criss-cross pattern.

Apply the material evenly. Using a paint grid in the application container is recommended.
Consumption: approx. 0.1 - 0.2 kg/m², depending on the substrate

3) Care treatment using StoDivers P 120
When the industrial flooring is clean and has cured, evenly apply a thin layer of care treatment. Apply the material using a pre-dampened mop. Leave the floor to dry sufficiently, approx. 20 - 30 min. Carry out the second application cycle at right angles (perpendicular) to the previous application.

In order to minimise seams, work in a criss-cross pattern. Avoid creating overlaps. Depending on the expected stress, several application cycles may be necessary. It is very important to observe the specified drying times between application cycles.
Consumption: approx. 30 - 50 ml/m² per application cycle

Drying, curing, ready for next coat	Dust-dry: after approx. 3 hours (at +23 °C) Ready for foot traffic: after approx. 12 hours (at +23 °C) Mechanically and chemically resistant: after approx. 7 days (at +23 °C)
Cleaning the tools	Clean with water immediately after use; hardened material can only be removed mechanically.
Notes, recommendations, special information, miscellaneous	The gloss level of the dead-matt sealing coat StoPur WV 200 is increased by adding StoDivers P 120. The layer thickness for sealing coats is normally < 0.5 mm and decreases as a result of mechanical use. This should be taken into account with regard to the required service life. StoPur WV 200 and the StoDivers P 120 care treatment are not plasticiser resistant! If glossy, coloured coatings are matted with StoPur WV 200 transparent, the basic colour shade will become lighter. This should be particularly observed when matting dark and bright colour shades. When applying StoPur WV 200 transparent it is possible that the colour shade differs from the original colour shade of the substrate coating. Roller marks might be visible, due to applying the sealer manually. Ensure sufficient ventilation when applying water-based coating systems. However, avoid draughts. Different layer thicknesses, too high humidity, and too low temperatures can lead to visual defects (differences in the gloss levels). General application instructions can be found at www.stocretec.de (Products) and

Technical Data Sheet
StoPur WV 200 transparent

in the latest issue of the "Technical Data Sheets" manual, in the appendix.

Delivery

	Article number	Designation	Container
	04446/001	StoPur WV 200 Set transparent	8.07 kg set

Storage

Storage conditions	Store in dry and frost-free conditions; avoid direct sunlight.
Storage life	In the original container until ... (see packaging).

Designation

Product group	Sealer
Safety	This product is subject to compulsory designation in accordance with the current EU directive. You will receive an EU Safety Data Sheet with your first order. Please observe the information regarding the handling of the product, its storage, and disposal.

Special notes

The information in this Technical Data Sheet serves to ensure the product's intended use, or its suitability for use, and is based on our findings and experience. Users are nevertheless responsible for establishing the product's suitability and use.
Applications not specifically mentioned in this Technical Data Sheet are permissible only after prior consultation. Where no approval is given, such applications are at the user's own risk. This applies in particular when the product is used in combination with other products.

When a new Technical Data Sheet is published, all previous Technical Data Sheets are no longer valid. The latest version is available on the Internet.

StoCretec GmbH
Gutenbergstr. 6
D-65830 Kriftel

Tel.: +49 6192 401-104
Fax: +49 6192 401-105
stocretec@sto.com
www.stocretec.de

Technical Data Sheet

S-Protect WS405

Aqueous emulsion on organofunctional silanes for the water repellent impregnation of absorbent, mineral substrates

Product Description

- Milky white emulsion based on organofunctional silane systems
- A hydrophobic agent for mineral building materials with high porosity
- Flash point 72°C
- Viscosity at 20 °C: 15 mPa•s
- pH-value (20°C) 6-8

Intended Use

- suited for the water repellent impregnation of porous, mineral substrates
- penetrates deeply in porous, mineral substrates
- shows no formation of sticky silicone films
- vapour permeable protection
- reduces significantly the uptake of water
- reduces the uptake of water soluble pollutants (eg. chlorides)
- protects hair like cracks of up to 0.3mm

Application

The amount of S-Protect WS405 to be applied is dependent on the absorbance of the substrate and the desired penetration depth.
Very absorbent substrates may require up to 1 Litre/m^2 of S-Protect WS405

Application Method

Low pressure spray with block brush to control
Airless spray or Silane pump

Substrate	**Application Rate**
Lightweight Concrete (Poren Panel & Block)	min. 250 ml/m^2
Clinker Masonry	min. 240 ml/m^2
Sand Limestone	min. 400 ml/m^2
Mineral Plaster	min. 200 ml/m^2

The substrates to be treated should be dry and clean. Acceptable surface cleaning methods include honing, chemical cleaning and water blasting. During application outside temperature and surfaces temperatures should be between +5°C and +40°C. S-Protect WS405 should not be applied during strong wind or when rain is imminent.
S-Protect WS405 is applied in a full flood coat normally in 2 coats. On vertical surfaces the solution should run down in visible descending wet curtain achieved by allowing the solution to flow against the surface of the building material without pressure. Most liquid delivery devices eg. airless pump or backpack sprayer are suitable. The material should not be atomized or applied with a brush.
The water repellent effect of S-Protect WS405 can develop within minutes depending on the substrate and weather conditions. The impregnation should be applied continuously and without a break to avoid overlapping.
A substrate which has already been treated will repeal any reapplied aqueous silane/siloxane emulsion solution. Non-absorbent sustrates such as window frames, window sills, plastic fittings, window glass, etc., should be covered before application. Surfaces which accidentally come into contact with S-Protect WS405 can be cleaned with alcohol (spirit) or aqueous chemical solution. Cleaning should be carried out as quickly as possible (within a few hours), otherwise formation of a silicone resin film can make cleaning more difficult. Silicone resin films are best removed using ethanol (or spirit). Plant life should be protected from overspray.
S-Protect WS405 reacts with the interfaces in pores and capillaries for the mineral surface and forms an invisible, water repellent interfacial compounds To determine the exact amount to be applied a sample is required as the porosities of mineral materials varies.
Before application read the Material Safety Data Sheet thoroughly for safely and toxicological data as well as for information on proper transportation, storage and use. The Material Safety Data Sheet is available upon request from Stoanz Ltd.

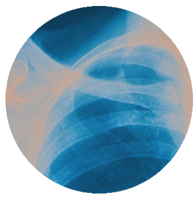
Packaging, Storage and Handling

S-Protect WS405 is supplied in ready-to-use 20 Litre container.
S-Protect WS405 is not resistant to frost and should be stored at temperatures between +3°C and +40°C. S-Protect WS405 has a shelf life of 12 months if stored in originally sealed containers.

HolmesConsultingGroup

REPORT

STRUCTURAL AND CIVIL ENGINEERS



78- 82 BYRON ST

BUILDING A

PREPARED FOR

STEWART FAMILY TRUST

111702

JUNE 2014

78 BYRON ST – DETAILED SEISMIC ASSESSMENT REPORT

BUILDING A – SHEER ADVENTURE WORKSHOP

Prepared For:
STEWART FAMILY TRUST

Date:	27 June 2014
Project No:	111702
Revision No:	1

Prepared By:

David J Herman
DESIGN ENGINEER

Reviewed By:

Richard Seville
PROJECT DIRECTOR

Holmes Consulting Group LP
Christchurch Office

REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
27 June 2014	1	Information

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This report covers the structural damage sustained by the Sheer Adventure Workshop building at 78 Byron St, as a result of the series of earthquakes including the Darfield Earthquake that struck at 4:36am on 4th September, 2010 and the Lyttelton Earthquake that struck at 12.51 pm on the 22nd of February, 2011. The Sheer Adventure Workshop building consists of the original western warehouse structure and the eastern lean-to addition.

The lateral capacity of the building in the east-west direction prior to the earthquakes was found to be approximately 35% of current code. The capacity in the east-west direction was limited by the combined axial loading and bending capacity of the steel portal frames. The lateral capacity of the building in the north-south direction was found to be approximately 70% of current code and was limited by shear in the central ungrouted concrete block infill walls. The external wing-wall extension in the northwest corner of the original structure was found to be 15% of current code and was limited by the out-of-plane capacity of the wall at its base.

The level of shaking experienced at the site is estimated from the Geonet strong motion data recorded at monitoring sites around Christchurch and is related to the fundamental periods of the building for the Lyttelton Earthquake. Given the strong motion data available, it is likely that this earthquake produced accelerations significantly in excess of the design spectra for this building.

Preliminary and detailed observations have been made of the damage sustained as a result of the earthquakes. This report summarises the findings of these detailed observations and provides recommendations regarding the repair work required.

General damage observed consists of cracking and differential settlement of the concrete slab, diagonal cracking along the mortar joints of the concrete masonry unit block walls throughout the original western structure, and failure of a roof bracing strap.

As a result of the earthquakes the building's capacity has likely been reduced in the north-south direction due to diagonal cracking along mortar joints in some of the infill wall panels. The remaining uncracked panels provide the residual lateral capacity in the north-south direction and we estimate the remaining capacity to be greater than 33% of current code. The building's capacity in the east-west direction remains at approximately 35% of current code. The wing-wall extension in the northwest corner of the original structure remains at 15% of current code.

Repairs are recommended in the report and include repairing and repointing of the concrete blockwork and infill wall mortar joints, repair of spalled concrete at portal column bases and beam joints, removing and replacing the slab-on-grade in approximately 40% of the building, and replacing the failed roof bracing tension strap. Note that replacement of slab sections and wall repairs may require the tenants to relocate equipment and operations to the unaffected areas of the building. The northwest wing wall extension should be deconstructed. A fire engineer should be engaged to determine if the wall needs to be replaced. If so, the new wall should be designed for 100% of current loading standard.

Following the repairs recommended herein, the lateral load resisting performance of the building will be restored to its pre-earthquake condition. As noted above, this capacity is limited by the capacity of the steel portal frames in the east-west direction (35% DBE) and the capacity of the infill walls in the north-south direction (70% DBE).

The following further investigations are recommended to confirm analysis assumptions and complete observations of areas of potential damage:

- To confirm foundation assumptions, a geotechnical report should be completed for the site.
- One location of the slab should be broken up along the internal north-south running block wall to confirm block wall foundation and steel portal frame column foundation assumptions.
- The concrete encasement around the steel portal frame column should be broken out at the slab level and the beam-column joint level at several locations to determine if any damage has occurred to the steel portal frames.

All locations should be confirmed with the engineer prior to implementation of any destructive investigations.

This report is considered a live document and will be updated throughout the course of the project with the final report issued once the repairs have been completed.

1. INTRODUCTION

Holmes Consulting Group has been engaged by the Stewart Family Trust to complete a structural review following the Canterbury Earthquakes.

The Darfield Earthquake of 4:36am on 4th September, 2010 and the Lyttelton Earthquake of 12.51 pm on 22nd of February, 2011 have subjected the building to strong ground motions which significantly exceed the full design earthquake load for buildings of this nature. Consequently it is important that a full evaluation is performed.

1.1 PURPOSE

The purpose of this study was to:

- review the impact of the earthquakes on the building
- determine the seismic capacity of the building
- map typical damage around the building
- identify those items requiring repairs or replacement
- specify concept repairs to comply with Christchurch City Council regulations

The overall objective is to ensure that the building is repaired and opened for tenants in as timely and smooth a fashion as possible.

1.2 SCOPE OF WORK

The CERA detailed engineering evaluation procedure is currently in draft form and specific requirements for re-occupation of buildings or applications for Building Consents are not yet certain. As such, this report covers the following cope:-

- Review the available structural drawings to determine the building structural systems and predict areas of likely damage.
- Inspect a sufficient amount of the building structure to be able to make a determination of the behaviour of the building in the earthquake, and to map damage to the structure.
- Repair of damage caused directly by the earthquake.
- Adoption of the new loadings standard (Z=0.3).
- Assessment of Critical Collapse Hazards.
- Prepare a report detailing the proposed repairs required including extent and details.

The likely extent of repairs outlined herein is based on our observations described herein and does not consider any potential changes to the minimum design load levels other than those described above. Further repairs may also be identified during the course of conducting detailed observations.

It should be noted that even after the detailed observations and analysis outlined above, it is difficult to accurately quantify the residual capacity remaining in the structure following this significant earthquake. As such, it is likely that we may never be able to categorically state that the building is as good as it was before the earthquake.

1.3 LIMITATIONS

Findings presented as a part of this project are for the sole use of the Stewart Family Trust, its insurer, and the Christchurch City Council in its evaluation of the subject property. The findings are not intended for use by other parties, and may not contain sufficient information for the purposes of other parties or other uses.

Our observations have been visual only and limited to representative samples, as described in our record of observations. Our observations have been restricted to structural aspects only. Waterproofing elements, electrical and mechanical equipment, fire protection and safety systems, service connections, water supplies and sanitary fittings have not been inspected or reviewed, and secondary elements such as windows and fittings have not generally been reviewed.

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. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

2. STATUTORY REQUIREMENTS

2.1 BUILDING ACT

When dealing with existing buildings there are a number of relevant sections of the Building Act [1] that need to be considered in relation to the building's structure and strength.

Section 112 - Alterations to Existing Buildings

Section 112 of the Building Act requires that a building subject to an alteration continue to comply with the relevant provisions of the Building Code to at least the same extent as before the alteration.

Essentially this section means that the building may not be made any weaker than it was, as a result of any alteration.

Section 122 – Meaning of Earthquake Prone Building

Section 122 of the Building Act 2004 deems a building to be earthquake prone if its ultimate capacity (strength) would be exceeded in a "moderate earthquake" and it would be likely to collapse causing injury or death, or damage to other property.

The Building Regulations (2005) define a moderate earthquake as one that would generate loads 33% as strong as those used to design an equivalent new building.

Section 124 – Powers of Territorial Authorities

If a building is found to be earthquake prone, the territorial authority has the power under section 124 of the Building Act to require strengthening work to be carried out, or to close the building and prevent occupancy.

Section 131 – Earthquake Prone Building Policy

Section 131 of the Building Act requires all territorial authorities to adopt a specific policy on dangerous, earthquake prone, and unsanitary buildings.

2.2 BUILDING CODE

The Building Act requires all new building work to comply with the New Zealand Building Code [2] which outlines the performance standards required for new building work. The Department of Building and Housing also publishes Compliance Documents which may be used to establish compliance with the Building Code.

Following the Lyttelton Earthquake, an amendment to the Compliance Document B1 Structure was published on the 19th May, 2011. This amendment contained changes to the seismic design loads for Canterbury including:-

- 36% increase in the basic seismic design load (Z) for Christchurch (new Z=0.3)
- 85% increase in the basic seismic design load (Z) for Akaroa (new Z=0.3)
- Increased serviceability limitations for new buildings

As a result, a building constructed last year to comply with the Building Code could now have a capacity of just 73% of the new load levels.

