupture in masonry wall at the intersection of grid lines 3/B. It appears that the rupture occurred at the vertical joint between older 140mm thick wall and newer 190mm thick wall, which leads to conclusion that the separation is a result of insufficient coupling of these two walls (PHOTO 43-44). As to be expected the crack has formed in the older, weaker 140mm wall.
16) A number of roof supporting elements in Stage 1 building (grid 2/B-C) have been compromised by fire. While such damage has likely negligible effect on seismic performance of the building, collapse hazard due to gravitational loads such as roof access and snow load may be considerable (PHOTO 45-46).

6. Available Information and Assumptions

6.1. Available Information

Following our inspections on 7 May 2012 and 18-20 March 2013 SKM carried out Quantitative Detailed Engineering Evaluation using the following information:

- SKM site measurements and inspection findings for the building.
- Intrusive investigation and reinforcement detection
- Geotechnical desk study (in absence of detailed geotechnical investigation, assumption have been made).

No drawings of the building were available.

6.2. Survey

Level or verticality survey was not considered necessary at this time.

6.3. Design Criteria and Assumptions

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

- The building was built according to the drawings and according to good practice at the time of construction
- The soil on site is class D as described in AS/NZS1170.5:2004, Clause 3.1.3, Soft Soil. This is a conservative assumption based on the desktop study. The ultimate bearing capacity on site was estimated to be in order of 100 kPa and is subject to confirmation by geotechnical engineer.
- 50 year 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
- The following criteria were used for the assessment of the building:

North-south direction	East-west direction
Period, T = 0.4s Ductility, μ = 1.25 Cd(T) = 0.73	Period, T = 0.4s Ductility, μ = 1.25 Cd(T) = 0.73

- The following material properties were estimated and used in the analyses:

Table 2: Material Properties

Material	Nominal Strength	Structural Performance
Concrete	f_c' = 30MPa	Sp = 0.9
Masonry – Concrete Blocks	f\`$_m$ = 12MPa (type B)	Sp = 1.0 (unreinforced) Sp = 0.9 (reinforced)
Mortar used in masonry	f\`j = 5.5 MPa (medium hardness)	Sp = 1.0 (unreinforced) Sp = 0.9 (reinforced)
Reinforcing steel	f_y = 250MPa	Sp = 0.9

- Representative locations were selected for intrusive investigation in Stage 1 & Stage 2 buildings to establish structural configuration of foundations, masonry walls and bond beams. Such information was considered as being applicable for all other similar elements in the buildings considering different times of construction between the sections of the building (for instance size of reinforcement was visually checked at one location and rebar detector was used to confirm presence of reinforcement in similar elements elsewhere assuming the same bar diameter was used).

The detailed engineering evaluation 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 issues that could affect the performance of the building such as corrosion of metallic elements 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 building. Other aspects such as building services are not covered.

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].

[3] http://resources.ccc.govt.nz/files/EarthquakeProneDangerousAndInsanitaryBuildingsPolicy2010.pdf

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 3. 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].

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[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 relevant current design standards and codes of practice pertinent to determining %NBS of building structures are primarily:

- AS/NZS 1170 Structural Design Actions
- NZS 3101:2006 Concrete Structures Standard
- NZS 3603:1993 Timber Structures Standard
- NZS 3604:2011 Timber-framed Buildings
- NZS 4230:2004 Design of Reinforced Concrete Masonry Structures
- AS 3700:2011 Masonry Structures (Structural Design of Unreinforced Masonry)

7. Results and Discussion

7.1. Critical structural weaknesses and collapse hazards

No critical structural weaknesses have been identified in this building.

7.2. Analysis Results

The equivalent static method as defined in NZS1170.5, clause 6.2, was used to calculate the appropriate seismic loads to apply to the building in order to analyse the response of the building and calculate the capacity.

The results of the analysis are reported in the following table, expressed as %NBS. The results below are calculated for the building in its damaged state. The building results have been broken down into seismic resisting elements, locations and actions as practicable.

Table 4: DEE Results

Seismic Resisting Element	Action	Note / worst case	Seismic Rating %NBS
Masonry walls	In plane response (vertical bracing)	Rocking failure mode in hollow core masonry wall at 2/B-C	52% NBS
	Out-of-plane response	Hollow core masonry wall at 2/B-C	51% NBS
Bond beams	Horizontal flexure	Bond beam at 3/A-B	68% NBS
Roof Diaphragm	Roof plane bracing	-	100% NBS
Foundations	Ground pressure at toe due to overturning moment	2/B-C	77% NBS* (based on estimated ground properties)

* Assumptions were adopted for this calculation. The capacity given is only provisional and is to be reviewed upon obtaining likely ground properties from the geotechnical engineer.

Geotechnical investigation, as recommended in the previous Qualitative Detailed Engineering Evaluation report (Section 5.4 Geotechnical Conditions) issued by SKM on 20 August 2012 should be carried out to validate assumptions made during the assessment.

7.3. Conclusions and Recommendations

The capacity of the building has been provisionally calculated as 51% NBS. The critical elements in the building with a low capacity are the unreinforced hollow block walls to Stage 1 building.

The capacity of 51% would lead to the building being considered as in the category 'moderate risk buildings' which are acceptable legally, but recommended to be improved.

Assumption was made for the verification of the foundation/ground bearing capacity and we recommend that the soil properties used for the calculations are reviewed and confirmed by a geotechnical engineer to fully validate the outcome of the quantitative assessment.

If it is determined that the building should be repaired there are a number of issues which will need to be investigated and associated documents prepared in order to submit a building consent application. These issues will need to be considered during the initial phase of strengthening works. Listed below are the likely items the council may require to be explored:

- A geotechnical investigation will be required and associated factual and interpretive geotechnical reports prepared – the geotechnical reports will be required to enable completion of the strengthening design.
- A fire report will be required and all necessary upgrades to egress routes, emergency lighting and specified systems will need to be undertaken.
- An emergency lighting design will be required to meet the provisions noted in the fire report.
- A disabled access summary will be required including provision for disabled facilities.
- The site amenities (toilets and the like) will need to be reviewed to ensure that there are sufficient facilities for the expected number of people on site.
- Landscaping will need to be considered although we do not anticipate that any modifications will be required since you will not be adjusting the footprint area of buildings on site and will likely only be required for the new build option.

8. Conclusion

SKM carried out a quantitative DEE of library building located in North Hagley Park near Lake Albert, in Christchurch Central. This assessment concluded that the building may be classified as a category 'moderate risk building' which is acceptable legally, but recommended to be improved.

The classification is subject to confirmation of ground properties estimated for the assessment.

Table 5: Quantitative assessment summary

Description	Grade	Risk	%NBS	Structural Performance
Lake Albert Toilets	C	Moderate	51 % NBS*	Legally acceptable. Improvement recommended

* The classification is subject to confirmation of ground properties estimated for the assessment.

Our key findings and recommendations are:

a) There is no damage to the building that would cause it to be unsafe to occupy.

b) Barriers around the building are not necessary.

c) A geotechnical site investigation be undertaken to confirm subsoil properties and allowable bearing pressures – more precise information about current ground bearing capacity may lead to different overall rating of the building, however we don`t expect the result would significantly differ from the current rating.

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. SITE INSPECTION REPORT PHOTOS

Site Inspection 07 May 2012	
	
PHOTO 1: East elevation	PHOTO 2: North elevation
	
PHOTO 3: West elevation	PHOTO 4: South elevation

PHOTO 5: Step cracking along mortar joints on the east wall.

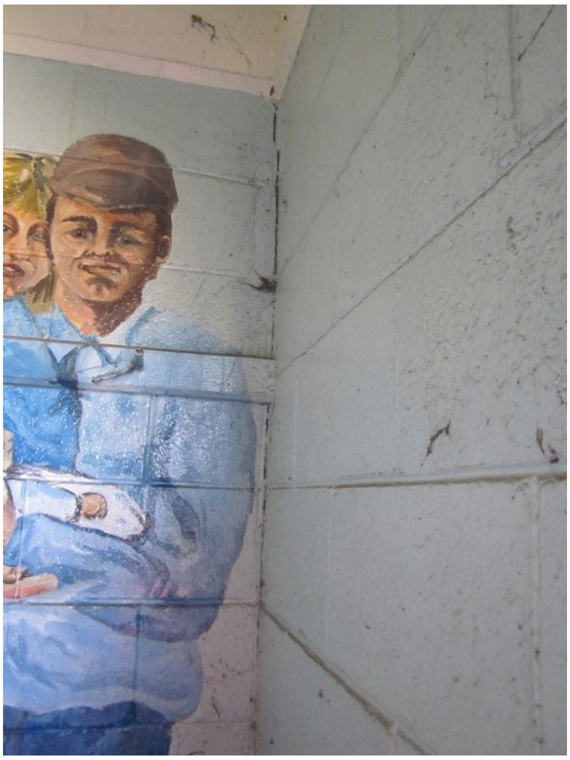


PHOTO 6: Crack between internal and external masonry wall inside the southern-most storage area.

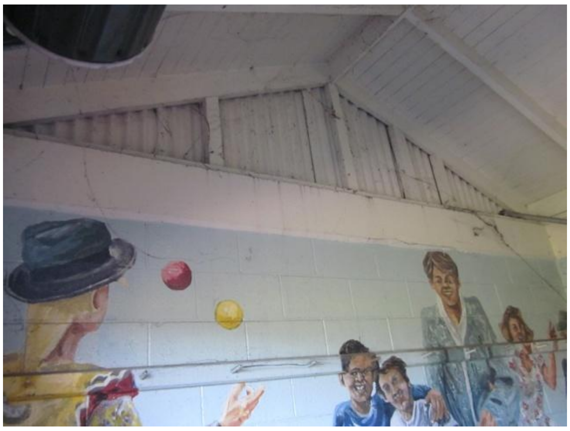


PHOTO 7: Internal south wall of the southern-most storage area.

PHOTO 8: Internal north wall of the southern-most storage area, showing removal of corrugated material.

	
PHOTO 9: Joint between masonry wall and roller door for southern-most storage area.	PHOTO 10: Timber-framed roof inside storage area.
	
PHOTO 11: Central storage area.	PHOTO 12: Crack between internal and external masonry walls inside the central storage area.

PHOTO 13: Central ceiling connection on the internal north wall of the central storage area.

PHOTO 14: Step cracking along mortar joints on the east wall.

PHOTO 15: Gap opening up between masonry wall and timber ceiling elements above locked toilet on east side of building.

PHOTO 16: Interior of locked toilet.

	
PHOTO 17: Gap opening up between masonry wall and internal wall cladding in locked toilet.	PHOTO 18: Gap opening up between interior wall and ceiling cladding and timber rafter in locked toilet..
	
PHOTO 19: Gap opening up at internal wall joint in locked toilet.	PHOTO 20: Spalling of concrete section of wall on east side.

	
PHOTO 21: Existing low quality pointing on northeast corner.	PHOTO 22: Gap opening up between interior cladding and masonry wall in north toilets (typical for both entrances).
	
PHOTO 23: Gap opening up at interior wall cladding joints in north toilets.	PHOTO 24: Gap opening up at wall and roof cladding at north toilet entrance.

