Sockburn Depot – North East End Store

BU 1531-008 EQ2

Detailed Engineering Evaluation

Quantitative Assessment Report

CHRISTCHURCH CITY COUNCIL

Christchurch City Council

Sockburn Depot – North East End Store Detailed Engineering Evaluation Quantitative Assessment Report

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Sockburn Depot – North East End Store
BU 1531-008 EQ2

Detailed Engineering Evaluation
Quantitative Report – SUMMARY
Final

149 Main South Road, Christchurch,

Background
This is a summary of the quantitative report for the building structure at the north east end of the Sockburn Depot, 149 Main South Road, and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, and visual inspections on 8 February 2012 and 13 July 2012.

Key Damage Observed
Key damage observed includes:

- Significant damage to the eastern timber stud wall. This damage may be historic;
- Deterioration of the connection detail joining the roof bracing to timber purlin at the western end;
- Significant elongation of the steel diagonal roof bracing;
- Differential lateral movement of the steel portal frames.

Critical Structural Weaknesses
The following Critical Structural Weakness has been identified:

a) Due to damage to the diagonal bracing elements in the eastern wall, the north-south lateral loads must be resisted by the portal frame columns bending out-of-plane. Due to the small column size this results in excessive longitudinal displacements.

Indicative Building Strength (from quantitative assessment)
Based on the information available, and from undertaking a quantitative assessment, the building's original capacity has been assessed to be in the order of 25% NBS and post-earthquake capacity in the order of 25% NBS. The building is therefore classed as an earthquake prone building.

Recommendations
It is recommended that:

a) A strengthening works scheme is developed to increase the seismic capacity of the building to at least 67% NBS at IL2; this will need to consider compliance with accessibility and fire requirements.

Contents

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council to undertake a detailed seismic assessment of the North East End Store at the Sockburn Depot, located at 149 Main South Road, Christchurch 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 in accordance with the Building Act 2004.

The seismic assessment and reporting have been undertaken based on the qualitative and 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	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 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).

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[i] 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 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/Territorial Authority 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.

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

4 Background Information

4.1 Building Description

The Store located in the north east end of the Sockburn Depot is a single storey portal frame structure. The Sockburn Depot site is located at 149 Main South Road, Sockburn. The original date of construction of the Store is unknown, but is expected to be prior to the 1970s.

The building and wider Sockburn Depot site backs onto Blenheim Road. For the purposes of this report we will refer to the longitudinal direction as north-south and the transverse direction as east-west (parallel to Blenheim Road).

4.2 Gravity Load Resisting System

The building has five steel portal frames spanning the transverse (east-west) direction.

The lightweight roof cladding is supported by 150x50mm timber purlins spanning between the tapered steel portal frames.

Ground floor dead and live loads are carried directly by the compacted gravel floor.

The foundations appear to consist of an unknown length of steel parallel flange channel section driven into the ground, with the portal frame columns welded to these sections.

The building has roof and wall cladding of corrugated iron and timber framing.

4.3 Lateral Load Resisting System

The lateral load resisting system in the transverse (east-west) direction consists of steel portal frames acting in-plane. There are five steel portal frames, with diagonal roof bracing elements in both the north and south end bays.

The lateral loads on the roof are transferred through the timber purlins to the moment resisting steel portal frames. Acting in-plane, the steel portals transfer the load to the foundations.

The end walls consist of corrugated iron and timber framing, with a diagonal timber brace member to transfer east-west lateral loads down to the base of the building.

The lateral load resisting system in the longitudinal (north-south) direction consists of the steel portal frames cantilevering out-of-plane, with diagonal bracing in the plane of the roof. In places this diagonal bracing has been cut off short of the western wall in order to allow for the installation of roller shutter doors. This results in an incomplete primary load path, however an indirect load path still exists via out-of-plane bending of purlins and the portal frame rafter.

The diagonal timber bracing elements in the eastern perimeter wall have significant historical damage, compromising the load path for resisting north-south lateral loads. Due to insufficient load paths the portal frames must resist loads out-of-plane by cantilevering from the base. This response is undesirable, and will be indicative of reduced seismic performance due to increased seismic displacements.