2.3 CHRISTCHURCH CITY COUNCIL POLICY

In 2006 the Christchurch City Council (CCC) adopted their Earthquake-Prone, Dangerous and Insanitary Building Policy [3], which was subsequently amended (under urgency) following the 4th September 2010 Darfield Earthquake.

The 2010 amendment outlines a process of identifying Earthquake Prone Buildings due to commence from 1 July 2012. Owners of Earthquake Prone Buildings identified through this process would have between 15-30 years to strengthen the building to a target of 67% of current code as outlined in Section 2.3.3.

Section 2.3.3 – Taking Action on Earthquake-prone Buildings

> … As noted in section 2.3.1 of this Policy, the Council will determine the level of strengthening required to reduce or remove the danger on a building-by-building basis. It will be guided by the Recommendations of the New Zealand Society of Earthquake Engineers that 67% of Full Code Levels is a reasonable target level of strengthening to reduce the risk posed by existing buildings…

The CCC's 2010 policy also includes the following section covering the repair of buildings damaged by an earthquake:

Section 2.3.6 – Buildings Damaged by an Earthquake

> Buildings may suffer damage in a seismic event. Applications for a building consent for repairs will be required to ensure structural strength. The Council will follow sections 2.3.1 and 2.3.3 of this Policy in determining the level of strengthening required for each building.
>
> If a building consent application for repairs is not made and/or the repair work is not completed within a timeframe that the Council considers reasonable the Council reserves the right to serve notice under section 124(1) of the Building Act 2004 to require the work to be done.

The judgement of a recent case before the High Court of New Zealand (CIV 2012-409-2444 [2013] NZHC 51) states that "…territorial authorities may not use section 124 notices to advance a policy of increasing building capacity to a level above 34% of the NBS. However, they are not prevented from requiring work to reduce or remove specific vulnerabilities capable of causing injury, death or property damage where the subject building is also under 34% of the NBS"

While strengthening earthquake prone buildings to levels above 34% is desirable and recommended, this judgement indicates that the Christchurch City Council does not have the authority to require earthquake prone buildings to be strengthened beyond 34% of the NBS.

2.4 CANTERBURY EARTHQUAKE RECOVERY AUTHORITY (CERA)

The Canterbury Earthquake Recovery Authority (CERA) was established on 28th March, 2011 to take responsibility for the recovery of Christchurch by means of the Canterbury Earthquake Recovery Act 2011 [4] which was passed on 18 April, 2011. Under this act, the CEO (of the Canterbury Earthquake Recovery Agency, CERA) has wide powers in respect of verifying building safety and requiring demolition or repairs. Particularly relevant sections are;

Section 38 – Works

(4) If the chief executive gives written notice to an owner of a building, structure, or other erection on or under land that demolition work is to be carried out there, -

(a) the owner must give notice to the chief executive within 10 days after the chief executive's notice is given stating whether or not the owner intends to carry out the works and, if the owner intends to do so, specifying a time within which the works will be carried out; and

(b) if the owner fails to give notice under paragraph (a) or the chief executive is not satisfied with the time specified, or the works are not carried out in the time specified or otherwise agreed, then –

(i) the chief executive may commission the carrying out of the works; and

(ii) in the case of the demolition of a building to which section 40(1) or (2) refers, the chief executive may recover the costs of carrying out the work from the owner of the dangerous building in question; and

(iii) the amount recoverable becomes a charge on the land on which the work was carried out.

Section 51 – Requiring Structural Survey

The chief executive may require any owner, insurer, or mortgagee of a building that he or she considers has or may have experienced structural change in the Canterbury earthquakes to carry out a full structural survey of the building before it is re-occupied for business or accommodation by the owner, a tenant, or any member of the public.

With regard to Section 51, we understand that it is likely that CERA will require a detailed engineering evaluation to be carried out for all buildings not exempt from the Earthquake Prone Building Legislation. At this stage it is not clear whether the detailed evaluation will be required prior to re-occupation.

CERA has recently published a draft procedure for the detailed engineering evaluation. Depending on the outcome of an initial qualitative assessment for a building, a further detailed quantitative assessment may be required. In addition to repair of earthquake damage, strengthening may be required in order to achieve compliance with the performance levels outlined in this evaluation procedure.

Typically the evaluation procedure defaults to achieving the minimum standard set out by the Earthquake Prone Building legislation, which has generally been accepted as achieving an ultimate limit state capacity equivalent to at least 33% current code.

This section discusses the form and capacity of the building prior to the Canterbury Earthquakes. The Sheer Adventures Workshop building at 78 Byron Street was constructed in two different time periods. The main structure was constructed in the 1960's with an addition along the eastern side of the building constructed in 1988. The original structural design for the 1988 addition was completed by Buchanan and Fletcher Ltd. Designers of the 1960's section are unknown. The street facing side of the structure is shown in Figure 3-1.

The information available for the review included the structural drawings for the eastern addition. Construction documents for the original warehouse structure were not available. A geotechnical assessment has not been conducted for this site at the time of this report.

Figure 3-1: Sheer Adventure Workshop – 78 Byron St

3.1 BUILDING FORM

The Sheer Adventure Workshop overall building footprint is approximately 28 m by 26 m. The building is composed of two sections built in separate time periods. The building is located adjacent to buildings on its eastern and southern elevations. The building is structurally separate from the neighbouring buildings. There may be some small separation gap between the buildings, however existing external façade elements and internal linings prevented the direct observation of the gap. There was originally a link (door opening) between the eastern addition and the building to the south. This has since been filled in with a fire rated concrete block wall. The block wall appears to be attached to the southern building and separated from this building. The overall building footprint is illustrated in Figure 3-2.

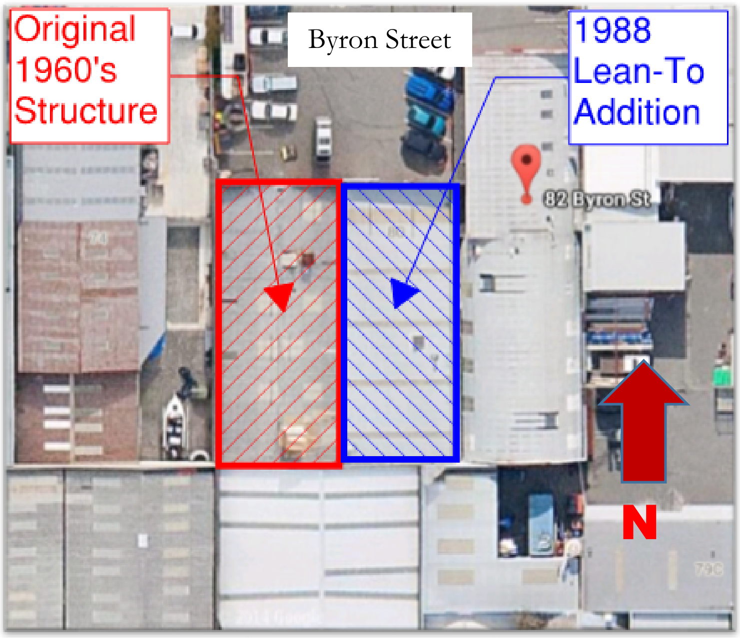


Figure 3-2: Sheer Adventure Workshop – Original Structure and Eastern Addition

The original structure, the western section, was designed and constructed in the 1960's (Figures 3-3 and 3-4). The building was likely designed to predecessor standards of the current NZ Building Code, comprising principally NZS1900:1965. The building is a single story structure, with an internal timber framed mezzanine at the north end of the building. The structure is an open warehouse with steel portal frames running across the building in the east-west direction. The portal frames are composed of I-sections with the columns encased in concrete. Infill walls around the west, south, and original 1960's eastern perimeter of the building are constructed of ungrouted concrete blockwork. There is a solid bond beam along the top of these walls that appears to be reinforced concrete. The roof is composed of diagonal sarking on timber framing with numerous skylights. There is no foundation information available to review however given the type of building and damage observed, the likely construction is concrete slab on grade with shallow pads under portal legs and shallow strip foundations under the walls.

The lean-to addition, the eastern section, was designed and constructed in 1988 (Figures 3-3 and 3-5). This addition was likely designed to the NZS4203 (1984) loadings standard. The structure consists of glulam cantilevered L-shaped frames which span in the east-west direction and are pinned to the original structure's reinforced concrete bond beam. The roof is composed of light gage metal roofing and insulation supported by timber purlins. The south, north, and east walls are composed of light timber framing. A connection doorway to the building adjacent to the south wall has been in-filled with a grout filled concrete block wall. The wall appears to be tied into the structural concrete blockwork walls of the adjacent structure, and are therefore not a part of this building. There is a short height timber framed mezzanine at the north end.

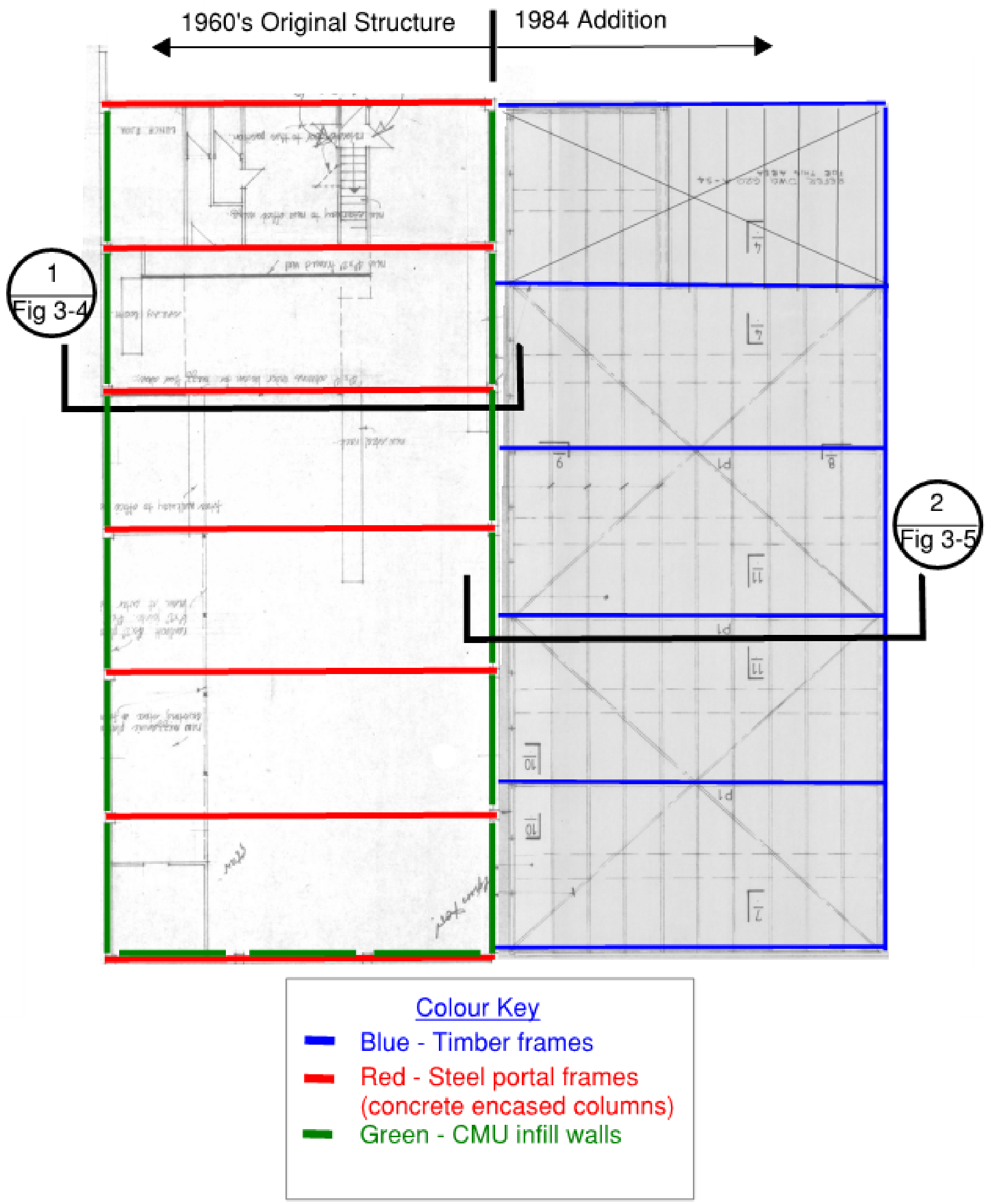


Figure 3-3: Sheer Adventure Workshop – Plan View of Building

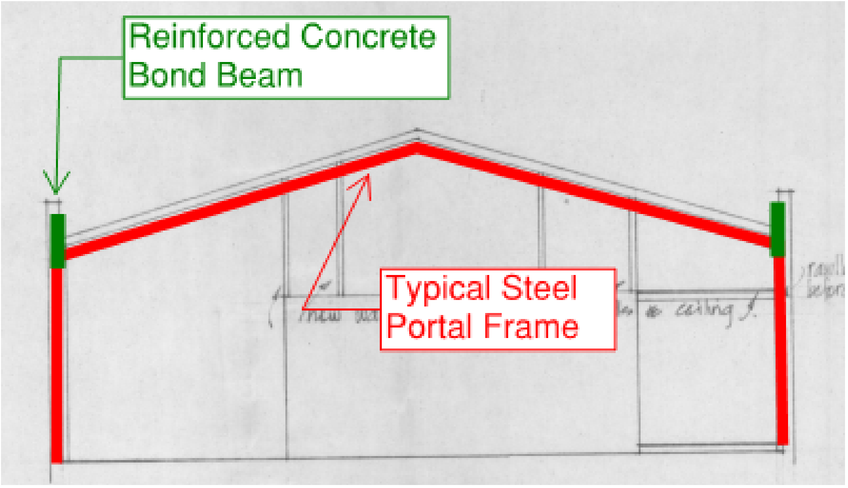


Figure 3-4: Section 1 - 1960's Original Steel Building Cross Section

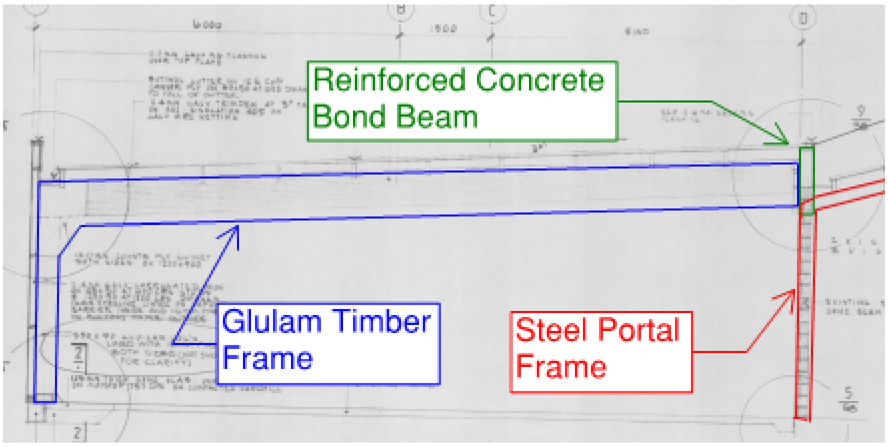


Figure 3-5: Section 2 - 1980's Eastern Addition

3.2 LATERAL LOAD RESISTING SYSTEM

In the east-west (across) direction lateral loads for the original structure at the roof level are distributed by the diagonal timber sarking and are resisted, typically, by the steel portal frames. Lateral loads at the roof level from the eastern lean-to addition are distributed to the timber glulam frames via the purlins. The lateral loads from the walls are transferred directly to the timber glulam frames. The timber glulam frames in the addition and the original steel portal frames are connected by the concrete bond beam of the original structure. The timber glulam frames and original steel portal frames are offset, as shown in Figure 3-3. Lateral loads are resisted by a combination of frame action in the glulam frame and the steel portal frames.

