Canterbury
District Health Board
Te Poari Hauora ō Waitaha

CORPORATE OFFICE

Level 1
32 Oxford Terrace
Christchurch Central
CHRISTCHURCH 8011

Telephone: 0064 3 364 4160
Fax: 0064 3 364 4165
carolyn.gullery@cdhb.health.nz

15 June 2018

[redacted]

[redacted]

[redacted]

RE Official Information Act (the "Act") request CDHB 9833 (Part two response)

We refer to your email dated 18 April 2018 requesting the following information under the Official Information Act from Canterbury DHB.

1. **The most recent Detailed Engineering Evaluation (DEE) of the Riverside, Parkside buildings at Christchurch Hospital.**

I note that we provided you with a response for question 1 on 18 May 2018 i.e. The Detailed Seismic Assessment Update for the Parkside Building completed by Holmes Consulting Group and dated 30 March 2016, and the Detailed Seismic Assessment Update for the Riverside Central Building completed by Holmes Consulting Group and dated 20 December 2017.

2. **For all buildings at Burwood Hospital except the new building opened in 2016.**

There are a number of buildings that comprise the wider Burwood Hospital Campus, excluding the new Burwood Hospital building opened in 2016.

Canterbury DHB engaged Holmes Consulting to complete a full structural review of the wider Burwood Campus, and a series of reports have been compiled as part of this process. These included the following buildings / facilities:

Please find attached as **Appendices 1 – 15** the reports for:

1. Orthopaedic Outpatients;
2. Administration Building;
3. Boiler House;
4. Chapel;
5. Engineering Services Building;
6. Nurse Hostel West;
7. Orthopaedic Rehabilitation Unit;
8. Physical Medicine;
9. Spinal Unit;
10. Surgical Services Unit and Surgical Operating Suites;
11. Surgical Block;
12. Maori Health;
13. Birthing Unit and Minor Procedure Unit
14. Milner Lodge and
15. Tapper Units

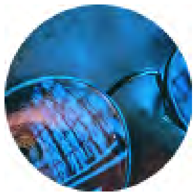
I trust that this satisfies your interest in this matter.

Please note that this response, or an edited version of this response, may be published on the Canterbury DHB website.

Yours sincerely

Carolyn Gullery
Executive Director
Planning, Funding & Decision Support

HolmesConsultingGroup



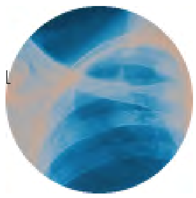
REPORT

STRUCTURAL AND CIVIL ENGINEERS

BURWOOD HOSPITAL CAMPUS

REPORT 19 - ORTHOPAEDIC OUTPATIENT/BSU HOSTEL

(FORMERLY SPINAL INJURIES HOSTEL)



PREPARED FOR

CANTERBURY DISTRICT HEALTH BOARD

106186.78

REPORT REV3 – 24 SEP 2013

BURWOOD HOSPITAL CAMPUS – DETAILED SEISMIC ASSESSMENT REPORT

REPORT 19 - ORTHOPAEDIC OUTPATIENT / BSU HOSTEL

Prepared For:
CANTERBURY DISTRICT HEALTH BOARD

Date: 24 Sep 2013
Project No: 106186.78
Revision No: 3

Prepared By:

Daniel Moore
PROJECT ENGINEER

Reviewed By:

Eric McDonnell
SENIOR PROJECT ENGINEER

Updated By:

Peter Grange
STRUCTURAL ENGINEER

Reviewed By:

Jenny Fisher
PROJECT DIRECTOR

Holmes Consulting Group LP
Christchurch Office

REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
01/05/2012	1	Interim results of quantitative assessment (Phase 3) for discussion (some on site investigations still to be completed)
23/05/2012	2	Interim results of quantitative assessment. Strengthening materials added to relevant sections. Figures 2-4 and 5-1 revised, misc revisions. Added Section 3.7, Additional Investigations required.
24/09/2013	3	Updated to include additional investigations and strengthening works completed.

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FIGURES PAGE

Holmes Consulting Group has been engaged by Canterbury District Health Board to complete a full structural review of the Burwood Hospital Campus following the Lyttelton Earthquake. A series of reports have been completed as part of this. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The individual building reports, like this one, should be read in conjunction with the base report and refer to the repair specification.