4.4 Survey

4.4.1 Post 22 February 2011 Rapid Assessment

A structural (Level 2) assessment of the above property was undertaken on 26 February 2011 by Opus International Consultants. The whole building was assessed during this inspection.

4.4.2 Further Inspections

Further inspections were undertaken by Opus International Consultants on 13 July 2012.

4.5 Original Documentation

No original construction drawings were available from the Christchurch City Council.

5 Structural Damage

The following damage has been noted:

5.1 Residual Displacements

There is some differential global displacement observed in the longitudinal direction.

5.2 Eastern Perimeter Wall

Some historical damage to the wall cladding and timber bracing struts was observed. We note that this damage has compromised the load path for resisting north-south lateral loads.

5.3 Diagonal Roof Bracing

One of the diagonal roof braces at the southern end appears to have elongated under seismic loading.

5.4 Non Structural Elements

Some historical damage and deterioration was observed to non-structural elements.

6 Detailed Seismic Assessment

The detailed seismic assessment has been based on the NZSEE 2006 [2] guidelines for the "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" together with the "Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure" [3] draft document prepared by the Engineering Advisory Group on 19 July 2011, and the SESOC guidelines "Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes" [5] issued on 21 December 2011.

6.1 Critical Structural Weaknesses

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 a building. We have identified the following CSW's for this building:

a) Lack of North-South Bracing

Due to damage to the diagonal bracing elements in the eastern wall, the north-south lateral loads must be resisted by the portal frame columns bending out-of-plane. Due to the small column size this results in excessive longitudinal displacements. The foundation details for these columns are unknown.

6.2 Quantitative Assessment Methodology

The assessment assumptions and methodology have been included in Appendix 3 of the report due to the technical nature of the content. A brief summary follows:

An equivalent static analysis was carried out on the building using the spectral values established from NZS1170.5 [1], with an updated Z factor of 0.3 (B1/VM1) and a ductility factor of 1.25. These analyses were used to establish the actions on the structural elements. Based on the actions determined from the analyses, an assessment of the building capacities was made.

6.3 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 may 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.

6.4 Quantitative Assessment

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 elements. This will be considered further when developing the strengthening options.

Table 2: Summary of Seismic Performance – μ = 1.25

Structural Element/System	**Failure Mode, or description of limiting criteria based on displacement capacity of critical element.**	**% NBS based on calculated capacity**
Moment resisting portal frames – north south direction (out-of-plane)	Flexural failure of the steel portal frame columns, cantilevering out-of-plane from the base.	25%
Building drift – longitudinal direction	The building drift at ultimate limit state exceeds deflection limits.	25%
Diagonal roof bracing – north south direction	The 50x50EA diagonal roof braces act in tension to transfer load between portals. The performance of the roof bracing will be governed by the capacity of the end connections.	<34%
Moment resisting portal frames – east west direction (in-plane)	Flexural failure, resulting in compression buckling failure of the moment resisting portal frame columns acting in plane.	86%
Portal frame foundations	Tensile failure of the foundations under uplift conditions. The foundation details are currently unknown, but likely do not have the capacity to resist uplift force demands.	Details unknown so no capacity could be calculated. Expected to be <67%

6.5 Discussion

Given the low seismic capacities described in Table 2 above, the building is considered earthquake prone in accordance with the Building Act 2004. The building is not expected to have a brittle failure mode or a global collapse mechanism.

The current seismic performance of the structure is limited by the lack of a reliable load path in the longitudinal (north-south) direction. This results in the moment resisting steel portal frames cantilevering out-of-plane, undergoing excessive displacements at ultimate limit state.

The roof bracing elements at the west end of the building have been cut off short of the column centrelines to accommodate the installation of the roller doors. This creates an indirect load path, inducing a load eccentricity and local bending of the purlin members and portal frame rafters.

7 Summary of Geotechnical Review

Following a site walkover and background investigations, a geotechnical desk study on the site was deemed unnecessary.