In the north-south direction in the original structure, lateral loads at the roof level are distributed by diagonal timber sarking to the reinforced concrete bond beam and resisted by the ungrounted concrete block infill walls between the portal frames. Lateral loads at the roof level of the lean-to addition are distributed by tension only steel strap bracing to the gib lined timber wall along the western edge of the building and to the reinforced concrete bond beam in the centre of the building. Lateral loads are resisted by the gib lined timber perimeter wall and the ungrouted concrete block infill walls below the reinforced concrete bond beam.

3.3 PRE-EARTHQUAKE BUILDING CAPACITY – DIRECT CODE COMPARISON

The original structure, the western building, was likely designed to the NZS1900:1965 loadings standard. The eastern addition, constructed in 1988, was likely designed to the NZS4203 (1984) loadings standard. Both standards have been replaced by NZS1170.5:2004 [8]. The recent amendment to the Building Code has further increased the load levels defined in NZS1170.

Based upon building occupancy, the Sheer Adventures Workshop has been considered as an Importance Level 2 (IL2) building in accordance with NZS1170.5:2004 [8]. For a building design life of 50 years, the associated return period of the Design Base Earthquake (DBE) is 500 years. Per the loadings standard for a warehouse structure as prescribed in this code (no post-disaster or special function), the risk factor for design is taken as R=1.0. The sub soil for the entire site has been assumed to be Soil Type D in accordance with NZS1170, which is typical of this location in Christchurch. The fundamental period of the structure was assumed to be 0.4s for out-of-plane and in-plane ungrouted concrete block walls. The fundamental period of the steel portal frame with lean-to frame addition was estimated with the Raleigh method to be approximately 1.0s.

Based upon the information available, we have assumed the following levels of ductility:

- in-plane lateral loads for frames – nominally ductile, μ= 1.25
- in-plane lateral loads for ungrouted concrete block walls – elastic, μ= 1.0
- Out-of-plane lateral loads for ungrouted concrete block walls – elastic, μ= 1.0

Comparisons of the load levels represented by the original loading standards to the current loading standard are plotted below for a building fundamental period of 0.4s. The seismic design loads from NZS1900:1965 are approximately 15% of current design loads. Seismic loads from NZS4203(1984) are approximately 35% of current design loads.

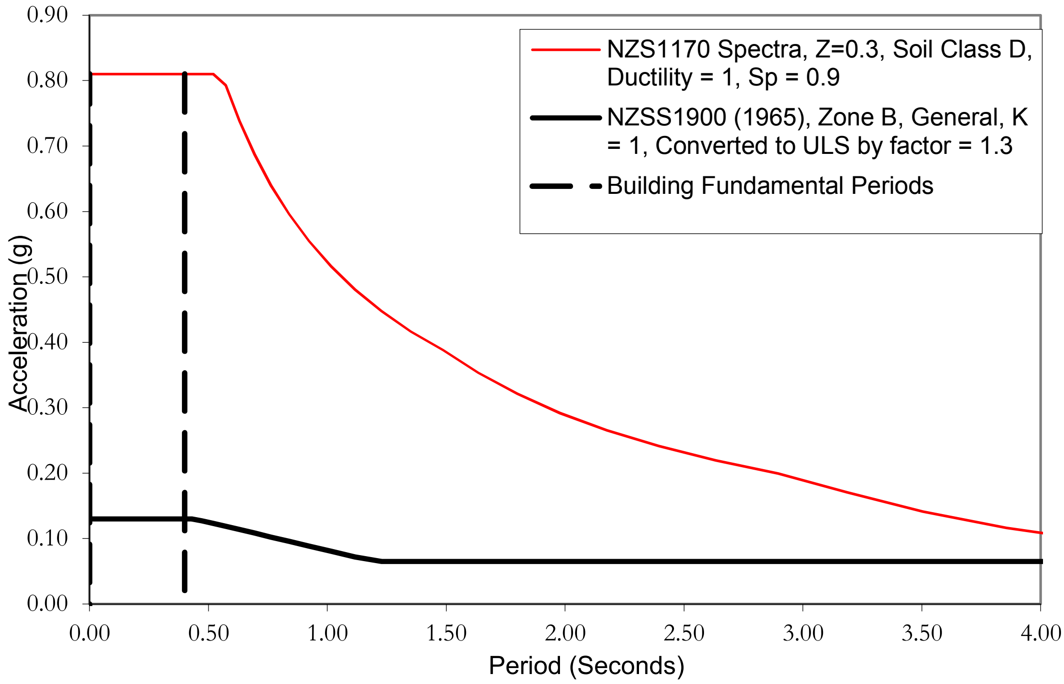


Figure 3-6: Comparison of Design Codes (NZSS1900 to NZS1170)

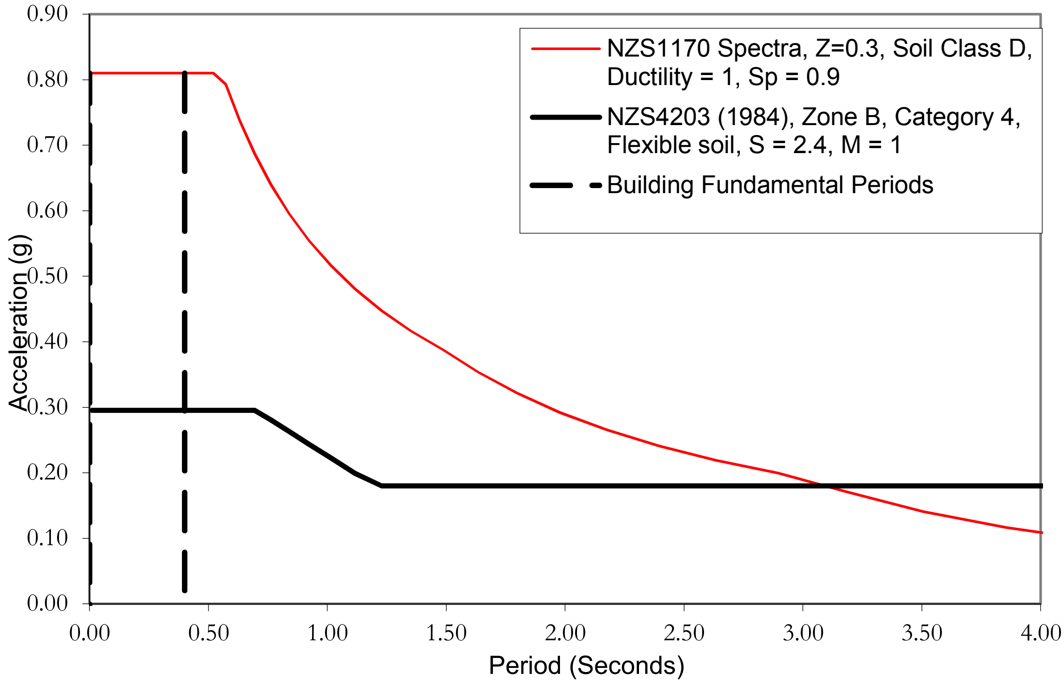


Figure 3-7: Comparison of Design Codes (NZS4203 to NZS1170)

3.4 PRE-EARTHQUAKE BUILDING CAPACITY – EQUIVALENT STATIC ANALYSIS TO NZS1170.5 (2004)

In addition to the direct code comparison provided above, an equivalent static analysis to NZS1170.5:2004 has been carried out to gain a better understanding of the buildings capacity when compared to current loading standards. The equivalent static analysis was carried out based upon the original construction documents available, incorporating on-site measurements and as-built observations.

The probable capacities have been calculated using the *New Zealand Society for Earthquake Engineering Guidelines for the Assessment of the Structural Performance of Buildings in Earthquakes* – NZSEE 2006 [13]. The guidelines allow some relaxation of the requirements for existing buildings when compared to what would be required for a new building. As a result existing buildings shown to achieve 100% of current code loading may not achieve the same level of seismic performance as a new building designed to achieve minimum compliance with the building code.

Account is also made of critical structural weaknesses. Critical structural weaknesses (CSW) are details, configurations, and building or site characteristics that could lead to increased damage levels in a building or the premature failure or collapse of all or part of a building.

To provide a comparison for each of the primary lateral components, the relative capacity of the elements have been assessed as a percentage of the demand imposed by the current loading code Design Basis Earthquake, and have been expressed as a %DBE. This includes checks for both strength and deflection requirements.

The demand on the various structural elements of the system were determined by performing a three dimensional analysis on the main framing elements. A three-dimensional model was constructed and analysed in an analysis program based on information contained on the available construction drawings and from on-site measurements. Based on the time period and type of building construction, some fixity in the portal frame foundations was assumed in the model.

To determine the design loads imposed by the current loading standard, the building natural periods were estimated to be 0.4s in the north-south direction and approximately 1.0s in the east-west direction. The fundamental period in the east-west direction was determined using the Raleigh Method described in the loading standard. From the structure's natural period and ductility, as listed in Section 3.3, the loading demand imposed by the current standard was determined and applied to the analysis model. The demand on the various structural components was obtained from the analysis results and compared to the calculated capacities to determine the %DBE of each element.

The calculated seismic capacity in the east-west direction of the Sheer Adventures Workshop is approximately 35% DBE. The limiting elements in the structure are the east-west steel and glulam portal frames. The table blow outlines the capacities of various elements in the structure.

The limiting factor in the north-south direction for the capacity of the Sheer Adventures workshop is the central ungrouted concrete block work infill walls which are at 70% DBE and limited by sliding shear of the infill walls.

Due to the flexible nature of the portal frames, large deflections are likely to occur, especially in at the western edge of the building. If the seismic gap with the adjacent building is insufficient, pounding is likely to occur.

To the north of the western concrete masonry wall of the original warehouse structure there is an extension of the concrete masonry block wall that appears to cantilever from the ground. There does not appear to be any dowel connection between the original wall and the extension. Assuming the wall is fully grouted with minimum reinforcement, the wall is at approximately 15% DBE and limited by out-of-plane loading.

Table 3-1: 78 Sheer Adventure Workshop – Seismic Assessment %DBE

Building Element	**%DBE (IL2)**	**Comments**
Ceiling Diaphragm – NS - EW	100 100	Diagonal sheathing in west side of building
Ceiling Tension Strap Bracing	95	Bracing is limited by tensile capacity
GluLam Portal Frames	35	Glulam frame is limited by combined bending and axial loading in the column
Steel Portal Frames	35	Steel portal frame is limited by combined bending and axial loading in beam element
Gib lined timber framed wall	70	Capacity limited by nail slip
Concrete Bond Beam	50	Capacity limited by out-of-plane flexural capacity due to assumed reinforcement layout.
Ungrouted concrete masonry block infill walls – in plane	70	Capacity limited by sliding shear
Ungrouted Concrete masonry block infill walls – face loading.	65	Capacity is limited by south wall block wall spanning between ground and roof
External Concrete Masonry Block wall – face loading	15	Capacity is limited by moment capacity at base of cantilevered wall

4.1 EARTHQUAKE SHAKING EXPERIENCED AT THE SITE

The Geonet Project, run by EQC and GNS Science, maintains the New Zealand National Seismograph Network which consists of a series of strong motion seismometers set up around New Zealand. The following image shows the location of the four closest monitoring stations to the building.

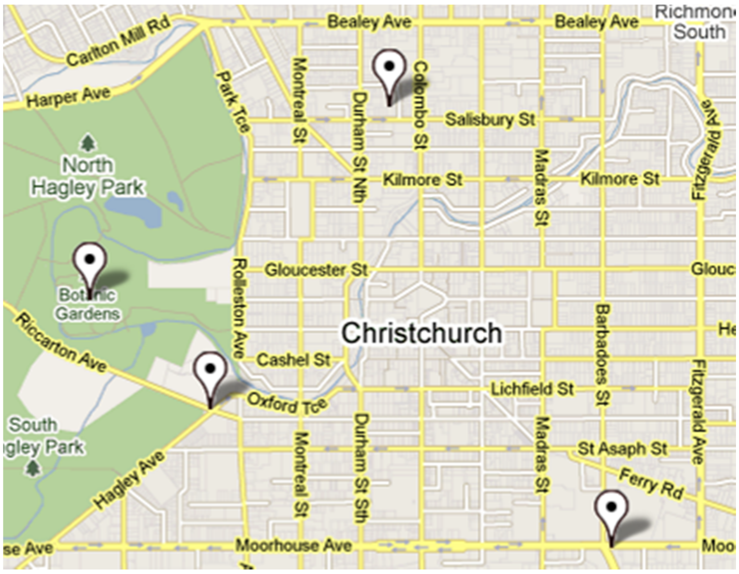


Figure 4-1: Location of Nearby Monitoring Stations

The strong motion shaking data resulting from the Darfield and the Lyttelton Earthquakes has been downloaded from these monitoring stations and processed to obtain acceleration response spectra (a response spectra essentially defines the peak response for a building subjected to the ground shaking, as a function of its fundamental period).

The accelerations recorded from the Lyttelton Earthquake are generally larger than those from the Darfield Earthquake, therefore these are presented and discussed in this section.