This report identifies the structural damage sustained by the Orthopaedic Outpatients/BSU Hostel building as a result of the series of Earthquakes, including the Darfield Earthquake that struck at 4:36am on the 4th September 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th June 2011 and the December Earthquake that struck at 3.18pm on the 23rd December 2011. The report summarises the effects of the damage on the lateral load capacity of the building, and provides structural repairs for the damage identified. The general form of the building, along with its capacity relative to current code levels, has been included for the buildings pre-earthquake undamaged state and post-earthquake state. Recommendations to increase the strength of the building to greater than 67% current code capacity have also been summarized.

The Orthopaedic Outpatient / BSU Hostel (formerly the Spinal Injuries Hostel) was designed in 1978 and constructed in the period there after. The building is a single storey, timber framed structure with a lightweight roof. There is a combination of high level gypsum board ceilings with exposed timber trusses and low level ceilings with either gypsum board linings or acoustic tile grid system. The external walls are predominantly of timber framed construction, clad with either 90mm concrete masonry veneer or weatherboard. Several 190mm masonry block 'wing' walls extend out from the building at various locations. A later single level extension (for the Orthopaedic Outpatient) is of similar construction.

The building has a suspended ground floor concrete slab to provide a crawlspace and routes for services under the building. A service duct and corridor connect through to the building to the North. The concrete slab is assumed to be formed by pre-cast Unispan floor planks with a 75mm topping slab reinforced with cold drawn wire mesh. The planks span onto concrete sub-floor walls which are in turn supported by continuous footings which are founded approximately 1000mm below the adjacent grade.

The information available for review was limited and included: Master Floor Plan from Canterbury District Health Board [3], partial architectural ground floor plan of the original building [4], original structural drawings [22], architectural drawings of the Orthopaedic Outpatient extension [5], partial plans of internal alterations that were carried out in 2006 and a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [6].

For the purposes of this assessment the Orthopaedic Outpatient / BSU Hostel has been considered as an Importance Level 2 building (IL2). If the building were to be assessed as an Importance Level 3 building, the seismic demand would increase by 30% (R=1.3) and as such the assessed capacities would be reduced proportionally. The IL3 capacities are shown in brackets.

Based upon a review of the drawings available, and the site investigations completed, the primary lateral forced resisting elements of the building were assessed at their pre-earthquake undamaged state. The majority of the Orthopaedic Outpatient / BSU Hostel building has been assessed to have a pre-earthquake capacity to resist approximately 25% of the demand imposed by the current loading code IL2 Design Basis Earthquake (DBE) in both the North-South (across the building) and East –West (along the building) directions (IL3 - 20%DBE).

Post-earthquake strengthening to isolated extensions has been implemented at the south end of the building to bring the assessed capacity of the building above 33% DBE (IL2). This includes the installation of two new plywood lined bracing walls. This strengthening was carried out to upgrade the external wall bracing to meet the minimum external wall bracing requirements of NZS3604:2011[11]. Due to further investigations uncovering insufficient bottom plates and fixings of the remaining walls however, the building is still limited at 25% DBE (IL3 - 20%DBE).

As a result of this, the Orthopaedic Outpatient / BSU Hostel is considered to be "Earthquake Prone" in terms of section 122 of the Building Act.

The Orthopaedic Outpatient / BSU Hostel building appears to have performed relatively well considering the age of the building and the seismic actions experienced at the site. Moderate damage has been noted to the gypsum wall and ceiling linings throughout the building. The damage is typified by cracking to the linings at the junction of wall to wall and wall to ceiling linings. Minor cracking has also been noted to the concrete masonry block veneer and the 190mm concrete masonry block walls.

Earthquake induced differential settlements have occurred at the Orthopaedic Outpatient / BSU Hostel building resulting in a worst case slope in the ground floor framing of 1:200. The settlement is not as severe as other areas of the campus and as a result much of the associated damage to the concrete sub-floor walls and the timber framed superstructure above has been more limited.

It is believed that the significant damage observed occurred during the 22nd February 2011 event. Further observations of the earthquake damage observed have been included in the body of this report.

The reduction in the lateral capacity of the building due to the earthquake damage observed is hard to quantify. As noted, the primary damage to the structure is to the sheet clad timber bracing walls. Although there is some reduction in strength of the bracing walls due to the damage noted, the primary affect is to the ongoing stiffness of the building. The reduced stiffness will result in larger lateral displacements during future seismic events and additional damage to interior linings and building contents.

There has also been some reduction in the capacity of the building as a result of the differential settlements noted, along with a reduction in the future differential settlement the building could absorb before severe distress to the structure occurs. In addition, while the resulting slopes in the ground floor slab are within the typical acceptable range for standard occupancy buildings, CDHB may wish to pursue re-levelling of the building due to the nature of the patient group occupying the building, and ongoing serviceability concerns.

