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# Sockburn Testing Station
# PRO 1530 B001

Detailed Engineering Evaluation

Qualitative Report

Version FINAL

538 Blenheim Road, Sockburn

INFRASTRUCTURE | MINING & INDUSTRY | DEFENCE | PROPERTY & BUILDINGS | ENVIRONMENT

# Sockburn Testing Station
# PRO 1530 B001

Detailed Engineering Evaluation
Qualitative Report
Version FINAL

538 Blenheim Road, Sockburn

Christchurch City Council


**Prepared By**
Cormac Joy

**Reviewed By**
David Lee


**Date**
16 September 2013

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# Qualitative Report Summary

**Sockburn Testing Station**

**PRO 1530 B001**

**Detailed Engineering Evaluation**

**Qualitative Report - SUMMARY**

**Version FINAL**

**538 Blenheim Road, Sockburn**

### Background

This is a summary of the Qualitative report for the building structure, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011 and visual inspections on 15 June 2012.

### Building Description

Due to the size of the structure and for the ease of understanding the report the building has been divided into four sections, refer to Figure 2. Sections one and four refer to the lightweight timber framed structures to the east and west of the larger sawtooth roof sections two and three.

Sections one and four have pitched roofs with lightweight metal cladding on timber sarking supported by timber rafters and purlins to section one and timber trusses for section four. The timber framed walls are clad with plasterboard internally. Section one has masonry block exterior walls and section four timber framed walls with weatherboard. The timber framed sections have concrete slab on grade floors with strip footings around the perimeter the masonry block wall areas have a cast in situ concrete slab.

Sections two and three consist of a concrete frames with infill masonry walls. Section two has a steel framed saw tooth roof with a lightweight metal cladding supported on timber purlins. Five lattice truss beams span longitudinally along the longer length of the saw tooth roof with steel bracing straps spanning diagonally across the roof. The shorter section of the roof with a steeper pitch has equal angle beams spanning vertically and smaller equal angles acting as the bracing diagonally. Section three consists of a similar saw tooth roof to section two. Lightweight metal cladding to the long stretch of roof is supported on timber purlins spanning between a concrete frames. No roof bracing exists along the long dimension of the roof. Two diagonal square hollow sections span across the shorter steeper section of the roof and are connected into the concrete frame. As no drawings were available foundation details are unknown.

**Key Damage Observed**

Key damage observed includes:-

Separation between concrete masonry and timber framed walls

Cracking and spalling of concrete to the top and bottom of the concrete columns

Cracking along blockwork mortar lines

**Critical Structural Weaknesses**

- Plan Irregularity (30% Reduction) (14% NBS)
- F Factor-damage to columns (30% Reduction) (10% NBS)

**Indicative Building Strength (from IEP and CSW assessment)**

Based on the information available, and using the NZSEE Initial Evaluation Procedure, the original capacity of the building has been assessed to be in the order of 14% NBS and post-earthquake capacity in the order of 10% NBS. Due to the nature of the damage that has occurred the post-earthquake capacity has been reduced. The buildings post-earthquake capacity excluding critical structural weaknesses and earthquake damage is in the order of 21% NBS.

The building has been assessed to have a seismic capacity in the order of 10% NBS and is therefore potentially Earthquake Prone.

**Recommendations**

It is recommended that:

- Based on the Christchurch City Councils policy, the building is deemed to be potentially earthquake prone. It is recommended that occupancy of the building is to remain unoccupied pending further detailed assessment.
- A quantitative assessment of the structure be undertaken to determine the seismic capacity and to develop potential strengthening concepts.
- Opening up works are carried out immediately to determine if the reinforcement in the columns has fractured where the cracking to the concrete has occurred.

# 1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the Sockburn Testing Station.

This report is a Qualitative Assessment of the building structure, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011.

A qualitative assessment involves inspections of the building and a desktop review of existing structural and geotechnical information, including existing drawings and calculations, if available.

The purpose of the assessment is to determine the likely building performance and damage patterns, to identify any potential critical structural weaknesses or collapse hazards, and to make an initial assessment of the likely building strength in terms of percentage of new building standard (%NBS).

At the time of this report, no intrusive site investigation, detailed analysis, or modelling of the building structure had been carried out. The building description is based on the visual inspection carried out on site.

# 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 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

## 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 any 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)) 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.

### 2.2.1 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.

**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.

**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 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;
- 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 33% 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.

## 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.

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

- Hazard Factor increased from 0.22 to 0.3 (36% increase in the basic seismic design load)
- 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.

# 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 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 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 | Unacceptable | Unacceptable |

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

Table 1 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.

| 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 |

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

# 4. Building Description

## 4.1 General

The Sockburn Testing Station is located at 538 Blenheim Road, Sockburn. The construction date of the structure is unknown but from site observation and type of structure is estimated to be in the 1960's. The timber framed ancillary sections one and four are thought to be additions at a later date. The site can be accessed from Blenheim Road to the south and the Main South Road to the north. An industrial estate is located to the eastern side of the building and storage areas to the western side. The building was previously used by Vehicle Testing New Zealand (VTNZ).

The site is predominantly flat with insignificant variations in ground levels throughout.

For the ease of understanding this report and due to the size of the building, the structure has been divided up into four sections for description and evaluation. The four sections of the building can be observed in Figure 2 below. No drawings of the structure were available.

In the descriptions below transverse applies to the east west direction parallel to Blenheim Road. Longitudinal is in the north/south direction normal to Blenheim Road.

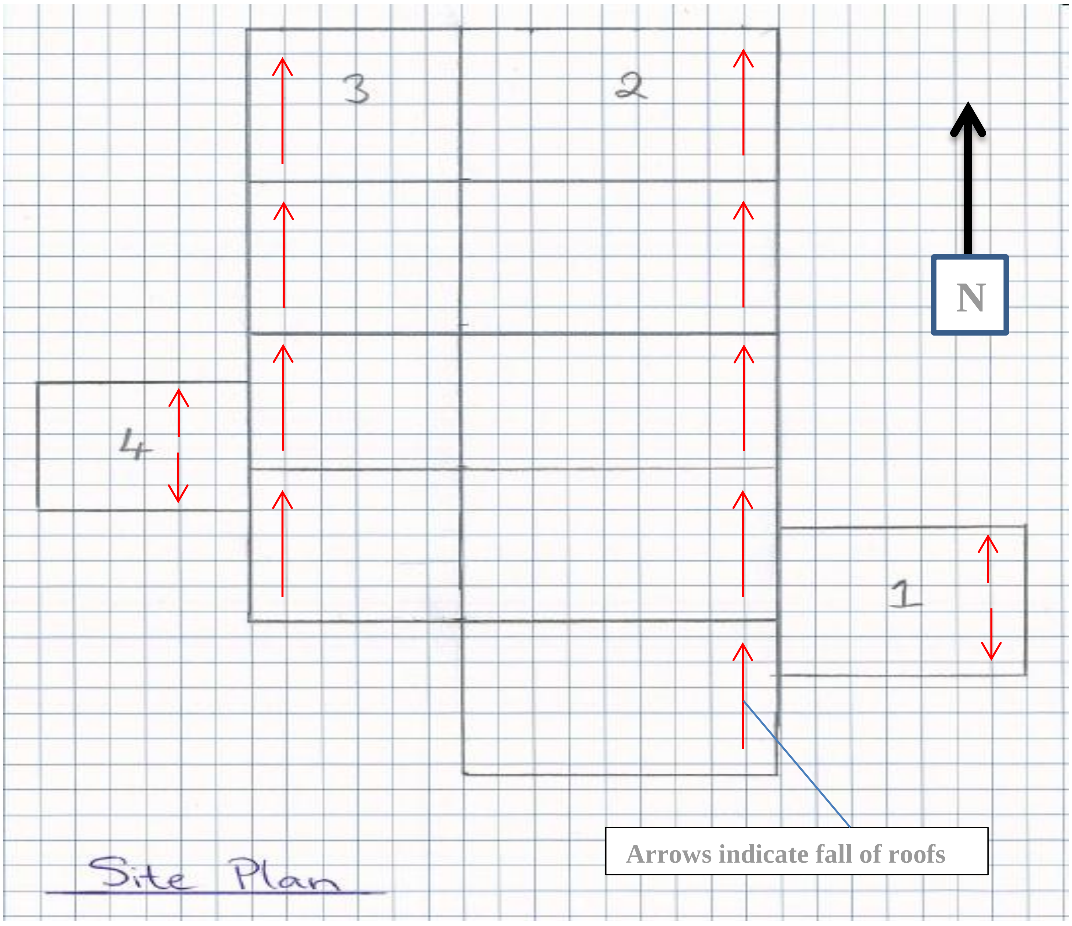



**Figure 2 Site Plan showing different sections of the building**

### 4.1.1 Section One

Section one refers to the office/reception area building to the eastern side of the main structure. The structure has a pitched roof with lightweight metal cladding on timber sarking supported by timber rafters and purlins. The timber framed walls are clad with plasterboard internally and clad with masonry blocks externally. The single storey construction has a concrete slab on grade floor with strip footings to the perimeter. The dimensions of the building are approximately 10.8m long 8.5m wide and 3.1m in height.