The following graphs plot the acceleration response spectra processed from the Geonet monitoring stations for the initial main shock of the Lyttelton Earthquake at 12:51pm on the 22nd February, as well as the elastic design spectra (NZS1170) for a new building constructed on the site. For reference the fundamental period of the building has been plotted on the graphs of the North-South and West-East directions respectively.

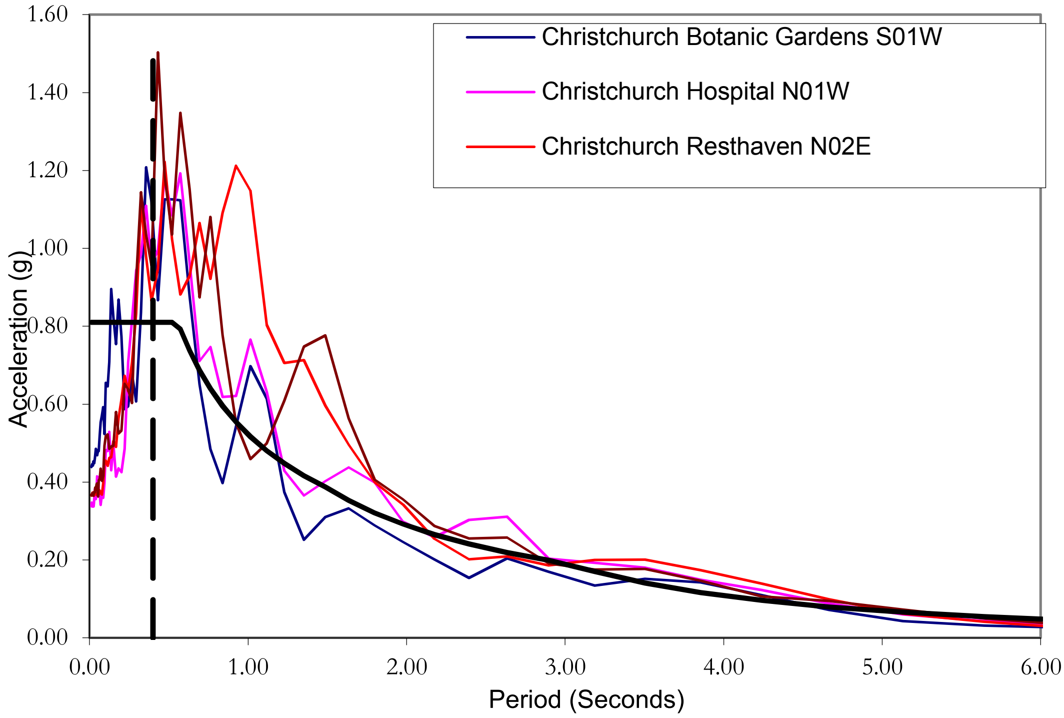


Figure 4-2: 5% Damped Spectra – North-South

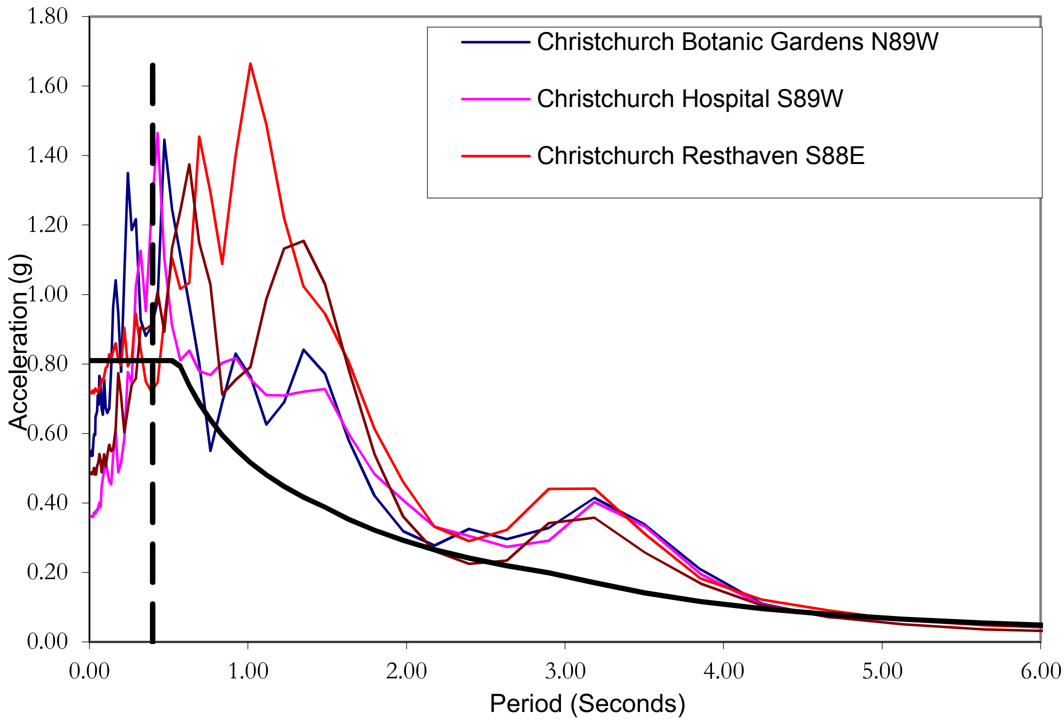


Figure 4-3: 5% Damped Spectra – East-West

It is apparent that in both directions there is significant variation in the shaking experienced at the different monitoring sites. This is due to the highly variable ground conditions around Christchurch.

The Shear Adventure Workshop building has been determined to have a fundamental period to less than 0.4 seconds (north-south) and (east-west). Based on the strong motion data downloaded, it is likely that the earthquake produced accelerations significantly in excess of the design spectra for this building.

Modern office buildings, typically designed to an importance level of 2 (IL2), are designed to resist an earthquake with a return period of 500 years at the Ultimate Limit State (ULS). The magnitude of the ground accelerations recorded in the CBD are in the order of 150% to 200% of the current design code at the time of the earthquake, and are some of the strongest ever recorded in the world. This intensity of shaking is roughly equivalent to an earthquake in Christchurch having a return period of 2500 years, deemed to be the Maximum Credible Event (MCE). At this level of shaking, modern buildings designed to the current codes in place at the time are expected to be at the point of collapse.

However it should also be noted that this earthquake was relatively short in terms of the strong shaking produced. The following plot of the earthquake record from the Christchurch Hospital monitoring station at 12:51 pm on the 22nd of February shows that the strong motion only lasted for a duration of approximately 5-10 seconds.

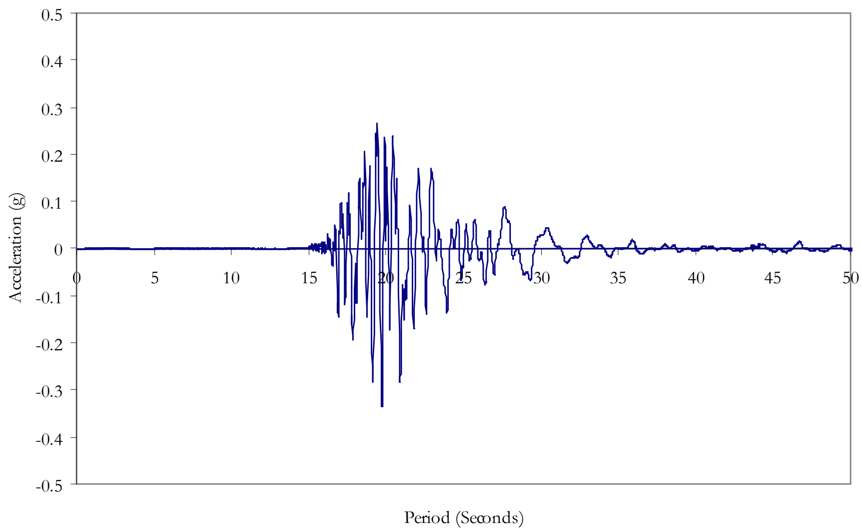


Figure 4-4: Earthquake Record from Christchurch Hospital Site

Because of this the building has only gone through a limited number of inelastic cycles. A full design earthquake for Christchurch (eg rupture of the Alpine Fault) is expected to have a significantly longer record of shaking, although the accelerations are not expected to be as strong. As an indication, rupture of the Alpine Fault is expected to contain in excess of 60 seconds of strong motion.

Due to the highly variable ground conditions around Christchurch, it is impossible to determine what the actual shaking experienced at the site was. However, based on the data described above it is likely that the shaking experienced by the building could have exceeded the current code design spectra for the building

4.2 PRELIMINARY INVESTIGATIONS

Preliminary investigations have been undertaken to ascertain areas of the building likely to be subject to damage, and therefore requiring specific attention during the detailed assessment. The areas identified for detailed inspection have been selected based on:-

- typical damage expected for buildings of this form;
- a review of the original drawings [6];
- review work undertaken to date;
- damage observed following the earthquakes.

In conjunction with a review of the structural drawings and previous damage review associated with this building the following areas were identified for potential damage:-

- Cracking of concrete block work wall mortar joints
- Distress at portal frame moment connections
- Differential settlement and lateral spreading of slab on grade
- Differential settlement and lateral spreading of walls and foundations

4.3 SUMMARY OF BUILDING DAMAGE

The following is a summary of the structural damage observed.

- In plane diagonal cracking along concrete block wall mortar joints
- Cracking along slab construction joints
- Differential settlement and lateral spreading of slab on grade
- Out of vertical walls at central block work wall and southern wall of eastern structure
- Broken tension only strap cross bracing in eastern structure

4.4 FURTHER INVESTIGATIONS REQUIRED

No geotechnical information is available for the building site. A geotechnical report should be completed to assess the ground conditions at the site.

Destructive investigations should be conducted to examine the condition of the steel portal frame beam to column connections ensure no damage occurred.

The connection between the reinforced bond beam and the steel portal frames as well as the steel portal frame foundations should be exposed and observed to confirm analysis assumptions.

5. DAMAGE OBSERVED & REPAIRS REQUIRED

5.1 TYPICAL DAMAGE OBSERVED & REPAIRS REQUIRED

Table 5-1 provides a photographic summary of the observed damage and typical repairs required. The table should be read in conjunction with Appendix A – Damage Photo Location Map and Appendix B – Record of Observations. The Repair Specification is attached in Appendix D.

Generally the aim of the repair work indicated is to restore the structure to its pre-earthquake state as far as practicable. Specifically the repair work described herein does not include the strengthening required to comply with the relevant regulations outlined in Section 2. It should be noted that more damage may be identified during the repair works and (if required) additional repair details will be specified accordingly.

The damage outlined herein does not include the necessary repair or replacement of architectural items, services, etc, or the removal of such items necessary to carry out structural observations. We recommend appropriate professionals be engaged to carry out scoping of these non-structural works.

Table 5-1: 78 Sheer Adventure Workshop - Photographic Summary of Primary Damage Observed & Repairs Required

Damaged Item	Location	Example	Recommended Repair
1. Slab on Grade			
1.1. Cracking and spreading at slab construction joints	South section of eastern structure		Chase channels each side of crack and install dowels across crack to prevent future lateral spreading of slab.
1.2. Differential settlement of slab	South section of eastern structure		Remove section of slab that has lifted. Re-level ground. Install new slab with epoxy dowels into adjacent slab sections.

Damaged Item	Location	Example	Recommended Repair
1.3. Differential settlement of slab	Southern wall at internal wall junction with slab in eastern structure.		Remove section of slab that has lifted. Re-level ground. Install new slab with epoxy dowels into adjacent slab sections.
2. Concrete Block Infill Walls			
2.1. Diagonal cracking along mortar joints in concrete block infill walls	Occurs along the west, south, and east walls of the original factory building (west structure)		Repair and repoint all mortar joint cracks from both the inside and outer faces.

Damaged Item	Location	Example	Recommended Repair
2.2. Vertical cracking in concrete block work wall	Occurs in north section of west wall		Remove and replace cracked concrete blocks. Repair and repoint cracked mortar joints.
2.3. External concrete block wing wall extension has moved creating a gap in the upper portion	External concrete wing wall at northwest corner of building		Remove wing-wall extension or strengthen for face loads. Provide temporary propping while the fire requirements are confirmed and the solution for the wall is determined.

Damaged Item	Location	Example	Recommended Repair
3. Western structure portal frames			
3.1. Cracking and spalling of concrete column to steel beam connection	Top of west portal frame column at west boundary wall		Remove concrete and observe beam to column connection to ensure no signs of distress. If no signs of distress, repair concrete encasement of column per repair specification.
3.2. Cracking and spalling of concrete column at slab level	Occurs at several locations in the western structure		Remove concrete and observe column at base to ensure no signs of distress. If no signs of distress, repair concrete encasement of column per repair specification.
4. Roof			
4.1. Broken roof tension strap	Eastern structure		Replace broken strapping. Refer to Section 5.2.

5.2 ROOF TENSION STRAP CROSS BRACING

In the eastern lean-to addition, a roof tension strap cross bracing member has failed in tension and snapped apart. This is likely due to the difference in stiffness of the eastern gib lined timber framed wall and the internal concrete blockwork infill walls. As determined in section 3, the capacity of the tension strap bracing is at 95% DBE.

The strap brace should be replaced. To improve the performance of the structure in future seismic events, the gib wall along the eastern boundary should be stiffened with additional bracing or additional strap bracing should be installed.

5.3 NORTHWEST WING WALL

In the northwest corner of the original 1960's warehouse structure, a concrete masonry block wall wing wall extension has, at some point, been added along the western boundary wall. This wall does not appear to be tied to the original wall or concrete bond beam and cantilevers from ground level. The out-of-plane capacity of the wall is 15% DBE. There are signs of lateral movement at the base of the wall and along the joint with the original wing wall end.

The wall extension may have been installed due to fire requirements. Temporarily prop the wall until the fire requirements are investigated and a strengthening solution can be determined.

5.4 DIFFERENTIAL SETTLEMENT REMEDIATION

In addition to the damage and repairs outlined in Table 5-1, differential settlement of the slab and foundations was identified onsite and in the levels survey. From the levels survey, the slab-on-grade and walls appear to have settled in the earthquake. The general pattern of settlement inside the western half of the building is settlement of the external wall foundations relative to the internal slab causing bowing in the slab. This is likely due to the heavier loads carried by the perimeter walls. The differential settlement along the wall foundations is typically less than 20mm. The differential settlement between the slab and the wall foundations is generally 35 mm or less. There is a high spot in the centre of the western half of the building.

In the eastern half of the building, settlement was generally relatively even throughout the interior with low spots at the south and north ends of the building. This is likely due to the light weight timber construction of the addition.