The minimum repairs required to reinstate the building to its pre-earthquake undamaged condition have been included in Section 4. This includes the repair and re-fixing of the wall and ceiling linings.

In addition to the repairs, recommended strengthening concepts to increase the seismic performance of the Orthopaedic Outpatient extension, and bring the assessed capacity above 67% DBE, have been included in section 5.

Despite the low % DBE to the isolated building extensions, timber framed buildings by their nature have built in redundancies and as such are unlikely to fail in a brittle manner. While the heavy plaster ceiling tile assembly has not been identified thus far as being damage (based upon the observations to date), the ceiling tiles assembly has been identified as the primary risk to building occupants as they could shake loose and dislodge during a significant earthquake. It is therefore our recommendation that the ceiling tiles be removed and replaced, as is suggested as part of the 67% DBE strengthening scheme.

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 Canterbury District Health Board to complete a full structural review of the Burwood Hospital Campus following the Lyttelton Earthquake. A series of reports have been completed as part of this. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The individual building reports, like this one, should be read in conjunction with the base report and refer to the repair specification.

The Burwood Hospital Campus base report covers the purpose and scope of the structural review. The current statutory requirements relevant to earthquake damaged buildings are outlined and the level of shaking experienced at the site estimated. Earthquake induced ground settlement damage across the campus is also discussed. The repair specification has been prepared to include repair details for typical damage observed in buildings on the Burwood Hospital Campus and is referred to as required in the specific building reports.

1.1 SCOPE OF WORK

This report is on the Orthopaedic Outpatient / BSU Hostel building, at Burwood Hospital, Mairehau Rd, Christchurch. The report identifies the general form of the structure, along with the gravity and lateral load resisting systems. Each component of the structural system was reviewed based upon the information available and any potential Critical Structural Weaknesses (CSW's) were noted.

The report also identifies the structural damage observed to date as a result of the series of Earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake, in particular, subjected the building to strong ground motions which significantly exceeded the current code loading demand for buildings of this nature.

The capacity of the Orthopaedic Outpatient / BSU Hostel building has been assessed relative to current code loading in the buildings pre-earthquake undamaged state and in its post-earthquake damaged state. The post-earthquake assessment summarizes the effects of the damage identified on both the gravity and lateral load resisting elements. Repair options to restore the buildings capacity to pre-earthquake levels for strength, durability and stiffness have been included. The repair options aim to maintain the buildings utility. Where required, strengthening options have also been provided.

1.2 LIMITATIONS

Findings presented as a part of this project are for the sole use of the Canterbury District Health Board (CDHB). 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 professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practising in this field at this time. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

Conclusions relate to the structural performance of the building under earthquake loads. We have not assessed the live load capacity of the floors, nor have we assessed the performance of non-structural components or building contents under earthquake loads.

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. Because all of the structure has not been available for detailed inspection or evaluation, this report is limited to those elements available and engineering judgement as to the likely condition of unseen elements. 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. PRE-EARTHQUAKE BUILDING CONDITION

This section discusses the form and earthquake resisting capacity of the building prior to the Darfield Earthquake.

The information available for review included: Master Floor Plan from Canterbury District Health Board [3], partial architectural ground floor plan of the original building [4], original structural drawings [22], architectural drawings of the Orthopaedic Outpatient extension [5], partial plans of internal alterations that were carried out in 2006 and a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [6].

2.1 BUILDING FORM

The Orthopaedic Outpatient / BSU Hostel (formerly the Spinal Injuries Hostel) was designed in 1978 and constructed in the period there after. An extension to the southeast corner of the Orthopaedic Outpatient Unit was added at a later date. Limited information about the original building and subsequent extensions and renovations was available at the time of writing this report.

Figure 2-1: BSU Hostel

The original building is a single storey, timber framed structure, approximately rectangular in plan of 50m x 25m. The roof is of lightweight timber construction with timber trusses supporting metal tray roofing. The timber trusses are predominantly supported on timber framing, except for the dining area where SHS post supports are used between external glazing. The ceilings are a combination of low level and high level gypsum board ceilings, and a ceiling tile assembly which runs along the building corridors. The high level ceilings are fixed to timber roof purlins spanning above exposed timber trusses. The low level gypsum board ceilings are fixed to timber ceiling joists spanning between external and internal walls. The ceiling tiles are a heavy plaster acoustical tile assembly with the light gauge steel supporting tracks fixed directly to the timber ceiling framing above. The extent of the ceiling types as well as a truss layout is indicated in Figure 2-2 below.