This is based on the knowledge that liquefaction is not predicted at the site (ECan Solid Facts map). The closest location of surface expression of liquefaction was recorded approximately 0.5km northwest, following the February 2011 earthquake. Surrounding residential areas have been zoned TC1, thus future land damage from liquefaction is considered to be unlikely.

The site is expected to be underlain by up to 5m of fine grained soils (sand and silt layers), and then predominantly gravel and gravelly sand (based on Soils and Foundations Ltd maps). Conversely, the ground water level is expected to be approximately 5.0m below ground level (Brown and Weeber groundwater maps).

8 Remedial Options

The building requires repair and strengthening, with a target of increasing the seismic performance to as near as practicable to 100%NBS, and at least 67%NBS. Our concept strengthening scheme to achieve this would include:

a) Repairing the timber frame iron-clad walls, and upgrading of the diagonal wall bracing members for resisting north-south loads.

b) Improving the diagonal roof bracing, particularly where member elongation has occurred, to reinstate the load path.

c) Investigation of the foundations and possible strengthening if required.

9 Conclusions

a) The seismic performance of the original building is governed by the moment resisting steel portal frames, which have an expected strength of 25%NBS in the longitudinal direction (north-south) and 86%NBS in the transverse direction (east-west). The building is therefore considered to be earthquake prone in accordance with the Building Act 2004.

b) A geotechnical desk study was not required for this site. This was based on the unlikely future land damage at the site from liquefaction, and the absence of observed ground damage during site investigations.

c) The building is considered earthquake prone in accordance with the Building Act 2004. The building is expected to have a non-brittle failure mode based on the performance of the moment resisting steel portal frames. It is considered that the building does not have a global collapse mechanism.

d) Strengthening the building to at least 67% is recommended. Strengthening works would include retrofit of the diagonal bracing elements of the building to upgrade the load path in the longitudinal direction for north-south seismic loads.

e) The foundation details are currently unknown, and a trial pit needs to be dug to investigate the existing foundations prior to undertaking any strengthening works.

10 Recommendations

a) Develop a strengthening works scheme to increase the seismic capacity of the building to at least 67% NBS. This will need to consider compliance with accessibility and fire requirements.

b) A quantity surveyor be engaged to determine the costs for either strengthening the building or demolishing and rebuilding.

11 Limitations

a) This report is based on an inspection of the building and focuses on the structural damage resulting from the 22 February 2011 Canterbury Earthquake and aftershocks only. Some non-structural damage is described but this is not intended to be a complete list of damage to 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 this time.

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

12 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 1 - Photographs

North East End Store – Sockburn Depot, 149 Main South Road		
No.	**Item description**	**Photo**
<u>General</u>		
1.	View of the northwest end of the building	
2.	Roller door access on the west face of the building	
3.	View of the southeast corner of the building	

4.	Internal view of the north end wall	
5.	Internal view of the south wall	
6.	Diagonal roof bracing (note they are cut off to allow installation of the roller door)	
7.	Portal Knee detail (east side) – Tapered Flange Beam (125TFB) with Parallel Flange Channel (100PFC)	

8.	Portal Knee detail (north west end) – Tapered Flange Beam Portal (125TFB) with Parallel Flange Channel Chord (150PFC) and Equal Angle Roof Brace (EA50x50)	
9.	Portal Frame - Tapered Flange Beam measure up	
10.	Timber framing with diagonal wall bracing	
11.	Timber framing with discontinuous/insufficient diagonal wall bracing	

Appendix 2 - Quantitative Assessment Methodology and Assumptions

A2.1. Reference Documents

- AS/NZS 1170.0:2002, *Structural design actions, Part 0: General principles*, Standards New Zealand.
- AS/NZS 1170.1:2002, *Structural design actions, Part1: Permanent, imposed and other actions,* Standards New Zealand.
- NZS1170.5:2004, *Structural design actions, Part 5: Earthquake actions – New Zealand,* Standards New Zealand.
- NZS 3404: Part 1:1997, *Steel Structures Standard, The Design of Steel Structures,* Standards New Zealand.
- NZSEE: 2006, *Assessment and improvement of the structural performance of buildings in earthquakes,* New Zealand Society for Earthquake Engineering.
- 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.