	
PHOTO 25: Interior roof layout in one of the two north toilets.	PHOTO 26: Gap opening up at connection between timber beams at roof level on the north side.
	
PHOTO 27: Gap opening up between joints in soffit lining on west side.	PHOTO 28: Existing impact damage to soffit lining on west side (not earthquake related).

	
PHOTO 29: Crack in external timber rafter at apex on south side.	PHOTO 30: Existing defect to end of timber fascia on south side.
Site Inspection 18-20 March 2013	
	
PHOTO 31: Exterior view of the property – south view	PHOTO 32: Exterior view of the property – north-east view

	
PHOTO 33: Exterior view of the property – north view	PHOTO 34: Exterior view of the property – south-west view
	
PHOTO 35: Interior view of the property – Roof to Stage 1 building (large storage room)	PHOTO 36: Interior view of the property – Roof to Stage 1 building (small storage room)
	
PHOTO 37: Interior view of the property – Roof to Stage 1 building (large storage room).	PHOTO 38: Interior view of the property – Roof to Stage 3 building.

	
PHOTO 39: Interior view of the property – Roof to Stage 2 building	PHOTO 40: Interior view of the property – Roof to Stage 2 building
	
PHOTO 41: Interior view of the property –Stage 2 building	PHOTO 42: View at roof overhang through hole in cementitious ceiling – Stage 1 building

	
PHOTO 43: Vertical rupture in wall junction at 3/B.	PHOTO 44: Detail of Photo 43.
	
PHOTO 45: Roof timber elements compromised by fire – Stage 1 building (grid line 2/B-C)	PHOTO 46: Detail of Photo 45.

Appendix A CERA Standardised Report Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	North Hagley Park Shelter Toilets (Lake Albert)	Reviewer:	Nick Calvert
	Unit / No: / Street	CPEng No:	141062
Building Address:	North Hagley Park, Christchurch Central	Company:	SKM
Legal Description:		Company project number:	ZB01276.117
		Company phone number:	03 940 4900
	Degrees / Min / Sec		
GPS south:	43 / 31 / 36.50	Date of submission:	24-Sep
GPS east:	172 / 37 / 28.40	Inspection Date:	18-20 March 2013
		Revision:	B
Building Unique Identifier (CCC):	PRK_1190_BLDG_011	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:		Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):	25	If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	5.00

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	3.50
Ground floor split?	no		Ground floor elevation above ground (m):	3.50
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	
Building height (m):	3.50		height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	76			
Age of Building (years):	45		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 framed	rafter type, purlin type and cladding	Assumed timber rafters & purlins and lightweight steel cladding
Floors:	concrete flat slab	slab thickness (mm)	Unknown
Beams:	none	overall depth x width (mm x mm)	None
Columns:	none	typical dimensions (mm x mm)	None
Walls:	unreinforced concrete masonry	thickness (mm)	200

Lateral load resisting structure

Lateral system along:	unreinforced masonry bearing wall - stone	**Note: Define along and across in detailed report!**	note wall thickness and cavity	190mm (140mm is unreinforced and hollow)
Ductility assumed, μ:	1.25			
Period along:	0.40	0.40 from parameters in sheet	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	estimated
Lateral system across:	unreinforced masonry bearing wall - stone		note wall thickness and cavity	200mm
Ductility assumed, μ:	1.25			
Period across:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	estimated

Separations:

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

Non-structural elements

Stairs:			
Wall cladding:	exposed structure	describe	Masonry walls
Roof Cladding:	Metal	describe	Corrugated sheeting
Glazing:			
Ceilings:	strapped or direct fixed		Timber sheeting
Services(list):	Water, sewerage		

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	partial	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:

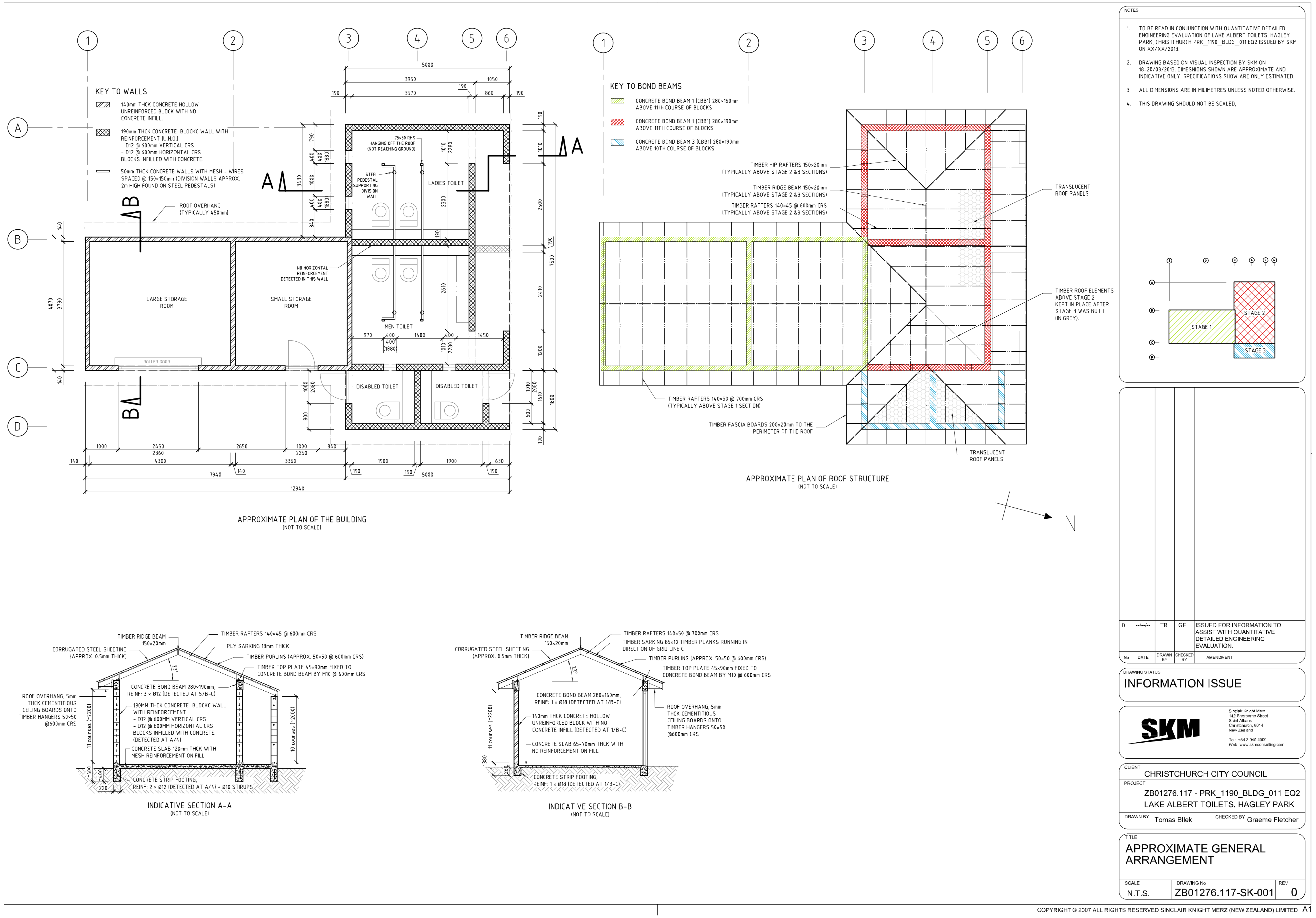
	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	Current damage noted will not diminish the capacity of the building.
	Describe (summary):	Cracking along mortar joints		
Across	Damage ratio:	0%		
	Describe (summary):	Cracking along mortar joints		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	yes	Describe:	Gaps opening up between masonry wall and timber ceiling elements.

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

Recommendations

	Level of repair/strengthening required:	minor non-structural	Describe:	
	Building Consent required:	no	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	Not an immediate collapse hazard.
Along	Assessed %NBS before:	51%	%NBS from IEP below	If IEP not used, please detail assessment methodology: Qualitative Assessment carried out includes NZSEE IEP (refer to SKM report).
	Assessed %NBS after:	51%		
Across	Assessed %NBS before:	51%	%NBS from IEP below	
	Assessed %NBS after:	51%		

Appendix B Building Sketches (18 March 2013)


APPROXIMATE PLAN OF THE BUILDING
(NOT TO SCALE)
APPROXIMATE PLAN OF ROOF STRUCTURE
(NOT TO SCALE)
INDICATIVE SECTION A-A
(NOT TO SCALE)
INDICATIVE SECTION B-B
(NOT TO SCALE)
INFORMATION ISSUE
CHRISTCHURCH CITY COUNCIL
ZB01276.117 - PRK_1190_BLDG_011 EQ2
LAKE ALBERT TOILETS, HAGLEY PARK
Tomas Bilek
Graeme Fletcher
APPROXIMATE GENERAL ARRANGEMENT
N.T.S.
ZB01276.117-SK-001
0
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Appendix C Structural Calculations

SINCLAIR KNIGHT MERZ
SKM

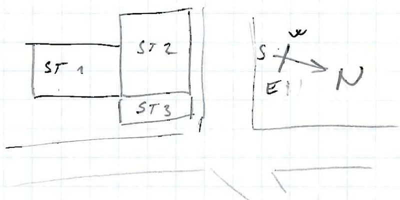
SKM

Job No. ZB 01276.117
Calc. Series
Client CCC
Page 0 - 1
Job Name LAKE ALBERT TOILETS
By Tomas Biker
Calcs Title QUANTITATIVE DEE
Date 22/3/2013

SCOPE

CALCULATE SEISMIC CAPACITY OF THE TOILET BLOCK LOCATED NEAR LAKE ALBERT, HAGLEY PARK, CHRISTCHURCH.

THE BUILDING WAS BUILT IN 3 STAGES AS SKETCHED BELOW



THE STAGE 1 BUILDING CONSISTS OF HOLLOW BLOCKWORK WALLS SUPPORTING TIMBER ROOF WITH LIGHT WEIGHT METAL SHEETING THROUGH CONCRETE BOND BEAMS RUNNING ON THE TOP OF ALL WALLS. ROOF DIAPHRAGM IS FORMED BY TIMBER PLANKS.

THE STAGE 2&3 BUILDING WERE BUILT AT LATER STAGE USING SIMILAR CONSTRUCTION, BUT WITH THE WALLS BEING THICKER, REINFORCED AND INFILLED WITH CONCRETE. ROOF DIAPHRAGM IS PROVIDED BY PLY SARKING.

THE BUILDING IS FOUND ON REINFORCED CONCRETE FOOTING AND GROUND BEARING SLAB (STAGE 1 UNREINFORCED, STAGE 2 & 3 REINFORCED WITH MESH)

NO DRAWING & SPECIFICATIONS WERE AVAILABLE.

CALCULATIONS ARE BASED ON SITE INSPECTION, BUILDING MEASURE UP AND INTRUSIVE INVESTIGATION IN MARCH 2013. → SEE PAGE D-1 FOR DRAWINGS BASED ON INVESTIGATIONS.