### 4.1.2 Section Two

The main vehicle testing station is split into two sections with a common middle masonry block wall, section two refers to the larger of the two structures as detailed in Appendix A and can be accessed from the office/reception area section one. The structure has a steel framed saw tooth roof with a lightweight metal cladding supported on timber purlins. Five lattice truss beams span longitudinally along the longer dimension of the saw tooth roof with bracing straps spanning diagonally across the roof, these can be noted in photographs 5 and 6, Appendix B. The shorter dimension of the roof with a steeper pitch has equal angle beams spanning vertically and smaller equal angles acting as the bracing diagonally. The saw tooth roof layout is shown below in Figure 3.

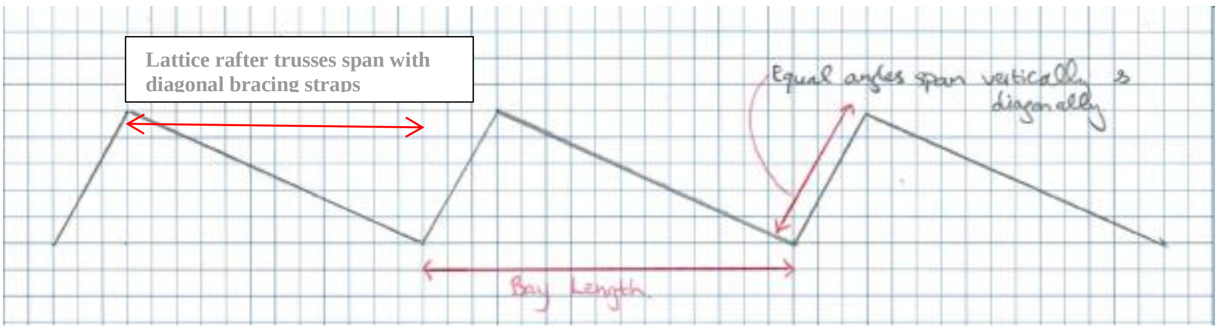



**Figure 3 Cross-section of saw tooth roof**

The saw tooth roof is supported on a concrete column and beam frame longitudinally with a cast in situ concrete infill above the bond beam and blockwork masonry below it. Transversely between the concrete columns an equal angle spans along the lower edge of the saw tooth roof. A low level foundation wall exists along the concrete framed walls to the east and west of the structure. The saw tooth roof has five sections that span 12.8m between the concrete column (420mm x 250mm) frames, one intermediate column (220mm x 220mm) is located between each 12.8m span. Two large door openings form the northern side of the structure with the southern end consisting of a combination of blockwork masonry and in situ concrete wall. Third and fourth door openings exist to the east and west sides of the southern most bay as indicated in Figure 4.

The floor consist of a concrete cast in situ slab, no drawings were available to determine the details of the foundations.

The dimensions of the building are approximately 64m long 13.6 wide with the height to the peak of the saw tooth roof 7.2 m and 4.9m to the bond beam of the concrete frame.

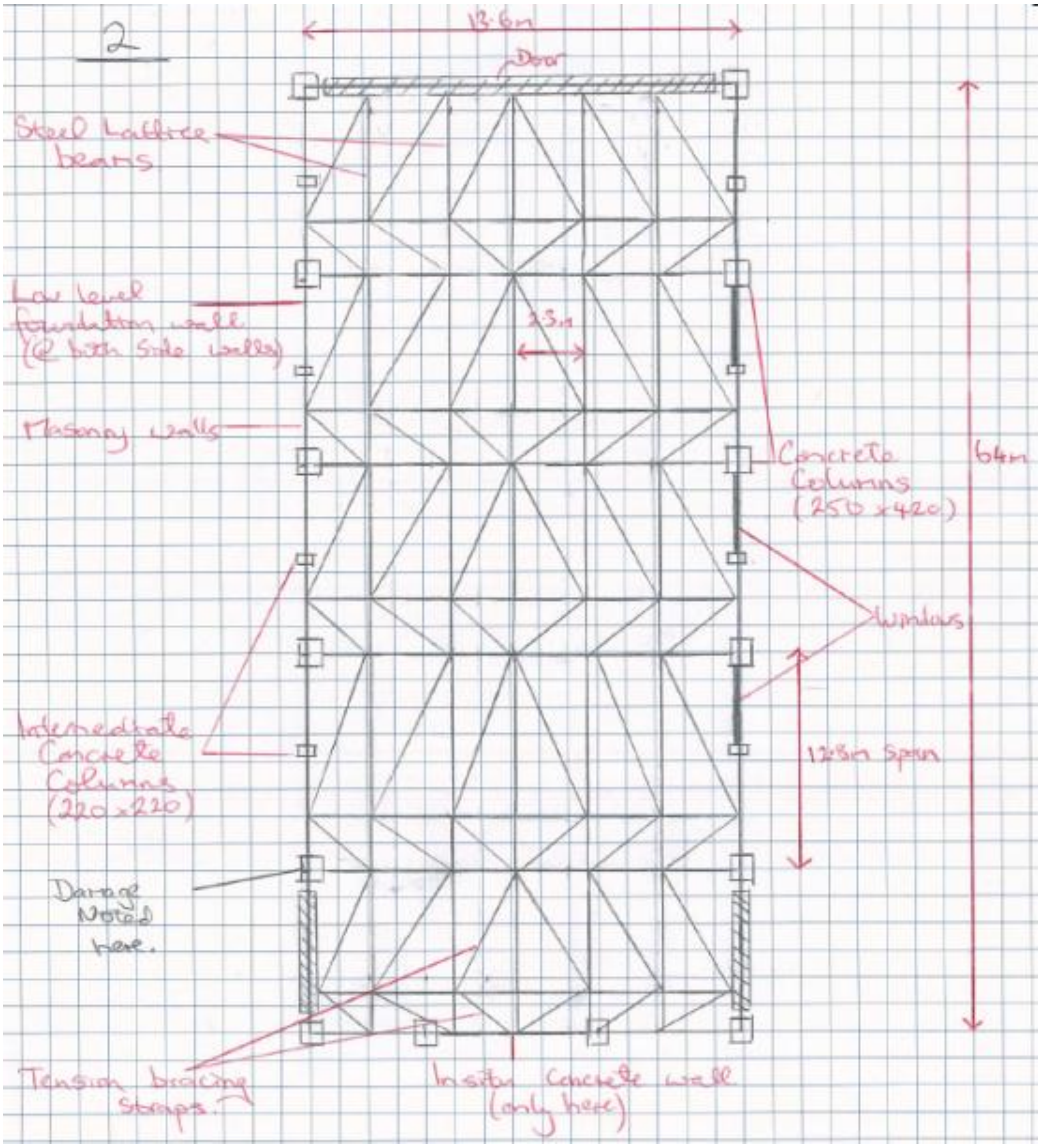



**Figure 4 Plan drawing of key structural elements to section two**

### 4.1.3 Section Three

Section three is of similar construction to section two. The roof consists of a similar saw tooth roof to section two. Lightweight metal roofing to the long dimension of the roof is supported on timber purlins spanning 5.05m between concrete frames as shown in photograph 10. No roof bracing exists along the long dimension of the roof. Two diagonal square hollow sections span across the shorter steeper section of the roof and are connected into the concrete frame as noted in photograph 10. Door openings form the north and south ends of the building.

The concrete frame infill consists of in situ concrete above the bond beam and masonry blockwork below. The concrete wall and frame to the east side is common to both sections two and three. Section three consists of four bays compared to five in section two.

The floor consists of a concrete cast in situ slab. As no drawings were available no details of the foundations are known. The structure is approximately 51.2 long 5.05m wide with the height to the peak of the saw tooth roof 7.2m and 4.9m to the bond beam of the concrete frame.

#### 4.1.4 Section Four

Section four is similar to section one and acts as an ancillary structure to section three. The lightweight metal roof cladding rests on timber sarking supported on timber trusses. Walls are timber framed internally clad with plasterboard and externally clad with timber weatherboard. Access to the building can be gained from section three as well as a roadside front entrance. The floor consists of a concrete flat slab on perimeter strip footings.

