Hagley Park South Implement Shed

PRK 1507 BLDG 020 EQ2

Detailed Engineering Evaluation

Quantitative Report

Christchurch City Council

Christchurch City Council

Hagley Park South Implement Shed

Detailed Engineering Evaluation Quantitative Report

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Hagley Park South Implement Shed
PRK 1507 BLDG 020 EQ2

Detailed Engineering Evaluation
Quantitative Report - SUMMARY
Final

Hagley Park South, Hagley Ave Christchurch

Background

This is a summary of the quantitative report for the Hagley Park South Implement Shed, and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on 12 March 2012, available drawings and calculations.

Key Damage Observed

No seismic damage was identified.

Critical Structural Weaknesses

The front masonry wall has been identified as a critical structural weakness. At a capacity of only 9% NBS, and no roof level diaphragm, there is no effective alternative load path for lateral loads along the front of the building.

Indicative Building Strength

Based on the site survey, and from undertaking a quantitative assessment, the building's capacity has been assessed to be 14% NBS as limited by the front unreinforced masonry wall. The building's post-earthquake capacity is particularly governed by the northernmost wall running in the east-west direction. The walls in the north-south direction are comparatively better with assessed capacities greater than 33% NBS.

The building has been assessed to have a seismic capacity of 14% NBS and is therefore classed as an Earthquake Prone building in accordance with the Building Act 2004.

Recommendations

It is recommended that:

(a) The building should not be used until strengthening works are carried out.

(b) The building should be cordoned off to a distance of 1 ½ times the height.

(c) Strengthening options be developed for increasing the seismic capacity of the building to at least 67% NBS.

Contents

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council (CCC) to undertake a detailed seismic assessment of the South Hagley Implement Shed located in Hagley Ave, Hagley Park South, following the M6.3 Christchurch earthquake on 22 February 2011.

The purpose of the assessment is to determine if the building is classed as being earthquake prone under the NZSEE classification system.

The seismic assessment and reporting have been undertaken based on the quantitative procedures detailed in the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011.

2 Compliance

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

2.1 Canterbury Earthquake Recovery Authority (CERA)

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

Section 38 – Works

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

Section 51 – Requiring Structural Survey

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

We understand that CERA 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). CERA have adopted the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011. This document sets out a methodology for both initial qualitative and detailed quantitative assessments.

It is anticipated that a number of factors, including the following, will determine the extent of evaluation and strengthening level required:

1. The importance level and occupancy of the building.

2. The placard status and amount of damage.

3. The age and structural type of the building.

4. Consideration of any critical structural weaknesses.

Any building with a capacity of less than 34% of new building standard (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67% as required by the CCC Earthquake Prone Building Policy.

2.2 Building Act

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

Section 112 - Alterations

This section requires that an existing building complies with the relevant sections of the Building Code to at least the extent that it did prior to the alteration.

This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

Section 115 – Change of Use

This section requires that the territorial authority (in this case Christchurch City Council (CCC)) is satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'.

This is typically interpreted by CCC as being 67% of the strength of an equivalent new building. This is also the minimum level recommended by the New Zealand Society for Earthquake Engineering (NZSEE).

Section 121 – Dangerous Buildings

This section was extended by the Canterbury Earthquake (Building Act) Order 2010, and defines a building as dangerous if:

1. 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
2. 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
3. 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
4. There is a risk that other property could collapse or otherwise cause injury or death; or
5. A territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

Section 122 – Earthquake Prone Buildings

This section defines a building as earthquake prone (EPB) 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 loads 33% of those used to design an equivalent new building.

Section 124 – Powers of Territorial Authorities

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

Section 131 – Earthquake Prone Building Policy

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

2.3 Christchurch City Council Policy

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

The 2010 amendment includes the following:

1. A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
2. A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;
3. A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
4. 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.

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

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

2.4 Building Code

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

On 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

- 36% increase in the basic seismic design load for Christchurch (Z factor increased from 0.22 to 0.3);
- Increased serviceability requirements.

2.5 Institution of Professional Engineers New Zealand (IPENZ) Code of Ethics

One of the core ethical values of professional engineers in New Zealand is the protection of life and safeguarding of people. The IPENZ Code of Ethics requires that:

Members shall recognise the need to protect life and to safeguard people, and in their engineering activities shall act to address this need.