CONTENT

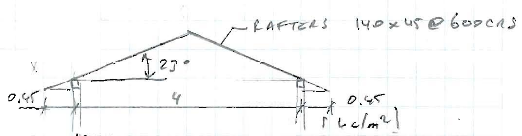
		SUMMARY OF OUTCOME [% NBS]
1	LOADING	
2	BRACING STRATEGY & CAPACITY	52 % NBS
3	MASONRY WALLS - OUT-OF-PLANE FLEXURE	51 % NBS
4	BOND BEAMS	68 % NBS
5	ROOF DIAPHRAGM	100 % NBS
6	FOUNDATIONS (SOIL BEARING CAPACITY NOT KNOWN)	[26-52 % NBS PROVISIONALLY]
D	DRAWINGS	

SKM

Job No. ZB01276.117
Calc. Series LOADING
Client CCC
Page 1-1
Job Name LAKE ALBERT TOILETS
By TB
Calcs Title LOADING
Date 19/3/2013

LOADING - SELF WEIGHT

ROOF (STAGE 2)



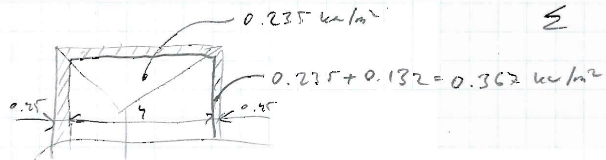
- RAFTERS $0.14 \times 0.045 \times \frac{1}{\cos 23°} \times 4.6 \times \frac{1000}{600}$ = 0.053
- SARKING PLY(18mm) $0.018 \times \frac{1}{\cos 23°} \times 6$ = 0.118
- PURLINS @ 600 crs $0.05 \times 0.05 \times \frac{1}{\cos 23°} \times 4.6 \times \frac{1000}{600}$ = 0.021
- METAL SHEATING (0.5) $0.005 \times \frac{1}{\cos 23°} \times 78.5$ = 0.043

Σ = 0.235 kN/m²

∴ ADDITION AT OVERHANG

- CEMENTITIOUS CEILING (5mm) 0.005×13 = 0.065
- TIMBER HANGER A+B $0.05 \times 0.05 \times (0.45 + 0.25) \times 4.6 \times \frac{1000}{600} \times \frac{1000}{450}$ = 0.030
- FASCIA BOARD $0.02 \times 0.18 \times 4.6 \times \frac{1000}{450}$ = 0.037

Σ = 0.132 kN/m²



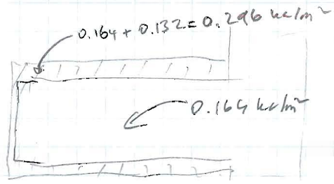
ROOF (STAGE 1)

- RAFTERS $0.14 \times 0.05 \times \frac{1}{\cos 23} \times 4.6 \times \frac{1000}{700}$ = 0.050
- SARKING - TIMB. $0.01 \times \frac{1}{\cos 23} \times 4.6$ = 0.05
- PURLINS @600crs $0.05 \times 0.05 \times \frac{1}{\cos 23°} \times 4.6 \times \frac{1000}{600}$ = 0.021
- METAL SHEATING $0.005 \times \frac{1}{\cos 23°} \times 78.5$ = 0.043

Σ = 0.164 kN/m²

∴ ADDITION AT OVERHANG

AS ABOVE = 0.132 kN/m²



SKM

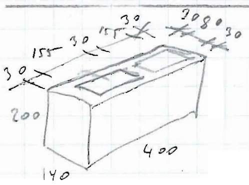
	Job No. ZB01276.117
	Calc. Series LOADING
Client CCC	Page 1-2
Job Name LAKE ALBERT TOILETS	By TB
Calcs Title LOADING	Date 20/3/2013

WALL 1 – 190mm THICK CONCRETE FILLED BLOCKS (TILED)

- CONCRETE BLOCK WITH CONCRETE FILL ... 0.19 × 24 = 4.56
- TILES - SAY ONE SIDE ONLY 0.01 × 22 = 0.22

Σ = 4.78 kN/m²

WALL 2 – 140 THICK HOLLOW BLOCKS (NO FINISH OR PAINT)



[WHOLE BLOCK] – 2× [HOLES]

V = 0.14 × 0.2 × 0.4 – 2× 0.08 × 0.2 × 0.155 = 0.00624 m³

WEIGHT OF ONE BLOCK = 0.00624 × 18.5 = 0.1154 kN

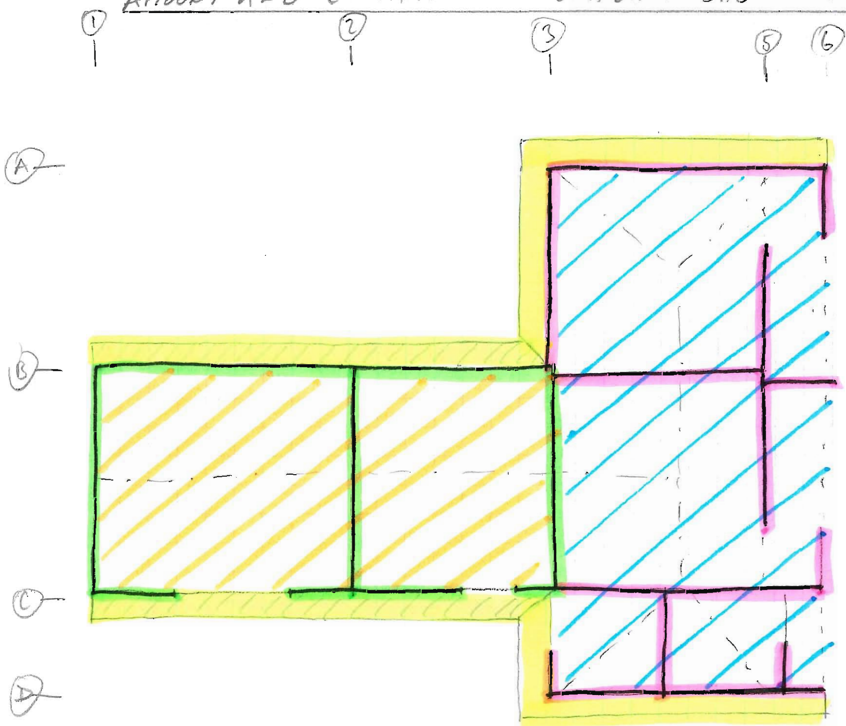
WEIGHT PER m²: 0.1154 × 12.5 = $\boxed{1.443 \text{ kN/m}^2}$

↑ 12.5 BLOCK PER m²

SKM

Job No. ZQ 01276.117
Calc. Series
Client CCC
Page 1 – 3
Job Name LAKE ALBERT TOILETS
By TB
Calcs Title LOADING – LOADING PLAN
Date

AMOUNT AND LOCATION OF LOADING SHOWN ON PLAN:



KEY

	WALL 1	– 4.78 kN/m²	(190 THK INFILLED BLOCK WITH REINF)
	WALL 2	– 1.443 kN/m²	(140 THICK HOLLOW BLOCK)
	ROOF + OVERHANG	– 0.296 kN/m²	(STAGE 1)
	ROOF + OVERHANG	– 0.367 kN/m²	(STAGE 2 & 3)
	ROOF	– 0.164 kN/m²	(STAGE 1)
	ROOF	– 0.235 kN/m²	(STAGE 2 & 3)

+ ALLOW FOR BOND BEAMS TO TOP OF THE WALLS
- $0.16 \times 0.28 \times 24 = 1.07$ kN/m (ST 1)
- $0.19 \times 0.28 \times 24 = 1.28$ kN/m (ST 2&3)

CALCULATIONS

PROJECT	CCC BU 1190_BLDG_011 EQ2 - Lake Albert Toilets		
PART OF STRUCTURE	Earthquake Loading - Design Action Coeficients	PROJECT No. ZB01276.117	REVISION 0
	BOTH DIRECTIONS	DATE 14 Mar 13	BY Tomas Bilek

Earthquake Loading to NZS 1170.5:2002

Spreadsheet Rev 0.1

This spreadsheet is for the calculation of equivalent static earthquake loads on structures. All references are to NZS 1170.5:2004 except where noted. As per NZS 1170.5 this spreadsheet is not applicable to bridges, tanks containing liquids, civil structures (dams and bunds etc) off-shore structures and soil retaining structures. It is recommended that the structure period is calculated but if not 0 seconds should be input to acheive conservative results. This spreadsheet is not applicable to parts of structures - see section 8 for design of parts.

INPUT

	Site location	Christchurch
3.1.3	Site Subsoil class	D
AS/NZS1170.0	Importance level	2
AS/NZS1170.0	Design life	50 years
AS/NZS1170.0	ULS Earthquake Annual probability of exceedance	1/500
AS/NZS1170.0	SLS Earthquake Annual probability of exceedance	1/25

CALCULATION

Structure period, T	0.4 s
Structural Ductility Factor, μ	1.25
ULS Structural Performance Factor from material code, S_p	

Note: Leave these S_p blank to use the values in NZS1170.5

Spectral shape factor, $C_h(T)$	3.00
Spectral shape factor, $C_h(0)$	3.00
Hazard factor, Z	0.3
ULS Return Period, R_u	1.00
SLS Return Period, R_s	0.25
ULS Near fault factor, N(T,D)	1.00
SLS Near fault factor, N(T,D)	1.00
ULS Elastic site hazard spectrum for horizontal loading, C(T)	0.90
SLS Elastic site hazard spectrum for horizontal loading, C(T)	0.23
ULS Elastic site hazard spectrum for vertical loading, $C_v(T)$	0.63
SLS Elastic site hazard spectrum for vertical loading, $C_v(T)$	0.16
ULS Structural Performance Factor, S_p	0.93
ULS Structural Performance Factor for sliding or toppling, S_p	1.0
SLS Structural Performance Factor, S_p	0.7
k_μ	1.14

ULS horizontal design action coefficient, $C_d(T_1)$	**0.73**
ULS horizontal design coefficient sliding or toppling, $C_d(T_1)$	**0.79**
ULS vertical design action coefficient, $C_{vd}(T_1)$	**0.58**
SLS horizontal design action coefficient, $C_d(T_1)$	**0.14**
SLS vertical design action coefficient, $C_{vd}(T_1)$	**0.11**

Filename : ZB01276.117 - CCC - Lake Albert Toilet - Earthquake Loading_0.xlsx 14/03/2013

SKM

		Job No.	ZB 01276.117
		Calc. Series	BRACING
Client	CCC	Page	2-1
Job Name	LAKE ALBERT TOILET	By	TB
Calcs Title	BRACING STRATEGY	Date	20/3/2013

PHILOSOPHY

CONSIDERING THE CONSTRUCTION OF THE BUILDING, LATERAL FORCES ARE LIKELY TO BE DISTRIBUTED THROUGHOUT THE BRACING ELEMENTS ACCORDING TO THEIR TRIBUTARY AREAS (USING BANDWIDTHS AS NOTED ON BRACING STRATEGY PLANS ON PAGES 2 & 3)

THE BUILDING CONSISTS OF 2 MAJOR LATERAL RESISTING SYSTEMS:
- 190 mm THICK REINFORCED BLOCKWORK WALL (STAGE 2 & 3)
- 140 mm THICK UNREINFORCED HOLLOW BLOCKWORK (STAGE 1)

SKM

Job No. ZB01276.117
Calc. Series BRACING
Page 2-2
By TB
Date 20/3/2013

Client CCC
Job Name LAKE ALBERT TOILETS
Calcs Title BRACING STRATEGY

SHEAR WALLS IN EAST-WEST DIRECTION

WALL 1 (190 THICK REINF. BLOCK WALL)
WALL 2 (140 THICK HOLLOW BLOCK – NO REINF.)