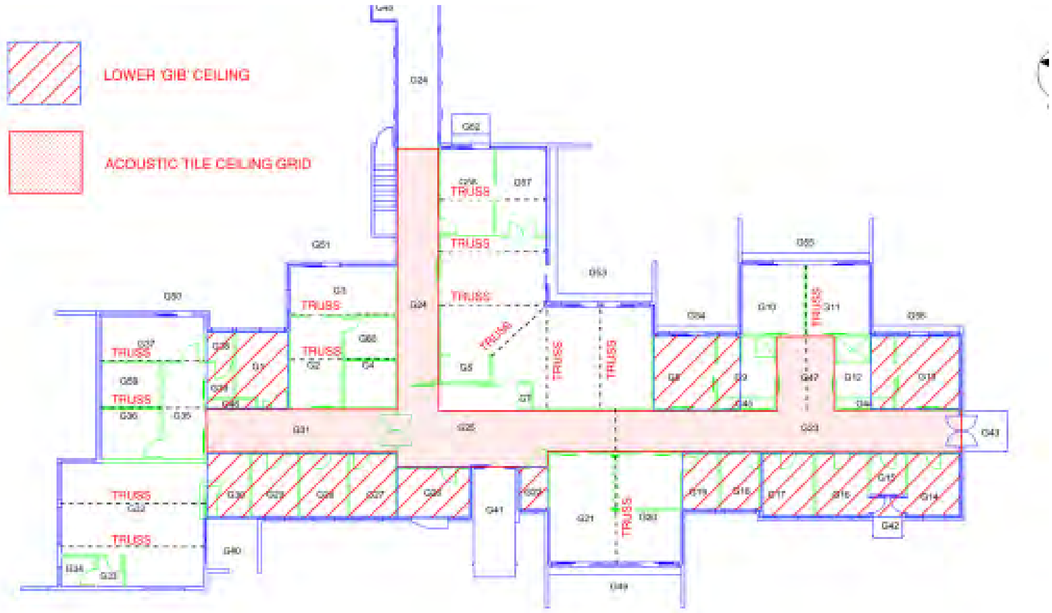


Figure 2-2: Ceiling and Truss Layout

The external walls are predominantly of timber framed construction with either 90mm reinforced concrete block masonry veneer or external weatherboard claddings. The external timber walls are typically framed with 100mm x 50mm studs and lined on their interior face with gypsum wallboard. There are also sections of exterior wall framed with 150mm x 50mm studs. The internal walls are clad on each face with gypsum wallboard, which extends up to the ceiling line. Several 190mm reinforced masonry block 'wing' walls extend out from the building at various locations. A later single level extension (for the Orthopaedic Outpatient) consists of a similar construction.

The concrete block veneer is 190mm thick, reinforced with MD12 bars at 800mm centres, each way and fixed to the timber framing with galvanized ties at 900mm centres horizontally and 400mm centres vertically. There is a 50mm cavity between the block veneer and the timber framed stud walls. The block 'wing' walls are typical reinforced with MD16 bars at 600 centres each way.

The building has a suspended ground floor concrete slab to provide a crawlspace and routes for services under the building. A service duct and corridor connect through to the building to the North. The elevated concrete slab is formed by pre-cast Unispan floor planks with a 75mm topping slab reinforced with cold drawn wire mesh (similar to the floor assemblies in the Spinal Injuries Unit and the Physical Medicine Unit). The planks span to the concrete sub-floor walls which are in turn supported by continuous concrete footings.

2.1 LATERAL LOAD RESISTING SYSTEMS

The lateral load resisting system of the building consists of timber stud bracing walls clad in gypsum wallboard. For the majority of the building gypsum board lined ceiling diaphragms are used to distribute the lateral loads into regularly spaced internal and external walls in each direction. Along the corridors, where the ceiling tile assembly is not capable of acting as a ceiling diaphragm, lateral loads are transferred to the top plates and supporting walls through direct bending of the ceiling framing. There is no diaphragm or bracing present in the roof plane.

Because there is no contiguous diaphragm present in the ceiling or roof frame, lateral loads are assumed to be distributed to the bracing walls on a tributary area basis.

At the ground floor level the precast floor units, along with the reinforced topping slab, act as a rigid diaphragm to distribute lateral loads to the concrete sub-floor walls and partial basement below.

The lateral load resisting system below the ground floor level is significantly stiffer than the timber or the steel framed portions of the superstructure above. As a result these portions of the superstructure have been treated as being de-coupled from the concrete sub-floor above for the purpose of this evaluation.