1.1 Giving Priority to the safety and well-being of the community and having regard to this principle in assessing obligations to clients, employers and colleagues.

1.2 Ensuring that responsible steps are taken to minimise the risk of loss of life, injury or suffering which may result from your engineering activities, either directly or indirectly.

All recommendations on building occupancy and access must be made with these fundamental obligations in mind.

3 Earthquake Resistance Standards

For this assessment, the building's earthquake resistance is compared with the current New Zealand Building Code requirements for a new building constructed on the site. This is expressed as a percentage of new building standard (%NBS). The loadings are in accordance with the current earthquake loading standard NZS1170.5 [1].

A generally accepted classification of earthquake risk for existing buildings in terms of %NBS that has been proposed by the NZSEE 2006 [2] is presented in Figure 1 below.

Description	Grade	Risk	%NBS	Existing Building Structural Performance	Improvement of Structural Performance: Legal Requirement	Improvement of Structural Performance: 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	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 required under Act)	Unacceptable	Unacceptable

Figure 1: NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE Guidelines

Table 1 below compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% risk of exceedance in 50 years (i.e. 0.2% in the next year). It is noted that the current seismic risk in Christchurch results in a 6% risk of exceedance in the next year.

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

3.1 Minimum and Recommended Standards

Based on governing policy and recent observations, Opus makes the following general recommendations:

3.1.1 Occupancy

– The Canterbury Earthquake Order[1] in Council 16 September 2010, modified the meaning of “dangerous building” to include buildings that were identified as being EPB’s. As a result of this, we would expect such a building would be issued with a Section 124 notice, by the Territorial Authority, or CERA acting on their behalf, once

[1] This Order only applies to buildings within the Christchurch City, Selwyn District and Waimakariri District Councils authority

they are made aware of our assessment. Based on information received from CERA to date, this notice is likely to prohibit occupancy of the building (or parts thereof) until its seismic capacity is improved to the point that it is no longer considered an EPB.

3.1.2 Cordoning

– Where there is an overhead falling hazard, or potential collapse hazard of the building, the areas of concern should be cordoned off in accordance with current CERA/Christchurch City Council guidelines.

3.1.3 Strengthening

– Industry guidelines (NZSEE 2006 [2]) strongly recommend that every effort be made to achieve improvement to at least 67%NBS. A strengthening solution to anything less than 67%NBS would not provide an adequate reduction to the level of risk.

– It should be noted that full compliance with the current building code requires building strength of 100%NBS.

3.1.4 Our Ethical Obligation

– In accordance with the IPENZ code of ethics, we have a duty of care to the public. This obligation requires us to identify and inform CERA of potentially dangerous buildings; this would include earthquake prone buildings.

4 Building Description

4.1 General

Figure 2: Location of Hagley Park South Implement Shed

The Hagley Park South Implement Shed is a single storey unreinforced concrete masonry shed with a timber framed lightweight corrugated iron roof. The building sits on a concrete slab-on-ground, located at the residence off Riccarton Ave.

The 3.5m x 6m store comprises the western end of the building. The remaining 11.5m x 6m eastern end comprises garages and general storage. The Implement Shed building has one side window and a front timber door opening. The garages have three front roller shutter doors and a set of rear timber doors.

The timber rafters are supported on the masonry walls and one central beam. The timber rafters in the garage are supported by the masonry walls, a steel truss and a timber beam. The steel truss is supported by two concrete columns.

There is timber stud wall framing at the front of the building in short wall lengths framing the roller door openings.

4.2 Gravity Load Resisting System

The roof framing is supported on the concrete block walls and concrete columns. The concrete block walls sit on a slab-on-ground with an unknown depth of concrete footings.

4.3 Seismic Load Resisting System

Seismic loads in both orthogonal directions are resisted by the unreinforced masonry walls as the primary structural elements of the building. The building has no ceiling, and as such in a seismic event there is limited capacity for load re-distribution between the walls and columns.

5 Survey

No copies of the design calculations or structural drawings have been obtained for this building.

The cover-meter survey found that there was no reinforcement in the masonry, and it appears to be ungrouted.

The building currently does not have an earthquake assessment placard.

6 Damage Assessment

The Implement Shed front wall has a crack, approximately 5mm thick, at the end of the concrete lintel which is likely a result of the recent earthquake events. All other damage is minor and appears to be age related.

