Christchurch City Council
PRO 3685 B001
Lyttelton Service Centre
33A London Street

QUANTITATIVE ASSESSMENT REPORT

FINAL

- Rev C
- 07 January 2014

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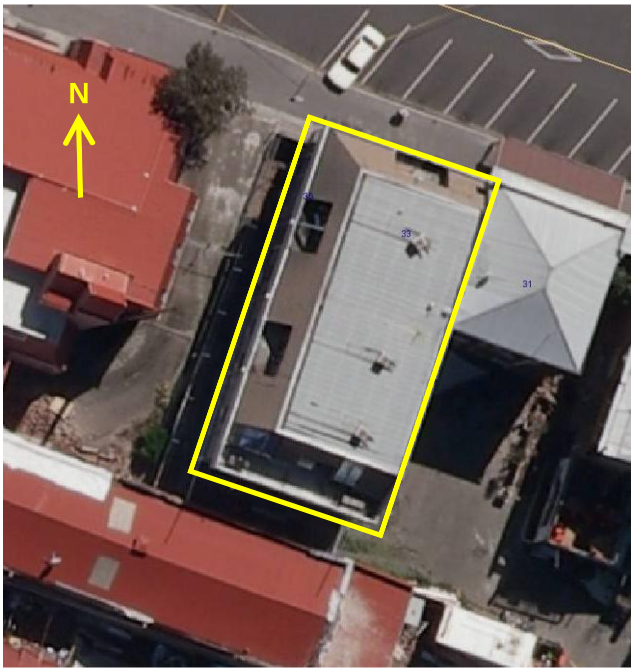
1. Executive Summary

1.1. Background

A quantitative Assessment was carried out on the building located at 33A London Street, Lyttelton. The Lyttelton Service Centre has two occupied storeys and an additional basement that currently acts as a car park. The building is primarily used as office space for a local government service centre. The basement and ground floor of the building are constructed from concrete gravity frames and concrete block shear walls. The ground and first floors consist of precast flooring units which are supported by the concrete frames and walls. The structure above level one is constructed from steel portal frames and has an external concrete block wall along the east side. The other external and internal walls on the upper floor are constructed from timber framing. The building is supported on concrete strip and pad foundations and has a concrete slab on grade at basement level.

Detailed descriptions outlining the buildings age and construction type is given in Section 5 of this report.



- **Figure 1 Aerial Photograph of 3 Thames Street**

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 30/03/2011, 24/06/2011, 29/06/2011 and 01/07/2011, available drawings and calculations.

1.2. Key Damage Observed

Key damage observed includes:-

- Damage to the concrete block wall in the south-east corner just below the level one floor slab, consisting of cracking along mortar joints and through block faces. Block faces had dislodged and fallen off in places exposing the concrete infill.
- Severe cracking to the concrete block work around one of the level 1 windows located on the south face of the building and the pilaster within the building. Cracks present along the mortar joints and through the block faces. Block faces have dislodged and fallen off in places. Propping has been installed to support the floor beam above previously supported on the pilaster.
- Eastern block wall has rotated at level one towards the adjacent building and in addition the brick wall of the adjacent building is leaning on the Service Centre Building. Horizontal crack at first floor level along the full length of the building from north to south. 100 to 200mm gap between the roof and the wall shows where the eastern wall has rotated away from the building. The wall rotation has caused the steel roof rafter fixings to have been pulled out of the wall. Propping has been installed to support the rafters. The propping continues down to the basement slab.
- Cracking to plasterboard wall linings where separation between the wall and the eastern block wall has occurred.
- Unreinforced masonry retaining wall present at basement level has collapsed. This has caused the ground at the entrance to subside and damaged the sandwich panel wall linings in the basement. The retaining wall damage level varies from complete collapse at the west end to bulging of the wall at the east end.
- Shop front glazing and doors badly damaged.
- Severe cracking to concrete blockwork around the stairs (at landing and to the wall under the first flight of stairs) and in the toilet. Diagonal cracking through block faces and cracking along mortar joints present. Spalling to block faces had also occurred.
- Glass panes to the partitions have been broken and in some areas partitions have been badly damaged.