### 4.2 Gravity Load Resisting System

#### 4.2.1 Section One

The gravity loads in the structure are resisted by the timber framed roof and side walls. The gravity load from the roof is transferred into the timber rafters and purlins and distributed out to the timber framed walls. The load is then transferred down to the concrete foundations and into the ground.

#### 4.2.2 Section Two

The gravity load resisting system for the section two consists of the concrete frame structure. The gravity load is transferred from the roof metal cladding into the timber purlins and steel roof structure. The saw tooth roof structure with the lattice truss beams and strap bracing will act similar to a truss system to transfer the roof load into the concrete side wall frame and to the equal angle beam spanning between the concrete columns. This equal angle acts as a drag member out to the concrete frame. For the steeper pitched part of the roof, the gravity load is transferred straight down through equal angle beams supporting the glazing to the horizontal equal angle. The concrete columns will then allow the transfer of the load down to the concrete foundations and into the ground.

#### 4.2.3 Section Three

Similar to section two the gravity load resisting system comprises of the concrete frame structure. The gravity load is transferred from the metal roof cladding into the timber purlins. The purlins directly transfer the load out to the concrete frame. For the steeper section of the roof the gravity loads are transferred through the steel square hollow sections. These enable further transfer of the gravity load from the ridge of the roof down to the concrete frame. The concrete columns then transfer the gravity load from the roof down to the ground via the concrete foundations.

#### 4.2.4 Section Four

Similar to section one the gravity load resisting system is provided by the timber framed roof and walls. The gravity loads are transferred from the lightweight metal cladding into the timber sarking roof. The gravity load is then transferred by the timber trusses out to the timber framed side walls and down to the ground via the concrete foundations.

## 4.3 Lateral Load Resisting System

### 4.3.1 Section One

Lateral loads acting on the structure are resisted by the plasterboard braced timber framed walls and the timber sarking acting as a diaphragm both transversely and longitudinally. The lateral roof load will be distributed to the side walls through the timber sarking. The plasterboard braced walls will then allow the transfer of the lateral load from the roof down to the concrete foundations and into the ground.

### 4.3.2 Section Two

Longitudinally the lateral loads are resisted by the lattice truss beams and the steel bracing straps to the roof and the concrete frame. When the roof is loaded longitudinally it will cause the beams to act as compression struts and the bracing straps to act as ties similar to a truss system. These loads are then transferred down to the beam spanning between the columns, this beam will go into bending to transfer the load to the concrete columns. To accommodate this load the columns have to act as cantilevers in order to transfer the shear load from the top of the columns down to the concrete foundations. In addition portal frame action is expected of the frames due to the base plate connection between the equal angle beams and the top of the concrete columns. The masonry blockwork adjacent to the concrete columns will offer further stiffness acting similar to a shear wall to allow the transfer of the shear load downwards to the foundations and into the ground.

When the building is loaded transversely the braces will again act as ties transferring the load down to the beam spanning between the concrete columns. Cantilever action is expected of the concrete columns in order to transfer this load down to the concrete foundations. The equal angle beam and concrete columns will offer some frame action to transfer the lateral load downward to the concrete foundations. The circular hollow sections spanning between the high level concrete panels will offer stability under a transverse lateral load.

### 4.3.3 Section Three

The lateral load resisting system for section three comprises of the tension/compression diagonal square hollow sections to the shorter span of the saw tooth roof and the concrete frame. When the building is loaded longitudinally the loads are transferred down from the roof through the diagonal bracing SHS's to the concrete frame and columns. The lateral load will be transferred through the concrete beams down to the concrete columns. The concrete columns through cantilever and frame action will transfer the load down to the ground via the concrete foundations.

When the structure is loaded transversely the tension/compression square hollow sections will transfer the load from the ridge line of the saw tooth roof to the concrete frame. The lateral load will also be transferred from the metal roof cladding to the timber purlins spanning between the concrete frames. The purlins will offer further lateral resistance transferring the load to the concrete structure which in turn will transfer the load down to the ground via the concrete foundations.

### 4.3.4 Section Four

The lateral load resisting system to section four, similar to section one consists of the plasterboard timber framed walls and timber sarking diaphragm to the roof. The lateral load is transferred from the metal roof cladding through the diaphragm action of the timber sarking to the timber framed side walls. The load is then transferred downwards to the ground through the concrete foundations.

# 5. Assessment

A visual inspection of the building was undertaken on 15 June 2012. Both the interior and exterior of the building were inspected. The building was observed to have no placard in place. The main structural components of the building were able to be viewed due to the exposed simple construction of the building.

The visual inspection consisted of observing the building to determine the structural systems and likely behaviour of the building during an earthquake.  The site was assessed for damage, including observing the ground conditions, checking for damage in areas where damage would be expected for the structure type observed and noting general damage observed throughout the building in both structural and non-structural elements.

The %NBS score is determined using the IEP procedure described by the NZSEE which is based on the information obtained from visual observation of the building.

# 6. Damage Assessment

## 6.1 Surrounding Buildings

Sockburn testing station is located in a commercial area with properties adjacent to the site on the east with car parking facilities to the north and south with a green area to the west. During the inspection there was no apparent damage to the surrounding buildings or adjoining properties.

## 6.2 Residual Displacements and General Observations

No residual displacements of the structure were noticed during the inspection of the building.

There was significant damage noted to the concrete columns in both sections two and three. Cracking and spalling of concrete has occurred particularly at the base of the columns and at the top where the equal angle roof beams are connected to the columns.

The concrete columns act as cantilevers under the lateral seismic load and were not able to carry the full extent of the seismic loads to the foundations resulting in the damage.

Cracking was noted to the masonry blockwork infill between the concrete frame structures to sections two and three.

Minor separation was noted between the plasterboard lined timber framed walls and blockwork masonry to sections one and four.

## 6.3 Ground Damage

No ground damage was observed during our inspection of the site.

# 7. Critical Structural Weakness

## 7.1 Short Columns

No short columns were observed during inspections of the building.

## 7.2 Lift Shaft

The building does not contain a lift shaft.

## 7.3 Roof

No critical structural weaknesses were observed in the roof structure. Roof strap bracing exists to section two only with sarking acting as diaphragm bracing to sections one and four.

## 7.4 Plan Irregularity

Plan irregularity exists in section three due to the spacing of lateral load resisting elements transversely being greater than two times the building width. This is accommodated for in the CERA Initial Evaluation Form as 'significant' plan irregularity in accordance with the NZSEE guidelines.

## 7.5 Staircases

No staircases exist in the structure.

## 7.6 Liquefaction

No liquefaction was observed on site.

# 8. Geotechnical Consideration

## 8.1 Site Description

The site is situated in the suburb of Sockburn, in western Christchurch, and is relatively flat at approximately 20m above mean sea level. It is approximately 600m northeast of an unnamed tributary of the Heathcote River, 1.7km south of the Avon River, and 16km west of the coast (Pegasus Bay).

## 8.2 Published Information on Ground Conditions

### 8.2.1 Published Geology

The geological map of the area[1] indicates that the site is underlain by the following unit:

- Alluvial gravel, sand, and silt of historic river flood channels, of the Yaldhurst Member, sub-group of the Springston Formation, Holocene in age.

Immediately to the east of the site the map indicates dominantly sand and silt overbank deposits, being alluvial soils of the Yaldhurst Member, sub-group of the Springston Formation, Holocene in age.

Figure 72 from Brown & Weeber[1] indicates that groundwater is likely to be 4m below the surface, and the site is on the boundary of low and no liquefaction susceptibility.

### 8.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that three boreholes with the correct lithographic logs are located within 200m of the site (see Table 2).

These indicate the area is underlain by sandy gravels, with a silt layer at ~6m, underlain by layers of gravels separated by clay lenses at depth. The log to the east (M35/14605) indicates sand and silt is present from the surface to approximately 3m.

One log indicates groundwater was encountered at 4.7m.