Slopes, throughout the building, are typically around 0.50% (1:200). In aa localised area towards the south of the eastern lean-to addition, the slope is approximately 0.8% (1:130). Near this location, the slab has cracked along a construction joint. The differential settlement across the construction joint is approximately 10 mm.

5.4.1 Repair Work to Return the Building to Original Level

Based on these observations, the settlements affecting the foundations and walls may be acceptable and not require re-levelling. The bowing in the ground floor slab can be addressed by locally demolishing and replacing the existing slab-on-grade. The areas of slab to be replaced include the southern end of both the western and eastern parts of the building as well as the northern area of the eastern half of the building. Refer to Appendix C for the extent of slab replacement.

The existing slab-on-grade to be demolished should be replaced with a new 125mm slab on DPM on sand binding on 150 mm of compacted hard fill. Allow for reinforcing in the slab of 12mm diameter deformed reinforcing bars at 300mm centres each way. The demolition is to be such that the dowels connecting the foundation beam to the slab remain in place and are cast into the new slab. Allow to epoxy inject D12 starters at 300 mm into the existing slab and foundations around the perimeter of the new slab.

Note that this option will return the structure to its pre-earthquake level and lateral capacity, but does not address the mitigation or prevention of differential settlements occurring in future seismic events. Differential settlements could reduce the lateral capacity of the structure. However, as indicated above, there has not been significant differential settlement between existing foundations in recent events. The mitigation of future differential settlement should be considered in any future strengthening scheme.

6. POST-EARTHQUAKE BUILDING CAPACITY

6.1 POST-EARTHQUAKE BUILDING CAPACITY

In its damaged state following the earthquakes, we consider the Sheer Adventures workshop to have no reduction in gravity load resistance. The overall lateral load resisting capacity of the building in the north-south direction has been affected due the snapping of the eastern roof tension strap bracing and due to diagonal cracking along the mortar joints of the ungrouted concrete blockwork infill walls. The roof tension strap bracing that has snapped has reduced the roof bracing capacity from 95% to 75% DBE. Diagonal cracking along the mortar joints of the concrete blockwork walls has reduced the lateral load resisting capacity of the walls to approximately 35% DBE. This has been determined by considering the blockwork infill wall panels that remain uncracked as well as accounting for some contributions to capacity from the damaged walls per NZSEE 2006. Distribution via the reinforced concrete bond beam of the lateral loads to undamaged infill wall segments is likely to occur.

The overall lateral load resisting capacity of the building in the east-west direction has not been significantly reduced due to the damage observed after the earthquakes.

Following the recommended repair of the structural damage, the lateral load resisting performance of the structure will be restored. The capacity of the primary structural system in the east-west direction will approximately 35% of DBE and in the north-south direction will be 75% DBE.

Following the recommended repair of the consequential damage the durability performance of the structure should be restored. However this should continue to be monitored throughout the life of the building to ensure that the sealants are maintained to prevent moisture penetration, and that any future cracking is repaired.

7. REFERENCES

1 *Building Act 2004*, New Zealand Government

2 *New Zealand Building Code*, New Zealand Government

3 *Earthquake-Prone Dangerous and Insanitary Buildings Policy 2010*, Christchruch City Council, 2010

4 *The Canterbury Earthquake Recovery Act 2011*

5 78-82 Byron Street – EQ Damage and Repair, Structex, September 2013

6 Temporary Warehouse 82 Byron St Christchurch, Holmes, Wood & Poole, April 1975

7 Survey Report for Building Survey at 78 & 82 Byron St, Christchurch, Staig & Smith, August 2013

8 *Structural Design Actions part 5: Earthquake Actions – New Zealand, NZS 1170.5:2004*, Standards New Zealand, 2004

9 *Timber Framed Buildings, NZS 3604:2011*, Standards New Zealand, 2011

10 *Steel Structures Standard, NZS 3404:1997*, Standards New Zealand, 1997

11 *Timber Structures Standard, NZS 3603:1993*, Standards New Zealand 1993

12 *Design of Reinforced Concrete Masonry Structures, NZS 4230:2004*, Standards New Zealand, 2004

13 *Assessment and Improvement of the Structural* Performance of Buildings in Earthquakes, NZSEE-2006, New Zealand Society for Earthquake engineering, 2006

APPENDIX A

Damage Map

HolmesConsultingGroup

Project Name: *78 Byron Street DEE Building A*

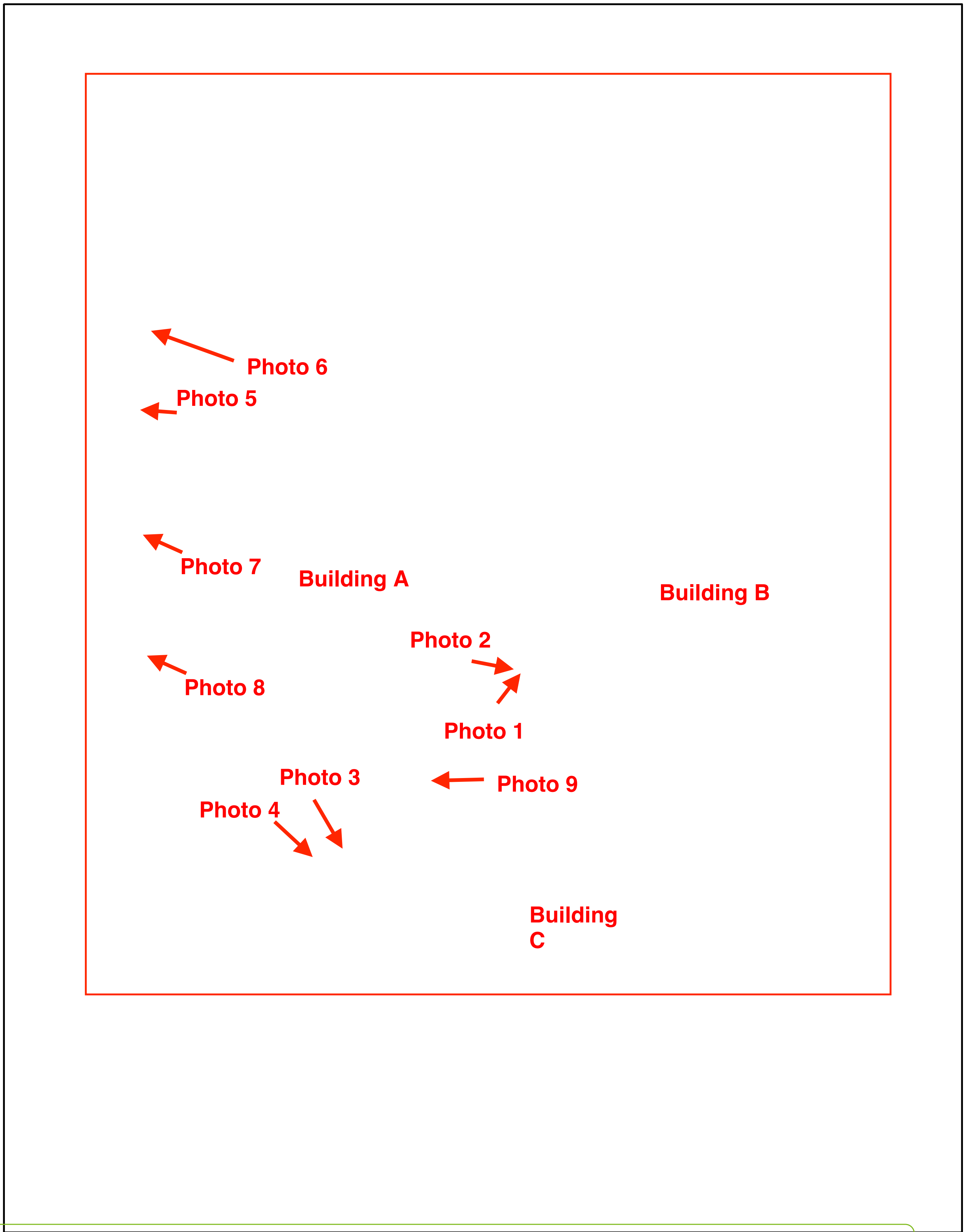
Project No: *111702*

Calcs By: *DJH*

Date: *24/06/14* Page No: *1*

Sketch No: *APA-SK01* Revision: *1*

CALCS/SKETCHES



APPENDIX B

Record of Observations

Damaged Item	Location	Example	Recommended Repair
1. Slab on Grade			
1.1. Cracking and spreading at slab construction joints	South section of eastern structure See damage map photo 1.		Chase channels each side of crack and install dowels across crack to prevent future lateral spreading of slab.
1.2. Differential settlement of slab	South section of eastern structure See damage map photo 2.		Remove section of slab that has lifted. Re-level ground. Install new slab with epoxy dowels into adjacent slab sections.

Damaged Item	Location	Example	Recommended Repair
1.3. Differential settlement of slab	Southern wall at internal wall junction with slab in eastern structure. See damage map photo 3.		Remove section of slab that has lifted. Re-level ground. Install new slab with epoxy dowels into adjacent slab sections.
2. Concrete Block Infill Walls			
2.1. Diagonal cracking along mortar joints in concrete block infill walls	Occurs along the west, south, and east walls of the original factory building (west structure) See damage map photo 4.		Repair and repoint all mortar joint cracks from both the inside and outer faces.

Damaged Item	Location	Example	Recommended Repair
2.2. Vertical cracking in concrete block work wall	Occurs in north section of west wall See damage map photo 5.		Remove and replace cracked concrete blocks. Repair and repoint cracked mortar joints.
2.3. External concrete block wing wall extension has moved creating a gap in the upper portion	External concrete wing wall at northwest corner of building See damage map photo 6.		Remove wing-wall extension or strengthen for face loads. Provide temporary propping while the fire requirements are confirmed and the solution for the wall is determined.

Damaged Item	Location	Example	Recommended Repair
3. Western structure portal frames			
3.1. Cracking and spalling of concrete column to steel beam connection	Top of west portal frame column at west boundary wall See damage map photo 7.		Remove concrete and observe beam to column connection to ensure no signs of distress. If no signs of distress, repair concrete encasement of column per repair specification.
3.2. Cracking and spalling of concrete column at slab level	Occurs at several locations in the western structure See damage map photo 8.		Remove concrete and observe column at base to ensure no signs of distress. If no signs of distress, repair concrete encasement of column per repair specification.
4. Roof			
4.1. Broken roof tension strap	Eastern structure See damage map photo 9.		Replace broken strapping.

APPENDIX C

Conceptual Repair Plan

HolmesConsultingGroup

Project Name: *78 Byron Street DEE - Building A*

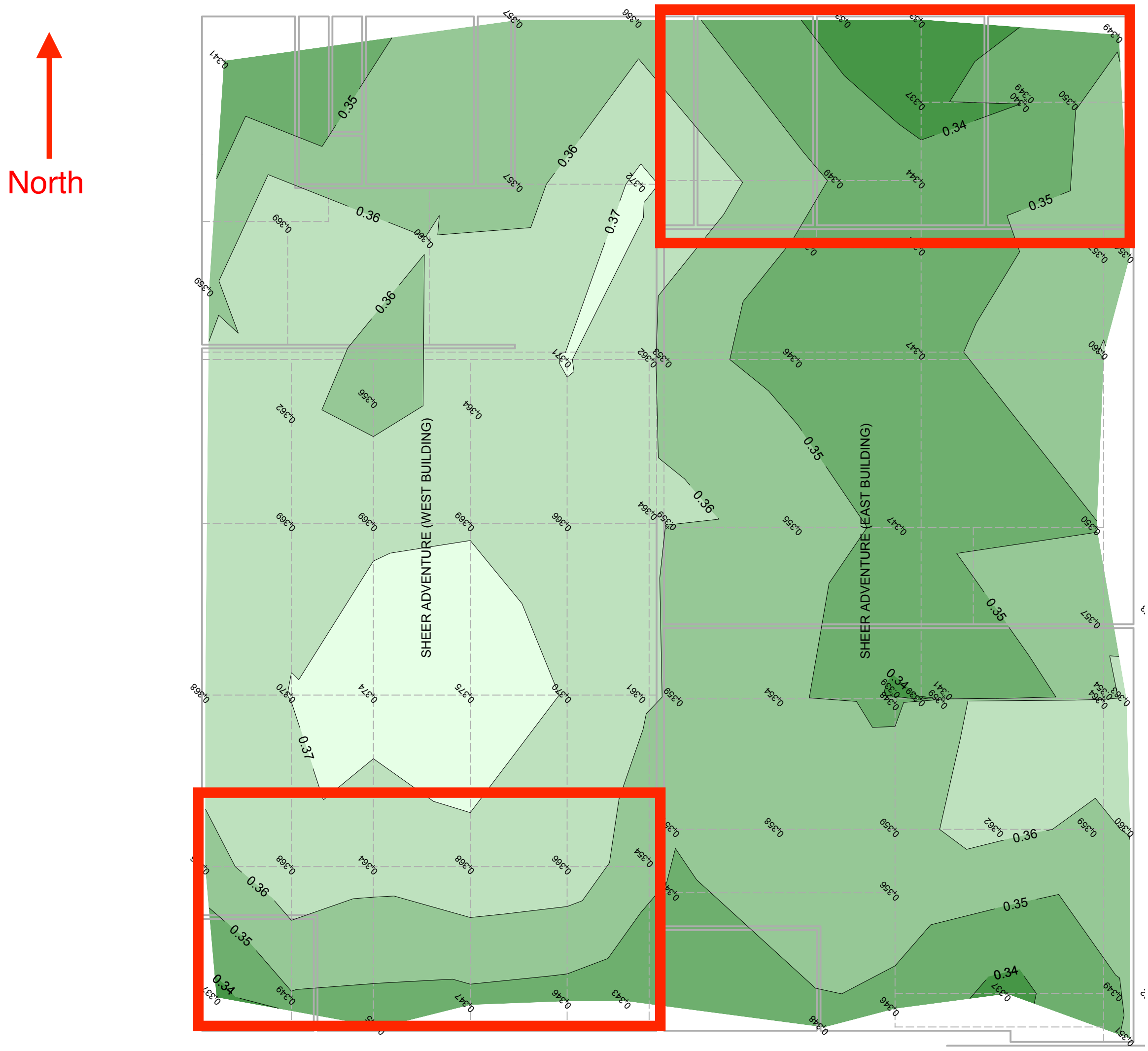
Project No: *111702*

Calcs By: *DJH*

Date: *24/06/14* Page No: *1*

Sketch No: *APC-SK01* Revision: *1*

CALCS/SKETCHES



- Area of slab to be broken out and replaced with D12 starters epoxied into existing slab or foundations/walls to remain.