1.3. Critical Structural Weaknesses

- No potential critical structural weaknesses were found

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.
- No geotechnical investigation has been undertaken. We have based this report on our knowledge of the site and the absence of liquefaction ejecta on the site.
- 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 22%NBS and post earthquake capacity in the order of 10%NBS. No critical structural weaknesses were found in the building.

The building has been assessed to have a seismic capacity in the order of 10% NBS and is therefore likely to be 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 remain yellow until the building is repaired.
b) We consider that barriers around the collapsed footpath and retaining wall are necessary (these are already in place).

2. Introduction

Sinclair Knight Merz were engaged by Christchurch City Council to carry out a Quantitative Assessment of the seismic performance of the building located at 33A London St, Lyttelton.

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].

A qualitative assessment dated October 2011 was prepared which identified that the seismic capacity of the building was likely to be less than 34% of the New Building Standard (NBS). A quantitative assessment was recommended to confirm the initial assessment findings and to determine a more accurate seismic rating of the building.

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[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 building is located at 33A London Street, Lyttelton. The building has two occupied storeys and an additional basement that currently acts as a car park. The building is primarily used as office space for a local government service centre. The basement and ground floor of the building are constructed from concrete gravity frames and concrete block shear walls. The ground and first floors consist of precast flooring units which are supported by the concrete frames and walls. The structure above level one is constructed from steel portal frames and has an external concrete block wall along the east side. The other external and internal walls on the upper floor are constructed from timber framing. The building is supported on concrete strip and pad foundations and has a concrete slab on grade at basement level. There is a sloping basement below the ground floor, which forms the bottom storey.

The building is classified as Importance Level 2. This level of importance is described as 'normal' with medium consequence for loss of human life, or considerable economic, social or environmental consequence of failure.

5.2. Gravity Load Resisting system

Our evaluation was based on the original drawings of the building dated April 1986 by Arch Goodwin Design (architects) and Tyndall & Hanham (Engineers). The structural drawings show most of the structural members, their materials and the rigor of the detailing. It is assumed that the building was designed shortly before and constructed shortly after the date on the construction drawings.

Gravity loads are transferred through the building through bending in the purlins, and rafters. The rafters are supported from the reinforced masonry wall to the east and directly from the floor to the west. The Unispan floor slabs span between precast concrete beams or filled reinforced concrete masonry walls. The precast concrete beams are supported by concrete columns or filled reinforced concrete masonry walls. The columns and walls are supported on pad and strip foundations.

5.3. Seismic Load Resisting system

It appears that lateral loads on the roof structure are intended to be resisted by steel frame action across the building (east-west direction) and along the building all loads are braced back to the full height eastern block wall. For the basement and ground floor levels all the lateral loads are taken by block shear walls. The concrete floor acts as a diaphragm transferring loads to block shear walls. The concrete beams and columns take the gravity loads of the structure. The walls are restrained from overturning by the strip foundations and their own self weight.

5.4. Building Damage

SKM undertook inspections on 30/03/2011, 24/06/2011, 29/06/2011 and 01/07/2011. The following areas of damage were observed during the time of inspection:

External Damage:

- Damage to the concrete block wall in the south-east corner just below the level one floor slab was present. This damage consisted of cracking along mortar joints and through block faces. Block faces had dislodged and fallen off in places exposing the concrete infill.
- Severe cracking to the concrete block work around one of the level 1 windows located on the south face of the building. Cracks present along the mortar joints and through the block faces. Block faces have dislodged and fallen off in places.
- Diagonal crack present to the top of one of the circular columns in the carpark – note it is considered unlikely that this damage is earthquake related, however repairs should be undertaken to check corrosion of the column reinforcing.
- Eastern block wall has rotated at level one towards the adjacent building, in addition the adjacent brick wall is leaning on the Service Centre Building. This has lead to the following damage:
  - The steel roof rafter fixings have been pulled out of the wall. Propping has been installed to support the rafters. The propping continues down to the basement slab.
  - Cracking to plasterboard wall linings where separation between the wall and the eastern block wall has occurred.
  - All wall linings and other fixtures have been removed at level 1 within 2m of the damaged wall to allow demolition and make safe work to the damaged wall.
- Unreinforced masonry retaining wall present at basement level has collapsed. This has caused the ground at the entrance to subside and damaged the sandwich panel wall linings in the basement. The retaining wall damage level varies from complete collapse at the west end to bulging of the wall at the east end.
- Shop front glazing and doors badly damaged.
- There was no liquefaction or land movement visible at the site other than the retaining wall collapse.