2.2 PRE-EARTHQUAKE BUILDING CAPACITY

2.2.1 Code Comparison

The building capacity under earthquake actions discussed in this section is compared to the capacity that a similar building would be designed to today. A new building would be designed to the *Structural Design Actions Standard, Part 5: Earthquake Actions – New Zealand*, NZS 1170.5:2004 [9] (incorporating the amendments made to this standard as a result of the Lyttelton Earthquake as outlined in the Amendment 10 of the Building Code [8]) and/or the New Zealand Standard *Timber Framed Buildings*, NZS 3604:2011[11]. The implications of the amendments following the Lyttelton Earthquake are discussed more fully in the Burwood Hospital Campus Base Report however, for a building of this type the amendments essentially result in an increase to the design loads of 36 % when compared to pre-earthquake NZS 1170.5:2004 [9] design levels and by 67% when compared to pre-earthquake, NZS3604:2001 [11], design levels.

It is reasonable to assume that when the building was originally designed in 1978, the likely loading standard referenced at the time was either the *New Zealand Standard Model Building Bylaw – Chapter 8, Basic Design Loads*, NZSS 1900:1965 [10] and/or the NZSS95:1944 *New Zealand Standard Model Building By-Law for Light Timber Framed Construction*. The NZSS 1900:1965 [10] loading standard is referenced on a partial architectural Ground Floor Plan in regard to bracing requirements. When these By-Laws were written, neither the seismology of the different areas within New Zealand, or the impact this could have on buildings was as well understood as it is today. Along with an increase in the seismic demands required by the change in the loading code over this period, the seismic detailing requirements have also progressed significantly resulting in more ductile and better performing buildings.

The current code requires a new building to be designed for an earthquake, known as the Design Basis Earthquake (DBE), which is based upon the buildings physical location, local soil conditions, building type, fundamental period and importance level.

While the original structural drawings were available for review, neither the calculations nor the specifications were available, so the exact design and loading assumptions originally made are unknown. For the purposes of this report seismic loading assumptions have been made based on a detailed review of the drawings available and physical observations of the building.

The Orthopaedic Outpatient / BSU Hostel is not regarded as an essential hospital facility by the CDHB and is therefore classified as an Importance Level 2 building in accordance with NZS 1170:2004 [10] The associated return period of the NZS 1170:2004 [9] DBE is 500 years, with an associated risk factor for design of R = 1.0. The sub soil for the site is taken as Soil

Type D, which is consistent with the findings of a post-earthquake geotechnical investigation [6].

As the superstructure of the Orthopaedic Outpatient / BSU Hostel is timber framed, and has been assigned a standard importance level (IL2), it has been assessed to NZS 3604:2011[11]. The requirements of NZS 3604:2011[11] incorporate the DBE earthquake for the specific site conditions. The bracing output is roughly equivalent to a NZS 1170:2004 [9] analysis assuming an Importance Level 2 building, a risk factor, R = 1.0, and a wall bracing ductility factor, μ=3.5.

Based upon the period of construction the concrete floor diaphragm and subfloor walls have been assumed to have nominal ductility, and as such the reinforced concrete walls have been assigned a ductility factor of μ=1.25. The timber framed superstructure has been assigned a ductility factor of μ=3.3 based on the existing timber properties and strength values of the diaphragm and shear walls.

A comparison between the Design Basis Earthquake (DBE) of NZSS 1900:1965 and NZS 1170:2004 for the site is plotted below. Based upon a fundamental building period below 0.50 seconds, the seismic demands on the timber framed superstructure and the concrete sub-floor structure have increased by approximately 210% and 560% respectively.