SKM

Job No. ZB01276.117
Calc. Series BRACING
Client CCC
Page 2 - 3
Job Name LAKE ALBERT TOILETS
By TB
Calcs Title BRACING STRATEGY
Date 20/3/2013

SHEAR WALLS IN NORTH-SOUTH DIRECTION

SKM

Job No. ZB01276.117
Calc. Series
Client CCC
Page 2-4
Job Name LAKE ALBERT TOILET
By TB
Calcs Title BRACING CAPACITY
Date 20/3/2013

CHECK EW2 – 4m LONG – FOR IN PLANE SHEAR

∴ LOADING

MASS IN TRIBUTARY AREA (i.e. TAKEN BY THE WALL EW2)

• WALLS — OVERALL LENGTH [m] — ASSUMED HEIGHT [m] — UNIT WEIGHT
WALL 2 — 4+4+4 = 12m × 1.2 × 1.443 = 20.8 kN

• CONCRETE BOND BEAM (4+4+4) × 0.28 × 0.16 × 24 = 12.9 kN

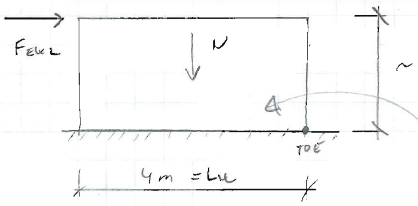
• ROOF – INTERNAL (4×4) × 0.164 = 2.7 kN
OVERHANG 2 × 4 × 0.45 × 0.296 = 0.6 kN

W_{EW2} Σ = 37 kN

∴ HORIZONTAL SEISMIC FORCE

$F_{EW2} = C_d(T_1) \times W_{EW2} = 0.73 \times 37 = \underline{27\ kN}$

∴ ANALYSE WALL



bw
140 mm THICK BLOCKWORK WALL
UNREINFORCED, NO INFILL (HOLLOW)
ASSUME – (f'_b = 26.9 MPa) (BRICK/BLOCK)
f'_j = 5.5 MPa (MORTAR)

• USE APPROACH DETAILED IN "ASSESSMENT AND IMPROVEMENT OF UNREINFORCED MASONRY BUILDINGS FOR EARTHQUAKE RESISTANCE", SECTION 8 – IN PLANE WALL RESPONSE. (AIUMB) – DRAFT 02/2011

→ CHECK CAPACITY IN DIAGONAL TENSION FAILURE MODE. (V_{dt})
→ CHECK CAPACITY IN ROCKING FAILURE MODE (V_r)
→ CHECK CAPACITY IN BED JOINT SLIDING FAILURE MODE (V_s)

CAPACITY IN TOE CRUSHING FAILURE MODE IS UNLIKELY TO OCCURE IN NZ (ACCORDING TO AIUMB)

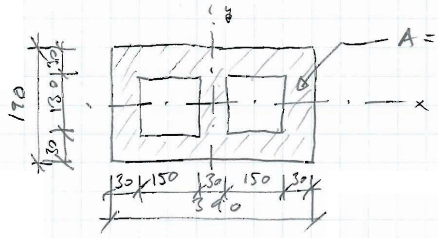
MINIMUM OF THE ABOVE WILL GOVERN THE CAPACITY

SKM

Job No.
Calc. Series
Client
Page 2-5
Job Name BLOCK "20 SERIES" & "15 SERIES"
By TB
Calcs Title SECTION PROPERTIES
Date 21/03/2013

190 mm THICK BLOCK WORK WALL (HOLLOW)

CALCULATE SECTION PROPERTIES ABOUT AXIS X



$A = 390 \times 190 - 2 \times 150 \times 130 = 35\ 100\ mm^2$

AREA PER METER OF WALL

$A_{1m} = 35\ 100 \times \frac{1000}{390} = 90\ 000\ mm^2$ /m OF WALL

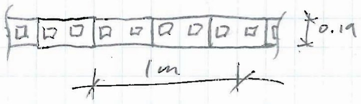
MOMENT OF INERTIA (I)

$I_x = \frac{1}{12} \times 390 \times 190^3 - 2 \times \frac{1}{12} \times 150 \times 130^3 = 167\ 992\ 500\ mm^4$

SECTION MODULUS (W)

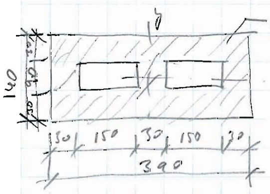
$W_x = \frac{I_x}{190/2} = \frac{167\ 992\ 500}{190/2} = 1\ 768\ 342\ mm^3$

SECTION MODULUS PER 1m LONG WALL



$W_{x,1m} = 1\ 768\ 342 \times \frac{1000}{390} = 4\ 534\ 210\ mm^3$ PER m OF WALL

140 mm THICK BLOCKWORK WALL (HOLLOW)



$A = 390 \times 140 - 2 \times 150 \times 90 = 27\ 600\ mm^2$

$A_{1m} = 27\ 600 \times \frac{1000}{390} = 70\ 770\ mm^2$ /m OF WALL
$= 0.07077\ m^2$

$I_x = \frac{1}{12} \times 390 \times 140^3 - 2 \times \frac{1}{12} \times 150 \times 90^3 = 70\ 955\ 000\ mm^4$

$W_x = \frac{I_x}{140/2} = \frac{70\ 955\ 000}{70} = 1\ 013\ 643\ mm^3$

$W_{x,1m} = 1\ 013\ 643 \times \frac{1000}{390} = 2\ 599\ 084\ mm^3$ /m OF WALL

SINCLAIR KNIGHT MERZ

SKM

SKM

Job No. ZB 01276.117
Calc. Series
Client CCC
Page 2-6
Job Name LAKE ALBERT TOILETS
By TB
Calcs Title BRACING CAPACITY
Date 21/3/2013

CAPACITY IN DIAGONAL TENSION FAILURE MODE (V_{dt})

WALL FLANGES NOT TAKEN INTO ACCOUNT:

$$V_{dt} = 0.54 \; b_w \; L_w \; \xi \; f_{dt} \sqrt{\left(1 + \frac{\sigma_{avg}}{f_{dt}}\right)} \qquad \text{(EQ 8-8b)}$$

- $b_w = 0.14$ m (WIDTH OF WALL)
- $L_w = 4$ m (LENGTH OF WALL)

⇒ VOIDS TO BE DEDUCED ⇒ USE EFFECTIVE (BEDDED AREA) ✓ 0

- $\xi \rightarrow h/L_w = 2.6/4 = 0.65 \Rightarrow \xi = 1.05$ (TAB 8-1)

- DIAGONAL TENSION STRENGTH (f_{dt})

$$f_{dt} = \frac{1}{2}\left(c + \frac{N_t}{A_w} \times 0.8 \mu_f\right) \qquad \text{(EQ 8-5)}$$

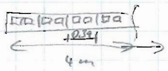
WHERE $c = 0.045 f'_j = 0.045 \times 5.5$ MPa (EQ 2-6)
(COHESION)
$= 0.2475$ MPa

ASSUME MEDIUM HARDNESS (TAB 2-4)
(TYPICAL AVERAGE FOR NZ)

$\mu_f = 0.65$ (EQ 2-7)
(COEF. OF FRICTION)

$N_t = 0$ (AXIAL LOAD ON THE TOP OF WALL IS NEGLIGIBLE ⇒ Ø)

$A_w = (390 \times 140 - 2 \times 150 \times 90) \times \frac{4000}{390} = 283\,077$ mm²
(EFFECTIVE AREA OF WALL PLAN) $= 0.283$ m²



$$\sigma_{AVG} = \frac{N_t}{A_w} = \frac{0}{0.283} = \emptyset \text{ MPa}$$

$$= \frac{1}{2}\left(0.2475 + \frac{0}{0.283} \times 0.8 \times 0.65\right) =$$

$= 0.12375$ MPa

A_w (EFFECTIVE, BEDDED AREA)

$$V_{dt} = 0.54 \times 0.283 \times 1.05 \times 0.12375 \sqrt{1 + \frac{0}{0.315}}$$

$V_{dt} = 0.0198$ MN $= 19.8$ kN

SKM

Job No. ZB01276.117
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Client CCC
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Job Name LAKE ALBERT TOILETS
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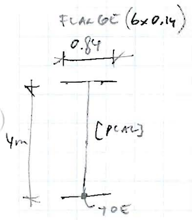
Calcs Title BRACING CAPACITY
Date 21/3/2013

CAPACITY IN ROCKING FAILURE MODE (V_r)

$$V_r = \frac{N_b}{h}\left[a_i - \frac{L_{er}}{3}\right] \qquad (EQ\ 8\text{-}a)$$

$$N_b = W_w + W_f + N_t \qquad (EQ\ 8\text{-}4)$$



$$W_w = A_w \times 2.6 \times 18.5 = 0.283 \times 2.6 \times 18.5 = 13.6\ kN$$

m²/m (AP)

$$W_f = 0.84 \times 0.07077 \times 2.6 \times 18.5 = 2.86$$

$N_t = \emptyset$ (NO LOAD APPLIED ON THE TOP OF THE WALL)

$$= 13.6 + 2.86 + 0 = 16.46\ kN$$

$h = 2.6m$ (HEIGHT OF THE WALL)

$$a_i = \frac{[\Sigma W_{fi} \times d_{fi}] + 0.5 \times W_w \times L_w}{[\Sigma W_{fi}] + W_w} \qquad (EQ\ 8\text{-}6)$$

$$= \frac{2.86 \times 4 + 0.5 \times 13.6 \times 4}{2.86 + 13.6} = 2.347$$

$L_{er} = 0.1 \times L_w = 0.1 \times 4 = 0.4m$ (EFFECTIVE LENGTH OF WALL IN ROCKING Cl.8.4.2)

$$V_r = \frac{16.46}{2.6}\left[2.347 - \frac{0.4}{3}\right] = 14\ kN$$

SKM

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Job Name LAKE ALBERT TOILETS
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CAPACITY IN BED-JOINT SLIDING FAILURE MODE (V_s)

$V_s = L_w \times b_w\, c + 0.8\, \mu_f \times N_t$

↳ USE ONLY EFFECTIVE BEDDED AREA

$= 0.283 \times 0.2475 + 0.8 \times 0.65 \times \emptyset$

$V_S = 0.070$ MN $=$ 70 kN

NOMINAL SHEAR CAPACITY OF THE WALL EW2

$V_n = \min(V_{dt}, V_r, V_s) = \min(19.8, 14, 70)$

$V_n = 14$ kN $< F_{EW2} = 27$ kN FAILS

$\% NBS = \frac{14}{27} \times 100 = \boxed{52\%\ NBS}$

SKM

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CHECK NS6 - 3.7m LONG FOR IN PLANE SHEAR

∴ LOADING

MASS IN TRIBUTARY AREA (TAKEN BY THE WALL NS6)