A2.2. Analysis Parameters

The following parameters are used for the seismic analysis

- Site Soil Category D (deep and soft soil);
- Seismic Hazard Factor Z = 0.3;
- Return Period Factor R_u = 1.0 (Importance Level 2 structure, 50 year design life);
- Ductility Factor *μ = 1.25* (Nominally Ductile Structure – in accordance with requirements outlined in NZS1170.5:2004);
- Structural Performance Factor S_p =0.925.

A2.3. Material Properties

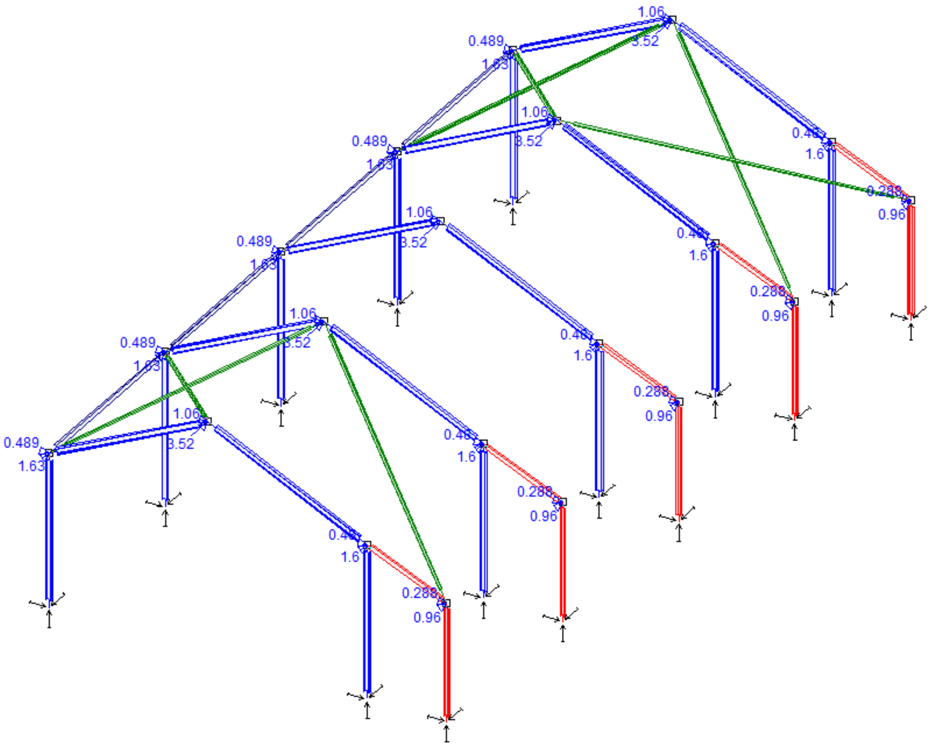
Table A1: Analysis Material Properties

Mild reinforcing nominal yield strength, *fy* (MPa) [(2)]	250
Probable steel yield strength, *fy (MPa)* [(1)]	270

Notes:
1. Based on guidance from *NZSEE 2006,* probable reinforcement yield strength is based on a value of 1.08 times the nominal yield strength (Cl. 7.1.1)
2. Based on guidelines from *Bridge Manual 2004,* characteristic yield strength of reinforcement for historical construction.

A2.4. Assessment Methodology

Equivalent Static Analysis



The building was analysed as being nominally ductile (*μ = 1.25).* The design actions were applied separately in each perpendicular direction, with 100% for the first axis plus 30% on the second axis, and then 30% in the first axis and 100% in the second axis, as required by NZS1170.5:2006 for nominally ductile and brittle structures (Clause 5.3.1.2). The building was assessed at Importance Level 2 (IL2).

Element force demands were extracted from the equivalent static analysis and compared to calculated capacities based on steel material properties assumed in Table A1. The results of these capacity to demand ratio checks are summarised in further detail in the report and presented as %NBS.