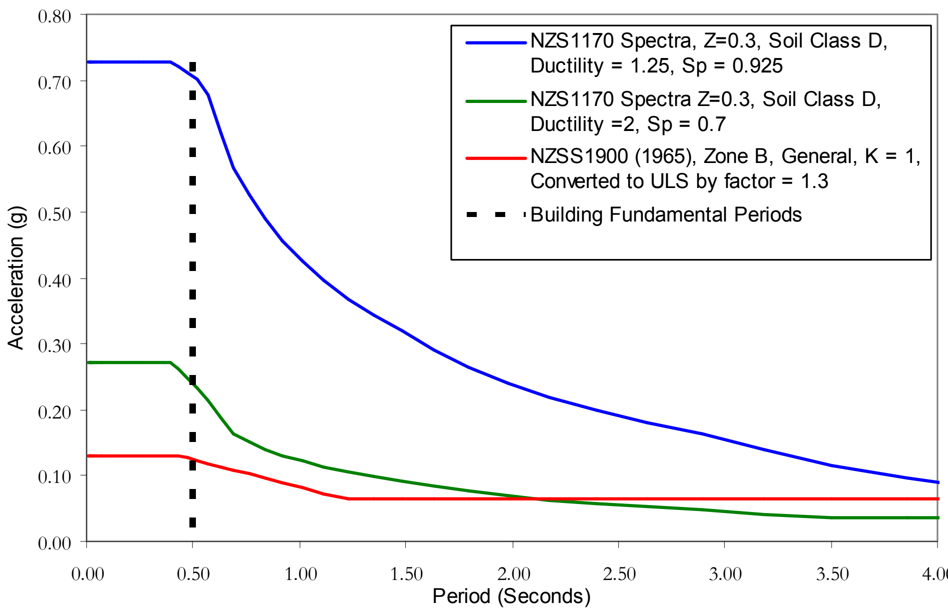


Figure 2-3: Comparison of Design Codes

2.2.2 Equivalent Static Analysis to NZS1170.5 & NZS 3604

In addition to the direct code comparison provided above, an equivalent static analysis to NZS1170.5: 2004 [9] & NZS 3604:2011 [11] has been carried out for the sub-structure and superstructure respectively in order to gain a better understanding of the buildings estimated 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.

Following the Lyttelton earthquake, a geotechnical report was conducted by Tonkin & Taylor, titled "Burwood Hospital Post Earthquake Geotechnical Assessment", dated June 2011 [6]. This report has been used to aid in the evaluation of the site conditions and the likely effect of the ground on the buildings past and future performance. The soil parameters described in the geotechnical report completed by Tonkin and Taylor has been used for the evaluation of the buildings existing foundation system.

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 [12]. 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. These are described in more detail in the Burwood Hospital Base report [1] and include strength governed elements such as short columns and deflection governed elements such as floor and stair elements with inadequate support seating.

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 the strength and deflection requirements.

As previously noted, the timber framed structure has been treated as being de-coupled from the ground floor slab and concrete sub-floor walls below. The timber framed structure has been evaluated using the bracing requirements of NZS 3604:2011 [11].

For the purpose of this evaluation of the timber frame portion of the structure several assumptions also had to be made in regards to the existing timber building properties. Specifically, the existing bracing capacities of interior and exterior walls are of primary concern. The expected strength values for these elements were taken from NZSEE 2006, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes* [12] and ASCE 41-06, *Seismic Rehabilitation of Existing Buildings* [13]. These values could be further refined through destructive investigations of the existing materials. The assumed diaphragm and shear wall expected strength values are as follows:

- Exterior Walls: Timber framed stud walls with gypsum wallboard cladding on the interior face. Expected strength = 1.5kN/m (30BU/m) with ductility, $\mu = 3.3$

- Interior Walls: Timber framed stud walls with gypsum wallboard cladding on each face. Expected strength = 3.0kN/m (60BU/m) with ductility, $\mu = 3.3$

- Ceiling Diaphragm: Timber ceiling joists with direct fixed gypsum wallboard (where it occurs). Expected strength = 1.5kN/m (30BU/m) with ductility, $\mu = 3.3$

The bracing requirements in NZS 3604:2011 assume a ductility factor, $\mu = 3.5$ for the bracing walls and diaphragms. To account for the less ductile existing walls outlined above, the wall bracing demands from NZS 3604:2011 have been factored up proportionally as required in our analysis. As a result of intrusive investigations moreover, the bottom plate and fixings into the concrete floor have been found to be insufficient. The capacities of the bracing elements have been in half to account for the shallow depth of the bottom plates and the small fixings.

As the building consists of stepped ceilings throughout a contiguous ceiling diaphragm across the building cannot be assumed, therefore re-distribution of loads to the bracing walls would be limited. The assessment of the bracing walls has been based on the tributary area supported by the wall lines.

Values for the bracing supplied by the reinforced concrete sub-floors walls have been taken from NZS 3604:2011. The foundations have been assessed at an ultimate bearing capacity of 150kPa, for factored loads, as per recommendations provided by Tonkin and Taylor.

The majority of the Orthopaedic Outpatient / BSU Hostel building has been assessed to have a pre-earthquake capacity to resist approximately 25% of the demand imposed by the current loading code Design Basis Earthquake (DBE) in the North-South direction (across the building) and approximately 25% DBE in the East –West (along the building) direction (IL3 - 20%DBE). At isolated extensions of the building, the external wall bracing has been upgraded to meet the minimum external bracing requirements of NZS 3604:2011 [11], as these sections of the building were assessed as low as approximately 15% DBE. See Figure 2-4 below for the specific locations where exterior bracing walls were upgraded with plywood walls.