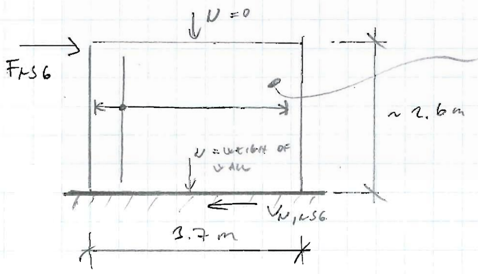
- WALLS
  - WALL 1 3.7+3.7+1.85 = 9.25m × 1.2 × 4.78 = 53 kN
  - WALL 2 1.85 = 1.85m × 1.2 × 1.443 = 3.2 kN
- CONCRETE BOND BEAM - ST2 (3.7+3.7+1.85) × 1.07 = 9.9 kN
- -ST1 1.85 × 1.27 = 2.4 kN
- ROOF - INTERNAL 3.7 × 4.8 × 0.235 = 4.2 kN
  - OVERHANG 1.85 × 0.45 × 0.367 = 0.3 kN

W_{NS6} Σ = 73 kN

∴ HORIZONTAL SEISMIC FORCE

$$F_{NS6} = C_d(T_1) \times W_{NS6} = 0.73 \times 73 = 53.3 \text{ kN}$$

∴ ANALYSE WALL AND CALCULATE CAPACITY (NZS 4230:2004)



190mm THICK VERTICALLY REINFORCED BLOCKWORK MASONRY (D12@600CRS)
ASSUME OBSERVATION TYPE B
f'_m = 12 MPa
f_y = 250 MPa

* HORIZONTAL REINFORCEMENT NOT DETECTED IN WALL "NS6"

CALCULATE $V_{n,NS6} = V_{n,NS6}\, b_w\, d = (v_m + v_p + v_s)\, b_w\, d$

- CONTRIBUTION OF AXIAL LOAD $v_p = \emptyset$
- CONTRIBUTION OF HORIZONTAL REINF $v_s = \emptyset$
- CONTRIBUTION OF MASONRY → SEE OVERLEAF (v_m)

SKM

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CALCULATE CONTRIBUTION OF MASONRY (V_m)

- $A_s = 6\ \phi 12 = 678.6\ mm^2$
- $\rho_w = A_s / (b_w d) = 678.6 / (190 \times 0.8 \times 3700) = 0.0012 > 0.0007$ ok.
- $C_1 = 33\ \rho_w \frac{f_y}{300} = 33 \times 0.0012 \times \frac{250}{300} = 0.033$
- $h_e / L_w = 2.6 / 3.7 = 0.703 \rightarrow C_2 = 0.42 [4 - 1.75 (h_e/L_w)]$
  $= 0.42 [4 - 1.75 (2.6/3.7)]$
  $= 1.164$
- $v_{bm} = 0.7\ MPa$ (BASIC SHEAR PROVIDED BY OBS. TYPE B)

$\underline{v_m} = (C_1 + C_2)\ v_{bm} = (0.033 + 1.164) \times 0.7 = \underline{0.8377\ MPa}$

OVERALL CAPACITY CHECK

- $v_n = v_m + v_p + v_s = 0.8377 + 0 + 0 = 0.8377\ MPa$

$\phi V_{n,NS6} = 0.85 \times v_n \times b_w \times d$
$= 0.85 \times 0.8377 \times 0.19 \times 0.8 \times 3.7$
$= 0.4\ MN = 400\ kN$

$\phi V_{n,NS6} = 400\ kN > F_{NS6} = 53.3\ kN$ O.K.

$\boxed{\gg 100\%\ NBS}$

SKM

Job No. ZB01276.117
Calc. Series MASONRY
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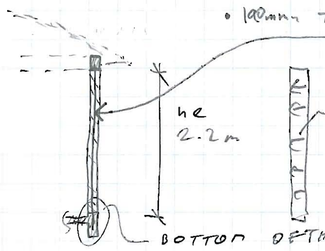
Page 3-1
Job Name LAKE ALBERT TOILETS
By TB
Calcs Title MASONRY WALLS
Date 21/03/2013

CHECK OUT-OF-PLANE BENDING IN WALL NS6



- 190mm THICK WITH D12 @ 600 SPACING VERTICALLY ($A_s = 188.5\ mm^2/m$)
- OBSERVATION TYPE B ($f'_m = 12\ MPa$) ; $f_y = 250\ MPa$

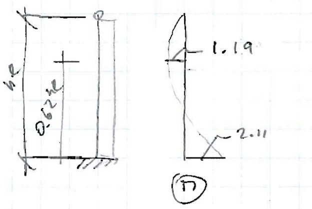
$w = [\text{WALL 1}] \times C\alpha(T_1) = 4.78 \times 0.73 = 3.5\ kN/m^2$

BOTTOM OF THE WALL FIXED (WALL REBAR EMBEDDED INTO FOUNDATIONS – FROM INTRUSIVE INVESTIGATION)

VERTICAL BENDING $M^*_b = -\frac{1}{8} w L^2 = \frac{1}{8} \times 3.5 \times 2.2^2 = 2.11\ kNm$

$M^*_{0.62} = \frac{9}{128} w L^2 = 1.19\ kNm$ (@ 0.62 he @ 1.36 m

AXIAL FORCE AT 0.62 he (@ 1.36m)



$N^*_{0.62} = 4.78 \times (2.2 - 1.36) = 4\ kN$

AT BASE

$N^*_b = 4.78 \times 2.2 = 10.5\ kN$

- CALCULATE OUT OF PLANE BENDING CAPACITY OF THE WALL @ 0.62 he

FOR N = 4 kN $\quad a = \frac{N_n + A_s f_y}{0.85 f'_m \times 1.0} = \frac{4 \times 10^{-3} + 188.5 \times 10^{-6} \times 250}{0.85 \times 12 \times 1.0}$

$= \frac{0.051125}{10.2} = 0.00501\ m$

$\phi M_n = 0.85 (N_n + A_s f_y)\left(\frac{t}{2} - \frac{a}{2}\right)$

$= 0.85 (4 \times 10^{-3} + 188.5 \times 10^{-6} \times 250)\left(\frac{0.19}{2} - \frac{0.00501}{2}\right)$

$= 0.004019\ MNm = 4.02\ kN > M^*_{0.62} = 1.19\ kNm$

O.K. %NBS $\boxed{> 100\%\ NBS}$

- @ BASE

FOR N = 10.5 kN $\quad a = 5.65\ mm$

$\phi M_n = 4.51\ kN > M_b = 2.11\ kNm$ O.K.

$\boxed{> 100\%\ NBS}$

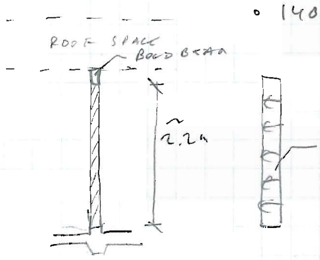
SKM

Job No. ZB 01276.117
Calc. Series
Client CCC
Page 3-2
Job Name LAKE ALBERT TOILET
By TB
Calcs Title MASONRY WALLS
Date 21/3/2013

CHECK OUT OF PLANE BENDING IN EQ2

• 140 mm THICK HOLLOW BLOCK – 20 LEAF.

SECTION PROPERTIES (PAGE 2-5)	PER BLOCK	PER m
A (AREA)	27 600 mm²	70 770 mm²
Zd (SECTION MODULUS)	1 013 643 mm³	2 599 084 mm³



$w = [\text{WALL 2}] \times C_a(T_n) = 1.443 \times 0.73 = 1.05\ kN/m^2$

$M^* = \frac{1}{8} \times 1.05 \times 2.2^2 = 0.635\ kNm$

* USE APPROACH GIVEN IN AS 3700

ϕM_{cv} = LESS OF : $\phi f_{fb}' Z_d + f_d Z_d$ (i)

OR $\phi 3.0 f_{fb}' Z_d$ (ii)

$\phi = 0.6$ (AS PER AS 3700)

• CALCULATE COMPRESSIVE STRESS AT MID HEIGHT (f_d)

WEIGHT OF WALL $1.443 \times 2.1 = 3.03\ kN$

BEDDED JOINT AREA $= 70\ 770\ mm^2$

$f_d = \frac{3.03 \times 10^{-3}}{70\ 770 \times 10^{-6}} = 0.0428\ MPa$

• FLEXURAL BOND STRENGTH (GOVERNED BY MORTAR)

$f_{fb} = 0.025 f_j$ (EQ 2-5, AIUMB)

$f_j = 5.5\ MPa$ (FOR MEDIUM HARDNESS – TAB 2.4 AIUMB)

$= 0.025 \times 5.5$

$= 0.1375\ MPa$

⇒ i) $\phi M_{cv} = 0.6 \times 0.1375 \times 2\ 599\ 084 \times 10^{-9} + 0.0428 \times 2\ 599\ 084 \times 10^{-9}$

$= 0.000\ 2144 + 0.000\ 1112 = 3.25 \times 10^{-4}\ MNm$

$= 0.325\ kNm$

ii) $\phi M_{cv} = 0.6 \times 3 \times 0.1375 \times 2\ 599\ 084 \times 10^{-9} = 6.43 \times 10^{-4}\ MNm$

$= 0.643\ kNm$

$\phi M_{cv} = 0.325\ kNm < M^* = 0.635\ kNm$ FAILS ⇒ $\boxed{51\%\ NBS}$

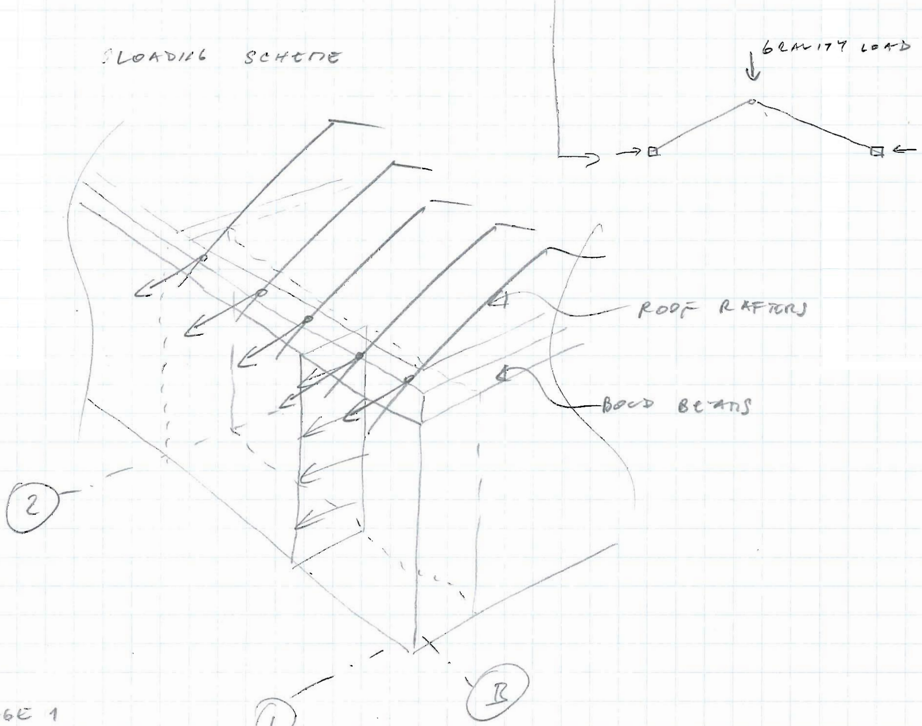
SKM

Client	CCC	Job No.	ZB01276.117
Job Name	LAKE ALBERT TOILETS	Calc. Series	BOND BEAMS
Calcs Title	BOND BEAMS	Page	4-1
		By	TB
		Date	21/03/2013

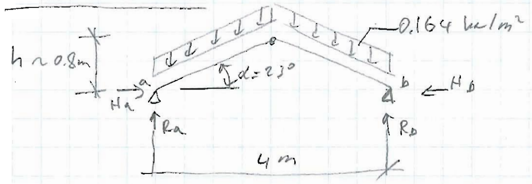
CHECK BOND BEAMS – STAGE 1

WORST CASE SCENARIO LIKELY OCCURES IN BEAM @ B/1-2, WHICH PROVIDES STABILITY TO MASONRY WALL BELOW AND TAKES HORIZONTAL REACTIONS FROM RAFTERS.