7 General Observations

Overall the building has performed better than expected under seismic loads for an unreinforced masonry structure. The building has sustained minor damage and continues to be operational.

Due to the non-intrusive nature of the original survey, many connection details could not be ascertained.

8 Detailed Seismic Assessment

8.1 Critical Structural Weaknesses

As outlined in the Critical Structural Weakness and Collapse Hazards draft briefing document, issued by the Structural Engineering Society (SESOC) on 7 May 2011, the term 'Critical Structural Weakness' (CSW) refers to a component of a building that could contribute to increased levels of damage or cause premature collapse of the building.

The front unreinforced concrete masonry wall was identified as a critical structural weakness in this building. At a capacity of only 9%NBS, and no roof level diaphragm, there is no effective alternative load path for lateral loads along the front of the building.

8.2 Seismic Coefficient Parameters

The seismic design parameters based on current design requirements from NZS1170.5:2004 and the NZBC clause B1 for this building are:

- Site soil class D, clause 3.1.3 NZS 1170.5:2004;
- Site hazard factor, Z=0.3, B1/VM1 clause 2.2.14B;
- Return period factor R_u = 1.0 from Table 3.5, NZS 1170.5:2004, for an Importance Level 2 structure with a 50 year design life;
- μ_{max} = 1.0

8.3 Detailed Seismic Assessment Results

A summary of the structural performance of the building is shown in the following table. Note that the values given represent the worst performing elements in the building, as these effectively define the building's capacity. Other elements within the building may have significantly greater capacity when compared with the governing element.

Table 2: Summary of Seismic Performance

Structural Element/System	Failure mode and description of limiting criteria	Critical Structural Weakness and Collapse Hazard	% NBS based on calculated capacity
Southernmost wall in the east-west direction i.e. along the building	Seismic shear capacity of block wall	No	42%
Northernmost wall in the east-west direction i.e. along the building	Seismic shear capacity of block wall	Yes	14%

Structural Element/System	Failure mode and description of limiting criteria	Critical Structural Weakness and Collapse Hazard	% NBS based on calculated capacity
Easternmost wall in the north-south direction i.e. across the building	Seismic shear capacity of block wall	No	54%
Westernmost wall in the north-south direction i.e. across the building	Seismic shear capacity of block wall	No	48%
Central wall in the north-south direction i.e. across the building	Seismic shear capacity of block wall	No	34%

8.4 Discussion of Results

Seismic loads in both principal directions are resisted by the unreinforced masonry walls. The front wall with a capacity of only 14% NBS is a critical structural weakness. Failure of this front wall would cause collapse of the roof and walls along the front of the building. There is no alternative load path for seismic lateral forces due to the lack of a roof level diaphragm or roof bracing.

The slab-on-ground appeared to be in reasonable condition.

As the building has an assessed capacity less than 34% NBS it is defined as Earthquake Prone in accordance with the Building Act 2004. We recommend that the CCC review any on-going usage of this building until such time that any required strengthening works have been undertaken.

8.5 Limitations and Assumptions in Results

Our analysis and assessment is based on an assessment of the building in its undamaged state. Therefore the current capacity of the building will be lower than that stated.

The results have been reported as a %NBS and the stated value is that obtained from our analysis and assessment. Despite the use of best national and international practice in this analysis and assessment, this value contains uncertainty due to the many assumptions and simplifications which are made during the assessment. These include:

- Simplifications made in the analysis, including boundary conditions such as foundation fixity;
- Assessments of material strengths based on limited drawings, specifications and site inspections;
- The normal variation in material properties which change from batch to batch;

- Approximations made in the assessment of the capacity of each element, especially when considering the post-yield behaviour.

9 Summary of Geotechnical Assessment

The full geotechnical report is attached as Appendix B.

9.1 *Expected Ground Conditions*

A review of the Environmental Canterbury (ECan) wells database showed six well logs in close proximity to the site, within 50m to 430m (refer to Site Location Plan in Appendix A). Material logs available from the borehole wells have been used to infer the ground conditions at the site as shown in Table 1 below.