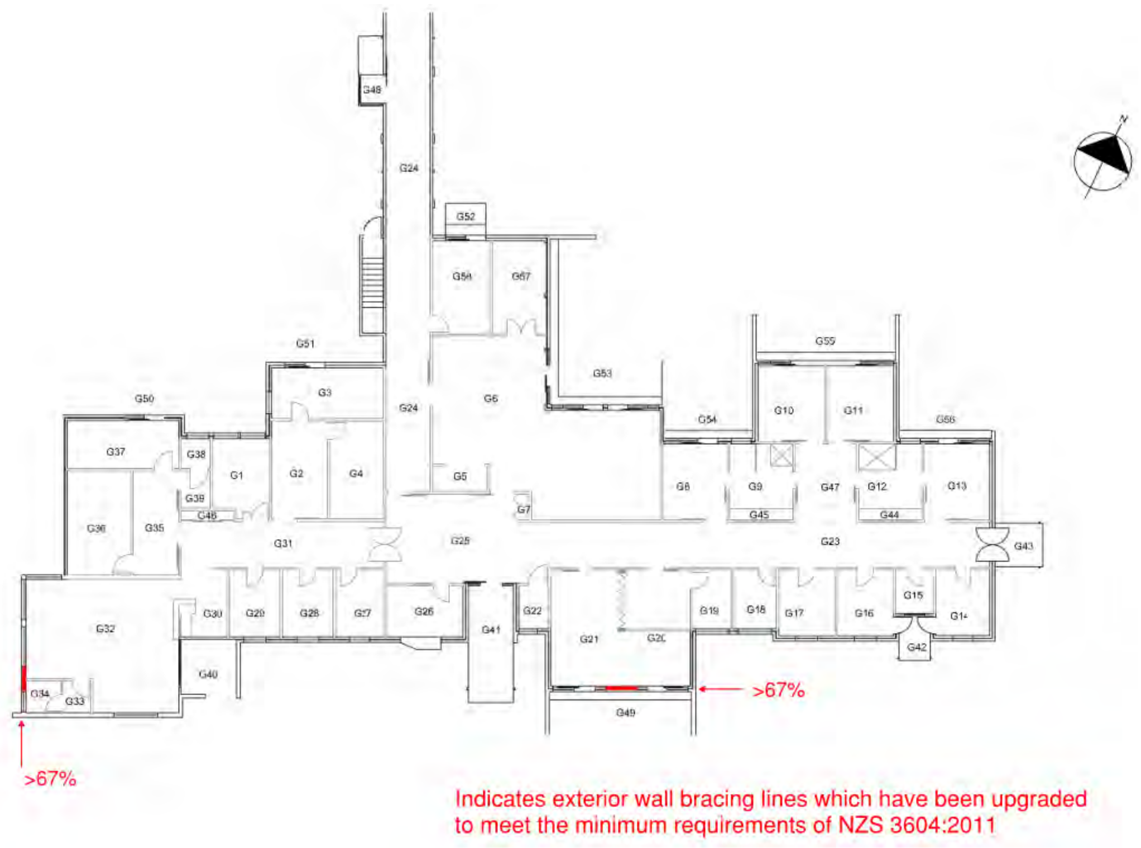


Figure 2-4: Walls upgraded to meet NZS3604

A summary of the %DBE for each primary element has been noted in Table 2-1 and Table 2-2.

Building Element	%DBE (IL2)	%DBE (IL3)	Comments
Ceiling Diaphragm - N-S - E-W	100% 100%	100% 100%	Based on 3604 maximum spacing of bracing elements and specific design of the Dining Area and Outpatients extension
Typical Wall Bracing - N-S - E-W	25% 25%	20% 20%	Limited by small ground floor fixings and shallow bottom plate found during further investigations
Isolated Exterior Bracing - N-S - E-W	40% 40%	30% 30%	Some isolated external walls have been upgraded with plywood bracing

Table 2-1: Timber Superstructure - Seismic Assessment %DBE

Building Element	%DBE (IL2)	%DBE (IL3)	Comments
Ground Floor Diaphragm - N-S - E-W	100% 100%	100% 100%	
Sub-floor bracing - N-S - E-W	100% 100%	100% 100%	
Foundations - N-S - E-W	100% 100%	100% 100%	

Table 2-2: Concrete Substructure – Seismic Assessment %DBE

If the building were to be assessed for an increased importance factor, IL3, the seismic demand would increase by 30% (R=1.3) and as such the assessed capacities would be reduced proportionally.