[1] Brown, L. J. & Weeber, J.H. (1992): *Geology of the Christchurch Urban Area*. Institute of Geological and Nuclear Sciences 1:25,000 Geological Map 1. IGNS Limited: Lower Hutt.

| Bore Name | Log Depth | Groundwater | Dist. from Site | Log Summary |
|---|---|---|---|---|
| M35/1859 | 101m | 4.7m | 80m W | 0 to 6m Gravel-dominated soils<br>6m to 9.5m Clay/Silt<br>9.5m to 38m Gravel & Sand<br>38m to 101m Gravel interlain by Clay |
| M35/1860 | 81m | - | 100m NW | 0 to 5m Gravel & Sand<br>5m to 9m Clay/Silt<br>9m to 35m Gravel & Sand<br>35m to 81m Gravel interlain by Clay |
| M35/14605 | 3.15m | - | 180m E | 0 to 1.52m Silt/Clay with Sand lenses<br>1.52m to 3.05m Grey Silt<br>3.05m to 3.15m Running Sand |

**Table 2 ECan Borehole Summary**

It should be noted that the boreholes were sunk for groundwater extraction and not for geotechnical purposes. Therefore, the amount of material recovered and available for interpretation and recording will have been variable at best and may not be representative. The logs have been written by the well driller and not a geotechnical professional or to a standard. In addition strength data is not recorded.

### 8.2.3 EQC Geotechnical Investigations

The Earthquake Commission has not undertaken geotechnical testing in the area of the subject site.

### 8.2.4 CERA Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has indicated the site is situated within the Green Zone, indicating that repair and rebuild may take place.

Land in the CERA green zone has been divided into three technical categories. These categories describe how the land in expected to perform in future earthquakes.

The site is indicated as being within the TC1 (grey) zone[2]. This means that future land damage from liquefaction is unlikely.

### 8.2.5 Post-Earthquake Liquefaction Observations

No evidence of liquefaction at the ground surface was observed or recorded in Sockburn or Hornby.

Aerial photography taken following the 22 February 2011 earthquake shows no signs of liquefaction outside the building footprint or adjacent to the site, as shown in Figure 3.

[2] CERA Technical Category Maps website, http://cera.govt.nz/maps/technical-categories/


**Figure 5 Post February 2011 Earthquake Aerial Photography[3]**

### 8.2.6 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated to comprise gravel-dominated soils to 6m overlying a band of silt from 6m to 9m, underlain at depth by gravel and sandy gravel layers interlain by clay/silt strata.

A thin surficial layer of sand and silt may be present at the site (as identified in Table 2), however this is more typically associated with the adjacent ‘sand and silt overbank deposits’ geological unit.

Groundwater levels at the site are expected to be approximately 4m to 5m below the surface.

## 8.3 Seismicity

### 8.3.1 Nearby Faults

There are many faults in the Canterbury region, however only those considered most likely to have an adverse effect on the site are detailed in Table 3 below.

[3] Aerial Photography Supplied by Koordinates sourced from http://koordinates.com/layer/3185-christchurch-post-earthquake-aerial-photos-24-feb-2011/

| Known Active Fault | Distance from Site | Direction from Site | Max Likely Magnitude | Avg Recurrence Interval |
|---|---|---|---|---|
| Alpine Fault | 130 km | NW | ~8.3 | ~300 years |
| Greendale (2010) Fault | 15 km | W | 7.1 | ~15,000 years |
| Hope Fault | 100 km | N | 7.2~7.5 | 120~200 years |
| Kelly Fault | 110 km | NW | 7.2 | ~150 years |
| Porters Pass Fault | 55 km | NW | 7.0 | ~1100 years |

**Table 3 Summary of Known Active Faults[4,5]**

The recent earthquakes since 4 September 2010 have identified the presence of a previously unmapped active fault system underneath the Canterbury Plains, including Christchurch City, and the Port Hills. Research and published information on this system is in development and not generally available. Average recurrence intervals are yet to be estimated.

### 8.3.2 Ground Shaking Hazard

New Zealand Standard NZS 1170.5:2004 quantifies the Seismic Hazard factor for Christchurch as 0.30, being in a moderate to high earthquake zone. This value has been provisionally upgraded recently (from 0.22) to reflect the seismicity hazard observed in the earthquakes since 4 September 2010.

The recent seismic activity has produced earthquakes of Magnitude-6.3 with peak ground accelerations (PGA) up to twice the acceleration due to gravity (2g) in some parts of the city. This has resulted in widespread liquefaction throughout Christchurch.

## 8.4 Slope Failure and/or Rockfall Potential

Given the site's location in Sockburn, global slope instability is considered negligible. However, any localised retaining structures or embankments should be further investigated to determine the site-specific slope instability potential.

## 8.5 Liquefaction Potential

The site is considered unlikely to be susceptible to liquefaction, due to the following reasons:

- Brown & Weeber (1992) indicating no to low liquefaction susceptibility;
- Anticipation of gravel-dominated subsoils and deeper groundwater levels (>4m);
- CERA's classification of the area as TC1, indicating future land damage from liquefaction is unlikely; and,
- No evidence of liquefaction is visible in the post-earthquake aerial photography.

[4] Stirling, M.W, McVerry, G.H, and Berryman K.R. (2002): "A New Seismic Hazard Model for New Zealand", *Bulletin of the Seismological Society of America*, Vol. 92 No. 5, June 2002, pp. 1878-1903.

[5] GNS Active Faults Database, http://maps.gns.cri.nz/website/af/viewer

## 8.6 Conclusions & Recommendations

This assessment is based on a review of the geology and existing ground investigation information, and observations from the Christchurch earthquakes since 4 September 2010.

The site appears to be situated on layers of gravel and sandy gravel. Associated with this the site is considered unlikely to liquefy.

A soil class of **D** (in accordance with NZS 1170.5:2004) should be adopted for the site.

In consideration of the findings of this assessment, further geotechnical investigation is not considered necessary for the assessment process at this site. However, any proposed future foundation works (repair or rebuild) should include geotechnical testing, as required by consenting guidelines.

# 9. Survey

No level or verticality surveys have been undertaken for this building at this stage as indicated by Christchurch City Council guidelines.

# 10. Initial Capacity Assessment

## 10.1 % NBS Assessment

Two separate IEP's have been carried out to cater for both forms of construction at the Sockburn Testing Station. These are the lightweight timber framed structures and saw tooth roof concrete frame structures. Following an IEP assessment, the critical buildings are two and three (concrete frame structures) and have been assessed as achieving 10% New Building Standard (NBS). Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the concrete frame buildings are considered potentially Earthquake Prone as they do not achieve above 33% NBS. Plan irregularity has been assessed as a critical structural weakness and thus reduces the overall % NBS. In addition the score has been adjusted using the F factor for sections two and three to consider the damage to the concrete columns which may result in a potential collapse hazard.

**Section One & Four**

| Item | %NBS | |
|---|---|---|
| Building (No CSW's) | 76 | |

**Section Two & Three**

| Item | %NBS | |
|---|---|---|
| Building excluding CSW's | 21 | |
| Plan Irregularity | 14 | (30% Reduction) |
| F Factor (damage to columns) | 10 | (30% Reduction) |

**Table 4 Indicative Building Capacities based on the NZSEE Initial Evaluation Procedure**

## 10.2 Seismic Parameters

The seismic design parameters based on current design requirements from NZS1170:2002 and the NZBC clause B1 for this building are:

- Site soil class: D, NZS 1170.5:2004, Clause 3.1.3, Soft Soil
- Site hazard factor, Z = 0.3, NZBC, Clause B1 Structure, Amendment 11 effective from 1 August 2011
- Return period factor $R_u$ = 1.0, NZS 1170.5:2004, Table 3.5, Importance Level 2 structure with a 50 year design life.

An increased Z factor of 0.3 for Christchurch has been used in line with recommendations from the Department of Building and Housing recommendations resulting in a reduced % NBS score

## 10.3 Expected Structural Ductility Factor

For buildings one and four a structural ductility factor of 2.0 has been taken both longitudinally and transversely. This is based on the lightweight timber framed structures with plasterboard clad walls.

For building two and three a structural ductility of factor 1.25 has been taken both longitudinally and transversely. This is based on the concrete masonry blockwork walls and the concrete frame.

## 10.4 Discussion of Results

For the lightweight timber framed buildings the results obtained from the initial IEP assessment are consistent with those expected for this type of a structure founded on Class D soils. Due to the structures having no critical structural weaknesses and a ductility of 2.0 it is reasonable to expect the structure not be regarded as Earthquake Prone or Earthquake Risk.

The concrete framed saw tooth roof structures would have been designed to standards with design loads significantly less than those required by the current loading standard and detailing requirements for ductile seismic behaviour that are present in the current standards. Due to the structure being a long narrow building a significant plan irregularity critical structural weakness has been applied reducing the overall % NBS by 30%. In addition the damage noted to the concrete columns has been assessed as a potential collapse hazard and the F Factor has been used to decrease the % NBS. When combined with the increase in the hazard factor for Christchurch to 0.3 it is reasonable to expect the building to be classified as potentially Earthquake Prone.