Table 2: Inferred Ground Conditions

Stratigraphy	Thickness (m)	Depth Encountered From (m) bgl
Silt and silty Sand	4.3-6.5m	Surface
sandy Gravel	5.0-7.0m	4.3-6.0m
Interbedded Sand and Silt	11.0 -12.0m	9.3-13.0m
Gravels (Riccarton Formation)	-	20.3-26.0m

A groundwater depth of approximately 2m below ground level has been extracted from groundwater depth contour maps (Environment Canterbury (2003) and Elder et al. (1991)).

9.2 *Aerial Photograph Records*

Aerial photographs taken after 4 September 2010 and 24 February 2011 do not show any significant surficial evidence of liquefaction, such as ejected sands and silts, in the vicinity of the site however evidence of liquefaction was identified approximately 330m east of the site.

9.3 *Site Walkover Inspection*

An inspection was carried out on the perimeter of the shed including the interior and exterior of the building and adjacent land areas by an Opus Geotechnical Engineer on 1 May 2011.

The following observations were made: (refer to Appendix B- Site Photos)

- Behind the shed is a footpath which has suffered cracks approximately 5mm wide in various places. (Refer Photograph 3 of Appendix B)
- The ground behind the shed has heaved and lifted the asphalt surfacing vertically by about 30mm with 20mm wide cracks. (Refer Photograph 4 of Appendix B)
- The garage floor adjacent to the shed showed signs of transverse and longitudinal cracks 3mm wide. (Refer Photograph 5 of Appendix B)
- No evidence of surface liquefaction was observed in the vicinity of the site.

9.4 *Liquefaction Hazard*

The 2004 ECan Solid Facts on Christchurch Liquefaction Study indicates that the Implement Shed at South Hagley Park is located within an area of moderate liquefaction ground damage potential based on a low ground water table. According to this study, ground damage from liquefaction is expected to be moderate and is likely to be affected by 100-300mm of ground subsidence.

The technical report “Foundations on Deep Alluvial Soils”, prepared for the Canterbury Earthquakes Royal Commission infers that the shed is situated approximately 900m south of a large area which suffered extensive liquefaction during the 22 February 2012 earthquake.

Tonkin and Taylor Aerial Reconnaissance indicated no observable liquefaction noted on site after the 4 Sept 2010 and 22 Feb 2011 events. However, latest mapping update of the area following the 13 June 2011 aftershock indicated liquefaction has occurred at the site.

9.5 *Discussion and Conclusions*

The land surrounding the Implement Shed has suffered minor damage due to the sequence of earthquake events following the 4 September 2010 Canterbury Earthquake

The site is identified to be within the area of moderate ground damage and liquefaction potential, there was minor evidence of ground deformation observed during the site walkover inspection.

The existing concrete foundations of the Implement Shed are considered appropriate. However, CCC will have to accept that in future seismic events there is a risk of minor differential settlement.

GNS Science indicates an elevated risk of seismic activity is expected in the Canterbury region as a result of the earthquake sequence following the 4 September 2010 earthquake. Recent advice (Geonet) indicates there is a 14% probability of another Magnitude 6 or greater earthquake occurring in the next 12 months in the Canterbury region. Liquefaction damage similar to what has occurred is expected in such an event, depending on the location of the epicentre. It is expected that the probability of occurrence is likely to decrease with time, following periods of reduced seismic activity.

9.6 *Geotechnical Recommendations*

The existing concrete slab foundations appear to have performed reasonably well and are considered suitable.

No further geotechnical investigations are recommended.

10 Remedial Options

The building has a calculated capacity of less than 34% NBS as limited by the shear capacity of the unreinforced concrete masonry walls.

Strengthening of the wall and roof bracing system of the building is required to achieve a capacity greater than 67%NBS. If continued reliance is to be made on the unreinforced masonry in the strengthening, then testing would need to be undertaken to accurately determine the masonry properties.

11 Conclusions

(a) The building has a seismic capacity of 14% NBS and is therefore classed as an Earthquake Prone building in accordance with the Building Act 2004.

(b) The front masonry wall is a critical structural weakness with failure likely to cause roof and wall collapse along the front of the building.

(c) Strengthening work is required to increase the overall building capacity to at least 67% NBS.

(d) The existing foundations have performed satisfactorily, and no further geotechnical testing is required.

12 Recommendations

(a) The building should not to be used until strengthening works are carried out.