APPENDIX D

Building Alignment Survey

ENHANCING THE LAND

SURVEY REPORT FOR BUILDING SURVEY AT
78 & 82 BYRON ST, CHRISTCHURCH

Introduction
Staig and Smith Ltd were engaged to carry out a survey of the buildings at 78 & 82 Byron St. The site comprises four individual buildings; the eastern and south-eastern buildings are tenanted by Socks Pacific and the two western buildings are tenanted by Sheer Adventure.

Diagram of the buildings surveyed.

The aim of the survey was to measure the building following the recent significant earthquakes in Christchurch.

The work involved surveying the following elements:

- Floor levels and gradients
- Internal column verticality

Following the survey, a set of plans have been prepared detailing the results from the survey.

Methodology
The survey was carried out on the 28th and 29th August 2013.

<u>Levels</u>
Levels were surveyed with a Leica 150M Sprinter Digital Level (SN 2111297). This instrument has a measuring standard deviation of 0.6mm (electronic) and 1.2mm (optical) at 30m. Closed level loops were observed. Levels were positioned relative to the walls. In the Socks Pacific showroom the level positions were based on a grid using

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Surveying, Planning, Engineering & Resource Management Consultants · e. enquiries@staigsmith.co.nz · www.staigsmith.co.nz
81 Selwyn Place, PO Box 913, Nelson 7040 · 12 Bealey Avenue, Christchurch 8014 · p. 0800 807 818

the portals and concrete slabs. In the remainder of the Socks Pacific the level positions were based on a 3x3 grid on each individual concrete slab. In the Sheer Adventure buildings the level positions were based on the concrete slab edges.

Contours have been provided at 10mm (0.01m) intervals.

Assumed datums have been used for the level surveys and origin marks are shown and labelled on the plans.

Any noticeable cracks or steps in the slab were surveyed and have been shown on the plan. The thickness of the different floor coverings was measured and levels have been adjusted so that all levels are in terms of the concrete surface.

Verticality Alignment

Verticality measurements were made with a Leica TCRP 1202 Robotic Total Station (SN 227215). This instrument has measuring standard deviations of 2" for angle and 2mm+2ppm for prism distance and 3mm+2ppm for reflectorless distance.

Measurements were made directly onto the walls/columns using reflectorless EDM measurements from the total station. The top and bottom of the columns/walls were measured wherever practical.

Analysis

Socks Pacific Northern Building

The floor contours and levels indicate that the central part of the floor is generally lower than the outside of the floor, with particularly low areas seen at the central southern, central middle and central northern parts of the floor. There are noticeably high areas at the central western and central eastern parts of the floor. Note the very outer edge of the floor, which is outside the main floor slabs, drops away significantly.

The floor slope plan shows maximum slopes adjacent cracks and on the outer edge of the floor as expected. Large areas of the floor, mainly in the outer areas, have gradients of 0.5% or more.

No verticality measurements were made in this building.

Socks Pacific Southern Building

The floor contours and levels indicate that the central part of the floor is generally higher than the outside of the floor. There is a noticeably low area at the south-eastern corner of the floor.

The floor slope plan shows that the majority of the floor has gradients of 0.5% or more, particularly the southern part of the floor.

The verticality values show maximum deviations from vertical near the southeast and southwest corners of the building.

Sheer Adventure Eastern Building

The floor contours and levels indicate that the central part of the floor is generally higher than the outside of the floor, as well as low spots on the central northern and central southern edges of the floor. There is a noticeably high area near the south eastern corner of the floor.

The floor slope plan shows maximum slopes adjacent cracks as expected. Isolated areas of the floor, mainly in the outer areas, have gradients of 0.5% or more.

The verticality values show significant deviations from vertical on the eastern wall of the building. Note that this wall, including the columns, is lined so measurements have not been made directly to the structural elements.

<u>Sheer Adventure Western Building</u>
The floor contours and levels indicate that the central part of the floor is generally lower than the outside of the floor. The southern edge and north-western corner of the floor are noticeably lower than the rest of the floor.

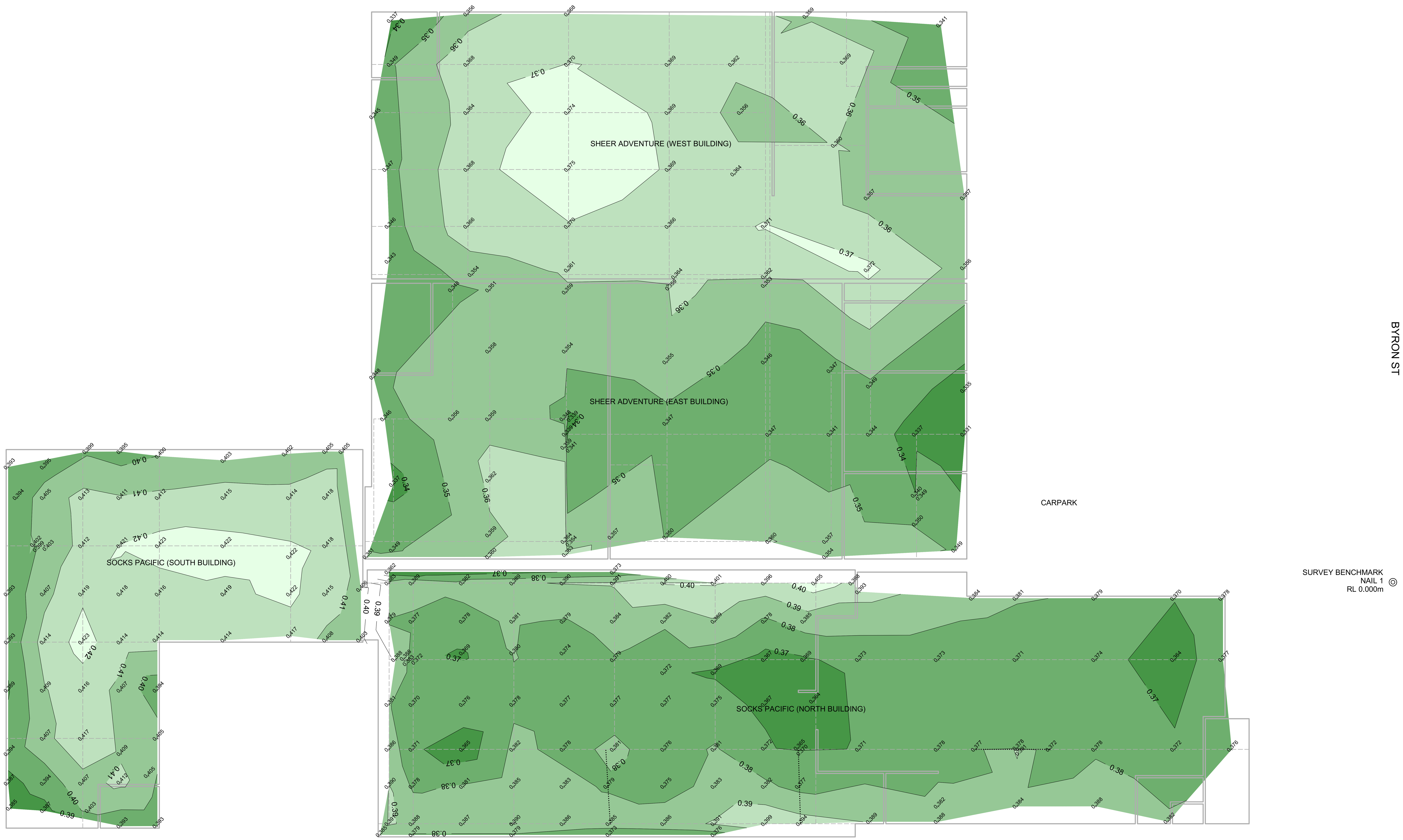
The floor slope plan shows maximum slopes at the interface to the adjacent building to the east. The southern and northern areas of the floor generally have gradients of 0.5% or more.

The verticality values show significant deviations from vertical on the southern end of the eastern wall and along the southern wall.

Conclusion
The survey was successfully carried out and plans drafted showing graphically the results of the survey. The plans show areas that deviated from level and verticality. The plans and report can be used to assist further structural analysis of the building.

Report Prepared by
<u>STAIG AND SMITH LTD</u>

Ephraim Jacobsen
Licensed Cadastral Surveyor



DO NOT SCALE

ORIGINAL SIZE A1

Notes

(1) VERIFY ALL DIMENSIONS ON SITE.
(2) DO NOT SCALE FROM DRAWING.
(3) LEVELS IN TERMS OF: ASSUMED DATUM
ORIGIN OF LEVELS: SURVEY BENCHMARK NAIL 1
(IN ASPHALT ON WEST SIDE OF EASTERN ENTRANCE)
RL 0.000m
(4) ORIENTATION IS APPROXIMATE ONLY AND IS NOT IN TERMS OF TRUE NORTH.
(5) THE FLOOR PLAN HAS BEEN OBTAINED FROM ON-SITE MEASUREUP. THE WALL LAYOUT IS INDICATIVE ONLY AND MUST NOT BE USED FOR STRUCTURAL ANALYSIS.
(6) LEVELS WERE TAKEN ON THE CONCRETE FLOOR SURFACE. WHERE A CARPET OR VINYL SURFACE WAS PRESENT, LEVELS WERE ADJUSTED FOR THE COVERING THICKNESS.
(7) THE COLUMN/WALL VERTICALITY ARROWS SHOW THE DIRECTION OF THE TOP OF THE COLUMN/WALL RELATIVE TO THE BOTTOM.
(8) THE COLUMN/WALL VERTICALITY ARROWS SHOW THE DIRECTION OF THE TOP OF THE COLUMN/WALL RELATIVE TO THE BOTTOM.
(9) VERTICALITY DEVIATIONS ARE FOR CONCRETE WALLS, UNLESS STATED OTHERWISE
(10) FIELDWORK CARRIED OUT ON 28TH-29TH AUGUST 2013.

ISO CONTOUR LEGEND - INTERVAL 0.010M

HIGH AREAS

LOW AREAS

KEY

STEP/CRACK IN CONCRETE SLAB
EDGE OF CONCRETE SLAB
TOP OF BANK (TB)
42.5 EXISTING CONTOURS
BUILDING WALL
TREE
SPOT HEIGHT
STREET LIGHT
POWER BOX
POWER POLE
TELEPHONE UPSTAND
HYDRANT
WATER VALVE
WATER METER
MANHOLE
SANITARY SEWER MANHOLE (SSMH)
STORMWATER MANHOLE (SWMH)
STANDARD SUMP
YARD SUMP
INVERT
FENCE
GATE
POWER LINE

AMENDMENT	DATE

STAIG & SMITH LTD
12 Bealey Avenue
Christchurch
Ph: 0800 807 818
Fax: 03 548 4427
e-mail: enquiries@staigsmith.co.nz
SURVEYING-PLANNING-ENGINEERING-RESOURCE MANAGEMENT

Design:
Drawn: EDJ
Checked: SO
Approved: SO
Issue Date: 10 Sept 2013

Job Title

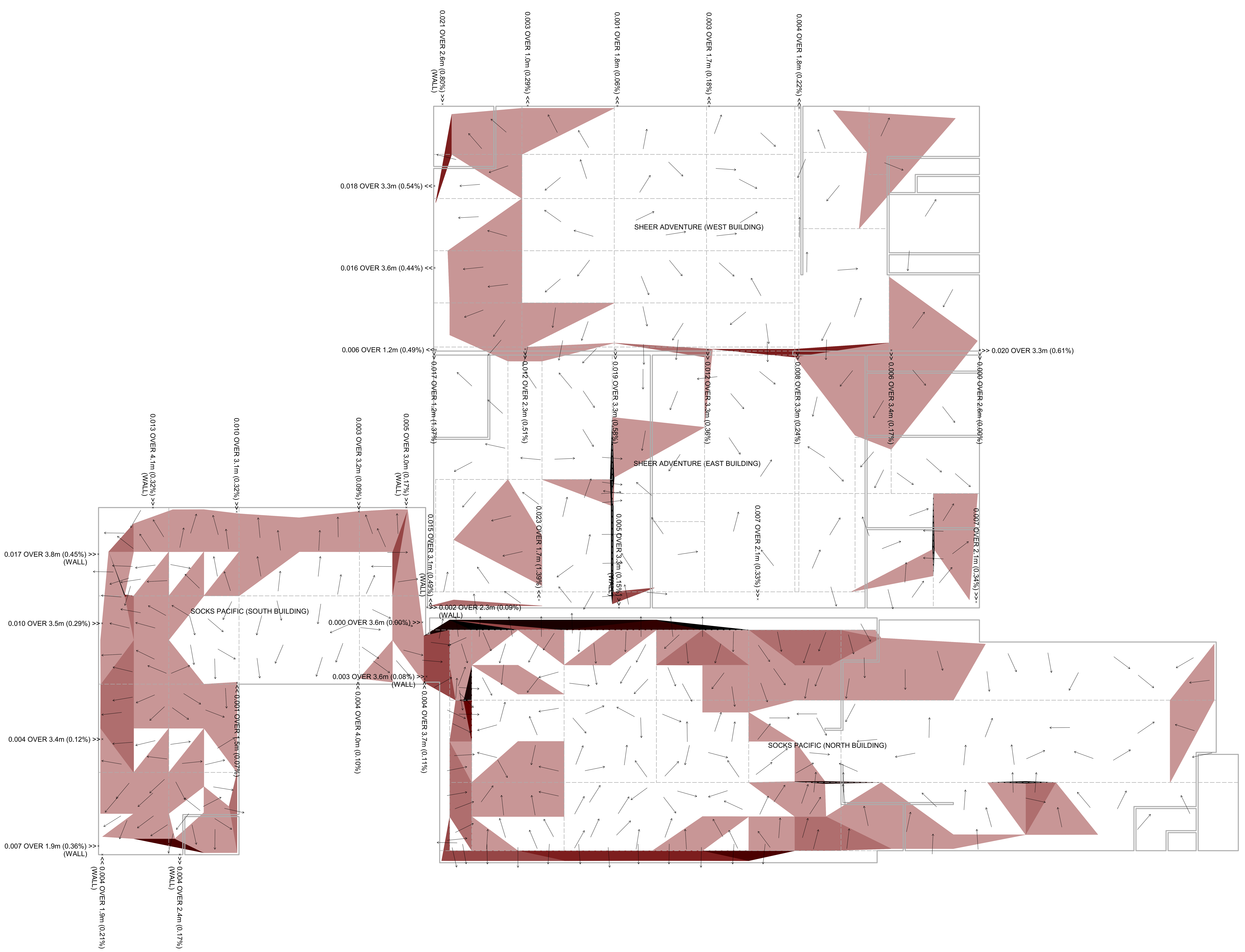
Structex Harvard Ltd
78 & 82 Byron St
Christchurch

Drawing Title

Floor Level Plan

Scale: 1:100 at A1
Issue. 1
Job No: 11417
Sheet 1 of 2

Y:\jobs\11400\11417 Structex - Byron St\CAD\12D\11417 Building Survey



DO NOT SCALE

ORIGINAL SIZE A1

Notes

(1) VERIFY ALL DIMENSIONS ON SITE.
(2) DO NOT SCALE FROM DRAWING.
(3) LEVELS IN TERMS OF: ASSUMED DATUM ORIGIN OF LEVELS: SURVEY BENCHMARK NAIL 1 (IN ASPHALT ON WEST SIDE OF EASTERN ENTRANCE) RL 0.000m
(4) ORIENTATION IS APPROXIMATE ONLY AND IS NOT IN TERMS OF TRUE NORTH.
(5) THE FLOOR PLAN HAS BEEN OBTAINED FROM ON-SITE MEASUREUP. THE WALL LAYOUT IS INDICATIVE ONLY AND MUST NOT BE USED FOR STRUCTURAL ANALYSIS.
(6) LEVELS WERE TAKEN ON THE CONCRETE FLOOR SURFACE. WHERE A CARPET OR VINYL SURFACE WAS PRESENT, LEVELS WERE ADJUSTED FOR THE COVERING THICKNESS.
(7) THE COLUMN/WALL VERTICALITY ARROWS SHOW THE DIRECTION OF THE TOP OF THE COLUMN/WALL RELATIVE TO THE BOTTOM.
(8) VERTICALITY DEVIATIONS ARE FOR CONCRETE WALLS, UNLESS STATED OTHERWISE
(9) FIELDWORK CARRIED OUT ON 28TH-29TH AUGUST 2013.