LOADING SCHEME



STAGE 1
LOADING SCHEME



$R_a = R_b = \frac{0.164 \times 4}{\cos 23} / 2 = 0.356 \text{ kN/m}$

$H_a = H_b$

USE TRUSS ANALOGY TO CALCULATE TENSION IN BOTTOM CHORD FROM OVERAL BENDING MOMENT

$M = \frac{1}{8} \times \frac{0.164}{\cos 23} \times 4^2 = 0.356 \text{ kNm/m}$

$H_a = H_b = \frac{M}{h} = \frac{0.356}{0.8} = 0.445 \text{ kN/m}$

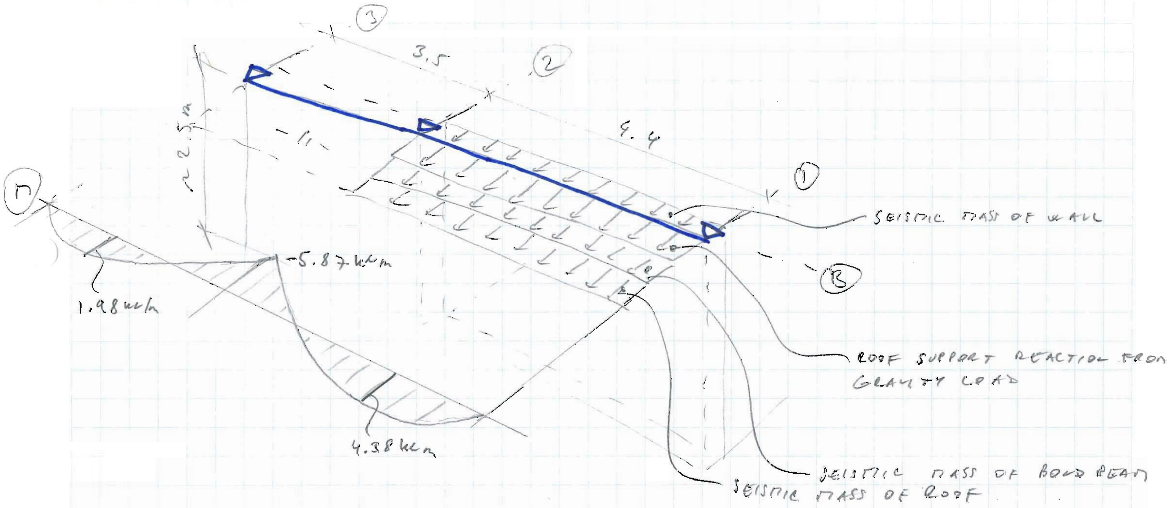
Ha IS RESISTED BY BOND BEAMS

SKM

Job No. ZB 01276. 117
Calc. Series BOND BEAMS
Client CCC
Page 6-2
Job Name LAKE ALBERT TOILETS
By TB
Calcs Title BOND BEAMS
Date 21/3/2013

ANALYSE BOND BEAM @ B/1-2 AND CHECK CAPACITY

- THIS IS LIKELY THE WORST CASE SITUATION



CALCULATE HORIZONTAL UDL ON BOND BEAM (AS SHOWN ABOVE)

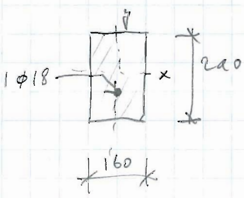
				$C_d(T_1)$	[kN/m]
SEISMIC	-	WALL	1.443 × 1.2	× 0.73	= 1.264
	-	BOND BEAM	0.29×0.16×24	× 0.73	= 0.813
	-	ROOF	2 × 0.164	× 0.73	= 0.239
	-	+OVERHANG	0.45 × 0.296	× 0.73	= 0.133

Σ = 2.45 kN/m

GRAVITY - ROOF HORIZ. REACTION = 0.445 kN/m

TOTAL HORIZ UDL = Σ = 2.895 kN/m

CHECK SECTION CAPACITY



290×160 RC BEAM - ASSUME $f_c = 30$ MPa
$f_y = 250$ MPa

$A_s = 1\phi18 = 254\ mm^2$

BENDING CAPACITY ABOUT AXIS Y

$$M_{u,y} = A_s \times f_y \times \left(d - 0.59\frac{f_y A_s}{f_c\ b}\right)$$

$$= 254\times10^{-6} \times 250 \times \left(0.08 - 0.59\frac{250\times254\times10^{-6}}{30\times0.29}\right) = 4.8\ kNm$$

$\phi M_{u,y} = 0.85 \times 4.8 = 4.1\ kNm < 5.87\ kNm$ FAILS

$\boxed{70\%\ NBS}$

SKM

Job No. ZB 01276.117
Calc. Series BOND BEAMS
Client CCC
Page 4-3
Job Name LAKE ALBERT TOILET
By TB
Calcs Title BOND BEAMS
Date 21/3/2013

TRY BOND BEAM @ 1/B-C (SAME SECTION AS PER B/1-2)

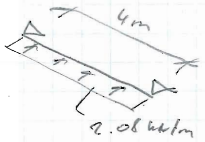
HORIZONTAL UDL ON BEAM

SEISMIC • WALL 1.443 x 1.2 x 0.73 = 1.264

• BOND BEAM 0.29 x 0.16 x 24 x 0.73 = 0.813

Σ = 2.08 kN/m

APPLY OVER SINGLE SPAN



$M_{max} = \frac{1}{8} \times 2.08 \times 4^2 = 4.16$ kNm

$\phi M_{u,y} = 4.1$ kNm $<$ 4.16 kNm FAILS

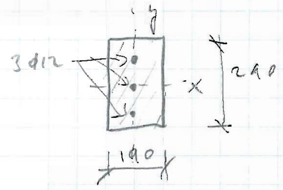
$\boxed{99\%\ NBS}$

SKM

Job No. ZB 01276.117
Calc. Series BOND BEAMS
Client CCC
Page 4-4
Job Name LAKE ALBERT TOILET
By TB
Calcs Title BOND BEAMS
Date 21/3/2013

CHECK BOND BEAMS @ STAGE 2 SECTION

CHECK IF SECTION CAPACITY IS EXCEEDED



290 x 190 RC BEAM – ASSUME $f_c = 30$ MPa, $f_y = 250$ MPa

BENDING CAPACITY ABOUT AXIS Y

$\phi M_{u,y} = \phi A_s f_y \left(d - 0.59 \frac{f_y A_s}{f_c b}\right)$

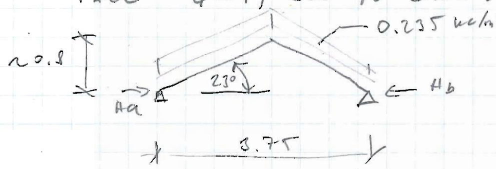
$= 0.85 \times 339.3\times10^{-6} \times 250 \left(0.095 - 0.59 \frac{250 \times 339.3\times10^{-6}}{30 \times 0.29}\right)$

$A_s = 3\phi 12 = 339.3\ mm^2$

$= 6.43$ kNm

ANALYSE BEAM @ 3/A-B

- HORIZONTAL REACTION FROM ROOF (SIMILAR METHOD AS PER PAGE 4-1) DUE TO GRAVITY LOADS



OVERALL MOMENT

$M = \frac{1}{8} \times \frac{0.235}{\cos 23°} \times 3.75^2 = 0.448$ kNm/m

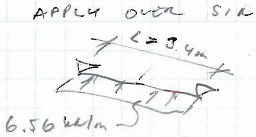
$H_a = H_b = \frac{M}{0.8} = \frac{0.448}{0.8} = 0.56$ kN/m

- SEISMIC LOADS FROM: [kN/m]

• WALL	4.78 × 1.2	× 0.73	= 4.187
• BOND BEAM	0.29 × 0.19 × 24	× 0.73	= 1.322
• ROOF	1.9 × 0.235	× 0.73	= 0.326
• OVERHANG	0.45 × 0.367	× 0.73	= 0.165
		Σ	= 6.0 kN/m

TOTAL HORIZONTAL UDL ON BEAM = 6 + 0.56 = 6.56 kN/m

APPLY OVER SINGLE SPAN (L = 3.4m)



$M_y^* = \frac{1}{8} \times 6.56 \times 3.4^2 = 9.48$ kNm $> M_{uy} = 6.43$ kNm

FAILS

$\boxed{68\%\ NBS}$

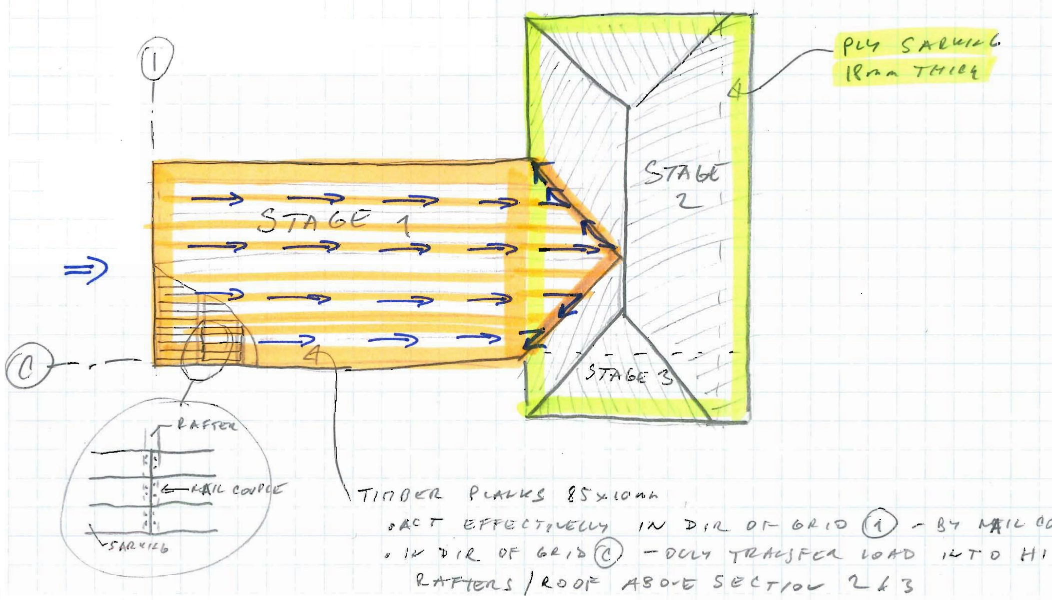
SKM

Client: CCC	Job No.: ZB 01276.117
	Calc. Series:
Job Name: LAKE ALBERT TOILETS	Page: 5-1
Calcs Title: ROOF DIAPHRAGM	By: TB
	Date: 22/2/2013

PHILOSOPHY

- ROOF ABOVE STAGE 2 & 3 IS CONSTRUCTED OF TIMBER RAFTERS COVERED BY PLY SARKING (18 mm MEASURED DURING INTENSIVE INVESTIGATION). FURTHER STABILITY IS PROVIDED BY ACTION OF HIP RAFTER STARTING FROM THE CORNERS OF PERIMETER WALLS.