Internal Damage:

- Severe cracking to concrete blockwork around the stairs (at landing and to the wall under the first flight of stairs) and in the toilet. Diagonal cracking through block faces and cracking along mortar joints present. Spalling to block faces had also occurred.
- Cracking along mortar joints to various block walls.

- The jointing between the plasterboard wall linings and the concrete column has moved causing the sealant to fall out.
- Severe cracking to the ground floor concrete block pilaster. Cracks present at mortar joints and through concrete block faces. Block faces have been displaced and fallen off in places. Propping has been installed to support the floor beam above previously supported on the pilaster.
- Hairline cracks present in gib wall linings.
- Horizontal crack present in the eastern block wall at first floor level. The crack runs along the full length of the building from north to south. This crack is reflected outside. 100 to 200mm gap between the roof and the wall shows where the eastern wall has rotated away from the building.
- Block work has spalled where the roof rafter fixings have been pulled out of the wall as a result of the wall rotating away from the building.
- Mechanical services in the ceiling have been displaced.
- Glass panes to the partitions have been broken and in some areas partitions have been badly damaged.
- Glass panes broken to the ranch slider on level one.
- Joint between the sloping ceiling lining and wall lining has opened up.

All these areas of damage are earthquake related and should be repaired.

6. Design Criteria

6.1. Available Information

Following our inspections on the30/03/2011, 24/06/2011, 29/06/2011 and 01/07/2011, SKM carried out a seismic review on the structure. This review was undertaken using the available information which was as follows:

- Structural drawings of the building dated April 1986 by Arch Goodwin Design (architects) and Tyndall & Hanham (Engineers).

6.2. Survey

No survey has been undertaken at this point, however minor settlement was visible and hence survey may be warranted.

6.3. Foundation and Ground Conditions

The ground conditions on site are assumed to be subsoil class C as described in AS/NZS1170.5:2004. This is a conservative assumption based on our desktop assessment of ground conditions at the site. The gravity columns are supported on bearing pads with shear walls on strip foundations.

Reference to the New Zealand Geological Map of the Christchurch area (1:250,000; compiled by Forsyth et al., 2008) indicates the study area is comprised of recent Loess sediments amongst materials of the Lyttelton Volcanic Group. Reclaimed sand consisting of fill is displayed at the port, further down the slope from the site at London Street and is not expected to be encountered.

Loess sediments of the Banks Peninsula region are characterised as yellow-brown windblown silts. These are commonly greater than 3 metres in thickness and can be in multiple layers. Lyttelton Volcanic Group materials consist of basalt and trachytic lava flows with breccias and tuffs. These are anticipated to have extensive thicknesses.

From reference to the topographic map, the site is on flat ground with steep (20°) slopes above and below.

6.3.1. Previous Ground Investigation Information

There is limited previous ground investigation for the site.

2008 - Environment Canterbury Well database – Lyttelton Wharf No.6

- Three borelogs drilled for groundwater quality testing.
- Logs display fill of reclaimed land, silts and basalts.

- Logs confirm the Loess and Lyttelton Volcanic Group materials (basalts encountered at 4.5 metres below ground surface).

6.3.2. Subsoil Class

Anticipated ground conditions for the site are Loess sediments overlying basalts of the Lyttelton Volcanic Group. With reference to the Environment Canterbury wells, which are situated 300m away from the site, the depth of the Loess can be expected to be at least 3 metres.

6.3.3. Bearing Capacity Estimate

Without detailed geotechnical knowledge of the subsoil, bearing capacity estimates are calculated using estimated soil parameters for Loess sediments. Estimates for cohesion and friction angle are taken from back analysis of Loess slopes in Lyttelton.