GRADIENT LEGEND - 0.5% INTERVAL

0.5 - 1.0%
1.0 - 1.5%
1.5 - 2.0%
2.0 - 2.5%
2.5 - 3.0%
3.0 - 3.5%
3.5 - 4.0%
OVER 4.0%

AMENDMENT	DATE

STAIG & SMITH LTD
12 Bealey Avenue
Christchurch
Ph: 0800 807 818
Fax: 03 548 4427
e-mail: enquiries@staigsmith.co.nz
SURVEYING-PLANNING-ENGINEERING-RESOURCE MANAGEMENT

Design:
Drawn: EDJ
Checked: SO
Approved: SO
Issue Date: 10 Sept 2013

Job Title

Structex Harvard Ltd
78 & 82 Byron St
Christchurch

Drawing Title

Floor Slope and Column Verticality Plan

Scale: 1:100 at A1 | Issue. 1

Job No: 11417 | Sheet 2 of 2

Y:\jobs\11400\11417 Structex - Byron St\CAD\12D\11417 Building Survey

APPENDIX E

Repair Specification

1.1 PRELIMINARY

Refer to the Preliminary and General Clauses of this Specification and to the General Conditions of Contract which are equally binding on all trades. This section of the Specification shall be read in conjunction with all other sections.

1.2 SCOPE

This Section consists of:-

1. Damage surveys.
2. Repair of cracks in reinforced concrete and blockwork.
3. Repair of concrete spalling.
4. Repair of cracks in unreinforced masonry.
5. Grouting of reinforcing bars and anchors into unreinforced masonry.

1.3 RELATED DOCUMENTS

In this section of the Specification reference is made to the latest revisions of the following documents:

The New Zealand Building Code		(BIA)
NZS 3103:1991	Specification for sands for mortars and plasters	(SCNZ)
NZS 3104:2003	Specification for Concrete Production	(SCNZ)
NZS 3109:1997	Specification for Concrete Construction	(SCNZ)
NZS 3112.4:1986	Methods of test for concrete Tests relating to grout	(SCNZ)
NZS 3121:1986	Specification for water and aggregate for concrete	(SANZ)
NZS 4210:2001	Code of Practice for Masonry Construction: Materials and Workmanship	(SANZ)
BS EN 459-1:2010	Building Limes Part 1: Definitions, Specifications and Conformity Criteria	(BS EN)
BS EN 1052-3:2002	Determination of Initial Shear Strength	(BS EN)
BS EN 459-2:2010	Building Limes Part 2: Test Methods	(BS EN)
NZSEE	Assessment & Improvement of the Structural Performance of Buildings in Earthquakes.	(NZSEE)

ASTM C 109-08	Standard Test Methods for Compressive Strength of Hydraulic Cement Mortars.	(ASTM)
ASTM C 1314-10	Standard Test Methods for Compressive Strength of Masonry Prisms.	(ASTM)
ASTM E488-90	Standard Test Methods for Strength of Anchors In Concrete and Masonry Elements.	(ASTM)

1.4 QUALITY ASSURANCE

1.4.1 General

It is the Contractor's responsibility to ensure that all work associated with this part of the contract is performed in accordance with the plans and specifications.

The Contractor's quality assurance procedures should encompass, but are not limited to, the following items:

1. Photographic record of damage observed
2. Recording of repairs completed
3. Daily recording of materials used
4. Mixing of epoxy/mortar/grout
5. Substrate surface preparation
6. Application of repair systems
7. Anchor hole location and embedment depth
8. Anchor and reinforcing steel placement
9. Testing frequency and reporting

The Contractor shall advise the Engineer in writing of the name of a suitably qualified and experienced representative to be responsible for ensuring that quality assurance procedures are being followed, prior to commencement on site.

Masonry shall be erected only under the direction of a Registered Mason specialising in the laying of masonry units. Before work commences on site, the Contractor shall advise the Engineer, in writing, the name of the Registered Mason who will be responsible for the masonry construction.

From time to time the Engineer may elect to audit the quality records. They shall be kept up to date and be made available for audit by the Engineer at all times during the construction of this project.

If so instructed, the Contractor shall forward copies of all or part of the records to the Engineer.

1.4.2 Inspection

The Engineer will review construction. Prior to grouting of anchor holes, the Engineer or his representative shall be notified and a reasonable opportunity given him to inspect prepared anchor holes.

Where necessary, the Engineer's instructions shall be carried out before grouting commences.

1.4.3 Producer Statement – Construction (PS3)

When the works are sufficiently complete that they are ready for application to the Territorial Authority for a Code Compliance Certificate, or otherwise at key handover dates for particular sections of the works, the nominated representative responsible for the quality assurance procedures for the Damage Repair will be required to certify to the main Contractor that all Damage Repair work has been carried out in full accordance with all Contract Documents and Contract Instructions in the form of a Producer Statement - Construction. This statement will be required to be completed prior to the issue of the Producer Statement – Construction Review by the Engineer for the whole or sections of the works as appropriate.

No Practical Completion Certificate shall be issued until such time as all the Producer Statements for the relevant section of the works have been received.

1.5 TESTING

The Contractor shall provide evidence of material compliance with the required testing as defined in this section of the Specification.

1.5.1 Mortar for Reinforced Concrete Blockwork

The Contractor shall provide evidence of testing for the following:

1. Mortar Compression Strength

Allow to test mortar compressive strength in accordance with NZS 4210:2001. Each test shall consist of a minimum of 3 specimens. Testing frequency shall be as follows:

(i) Test to be carried out for every 450 sq.m of wall constructed (measured on one face only) to be tested at an age of 28 days.

2. Masonry to Mortar Bond Strength

Allow to test masonry to mortar bond strength in accordance with NZS 4210:2001. Each test shall consist of a minimum of 3 specimens. Testing frequency shall be as follows:

(i) Test to be carried out for every 450 sq.m of wall constructed (measured on one face only) to be tested at an age of 7 days.

1.5.2 Unreinforced Masonry with Lime Mortar

The Contractor shall provide evidence of testing for the following:

1. Lime Chemical Requirements

Prior to the commencement of work allow to test the carbon dioxide content of the lime in accordance with Section 5.5 or 5.4 of BS EN 459-2:2010.

2. Soundness of Slaked Lime

Prior to the commencement of work allow to test soundness of slaked lime in accordance with Section 6.4.4 of BS EN 459-2:2010. Each test shall consist of a minimum of 3 specimens.

3. Mortar Compression Strength

Allow to test mortar compressive strength in accordance with ASTM C 109-08 (2008) except that the samples shall be left to cure at a temperature of 24±8 deg C and a relative humidity of less than 80%. Each test shall consist of a minimum of 3 specimens. Testing frequency shall be as follows:

(ii) Specimens prepared for testing prior to the commencement of works, to be tested at an age of 7, 28, 90, 180 and 360 days (total 15 specimens to be prepared).

(iii) Specimens prepared for every 50 sq.m of wall constructed (measured on one face only) to be tested at an age of 7 and 28 days.

4. Masonry Compression Strength

Prior to the commencement of work allow to test masonry compressive strength in accordance with ASTM C 1314-10 (2010) except that the samples shall be left to cure at a temperature of 24±8 deg C and a relative humidity of less than 80%. Each test shall consist of a minimum of 3 specimens. Testing frequency shall be as follows:

(i) Specimens prepared for testing prior to the commencement of works, to be tested at an age of 7, 28, 90, 180 and 360 days (total 15 specimens to be prepared).

5. Masonry Bed-Joint Shear Strength

Prior to the commencement of work allow to test initial masonry shear strength in accordance with BS EN 1052-3:2002. Each test shall consist of a minimum of 9 specimens as detailed in BS EN 1052-3:2002. Testing frequency shall be as follows:

(i) Specimens prepared for testing prior to the commencement of works, to be tested at an age of 28, 90, 180 and 360 days (total 12 specimens to be prepared).

Variation of Materials

If the repair works consist of structural elements with more than one type of masonry unit (i.e. red brick and Halswell Stone) the testing detailed above for masonry compressive strength and bed-joint shear strength shall be repeated for each masonry unit type.

Further tests shall be carried out if, and immediately when, the supply of the constituent materials varies during construction with at least three specimens per test. Tests shall also be conducted when requested by the Engineer.

The results of all tests shall be submitted upon request to the Engineer for review.

1.5.3 Unreinforced Masonry with Cement-Lime Mortar

The Contractor shall provide evidence of testing for the following:

1. Mortar Compression Strength

Allow to test mortar compressive strength in accordance with Appendix 2.A of NZS 4210. Each test shall consist of a minimum of 3 specimens. Testing frequency shall be as follows:

(i) Specimens prepared for testing prior to the commencement of works, to be tested at an age of 7, 28 and 90 days (total 9 specimens to be prepared).

(ii) Specimens prepared for every 50 sq.m of wall constructed (measured on one face only) to be tested at an age of 7 and 28 days (6 specimens to be prepared for every 50 sq.m).

2. Masonry Bed-Joint Shear Strength

Prior to the commencement of work allow to test initial masonry shear strength in accordance with BS EN 1052-3:2002. Each test shall consist of a minimum of 9 specimens. Testing frequency shall be as follows:

(i) Specimens prepared for testing prior to the commencement of works, to be tested at an age of 28 and 90 days (total 18 specimens to be prepared).

Variation of Materials

If the repair works consist of structural elements with more than one type of masonry unit (i.e. red brick and Halswell Stone) the testing detailed above for masonry compressive strength and bed-joint shear strength shall be repeated for each masonry unit type.

Further tests shall be carried out if, and immediately when, the supply of the constituent materials varies during construction with at least three specimens per test. Tests shall also be conducted when requested by the Engineer.

The results of all tests shall be submitted upon request to the Engineer for review.

1.6 SAFETY

The Contractor shall conform fully both on and off site with the provisions of the New Zealand Building Code in all matters related to construction safety, in particular with approved documents F1 (Hazardous Agents on Site), F2 (Hazardous Building Materials), F4 (Safety from Falling) and F5 (Construction and Demolition Hazards).

1.7 MATERIALS AND WORKMANSHIP

1.7.1 Materials

The Contractor shall adhere to all requirements of NZS 3104, NZS 3109 and NZS 4210, except where specified otherwise herein or instructed otherwise by the Engineer. A copy of this standard shall be kept on the site and relevant parts read with the following Clauses of this Specification.

Materials to be used in conjunction with brick or stone masonry shall be selected to minimise the effects of effloresence.

The Engineer may approve equivalent products that satisfy all of the requirements and show equality to the systems specified herein. Approval for the equivalent system shall be sought prior to submission of tender, refer also to the Submittals section below

1.7.2 Workmanship

All work shall be carried out by licensed applicators of the material manufacturer's.

Undertake all preparatory work necessary prior to application of the specified system to ensure proper bond and clean, true surfaces in the finished work.

All materials shall be mixed and applied in accordance with best trade practice and applied by skilled applicators to the manufacturer's recommendations.

All adjoining work shall be adequately protected during mixing and application and utmost care shall be taken not to damage surrounding fixtures and fittings. All damage consequent upon this operation shall be completely made good.

Remove debris at regular intervals and leave the completed work free from defects of all kinds.

1.7.3 Completion

Clean all adjoining surfaces and fittings of any paint contamination. Replace all hardware without damage to it or the adjoining surface. Take away from the site all painting materials, equipment and rubbish leaving the surrounding area clean, tidy and undamaged.

1.8 DAMAGE SURVEYS

We have undertaken an initial assessment that has identified general forms of damage and repairs required. We have not been able to expose all critical elements for observation, nor have we conducted a detailed survey identifying each individual crack. At the request of the engineer the Contractor shall expose areas of the structure, in order to enable detailed observations to be made of critical areas.

The Drawings provide specific details of the primary structural repairs required. Repairs of more minor damage (such as cracking and spalling of concrete) shall be undertaken by the Contractor in accordance with this Specification, under the direction of the Engineer.

1.8.1 Crack Damage

The Contractor shall identify cracks to be repaired following the methodologies outlined in the following sections of this Specification. Following preparation but prior to epoxy injection or grouting, the Contractor shall contact the Engineer to arrange an inspection of the area to be repaired.

Cracks are to be repaired in various elements as identified in site reports or on structural drawings.

Records should be kept of repaired cracks and should include details of:-

1. Location
2. Crack width
3. Crack length
4. Volume of material (epoxy/grout) used

1.8.2 Spalling Damage

The Contractor shall identify areas of spalled concrete to be repaired following the methodologies outlined in the following sections of this Specification. Following preparation but prior to application of the repair mortar, the Contractor shall contact the Engineer to arrange an inspection of the area to be repaired.