## 10.5 Occupancy

As the structure achieves under 33% NBS, it deemed to be a potentially Earthquake Prone structure in accordance with the NZSEE guidelines. In addition, a potential collapse hazard exists due to the damage sustained by the concrete columns in recent seismic activity. Due to this hazard and the building being regarded as potentially Earthquake Prone in accordance with the Christchurch City Councils (CCC) policy, occupancy of the structure is prohibited.

# 11. Initial Conclusions

The overall building has been assessed to have a seismic capacity in the order of 10% NBS and is therefore potentially earthquake prone. In accordance with CCC policy regarding potentially Earthquake Prone buildings, it is recommended that this building remains unoccupied subject to further investigation and/or strengthening. The damaged concrete columns are likely to be critical structural elements as they act as cantilevers for lateral seismic loads for sections two and three.

# 12. Recommendations

The damage to the building during recent seismic activity in Christchurch has caused damage to the reinforced concrete columns in the building.

As the building has achieved less than 33% NBS following an initial IEP assessment it is regarded as potentially Earthquake Prone. As a result, we recommend that further detailed assessment of the structure is undertaken and if necessary, strengthening options explored.

It is recommended that opening up works are carried out to determine if the reinforcement in the columns has fractured where the cracking to the concrete has occurred.

# 13. Limitations

## 13.1 General

This report has been prepared subject to the following limitations:

- No intrusive structural investigations have been undertaken.
- No intrusive geotechnical investigations have been undertaken.
- No level or verticality surveys have been undertaken.
- No material testing has been undertaken.
- No calculations, other than those included as part of the IEP in the CERA Building Evaluation Report, have been undertaken. No modelling of the building for structural analysis purposes has been performed.

It is noted that this report has been prepared at the request of Christchurch City Council and is intended to be used for their purposes only. GHD accepts no responsibility for any other party or person who relies on the information contained in this report.

## 13.2 Geotechnical Limitations

This report presents the results of a geotechnical appraisal prepared for the purpose of this commission, and for prepared solely for the use of Christchurch City Council and their advisors. The data and advice provided herein relate only to the project and structures described herein and must be reviewed by a competent geotechnical engineer before being used for any other purpose. GHD Limited (GHD) accepts no responsibility for other use of the data.

The advice tendered in this report is based on a visual geotechnical appraisal. No subsurface investigations have been conducted. An assessment of the topographical land features have been made based on this information. It is emphasised that Geotechnical conditions may vary substantially across the site from where observations have been made. Subsurface conditions, including groundwater levels can change in a limited distance or time. In evaluation of this report cognisance should be taken of the limitations of this type of investigation.

An understanding of the geotechnical site conditions depends on the integration of many pieces of information, some regional, some site specific, some structure specific and some experienced based. Hence this report should not be altered, amended or abbreviated, issued in part and issued incomplete in any way without prior checking and approval by GHD. GHD accepts no responsibility for any circumstances, which arise from the issue of the report, which have been modified in any way as outlined above.

Appendix A

# Photographs

**Photograph 1: East elevation showing Sections one and two**

**Photograph 2: South elevation showing Sections two, three and four**

**Photograph 3: Separation between masonry blockwork and timber framed walls to Section one**

**Photograph 4: Timber framed roof with timber sarking to Section one**

**Photograph 5: Interior structure of Section two showing concrete frame structure with infill masonry**

**Photograph 6: Roof structure to Section two shows lattice truss beams spanning longitudinally with diagonal strap bracing**

**Photograph 7: Cracking and spalling of the concrete has occurred at the connection between the roof steel beams and the top of the concrete column**

**Photograph 8:  Cracking at the base the column indicates bending and movement**

**Photograph 9: Externally cracking is evident to the concrete column and masonry blockwork**

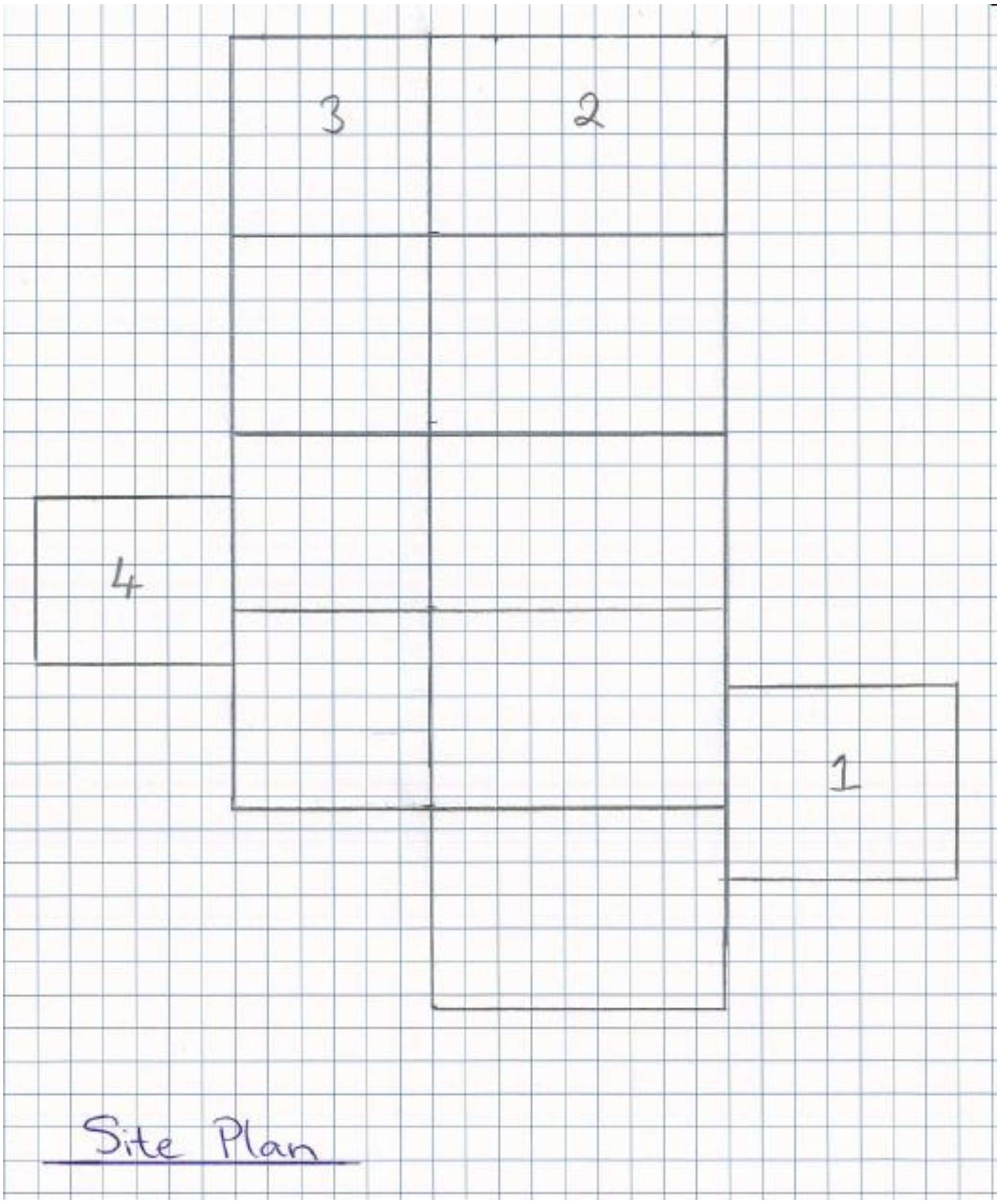
**Photograph 10: Section three has SHS diagonal sections acting as compression/tension members to transfer load, no strap bracing is present**