Material	**Cohesion** (kPa)	**Φ Friction angle** (°)
Loess	5	30

- **Table 1 Estimated Loess soil parameters.**

Footing Width (mm)	**Ultimate bearing capacity (kPa)**	**Serviceable load (kPa)**
400 (Strip footing)	199	66
600 (Strip footing)	223	74
1300 (Square footing)	308	103
1700 (Square footing)	356	119

- **Table 2 Bearing capacity estimates for each footing type of the service centre.**

6.3.4. Geotechnical Investigation Recommendations

Site observations of earthquake damage in the Lyttelton area suggest that no liquefaction or lateral spreading has occurred on site. However it is likely that over 100mm of settlement has occurred to the building. It is recommended that geotechnical site investigation is undertaken in order to obtain reasonable soil parameter estimates for bearing capacity and to confirm the seismic subsoil class.

6.4. 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. We have reviewed the building and from our visual inspection the structure appears to be built in accordance with the drawings.
- 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
- The following ductility criteria used in the building:

- **Table 2: Assumed Building Ductility**

Ductility of Building in Current State	Ductility of Building in Strengthened State
2.00[3]	2.00

The ductility noted above has been used on the basis that the lateral load resisting elements of the building are reinforced and are detailed such that they should be able to achieve limited ductility but are unlikely to be able to achieve full ductility.

- The following material properties were used in the analyses:
- **Table 3: Material Properties**

Material	Nominal Strength	Structural Performance
Structural Steel	f_y = 300MPa	S_p = 0.9
Masonry (reinforced)	f_m = 12MPa	S_p = 1.0
Masonry (unreinforced)	f_m = 5.7MPa[4]	S_p = 1.0
Concrete	f_c' = 25MPa	S_p = 1.0

[3] Ductility for the unreinforced and unfilled concrete block walls was taken as 1.0

[4] Value obtained from "Assessment and Improvement of Unreinforced masonry Building for Earthquake Resistance" – Draft 02/2011

Material	Nominal Strength	Structural Performance
Timber - Unknown	f_b = 10MPa & f_c = 15MPa	S_p = 1.0

- The building was built according to the drawings and according to good practice at the time. We have reviewed the building and from our visual inspection the structure appears to be built in accordance with the drawings.

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.

6.5. 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[5].

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

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

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[6]. 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[7]. This timeframe is likely to be adjusted by CERA and **Error! Reference source not found.** below contains the likely new recommendations.

Table 4: 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.

[6] NZSEE 2006, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes*, p 2-2

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

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 is thought to have a plan irregularity caused by the layout of the walls. This irregularity has been accounted for in the recorded %NBS below.

7.2. Analysis Results

The loadings for the building have been calculated using the equivalent static method. Even though the building slightly exceeds the 10m height criteria, and may also slightly exceed the 0.4 second period criteria, it was considered regular under Section 6.1.3.1(c), which meant that equivalent static method was appropriate. The analysis assumes that the concrete beams and columns take gravity loads only and do not offer any lateral load resisting capacity.

The shear distribution to the walls has been based on the relative elastic stiffness of the walls, although a significant amount of moment redistribution was required in order to optimise the building capacity as part of our subsequent design calculations. While an equivalent static analysis was considered reasonable for initial concept design purposes, the subsequent need for significant seismic force redistribution served to increased the torsional irregularity of the building at ultimate limit state, so we recommend that seismic analysis be undertaking using the modal response spectrum method for any subsequent detailed design. Modal response spectrum design will allow increased accuracy in modelling the building and determining the forces, which will be necessary due to under-strength problems and torsional irregularity that were discovered during our quantitative analysis.

7.3. Concrete Block Masonry Shear Walls

Most of the block walls have under capacity due to torsional irregularity which increases the demand for these walls. The walls which appear to have sufficient capacity are the eastern full height wall and the wall across the building on grid line D. The remaining walls need to be strengthened to meet the current design standards. The walls also have inadequate self weight to resist seismic overturning and hence the footings require modification.

7.4. Foundations

The analysis indicates that sliding and bearing are the major issues with the foundations of the structure. The strip footings for the block walls do not appear to have enough bearing capacity when overturning in plane of the walls is considered. For strengthening of the foundations the length of the walls for basement and first floor level have been increased to more widely spread the overturning forces in the walls.

7.5. Top Storey & Roof

The analysis indicates that the roof level steel structure does not have sufficient roof bracing to carry the necessary diaphragm actions. Additional roof and wall bracing will be required to strengthen the roof structure.