Spalled concrete is to be repaired in various elements as identified in site reports or on structural drawings.

Records should be kept of repaired spalling and should include details of:-

1. Location
2. Approximate spalled area
3. Volume of material (repair mortar) used

1.8.3 Verticality Survey

At the request of the Engineer, the Contractor shall undertake a verticality survey of the target building. The verticality survey shall ascertain whether there is any residual displacement or twist of the building. The Contractor shall submit their proposed methodology to the Engineer for approval prior to undertaking the survey.

1.9 REPAIR OF CRACKS IN REINFORCED CONCRETE AND BLOCKWORK

The following sections of the Specification detail the procedures to be followed when repairing cracks in reinforced concrete and reinforced concrete blockwork.

Cracks less than 0.2mm wide are considered to be superficial and do not require specific structural repair.

1.9.1 Repair of Hairline Cracks (< 2mm)

Where possible at the direction of the Engineer, cracks between 0.2mm and 2mm shall be repaired by injection of epoxy resin.

Where access to seal around the element being repaired is possible, repair the crack using a low viscosity epoxy resin such as Sikadur Injectokit – LV or Sikadur 52.

Where access is not possible to prevent grout loss, repair the crack with a thixotropic epoxy resin such as Sikadur Injectokit – TH.

Seal and prepare the surface being repaired and inject the epoxy resin in accordance with the manufacturers instructions.

Alternative products of equivalent properties may be acceptable but must be submitted to the Engineer for approval at the time of tender.

1.9.2 Repair of Large Cracks (< 5mm)

Where possible at the direction of the Engineer, cracks between 2mm and 5mm shall be repaired by injection of Sikadur 52.

Seal and prepare the surface being repaired and inject the epoxy resin in accordance with the manufacturers instructions.

Alternative products of equivalent properties may be acceptable but must be submitted to the Engineer for approval at the time of tender.

1.9.3 Repair of Very Large Cracks (> 5mm)

Advise the Engineer of any cracks larger than 5mm in width.

If the Engineer does not require any specific repair detail, cracks larger than 5mm shall be repaired by injection of Sikadur 42 / Sika Grout 212.

Seal and prepare the surface being repaired and inject the epoxy resin / cementicious grout in accordance with the manufacturers instructions.

Alternative products of equivalent properties may be acceptable but must be submitted to the Engineer for approval at the time of tender.

1.9.4 Repair of Reinforced Concrete Blockwork Mortar Joints

Repair mortar shall conform to NZS 4210, clause 2.2. Mortar shall have a 28 day compressive strength of at least 12.5 MPa. The 7 day masonry to mortar bond strength shall not be less than 200kPa. Sand shall comply with NZS 3103 with chloride levels not exceeding 0.04% by dry weight of sand. Appearance of new mortar shall match the existing. The contractor shall submit to the Engineer a mix design for the mortar prior to commencing work.

Damaged mortar beds shall be raked out to a minimum depth of 25 mm and re-pointed. Pointing shall be undertaken to match existing.

The Contractor shall carry out material testing as required in Section 1.5.

1.10 REPAIR OF CONCRETE SPALLING

The following sections of the Specification detail the procedures to be followed when repairing spalled concrete.

1.10.1 Repair of Shallow Spalling (<40mm thick)

At the direction of the Engineer break back to sound concrete. The depth of breakout on the edge of any repair area shall be a minimum of 10 mm and feather edges will not be accepted. To achieve this, the perimeter of the area to be repaired shall first be cut to a depth of 10 mm using a suitable tool.

Clean any exposed reinforcing using a wire brush. Prepare the exposed concrete surface and reinforcing in accordance with the manufacturers instructions, applying a primer such as Sika Monotop Primer as required.

Build up the required concrete profile using a high strength repair mortar, such as Sika Monotop Structural Mortar, and finish in accordance with the manufacturers instructions.

Alternative products of equivalent properties may be acceptable but must be submitted to the Engineer for approval at the time of tender.

1.10.2 Repair of Moderate Spalling (<80mm thick)

At the direction of the Engineer break back to sound concrete. The depth of breakout on the edge of any repair area shall be a minimum of 10 mm and feather edges will not be accepted. To achieve this, the perimeter of the area to be repaired shall first be cut to a depth of 10 mm using a suitable tool.

Clean any exposed reinforcing using a wire brush. Prepare the exposed concrete surface and reinforcing in accordance with the manufacturers instructions, applying a primer such as Sika Monotop Primer as required.

Build up the required concrete profile using a high build repair mortar, such as Sika Monotop High Build Mortar, and finish in accordance with the manufacturers instructions.

Alternative products of equivalent properties may be acceptable but must be submitted to the Engineer for approval at the time of tender.

1.10.3 Repair of Deep Spalling (>80mm thick)

At the direction of the Engineer break back to sound concrete. The depth of breakout on the edge of any repair area shall be a minimum of 10 mm and feather edges will not be accepted. To achieve this, the perimeter of the area to be repaired shall first be cut to a depth of 10 mm using a suitable tool.

Clean any exposed reinforcing using a wire brush. Prepare the exposed concrete surface and reinforcing in accordance with the manufacturers instructions.

Box and pour to the required concrete profile using a flowable repair mortar, such as Sika Monotop Microconcrete, and finish in accordance with the manufacturers instructions.

Alternative products of equivalent properties may be acceptable but must be submitted to the Engineer for approval at the time of tender.

1.11 REPAIR OF CRACKS IN UNREINFORCED MASONRY

The following sections of the Specification detail the procedures to be followed when repairing cracks in unreinforced masonry. The Contractor shall adhere to all requirements of NZS 4210, except where specified otherwise herein or instructed otherwise by the Engineer.

1.11.1 Unreinforced Masonry with Lime Mortar

At the direction of the Engineer, replace or repair cracked masonry units. New brick or stone masonry units shall be selected to match colour, density and texture of the existing.

Repair mortar shall be lime mortar to match the existing. Lime mortar shall be composed of aged slacked lime (lime putty), sand and water. The prescribed ratio of lime to sand shall be 1:3. Alternate mix designs may be used with the approval of the Engineer. Repair mortar should be batch mixed.

Lime used shall meet the requirements of BS 890:1995 and shall be slaked for a minimum of 3 months. Acceptable suppliers include:

1. Historic Building Conservation
   18 Soper Road
   Wingatui
   Dunedin
   03 489 0930

Alternate suppliers may be used with the approval of the Engineer.

Sand shall be Class A sand as defined in NZS 3103 with chloride levels not exceeding 0.04% by dry weight of sand.

Alkaline resistant mineral oxides can be added to enable colour matching. Dosage rates of mineral oxides shall limited such that they do not cause a reduction in mortar strength.

No cement products, modern additives or plasticisers are to be added to the mix unless approved by the Engineer.

The contractor shall submit to the Engineer a mix design for lime mortar prior to commencing work.

Masonry repairs shall be cured in accordance with NZS 4210, Sections 2.18 and 2.19.

The Contractor shall carry out material testing as required in Section 1.5.

1.11.2 Unreinforced Masonry with Cement-Lime Mortar

Repair of unreinforced masonry with cement-lime mortar shall be undertaken in accordance with NZS 4210.

At the direction of the Engineer, replace or repair cracked masonry units. New brick or stone masonry units shall be selected to match existing. Damaged mortar beds shall be raked out to a minimum depth of 25 mm and re-pointed. Pointing shall be undertaken to match existing.

Repair mortar shall be cement-lime mortar to match the existing. Mortar shall be composed of Portland cement, hydrated lime, sand and water, and shall be Durability Class M2 mortar as defined in Table 2.1 of NZS 4210.

Mortar shall have a 28 day compressive strength of at least 5 MPa. The 28 day masonry to mortar bond strength shall not be less than 200kPa.

Building lime shall comply with BS EN 459-1:20101 and shall be slaked with water to form hydrated lime before use in mortar. Sand shall be Class A sand as defined in NZS 3103 with chloride levels not exceeding 0.04% by dry weight of sand.

Repair mortar should be batch mixed. Hydrated lime shall not be omitted from the mix unless approved by the Engineer. Alkaline resistant mineral oxides can be added in accordance with NZS 4210 to enable colour matching.

The contractor shall submit to the Engineer a mix design for the mortar prior to commencing work.

Masonry repairs shall be cured in accordance with NZS 4210, Sections 2.18 and 2.19.

The Contractor shall carry out material testing as required in Section 1.5.

1.12 GROUT INJECTION OF CRACKS AND OPEN JOINTS IN UNREINFORCED MASONRY

The following sections of the Specification detail the procedures to be followed when repairing cracks in unreinforced masonry.

At the direction of the Engineer, replace or repair cracked masonry units. New brick or stone masonry units shall be selected to match colour, density and texture of the existing.

1.12.1 Materials

Grout shall be Centricrete MV or approved alternative.

Repair mortar shall be as specified in Section 1.10.

1.12.2 Methodology

Rake open joint to adequate depth to remove any loose mortar, but no less than 25 mm.

Flush joint, surfaces and cracks with clean water. Loose bricks shall be removed and reset with mortar

Injection holes are to be drilled at header joints or cracked brick, through to the inner wythe in each course. Bleed tubes shall be provided as required to ensure that any trapped air can escape. Flush bleed holes with water. Pressure inject grout to bond cracks in accordance with the manufacturers recommendations.

Re-point holes and open joints. Pointing shall be undertaken to match existing.

1.13 GROUTING OF BARS INTO UNREINFORCED MASONRY

The following sections of the Specification detail the procedures to be followed when grouting threaded rod and reinforcing bars into unreinforced masonry.

1.13.1 Shallow Embedment (< 750mm)

Grout shall be either non-shrink cement or epoxy based and shall be prepared in accordance with the manufacturers specifications. Acceptable products include:

1. Hilti HY150
2. Hilti CM 651
3. Sika Sikadur 52
4. Sika Grout 212
5. Ramset Epcon C6 (brick masonry only)
6. Ramset Premier Grout MP

The Contractor shall submit details of the proposed grout and grouting procedure to the Engineer for approval prior to the start of construction.

Anchor hole diameters are detailed in Table 1.

Table 1 Hole Diameters

Anchor Size (mm)	Hole Diameter (mm)	
	Hilti HY150 / Sikadur 52 / Epcon C6	Hilti CM 651 / Sika Grout 212/ Premier Grout MP
10	12	20
12	14	24
16	18	32
20	22	40
25	27	50

Holes shall be drilled using hammer drills and must be dry prior to filling with grout.

All holes shall be cleaned out using a stiff bristled wire bottlebrush and a compressed air source (with no oil bottle in the line) so that all dust and debris are removed from the side of the hole.

When this has been completed the Contractor is to notify the Engineer for inspection of the holes prior to placement of bars and grout.

The holes shall be partially filled with grout prior to inserting the reinforcing bar. Holes shall be filled from the bottom up (rather than pouring from the top). When a wall cavity is encountered, use a proprietary mesh anchor sleeve to bridge the cavity.

Bars shall be placed in the holes, given one turn to expel air voids and shall be fully supported (if necessary) and left undisturbed for at least 24 hours. After 24 hours horizontal bars installed at 15 degree slope can be bent horizontal.

After the bars have been placed in position, ensure that the grout fills the hole to the surface of the substrate. Top up holes if necessary.

The grouts shall be used strictly in accordance with the manufacturer's instructions.

When it is intended that the anchor be concealed in the completed state the bar shall be recessed a minimum of 35 mm beyond the exposed face of the masonry. Close the hole by mixing dust salvaged from the drill hole with a weakly cementicious mortar (eg Sika 212) and plug the hole to match existing.

1.13.2 Deep Embedment (> 750mm Vertical)

This section of the Specification details the procedure to be followed when grouting vertical bars into masonry walls. Grout shall be either non-shrink cement or epoxy based and shall be prepared in accordance with the manufactures specifications. Acceptable products for grouting include:

1. Hilti RE500
2. Sika Sikadur 52
3. Sika Grout 212
4. Ramset Epcon C6
5. Ramset Premier Grout MP

The Contractor shall submit details of the proposed grout and grouting procedure to the Engineer for approval prior to the start of construction.

Anchor hole diameters are detailed in Table 2.

Table 2 Hole Diameters

Anchor Size (mm)	Hole Diameter (mm) Hilti RE500 / Sikadur 52/ Epcon C6	Sika Grout 212/ Premier Grout MP
10	12	20
12	14	24
16	18	32
20	22	40
25	27	50

Holes may be drilled using hammer or core drills and must be dry prior to filling with grout.

If the brick wall is suspected to contain cavities that might cause significant grout loss the hole can be filled with expandable builders filler and re-drilled at 2mm oversize to remove the filler from the walls of the hole.

All holes shall be cleaned out using a stiff bristled wire bottlebrush and a compressed air source (with no oil bottle in the line) so that all dust and debris are removed from the side of the hole.

When this has been completed the Contractor is to notify the Engineer for inspection of the holes prior to placement of bars and grout.

The holes shall be partially filled with grout prior to inserting the reinforcing bar. Holes shall be filled from the bottom up (rather than pouring from the top). Grouting shall be conducted with a maximum lift height of 1.5m.

Bars shall be placed in the holes, given one turn to expel air voids and shall be fully supported (if necessary) and left undisturbed for at least 24 hours.

After the bars have been placed in position, ensure that the grout fills the hole to the surface of the substrate. Top up holes if necessary.

The grouts shall be used strictly in accordance with the manufacturer's instructions.

1.13.3 Control Tests – Grouted Anchors & Reinforcing Bars

Shallow embedment anchors installed in accordance with Section 1.11.1 above shall be tested in accordance with the following:

Torque Testing

One quarter of all new tension anchor bolts and reinforcing bars embedded in unreinforced masonry walls shall be tested using a torque calibrated wrench to the following minimum torques,

12mm diameter anchors or bars	54Nm
16mm diameter anchors or bars	68Nm
20mm diameter anchors or bars	100Nm

No slippage shall occur under the above torques.

Direct Tension Testing

A minimum of five direct tension tests shall be undertaken for each anchor size specified on the Structural Drawings. The load testing shall be undertaken as follows:

The masonry wall should support the test apparatus. The distance between the anchor and the test apparatus support should not be less than the wall thickness.

The tension test load reported should be the load recorded at 3 mm relative movement of the anchor and the adjacent masonry surface. For the testing of existing anchors, a preload of 1.5 kN shall be applied prior to establishing a datum for recording elongation.

Reports

Results of all tests shall be reported. The report shall include the test results as related to anchor size and type, location, embedment depth, wall thickness and joist orientation.