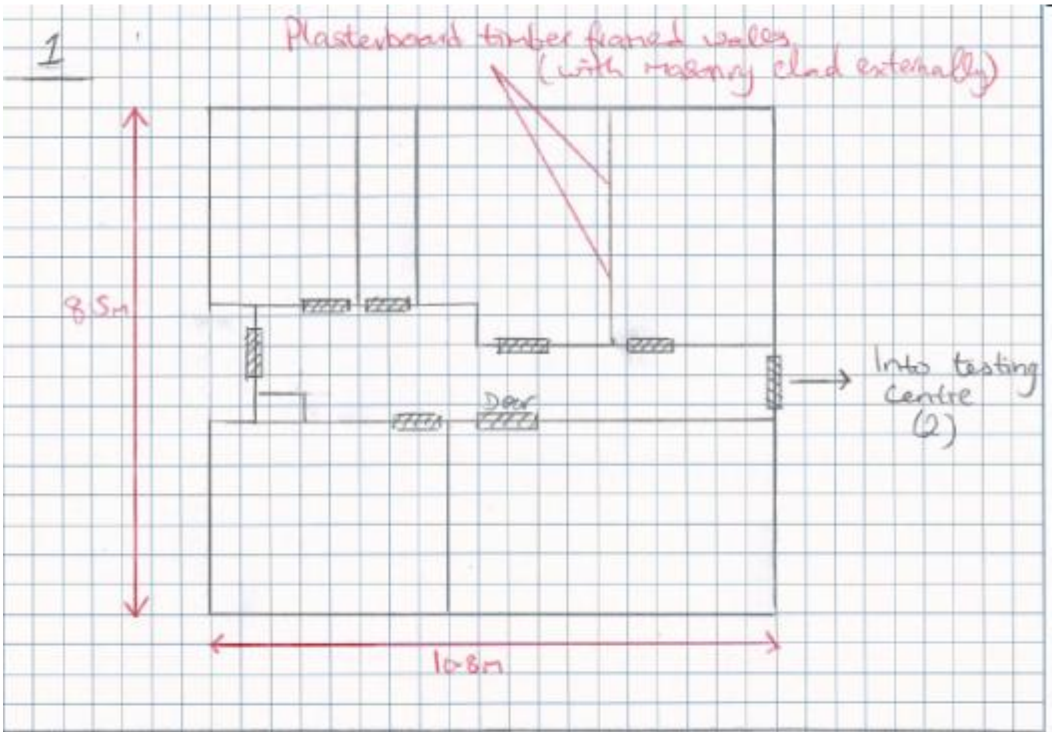
**Photograph 11: Cracking and spalling of concrete at column base**

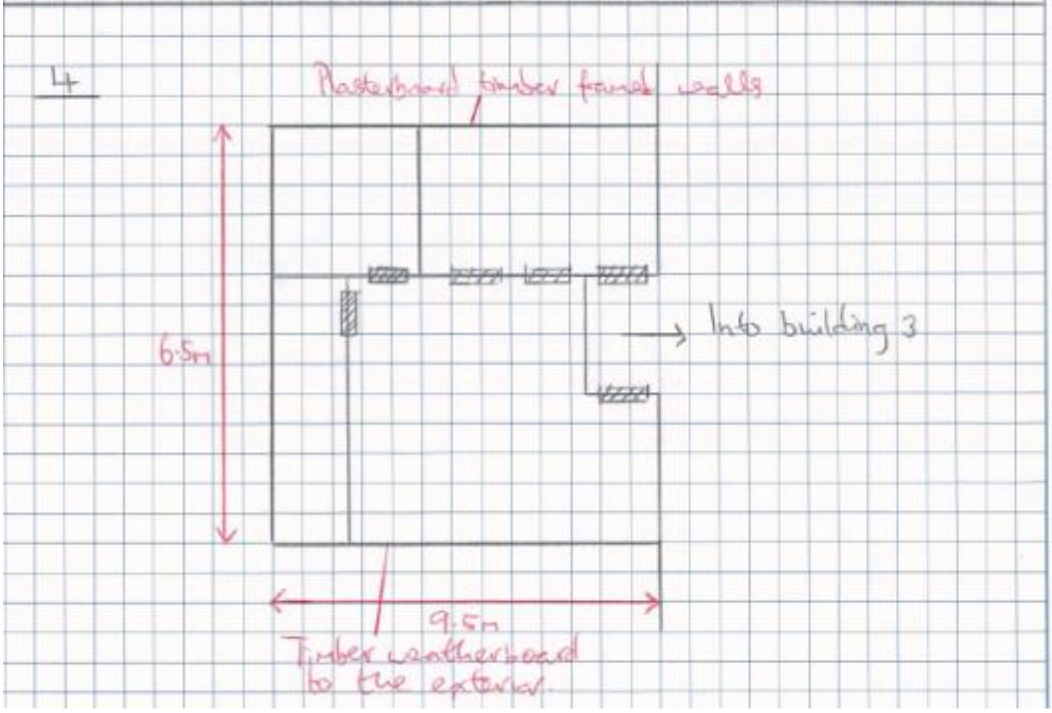
**Photograph 12: Timber truss roof in section four with timber sarking**

# Appendix B
# Existing Drawings/Sketches

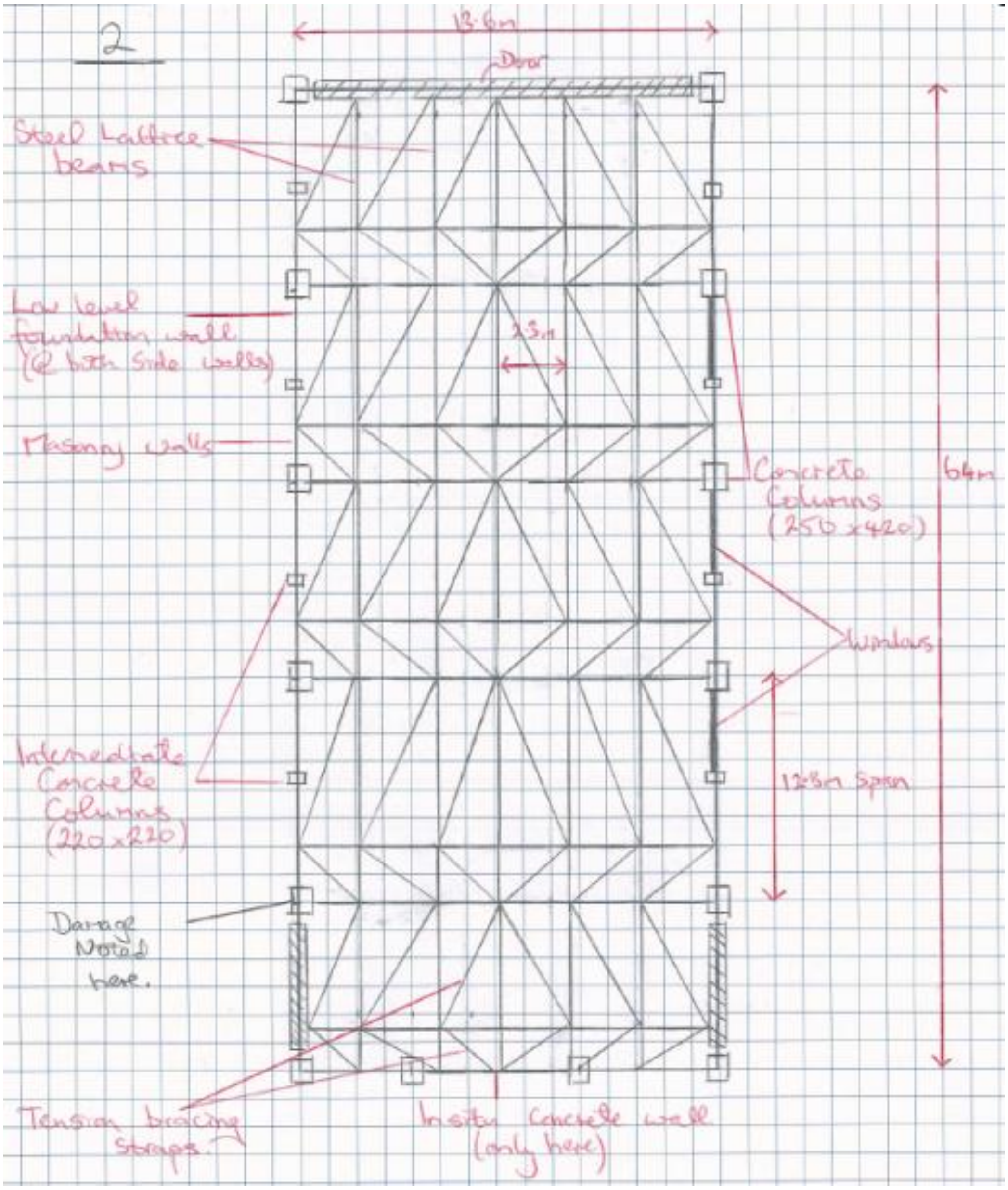
No drawings have been made available for this building. Shown below is a plan of the overall site with sketches of each building showing key structural elements.