7.6. Summary

The results of the analysis are reported in the following table as %NBS for those elements with capacity less than 100%NBS. The results below are calculated for the building in its damaged state and the results have been broken down into their seismic resisting elements. As the building has elements that are less than 34%NBS any item with a capacity less than 67%NBS will need to be strengthened so that the overall building capacity is greater than 67%NBS.

(%NBS = probable strength / new building standards)

- **Table 5: DEE Results**

STRUCTURAL ELEMENT	ISSUE OF CONCERN	COMMENTS
Masonry shear walls	Torsionally irregular and apparently under-strength	Current capacity 22%NBS. Widespread strengthening required in order to achieve 67%NBS.
Cantilever block wall on Grid 1 above 1st floor level.	Out of plane bending failure resulting in 5mm crack near 1st Floor level. Risk of pounding or collapse onto adjacent building.	Current capacity around 10%NBS. Replacement required, along with additional out-of-plane supports to achieve 67%NBS.
Shear wall strip footings	Narrow and lightly reinforced, leading to risk of overturning and bearing failure.	Current capacity 22%NBS. Further geotechnical investigation would be required prior to undertaking detailed design. Widened foundations appear to be required.
Unreinforced masonry basement wall collapse	Overturning failure	Current Capacity less than 20% NBS.
Severe cracking to concrete blockwork around the stairs	Supporting stairs and landing	Current capacity around 40% rebuild required.

STRUCTURAL ELEMENT	ISSUE OF CONCERN	COMMENTS
Roof rafters	Out of plane movement of wall has caused rafters to pull out from wall.	Current Capacity less than 20% NBS.
Severe cracking to the ground floor concrete block pilaster	Supporting suspended floor beams.	Current Capacity less than 20% NBS.

The above summary table shows that the building is an earthquake prone building (capacity less than 34% NBS. The limiting part of the structure is the concrete block east wall above first floor level.

Earthquake Prone Buildings are required to be strengthened to 67% of New Building Standard in accordance with the local authority policy.

7.7. Recommendations

The quantitative assessment carried out indicates that the building has a seismic capacity less than 34% of NBS and is therefore classed as being in the category of 'High Risk Buildings'. Strengthening of the building is required to bring it up to a minimum of 67% of NBS. We recommend that the following actions are taken:

- A survey is carried out to confirm whether relevelling is required
- Repair and strengthening options are considered to determine whether repair of the building is economically feasible
- The retaining wall to the north of the building should be repaired to prevent further deterioration of the wall, potential collapse of the roadway and the associated additional damage to the building that is occurring.

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 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.

8. Further Investigation

- A survey of the suspended floors is recommended to confirm the settlement that has occurred. It is likely that the survey will show that settlement has occurred and it is highly likely that the settlement was caused by the earthquakes because the settlement is clearly visible and unlikely to be tolerated in a new building.
- A full geotechnical investigation will be required prior to lodging a consent for the repairs and any design changes recommended in the geotechnical investigation will need to be incorporated in the detailed strengthening design
- A detailed strengthening design should be undertaken using modal analysis to confirm that the concept strengthening and the associated estimate is appropriate.
- A full strengthening and repair specification should be prepared accounting for the damage contained in the damage assessment report and strengthening as confirmed by the detailed design.
- Further investigation into pounding with the adjacent structure will be required, however it was not considered in this report due to the authors expectation that the neighbouring building will be demolished.

9. Conclusion

SKM carried out a quantitative assessment on the Lyttelton Service Centre building located at 33A London Street, Lyttelton. This assessment concluded that the building is classified as being a "High Risk building" and is likely Earthquake Prone.

Table 6: Quantitative assessment summary

Description	Grade	Risk	%NBS	Structural Performance
33A London St	E	High	10	Unacceptable. Improvement required.

Strengthening is required on the building to bring the seismic capacity up to at a minimum of 67% of NBS. Strengthening works required to the building is extensive and it may be more feasible to demolish and rebuild the building.