- ROOF ABOVE STAGE 1 IS CONSTRUCTED OF TIMBER RAFTERS COVERED BY TIMBER SARKING (PLANKS 85×10 mm) RUNNING PARALLEL TO GRID LINE C. WHILE THE DIAPHRAGM WORKS EFFECTIVELY BY ACTION OF "NAIL COUPLE" IN DIRECTION PARALLEL WITH GRID LINE 1, IT IS INEFFECTIVE IN DIRECTION PARALLEL TO GRID LINE C AS IT IS ONLY TRANSFERING DIAPHRAGM LOADS ONTO SUPPORTING ELEMENTS → SUCH LOADS ARE TRANSFERED BY TENSION & COMPRESSION INTO ROOF STRUCTURE ABOVE STAGE 2 BUILDING, WHERE THE FORCES ARE TAKEN TO THE SUPPORTING WALL BY TRIANGULATION OF HIP RAFTERS.



TIMBER PLANKS 85×10mm
- ACT EFFECTIVELY IN DIR OF GRID (1) - BY NAIL COUPLE
- IN DIR OF GRID (C) - ONLY TRANSFER LOAD INTO HIP RAFTERS / ROOF ABOVE SECTION 2 & 3

ROOF DIAPHRAGM WAS FOUND SUFFICIENT BY INSPECTION

$\boxed{100\%\ NBS}$

SKM

Client CCC	Job No. ZB01276.117
Job Name LAKE ALBERT TOILETS	Calc. Series FOUNDATIONS
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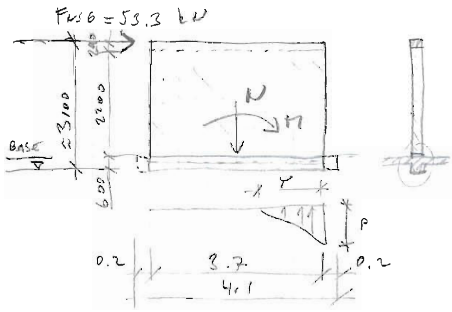
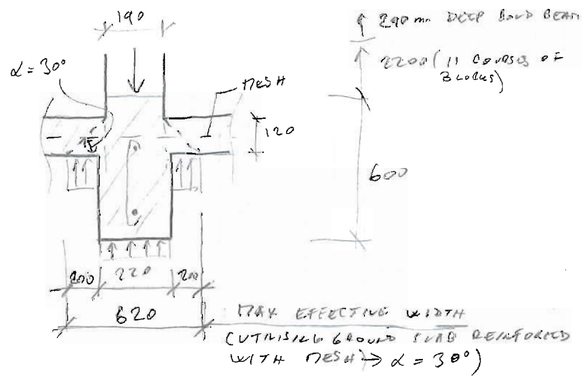
	Date 22/3/2013

CHECK GROUND PRESSURE UNDER MASONRY WALLS

(1) BY INSPECTION, MAX/MIN OVERTURNING MOMENT OCCURS IN WALL NS6 (B/3-5)





• OVERTURNING MOMENT (M)

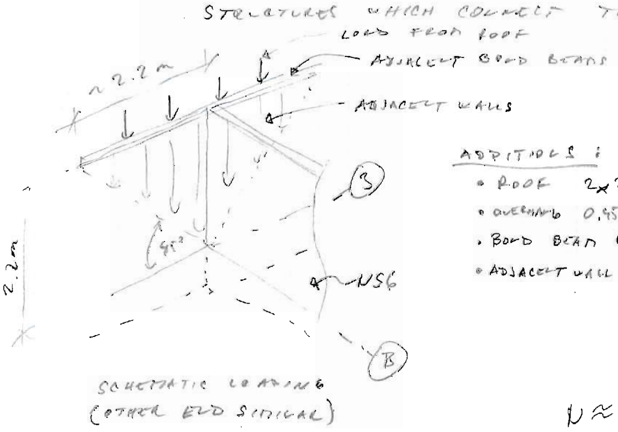
$$M = F_{N6} \times 3.1 = 53.3 \times 3.1 = 165 \text{ kNm}$$

• AXIAL LOAD (N)

SELF WEIGHT OF STRUCTURES

• MASONRY WALL	3.7 × 2.2 × 4.78	=	38.9
• BOND BEAM	0.29 × 0.19 × 3.9 × 24	=	5.15
• FOOTING	0.6 × 0.22 × 4.1 × 24	=	13
		Σ	57 kN

THIS IS TOO CONSERVATIVE, ADD MORE MASS FROM ADJACENT STRUCTURES WHICH CONNECT TO THE SUBJECT WALL



ADDITIONS:

• ROOF	2 × 2.2 × 3.7 × 0.235	=	3.83
• OVERHANG	0.45 × 2 × 0.367	=	0.33
• BOND BEAM	4 × 2.2 × 0.19 × 0.29 × 24	=	11.64
• ADJACENT WALL	$3 \times \frac{2.2 \times 2.2}{2} \times 4.78$	=	34.70
	$1 \times \frac{2.2 \times 2.2}{2} \times 1.443$	=	3.5
		Σ	54

$$N \approx 57 + 54 = 111 \text{ kN}$$

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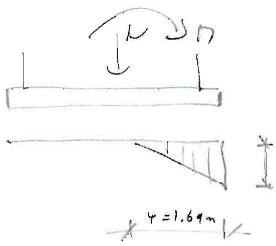
- $e = \frac{M}{N} = \frac{165}{111} = 1.49\ m \quad > D/6 = 4.1/6 = 0.68m$

HIGH ECCENTRICITY

- FOR HIGH ECCENTRICITY $Y = 3\left(\frac{D}{2} - e\right) = 3\left(\frac{4.1}{2} - 1.49\right) = 1.69m$

- MAX PRESSURE AT TOE

$$P = \frac{2N}{BY} = \frac{2 \times 111}{0.62 \times 1.69} = 212\ kPa$$



ALLOWABLE BEARING CAPACITY

SOIL PROPERTIES NOT KNOWN TO DATE → TESTING TO OBTAIN GROUND BEARING CAPACITY IS REQUIRED TO VALIDATE THIS CALCULATION (SCALAR TESTING FOR INSTANCE AT LEAST IN ABSENCE OF MORE COMPREHENSIVE INVESTIGATION.

FOR SAKE OF OBTAINING PROVISIONAL CAPACITY, ASSUME GBP IS IN RANGE OF 100 – 200 kPa.

ABP = 0.6 GBP → 0.6 × 100 = 60 kPa < 212 kPa
↑ SAFETY FACTOR → 0.6 × 200 = 120 kPa

∴ ABP IS LIKELY TO BE EXCEEDED

PROVISIONAL LIKELY CAPACITY IN RANGE OF 28 – 56 % NBS

! OBTAINING SOIL PROPERTIES WILL BE REQUIRED TO DETERMINE WHETHER THE CAPACITY IS MORE OR LESS THAN 33% NBS

SKM

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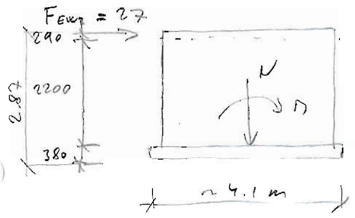
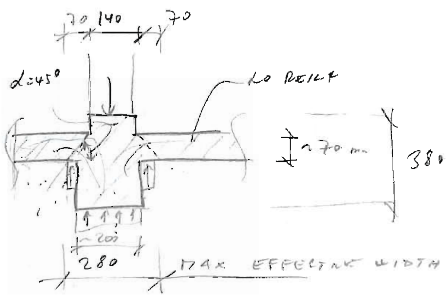
(2) TRY OVERTURNING MOMENT IN WALL EW2 (2/B-C)

- THIS WALL HAS EFFECTIVELY NARROWER FOUNDATIONS AND CONTRIBUTION OF SLAB IS LESS EFFECTIVE SINCE THERE IS NO MESH





OVERTURNING MOMENT (M)

$M = F_{EW2} \times 2.87 = 27 \times 2.87 = 77.5$ kNm

AXIAL LOAD AT BASE (N)

SELF WEIGHT OF STRUCTURES

• MASONRY WALL	4 × 2.2 × 1.443	=	12.7
• BOND BEAM	4 × 0.24 × 0.16 × 24	=	4.45
• FOOTING	4.1 × 0.2 × 0.38 × 24	=	7.48

$\Sigma = 24.63$ kN

ADDITIONAL SELFWEIGHT OF ADJACENT STRUCTURES (SIMILAR APPROACH AS PER NS-6 (TWO PAGES BACK))

2.2, 2.2

1.0, 2.2

• ROOF	2 × 2.2 × 4 × 0.164	=	2.89
• OVERHANG	4 × 2.2 × 0.45 × 0.296	=	1.17
• BOND BEAM	4 × 2.2 × 0.24 × 0.16 × 24	=	9.8
• ADJACENT WALL	$3 \times \frac{2.2 \times 2.2}{2} \times 1.443$	=	10.48
	$1 \times \frac{2.2 \times 1}{2} \times 1.443$	=	1.59

$\Sigma = 25.93$ kN

$N = 24.63 + 25.93 = 50.56$ kN

SKM

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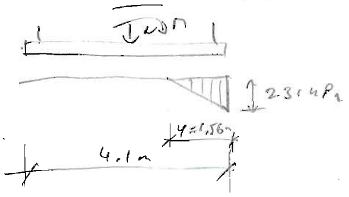
- $e = \frac{M}{N} = \frac{77.5}{50.56} = 1.53\ m \quad > \quad D/6 = 4.1/6 = 0.68$

HIGH ECCENTRICITY

- FOR HIGH ECCENTRICITY $y = 3\left(\frac{D}{2} - e\right) = 3\left(\frac{4.1}{2} - 1.53\right) = 1.56m$

- MAX PRESSURE AT TOE

$$P = \frac{2N}{By} = \frac{2 \times 50.56}{0.28 \times 1.56} = 231\ kPa$$



ALLOWABLE BARING PRESSURE

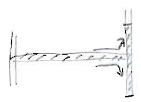
SIMILARILY TO FOUNDATIONS BELOW N56 (B/3-5), SOIL PROPERTIES UNKNOWN.