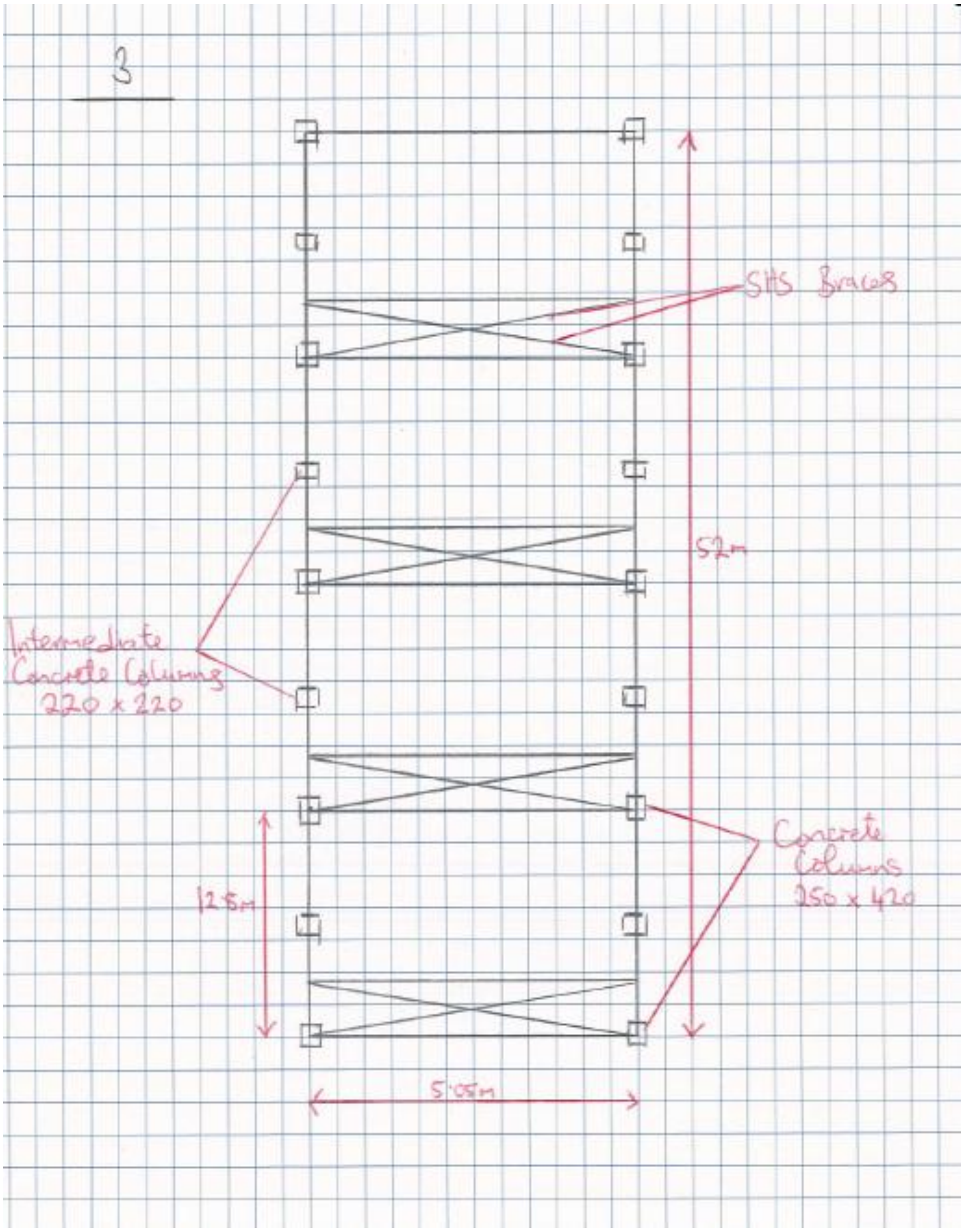



**Section One & Four**


2
13.6m
Door
Steel Lattice beams
Low level foundation wall (@ both side walls)
2.3m
Masonry walls
Concrete Columns (250 x420)
64m
Windows
Intermediate Concrete Columns (220 x 220)
12.8m Span
Damage Noted here.
Tension bracing straps
In situ Concrete wall (only here)

**Section Two**




**Section Three**

Appendix C

# CERA Building Evaluation Form

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

**Location**

| | | | | | |
|---|---|---|---|---|---|
| Building Name: | Sockburn Testing Station Building 2&3 | | | Reviewer: | David Lee |
| | Unit | No: | Street | CPEng No: | 112052 |
| Building Address: | 538 Blenheim Road | | | Company: | GHD |
| Legal Description: | | | | Company project number: | 513059650 |
| | | | | Company phone number: | |
| | Degrees | Min | Sec | | |
| GPS south: | | | | Date of submission: | |
| GPS east: | | | | Inspection Date: | 15/06/12 |
| | | | | Revision: | |
| Building Unique Identifier (CCC): | **BU 1530-001 EQ2** | | | Is there a full report with this summary? | yes |

**Site**

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

**Building**

| | | | | |
|---|---|---|---|---|
| No. of storeys above ground: | 1 | single storey = 1 | Ground floor elevation (Absolute) (m): | |
| Ground floor split? | no | | Ground floor elevation above ground (m): | |
| Storeys below ground | 0 | | | |
| Foundation type: | pads with tie beams | | if Foundation type is other, describe: | |
| Building height (m): | 7.20 | | height from ground to level of uppermost seismic mass (for IEP only) (m): | 6.2 |
| Floor footprint area (approx): | 870 | | | |
| Age of Building (years): | | | Date of design: | 1965-1976 |
| Strengthening present? | no | | If so, when (year)? | |
| | | | And what load level (%g)? | |
| Use (ground floor): | commercial | | Brief strengthening description: | |
| Use (upper floors): | | | | |
| 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 | Metal roof cladding on timber purlins supported on steel lattice beams (saw tooth roof) |
| Floors: | concrete flat slab | slab thickness (mm) | |
| Beams: | precast concrete | overall depth (mm) | |
| Columns: | cast-insitu concrete | typical dimensions (mm x mm) | |
| Walls: | fully filled concrete masonry | #N/A | |

Lateral load resisting structure

| | | | | |
|---|---|---|---|---|
| Lateral system along: | concrete frame with infill | **Note: Define along and across in detailed report!** | note total length of wall at ground (m): | |
| Ductility assumed, μ: | 1.25 | | wall thickness (m): | 200 |
| Period along: | 0.40 | ##### 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 frame with infill | | note total length of wall at ground (m): | |
| Ductility assumed, μ: | 1.25 | | wall thickness (m): | 200 |
| Period across: | 0.40 | ##### 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: | Metal | describe | |
| Glazing: | timber frames | | |
| Ceilings: | none | | |
| Services(list): | | | |

**Available documentation**

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

**Damage**

Site:
(refer DEE Table 4-2)

| | | | |
|---|---|---|---|
| Site performance: | | Describe damage: | |
| Settlement: | none observed | notes (if applicable): | |
| Differential settlement: | none observed | notes (if applicable): | |
| Liquefaction: | none apparent | notes (if applicable): | |
| Lateral Spread: | none apparent | notes (if applicable): | |
| Differential lateral spread: | none apparent | notes (if applicable): | |
| Ground cracks: | none apparent | notes (if applicable): | |
| Damage to area: | none apparent | notes (if applicable): | |

Building:

| | | | | |
|---|---|---|---|---|
| | Current Placard Status: | | | |
| Along | Damage ratio: | 29% | Describe how damage ratio arrived at: | |
| | Describe (summary): | Spalling & cracking of concrete to top of column | | |
| Across | Damage ratio: | 29% | | |
| | Describe (summary): | Spalling & cracking of concrete to top of column | | |
| Diaphragms | Damage?: | no | Describe: | |
| CSWs: | Damage?: | yes | Describe: | |
| Pounding: | Damage?: | no | Describe: | |
| Non-structural: | Damage?: | no | Describe: | |

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

**Recommendations**

| | | | | | |
|---|---|---|---|---|---|
| | Level of repair/strengthening required: | | | Describe: | |
| | Building Consent required: | | | Describe: | |
| | Interim occupancy recommendations: | do not occupy | | Describe: | |
| Along | Assessed %NBS before: | 14% | 10% %NBS from IEP below | If IEP not used, please detail assessment methodology: | |
| | Assessed %NBS after: | 10% | | | |
| Across | Assessed %NBS before: | 14% | 10% %NBS from IEP below | | |
| | Assessed %NBS after: | 10% | | | |

**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): 1965-1976

$h_n$ from above: 6.2m

Seismic Zone, if designed between 1965 and 1992: B

not required for this age of building

not required for this age of building

| | along | across |
|---|---|---|
| Period (from above): | 0.4 | 0.4 |
| (%NBS)nom from Fig 3.3: | 5.0% | 5.0% |

| | |
|---|---|
| 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 $(\%NBS)_{nom}$:** | **5%** | **5%** |