(b) The building should be cordoned off to a distance of 1½ times the height.

(c) Strengthening options be developed for increasing the seismic capacity of the building to at least 67% NBS.

13 Limitations

(a) This report is based on an inspection of the structure with a focus on the damage sustained from the 22 February 2011 Canterbury Earthquake and aftershocks only. Some non-structural damage is mentioned but this is not intended to be a comprehensive list of non-structural items.

(b) Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practicing in this field at the time.

(c) This report is prepared for the CCC to assist with assessing remedial works required for council buildings and facilities. It is not intended for any other party or purpose.

14 References

[1] NZS 1170.5: 2004, *Structural design actions, Part 5 Earthquake actions,* Standards New Zealand.

[2] NZSEE: 2006, *Assessment and improvement of the structural performance of buildings in earthquakes,* New Zealand Society for Earthquake Engineering.

[3] Engineering Advisory Group, *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure*, Draft Prepared by the Engineering Advisory Group, Revision 5, 19 July 2011.

[4] Engineering Advisory Group, *Guidance on Detailed Engineering Evaluation of Non-residential buildings, Part 3 Technical Guidance*, Draft Prepared by the Engineering Advisory Group, 13 December 2011.

[5] SESOC, *Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes*, Structural Engineering Society of New Zealand, 21 December 2011.

Appendix A – Photographs

Photo 1: North elevation of the Implement Shed

Photo 2: North elevation of the Implement Shed

Photo 3: North-west corner of the Implement Shed

Photo 4: Crack at edge of concrete lintel

Photo 5: Existing hole in block wall – ungrouted

Photo 6: Existing hole in block wall – ungrouted (close-up)

Photo 7: Interior of Implement Shed

Photo 8: Interior concrete column beside roller doors

Photo 9: Roof supports of Implement Shed

Photo 10: Roof framing of Implement Shed

Appendix B – Geotechnical Report

Appendix C – DEE Spreadsheet

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name: Hagley Park South Implement Shed
Unit No: Street
Building Address: Hagley Ave
Legal Description:
Degrees Min Sec
GPS south: 43 32 4.20
GPS east: 172 37 12.80
Building Unique Identifier (CCC):

Reviewer: Dave Dekker
CPEng No: 1003026
Company: Opus International Consultants Ltd
Company project number: 6-QUCC1.05
Company phone number: 03 363 5400
Date of submission: 11/09/2012
Inspection Date: 20/03/2012
Revision: Final
Is there a full report with this summary? yes

Site

Site slope:
Soil type:
Site Class (to NZS1170.5):
Proximity to waterway (m, if <100m):
Proximity to clifftop (m, if < 100m):
Proximity to cliff base (m,if <100m):

Max retaining height (m):
Soil Profile (if available):
If Ground improvement on site, describe:
Approx site elevation (m):

Building

No. of storeys above ground: 1 single storey = 1
Ground floor split?
Storeys below ground 0
Foundation type: strip footings
Building height (m): 2.50
Floor footprint area (approx):
Age of Building (years): 40

Ground floor elevation (Absolute) (m):
Ground floor elevation above ground (m):
if Foundation type is other, describe:
height from ground to level of uppermost seismic mass (for IEP only) (m):
Date of design:

Strengthening present? no
If so, when (year)?
And what load level (%g)?
Brief strengthening description:

Use (ground floor): other (specify)
Use (upper floors):
Use notes (if required): storage
Importance level (to NZS1170.5): IL2

Gravity Structure

Gravity System: load bearing walls
Roof: timber framed — rafter type, purlin type and cladding: timber and sheet metal cladding
Floors:
Beams:
Columns: cast-insitu concrete — typical dimensions (mm x mm)
Walls:

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along: concrete shear wall — enter wall data in "IEP period calcs" worksheet for period calculation
Ductility assumed, μ: 1.00
Period along: 0.20 ##### enter height above at H31 — estimate or calculation? estimated
Total deflection (ULS) (mm): — estimate or calculation?
maximum interstorey deflection (ULS) (mm): — estimate or calculation?

Lateral system across: concrete shear wall — enter wall data in "IEP period calcs" worksheet for period calculation
Ductility assumed, μ: 1.00
Period across: 0.20 ##### enter height above at H31 — estimate or calculation? estimated
Total deflection (ULS) (mm): — estimate or calculation?
maximum interstorey deflection (ULS) (mm): — estimate or calculation?