Appendix 3 – CERA DEE Spreadsheet

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name: Store North East End - Sockburn Depot
Unit | No: 149 | Street: Main South Road
Building Address:
Legal Description:
GPS south: Degrees 43 | Min 32 | Sec 18.96
GPS east: Degrees 172 | Min 33 | Sec 20.65
Building Unique Identifier (CCC): BU 1531-008-EQ2

Reviewer: Al Boyce
CPEng No: 209860
Company: Opus International Consultants Ltd
Company project number: 6-QUCCC.68
Company phone number: 6433635400
Date of submission: 20/09/2012
Inspection Date: 13/07/2012
Revision: Final
Is there a full report with this summary? yes

Site

Site slope: flat
Soil type: mixed
Site Class (to NZS1170.5): D
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): 15.00

Building

No. of storeys above ground: 1 (single storey = 1)
Ground floor split? no
Storeys below ground: 0
Foundation type: driven steel piles
Building height (m): 5.25
Floor footprint area (approx): 160
Age of Building (years):

Ground floor elevation (Absolute) (m): 15.00
Ground floor elevation above ground (m): 0.00
if Foundation type is other, describe:
height from ground to level of uppermost seismic mass (for IEP only) (m): 5.25
Date of design: 1935-1965

Strengthening present? no
Use (ground floor): other (specify)
Use (upper floors):
Use notes (if required): Storage
Importance level (to NZS1170.5): IL2

If so, when (year)?
And what load level (%g)?
Brief strengthening description:

Gravity Structure

Gravity System: frame system
Roof: timber framed
Floors: other (note)
Beams: steel non-composite
Columns: structural steel
Walls: non-load bearing

rafter type, purlin type and cladding
describe sytem
beam and connector type
typical dimensions (mm x mm)
0

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along: welded and bolted steel moment frame — 9
Ductility assumed, μ: — note typical bay length (m) 4.5m between portals
Period along: 0.71 — 0.49 from parameters in sheet — estimate or calculation? calculated
Total deflection (ULS) (mm): — estimate or calculation? calculated
maximum interstorey deflection (ULS) (mm): — estimate or calculation? calculated

Lateral system across: welded and bolted steel moment frame — 4.55
Ductility assumed, μ: 1.25 — note typical bay length (m) 4.5m between portals
Period across: 0.71 — 0.00 — estimate or calculation? calculated
Total deflection (ULS) (mm): — estimate or calculation? calculated
maximum interstorey deflection (ULS) (mm): — estimate or calculation? calculated

Separations:

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

Non-structural elements

Stairs:
Wall cladding: profiled metal — describe
Roof Cladding: Metal — describe
Glazing:
Ceilings:
Services(list):

Available documentation

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

Damage

Site: (refer DEE Table 4-2)

Site performance: — Describe damage:

Settlement:	none observed	notes (if applicable):
Differential settlement:	none observed	notes (if applicable):
Liquefaction:	none apparent	notes (if applicable):
Lateral Spread:	0-50mm	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: — Describe how damage ratio arrived at:
Describe (summary):

Across — Damage ratio: #DIV/0!
Describe (summary):

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

Diaphragms — Damage?: yes — Describe:
CSWs: — Damage?: no — Describe:
Pounding: — Damage?: no — Describe:
Non-structural: — Damage?: yes — Describe:

Recommendations

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

Along — Assessed %NBS before e'quakes: ##### %NBS from IEP below — If IEP not used, please detail assessment methodology: Quantitative Assessment
Assessed %NBS after e'quakes: 25%

Across — Assessed %NBS before e'quakes: ##### %NBS from IEP below
Assessed %NBS after e'quakes: 86%

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): 1935-1965 — h_n from above: 5.25m

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):	0.71	0.71
(%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, $(NBS\%)_b = (\%NBS)_{nom} \times A \times B \times C \times D \times E$ — **$\%NBS_b$:** **#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

3.5. Site Characteristics 1

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

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

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum		
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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