**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: | 0.30 |
| $Z_{1992}$, from NZS4203:1992 | 0.8 |
| Hazard scaling factor, **Factor B:** | 3.333333333 |

**2.4 Return Period Scaling Factor**

| | |
|---|---|
| Building Importance level (from above): | 2 |
| Return Period Scaling factor from Table 3.1, **Factor C**: | 1.00 |

**2.5 Ductility Scaling Factor**

| | along | across |
|---|---|---|
| Assessed ductility (less than max in Table 3.2) | 1.25 | 1.25 |
| Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3: | 1.14 | 1.14 |
| Ductiity Scaling Factor, **Factor D**: | 1.14 | 1.14 |

**2.6 Structural Performance Scaling Factor:**

| | | |
|---|---|---|
| Sp: | 0.925 | 0.925 |
| Structural Performance Scaling Factor **Factor E**: | 1.081081081 | 1.081081081 |

**2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom}$ x A x B x C x D x E**

| | | |
|---|---|---|
| **$\%NBS_b$:** | **21%** | **21%** |

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

| | | |
|---|---|---|
| **3.1. Plan Irregularity, factor A:** | significant | 0.7 |
| **3.2. Vertical irregularity, Factor B:** | insignificant | 1 |
| **3.3. Short columns, Factor C:** | insignificant | 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** | insignificant | 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 | 0.7 | 0.7 |
| Rationale for choice of F factor, if not 1 | s | |

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)** | **0.49** | **0.49** |
| **4.3 PAR x (%NBS)b:** PAR x Baselline %NBS: | **10%** | **10%** |
| **4.4 Percentage New Building Standard (%NBS), (before)** | | **10%** |

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

**Location**

| | | | | | |
|---|---|---|---|---|---|
| Building Name: | Sockburn Testing Station Building 1&4 | | | Reviewer: | David Lee |
| | Unit | No: | Street | CPEng No: | 112052 |
| Building Address: | 538 Blenheim Road, Sockburn | | | Company: | GHD |
| Legal Description: | | | | Company project number: | 513059650 |
| | | | | Company phone number: | |
| | Degrees | Min | Sec | | |
| GPS south: | | | | Date of submission: | |
| GPS east: | | | | Inspection Date: | 15/06/12 |
| | | | | Revision: | |
| Building Unique Identifier (CCC): | **BU 1530-001 EQ2** | | | Is there a full report with this summary? | yes |

**Site**

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

**Building**

| | | | | |
|---|---|---|---|---|
| No. of storeys above ground: | 1 | single storey = 1 | Ground floor elevation (Absolute) (m): | |
| Ground floor split? | no | | Ground floor elevation above ground (m): | |
| Storeys below ground | 0 | | | |
| Foundation type: | strip footings | | if Foundation type is other, describe: | Concrete slab on grade |
| Building height (m): | 3.10 | | height from ground to level of uppermost seismic mass (for IEP only) (m): | 2.9 |
| Floor footprint area (approx): | 92 | | | |
| Age of Building (years): | 51 | | Date of design: | 1976-1992 |
| Strengthening present? | no | | If so, when (year)? | |
| | | | And what load level (%g)? | |
| Use (ground floor): | commercial | | Brief strengthening description: | |
| Use (upper floors): | | | | |
| Use notes (if required): | | | | |
| Importance level (to NZS1170.5): | IL2 | | | |

Gravity Structure

| | | | |
|---|---|---|---|
| Gravity System: | load bearing walls | | |
| Roof: | timber framed | rafter type, purlin type and cladding | Metal roof cladding on timber purlins & rafters |
| Floors: | concrete flat slab | slab thickness (mm) | |
| Beams: | timber | type | |
| Columns: | | | |
| Walls: | | | |

Lateral load resisting structure

| | | | | | |
|---|---|---|---|---|---|
| Lateral system along: | lightweight timber framed walls | | **Note: Define along and across in detailed report!** | note typical wall length (m) | Lightweight timber framed walls with a masonry block cladding |
| Ductility assumed, μ: | | | | | |
| Period along: | 0.40 | 0.00 | | estimate or calculation? | estimated |
| Total deflection (ULS) (mm): | | | | estimate or calculation? | |
| maximum interstorey deflection (ULS) (mm): | | | | estimate or calculation? | |
| Lateral system across: | lightweight timber framed walls | | | note typical wall length (m) | |
| Ductility assumed, μ: | | | | | |
| Period across: | 0.40 | 0.00 | | 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: | plaster system | describe | Gypsum plasterboard lined |
| Roof Cladding: | Metal | describe | |
| Glazing: | timber frames | | |
| Ceilings: | light tiles | | |
| Services(list): | | | |

**Available documentation**

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

**Damage**

Site:
(refer DEE Table 4-2)

| | | | |
|---|---|---|---|
| Site performance: | | Describe damage: | |
| Settlement: | none observed | notes (if applicable): | |
| Differential settlement: | none observed | notes (if applicable): | |
| Liquefaction: | none apparent | notes (if applicable): | |
| Lateral Spread: | none apparent | notes (if applicable): | |
| Differential lateral spread: | none apparent | notes (if applicable): | |
| Ground cracks: | none apparent | notes (if applicable): | |
| Damage to area: | none apparent | notes (if applicable): | |

Building:

| | | | | |
|---|---|---|---|---|
| | Current Placard Status: | | | |
| Along | Damage ratio: | 100% | Describe how damage ratio arrived at: | |
| | Describe (summary): | No apparent damage | | |
| Across | Damage ratio: | | | |
| | Describe (summary): | No apparent damage | | |
| Diaphragms | Damage?: | no | Describe: | |
| CSWs: | Damage?: | no | Describe: | |
| Pounding: | Damage?: | no | Describe: | |
| Non-structural: | Damage?: | yes | Describe: | Slight separation between masonry blockwork & timber fr |

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

**Recommendations**

| | | | | | |
|---|---|---|---|---|---|
| | Level of repair/strengthening required: | | | Describe: | |
| | Building Consent required: | | | Describe: | |
| | Interim occupancy recommendations: | do not occupy | | Describe: | |
| Along | Assessed %NBS before: | 76% | 76% %NBS from IEP below | If IEP not used, please detail assessment methodology: | |
| | Assessed %NBS after: | | | | |
| Across | Assessed %NBS before: | 76% | 76% %NBS from IEP below | | |
| | Assessed %NBS after: | | | | |

**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): 1976-1992

$h_n$ from above: 2.9m

Seismic Zone, if designed between 1965 and 1992: B

not required for this age of building

not required for this age of building

| | along | across |
|---|---|---|
| Period (from above): | 0.4 | 0.4 |
| (%NBS)nom from Fig 3.3: | 16.0% | 16.0% |

| | |
|---|---|
| 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)$_{nom}$:** | **16%** | **16%** |

**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: | 0.30 | |
| | $Z_{1992}$, from NZS4203:1992 | 0.8 | |
| | Hazard scaling factor, **Factor B:** | 3.333333333 | |
| **2.4 Return Period Scaling Factor** | Building Importance level (from above): | 2 | |
| | Return Period Scaling factor from Table 3.1, **Factor C**: | 1.00 | |
| | | along | across |
| **2.5 Ductility Scaling Factor** | Assessed ductility (less than max in Table 3.2) | 2.00 | 2.00 |
| | Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3: | 1.57 | 1.57 |
| | Ductiity Scaling Factor, **Factor D**: | 1.00 | 1.00 |
| **2.6 Structural Performance Scaling Factor:** | Sp: | 0.700 | 0.700 |
| | Structural Performance Scaling Factor **Factor E**: | 1.428571429 | 1.428571429 |
| **2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom}$ x A x B x C x D x E** | **$\%NBS_b$:** | **76%** | **76%** |

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

| | | |
|---|---|---|
| **3.1. Plan Irregularity, factor A:** | insignificant | 1 |
| **3.2. Vertical irregularity, Factor B:** | insignificant | 1 |
| **3.3. Short columns, Factor C:** | insignificant | 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** | insignificant | 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 |

| | | Along | Across |
|---|---|---|---|
| **3.6. Other factors, Factor F** | For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum | 1.0 | 1.0 |
| | 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)** | | **1.00** | **1.00** |
| **4.3 PAR x (%NBS)b:** | PAR x Baselline %NBS: | **76%** | **76%** |
| **4.4 Percentage New Building Standard (%NBS), (before)** | | | **76%** |

**GHD**

226 Antigua Street, Christchurch 8011
T: 64 3 378 0900   F: 64 3 377 8575   E: chcmail@ghd.com



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