Separations:

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

Non-structural elements

Stairs:
Wall cladding:
Roof Cladding:
Glazing:
Ceilings:
Services(list):

Available documentation

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

Damage

Site: (refer DEE Table 4-2)
Site performance: — Describe damage:
Settlement: — notes (if applicable):
Differential settlement: — notes (if applicable):
Liquefaction: — notes (if applicable):
Lateral Spread: — notes (if applicable):
Differential lateral spread: — notes (if applicable):
Ground cracks: — notes (if applicable):
Damage to area: — notes (if applicable):

Building:
Current Placard Status:

Along — Damage ratio: 0% — Describe how damage ratio arrived at:
Describe (summary): minor crack

Across — Damage ratio: 0%
Describe (summary):

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

Diaphragms — Damage?: — Describe:
CSWs: — Damage?: — Describe:
Pounding: — Damage?: — Describe:
Non-structural: — Damage?: — Describe:

Recommendations

Level of repair/strengthening required: significant structural and strengthening — Describe: Replace URM
Building Consent required: yes — Describe:
Interim occupancy recommendations: do not occupy — Describe:

Along — Assessed %NBS before e'quakes: 14% ##### %NBS from IEP below — If IEP not used, please detail assessment methodology: Quantitative
Assessed %NBS after e'quakes: 14%

Across — Assessed %NBS before e'quakes: 34% ##### %NBS from IEP below
Assessed %NBS after e'quakes: 34%

IEP — **Use of this method is not mandatory - more detailed analysis may give a different answer, which would take precedence. Do not fill in fields if not using IEP.**

Period of design of building (from above): 0 — h_n from above: m
Seismic Zone, if designed between 1965 and 1992: — not required for this age of building
not required for this age of building

	along	across
Period (from above):	0.2	0.2
(%NBS)nom from Fig 3.3:		

Note:1 for specifically design public buildings, to the code of the day: pre-1965 = 1.25; 1965-1976, Zone A =1.33; 1965-1976, Zone B = 1.2; all else 1.0 — 1.00
Note 2: for RC buildings designed between 1976-1984, use 1.2 — 1.0
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0) — 1.0

	along	across
Final $(\%NBS)_{nom}$:	**0%**	**0%**

2.2 Near Fault Scaling Factor

Near Fault scaling factor, from NZS1170.5, cl 3.1.6: 1.00

	along	across
Near Fault scaling factor (1/N(T,D), **Factor A**:	1	1

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:
Z_{1992}, from NZS4203:1992
Hazard scaling factor, **Factor B:** #DIV/0!

2.4 Return Period Scaling Factor

Building Importance level (from above): 2
Return Period Scaling factor from Table 3.1, **Factor C**:

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)	1.00	1.00
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	0.00	0.00

2.6 Structural Performance Scaling Factor:

	along	across
Sp:	1.000	1.000
Structural Performance Scaling Factor **Factor E**:	1	1

2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom}$ x A x B x C x D x E — **$\%NBS_b$:** **#DIV/0!** **#DIV/0!**

Global Critical Structural Weaknesses: (refer to NZSEE IEP Table 3.4)

3.1. Plan Irregularity, factor A: 1

3.2. Vertical irregularity, Factor B: 1

3.3. Short columns, Factor C: 1

3.4. Pounding potential
Pounding effect D1, from Table to right 1.0
Height Difference effect D2, from Table to right 1.0
Therefore, Factor D: 1

Table for selection of D1	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Alignment of floors within 20% of H	**0.7**	**0.8**	**1**
Alignment of floors not within 20% of H	**0.4**	**0.7**	**0.8**

Table for Selection of D2	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Height difference > 4 storeys	0.4	0.7	1
Height difference 2 to 4 storeys	0.7	0.9	1
Height difference < 2 storeys	1	1	1

3.5. Site Characteristics 1

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum		
Rationale for choice of F factor, if not 1		

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)
List any: — Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

3.7. Overall Performance Achievement ratio (PAR) — **0.00** **0.00**

4.3 PAR x (%NBS)b: — PAR x Baselline %NBS: **#DIV/0!** **#DIV/0!**

4.4 Percentage New Building Standard (%NBS), (before) — **#DIV/0!**

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