PROVISIONALLY ASSUMING GBP 100 – 200 kPa (ABP 60 – 120 kPa)

PROVISIONAL LIKELY CAPACITY IN RANGE OF 26 – 52% NBS

04/04/2013
TB

THE ABOVE APPROACH IS SIMPLIFIED TO THE EXTENT THAT THE WALL & THE BELOW FOOTING IS TREATED AS AN ISOLATED STRUCTURE, IN REALITY THE GROUND PRESSURE WILL BE DISTRIBUTED INTO ADJACENT FOOTING BEFORE FAILURE OF THE GROUND AND BEFORE EXCESSIVE SETTLEMENT OCCUR.



SEE PAGES 5 TO 7 FOR APPROXIMATE ANALYSIS OF REDISTRIBUTION OF GROUND PRESSURES.

SKM

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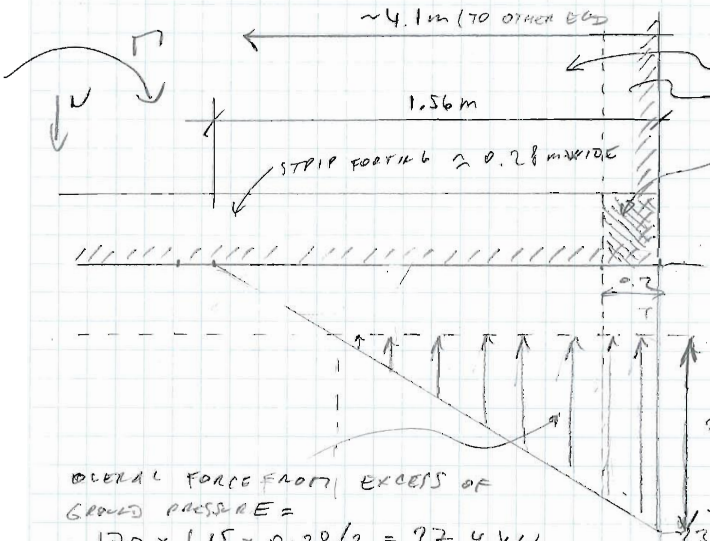
TRY IF REDISTRIBUTION INTO ADJACENT FOUNDATIONS POSSIBLE – EW2



ASSUME: GBP = 100 kPa →
→ ABP = 100×0.6 = 60 kPa

60 kPa (ALLOWABLE BEARING CAPACITY OF SOIL)

230−60 = 170 kPa (EXCESS)

230 kPa (PEAK GRO. PRESSURE)

OVERALL FORCE FROM EXCESS OF GROUND PRESSURE =
= 170 × 1.15 × 0.28/2 = 27.4 kN

(SEE IF THIS CAN BE TAKEN BY ADJOINING STRIP FOOTING, I.E AREA A_1 & A_2)

≈ 1.15m

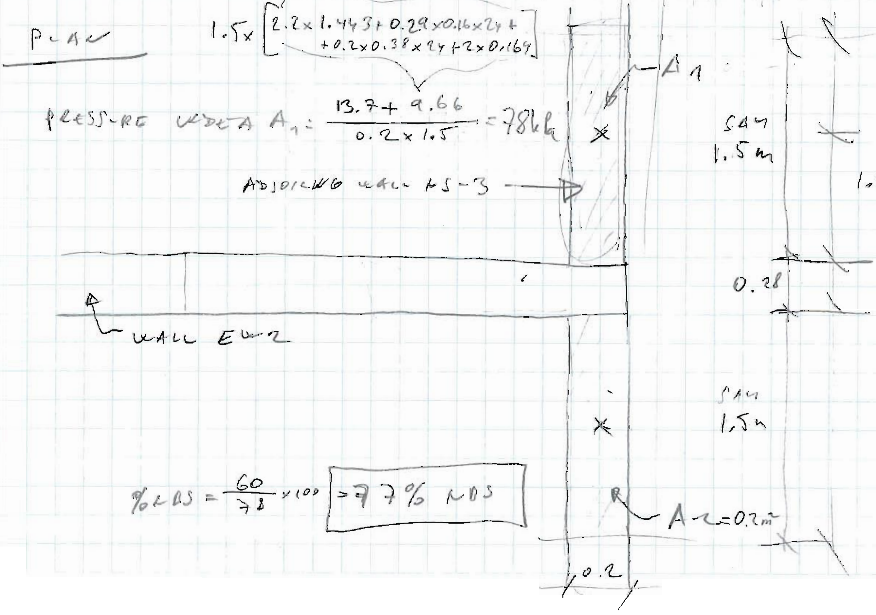
→ 27.4/2 = 13.7 kN EACH

6.44 [WEIGHT OF WALL, BOND BEAM, FOOTING & ROOF] PER METER

PLAN

$1.5 \times [2.2 \times 1.443 + 0.29 \times 0.16 \times 24 + 0.2 \times 0.38 \times 24 + 2 \times 0.169]$

PRESSURE UNDER A_1: $\frac{13.7 + 9.66}{0.2 \times 1.5} = 78$ kPa



% NBS = $\frac{60}{78} \times 100$ = 77% NBS

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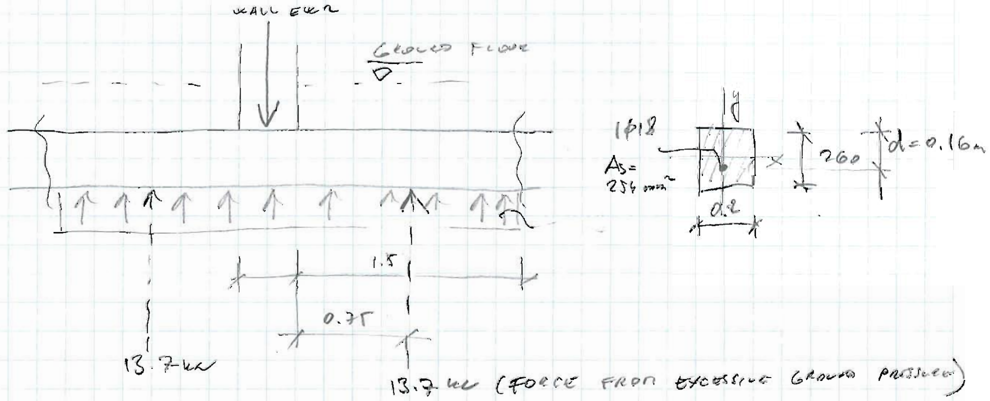
CHECK IF STRIP FOOTING CAN RESIST BENDING



BENDING $\underline{M^*} = 13.7 \times 0.75 = \underline{10.28 \text{ kNm}}$

BENDING CAPACITY

$$\phi M_{ux} = \phi \; A_s \times f_y \left(d - 0.59 \frac{f_y A_s}{f_c b} \right)$$

$$= 0.85 \times 254 \times 10^{-6} \times 250 \left(0.16 - 0.59 \; \frac{250 \times 254 \times 10^{-6}}{30 \times 0.26} \right)$$

$$= 8.37 \text{ kNm} \quad < \quad M^* = 10.78 \text{ kNm} \qquad \text{FAILS}$$

$$\% NBS = \frac{8.37}{10.28} \times 100 = \boxed{81\% \; NBS}$$

- THE CAPACITY OF FOUNDATIONS "UNDER WALL EW2" HAS BEEN EVALUATED TO BE IN ORDER OF 77% AFTER REDISTRIBUTION OF GROUND PRESSURES ASSUMING GROUND BEARING CAPACITY OF 100 kPa.

- THE ABOVE APPROXIMATION DOESN'T ACCOUNT FOR SERVICEABILITY LIMIT STATE, IT IS EXPECTED THAT THE BUILDING COULD BE SUBJECT TO SETTLEMENTS UNDER SEISMIC LOADS THAT MAY BE ACCOMPANIED WITH MINOR CRACKING IN WALLS AND FOUNDATIONS.

SKM

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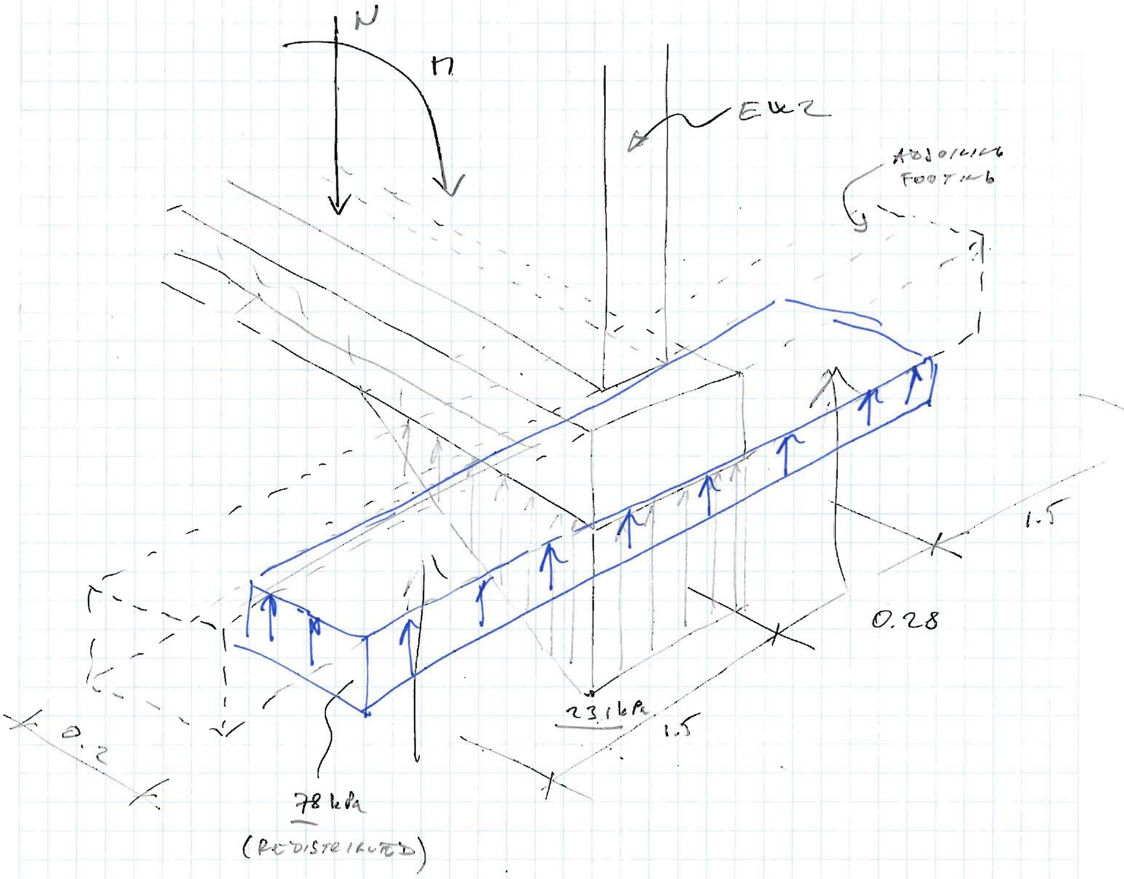

ISOMETRIC VIEW OF "T" JUNCTION AND REDISTRIBUTION OF EXCESSIVE GROUND PRESSURES UNDER WALL EW2