We make the following additional recommendations if the building is to be repaired:

- A full geotechnical investigation will be required prior to lodging a consent for the repairs and any design changes recommended in the geotechnical investigation will need to be incorporated in the detailed strengthening design
- A detailed strengthening design should be undertaken using modal analysis to confirm that the concept strengthening and the associated estimate is appropriate.
- A full strengthening and repair specification should be prepared accounting for the damage contained in the damage assessment report and strengthening as confirmed by the detailed design.
- Further investigation into pounding with the adjacent structure will be required, however it was not considered in this report due to the authors expectation that the neighbouring building will be demolished.

Brief discussion on the appropriateness of continued occupancy or the possible measures needed to be able to re-occupy

It is recommended that:

a) The current placard status of the building remain yellow until the building is repaired.

b) We consider that barriers around the collapsed retaining wall are necessary (these were in place at the time this report was written).

10. 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 predating 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.

11. Appendix 1 – CERA Report Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Lyttelton Service Centre	Reviewer:	N Calvert
	Unit / No: / Street	CPEng No:	242062
Building Address:	33 London Street, Lyttelton	Company:	Sinclair Knight Merz
Legal Description:		Company project number:	ZB01276.201
		Company phone number:	03 940 4900
	Degrees / Min / Sec		
GPS south:		Date of submission:	7-Jan
GPS east:		Inspection Date:	1/07/2011
		Revision:	C
Building Unique Identifier (CCC):	PR) 3685 B001	Is there a full report with this summary?	yes

Site

Site slope:	slope >1 in 5	Max retaining height (m):	4
Soil type:	gravel	Soil Profile (if available):	Bedrock overlain by around 3m of loess - detailed geotechnical required to confirm
Site Class (to NZS1170.5):	C		
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):	50.00

Building

No. of storeys above ground:	2 (single storey = 1)	Ground floor elevation (Absolute) (m):	50.00
Ground floor split?	no	Ground floor elevation above ground (m):	0.00
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):	11.6
Floor footprint area (approx):	240		
Age of Building (years):	25	Date of design:	1976-1992
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	parking	Brief strengthening description:	
Use (upper floors):	public		
Use notes (if required):			
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	frame system		
Roof:	steel framed	rafter type, purlin type and cladding	UB purlin, timber purlins and slate cladding
Floors:	precast concrete with topping	unit type and depth (mm), topping	75 Unispan and 75mm topping
Beams:	precast concrete	overall depth (mm)	400
Columns:	cast-insitu concrete	typical dimensions (mm x mm)	250 diameter
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):	31
Ductility assumed, μ:	1.25		wall thickness (m):	0.19
Period along:	0.40	0.08 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):	13.5
Ductility assumed, μ:	1.25		wall thickness (m):	0.19
Period across:	0.40	0.26 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:	timber	describe supports	concrete landings and floors
Wall cladding:	other light	describe	fibre cement boards
Roof Cladding:	Shingles or shakes	describe	
Glazing:	aluminium frames		
Ceilings:	light tiles		
Services(list):			

Available documentation

Architectural	Partial	original designer name/date	Arch Goodwin Design Ltd April 1986
Structural	full	original designer name/date	Tyndall and Hanham April 1986
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:	yellow		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	Engineering judgement of level of damage
	Describe (summary):	Damage insignificant in building capacity		
Across	Damage ratio:	75%		
	Describe (summary):	Damage insignificant in building capacity		
Diaphragms	Damage?:	no	Describe:	None
CSWs:	Damage?:	no	Describe:	Plan irregularity has been accounted for in assessed %NBS
Pounding:	Damage?:	no	Describe:	Adjacent building quite close but stiffness of building means that pounding unlikely
Non-structural:	Damage?:	yes	Describe:	Internal partitions damaged, curtain walling damaged, doors not shutting, potential relevelling required

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

Recommendations

Level of repair/strengthening required:	significant structural and strengthening	Describe:	Partial demolition and replacement as well as strengthening required.
Building Consent required:	yes	Describe:	Significant repair required
Interim occupancy recommendations:	do not occupy	Describe:	Yellow placard, building has been stabilised but there are no walls/handrails around stair

Along	Assessed %NBS before:	20%	%NBS from IEP below	If IEP not used, please detail assessment methodology: SKM quantitative Calculations
	Assessed %NBS after:	20%		
Across	Assessed %NBS before:	20%	%NBS from IEP below	
	Assessed %NBS after:	5%		