A review of the drawings available and site observations revealed no obvious Critical Structural Weaknesses (CSW's).

As a result of portions of the building being assessed at below 33% DBE, in its current state, the Orthopaedic Outpatient / BSU Hostel is considered to be "Earthquake Prone" in terms of section 122 of the Building Act. Christchurch City Council current policy requires that buildings identified as "Earthquake Prone" be strengthened to a target of 67% of current code requirements when seeking consent for repairs. The minimum strengthening required however, is to 33% DBE.

Despite the low % DBE of the bracing walls, timber framed buildings by their nature have built in redundancies and as such are unlikely to fail in a brittle manner.

Methodology to improve the seismic performance of the buildings and provide strengthening to achieve 67% DBE have been included in Section 5.

3. POST-EARTHQUAKE BUILDING CONDITION

This section covers the structural damage sustained by the Birthing Unit and Minor Procedures Unit, and its effect on the buildings capacity to resist seismic loads, as a result of the series of earthquakes which includes the Darfield Earthquake that struck at 4:36am on 4th September, 2010, the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011, the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake subjected the building to strong ground motions which significantly exceed the full design earthquake load for buildings of this nature and appears to have caused the majority of the earthquake damage observed, after the Darfield event.

3.1 THE LYTTELTON EARTHQUAKE

The fundamental period of the building is estimated to be approximately 0.4 seconds. 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 strong motion data presented in the Base Report, it appears the Lyttelton Earthquake produced ground shaking intensities of approximately 60-120% of the current Ultimate Limit State design spectra for an Importance Level 2 building.

It should be noted that the Lyttelton Earthquake was very short in terms of strong shaking produced with the strong motion only lasting for duration of 5-7 seconds. Rupture of the Alpine Fault is expected to contain 50 to 60 seconds of strong motion.

3.2 PRELIMINARY INVESTIGATIONS

Preliminary investigations were carried out to identify 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
- damage observed during 'Rapid' Structural Assessments following the Lyttelton Earthquake, June 13th aftershocks and subsequent events

The following areas were identified for potential damage:

- movement or damage to structure associated with ground movement and/or settlement
- cracking and joint failure of concrete sub-floor walls, service tunnels and foundations
- connections of timber roof framing to exterior timber stud walls

- distress and cracking of gypsum clad bracing walls and ceilings
- signs of distress at connection of interior and exterior stud walls to precast floor system below
- distress and cracking of reinforced concrete block veneer and connection to timber framing above
- cracking to masonry block 'wing' walls
- signs of distress at interfaces between different sections of the building

Rapid Level 2 assessments were carried out on the 24th February 2011[17] and on the 14th [18] and 15th June 2011 [19] following the June 13th earthquakes. Two additional Rapid Visual Structural Assessment was conducted on 24th December 2011 [20] and 5th January 2012 [21], following the 23rd December 2011 and 4th January 2012 events. These structural observations involved a complete walk around the exterior and throughout the interior of the building. The following primary areas of damage were identified from the damage assessments:

- minor cracking to internal linings at joints
- possible movement to external pavements
- minor cracking to external block work

Although no significant damage or settlement was noted in the earlier assessments, due to the building age and other damage observed throughout the hospital campus it was considered a more detailed inspection was required. The aim of the detailed inspections was to determine the full extent of the damage caused to the building, particularly those elements identified for potential damage above. These areas were targeted to identify if damage had occurred, and to what extent the damage had reduce the capacity of the buildings lateral load resisting system to resist future seismic events.

3.3 DETAILED STRUCTURAL OBSERVATIONS

Further detailed inspections have been carried out following the initial assessments to ascertain the full extent of structural damage. The majority of the detailed structural observations were completed on the 14th March 2012.

A full record of these observations can be found in Appendix A, with reference plans describing the location labelling used found in Appendix B. A full photographic record of the observations is available electronically on request. The detailed structural observation identified the following items:

- cracking or linings at the junction of wall to wall and wall to ceiling linings
- minor cracking to mortar lines of 190mm concrete masonry block and concrete masonry block veneer
- vertical separation of 190mm concrete masonry block from the adjacent concrete masonry block veneer
- post-earthquake 'bounce' in elevated precast floor assembly.

3.4 GEOTECHNICAL REVIEW

A review of the ground damage and conditions was carried out by Tonkin & Taylor for the Burwood Hospital Campus. A subsequent report titled *Burwood Hospital Post Earthquake Geotechnical* Assessment was issued in June 2011 [6]. The geotechnical review concluded that the settlement and damage to building foundations and slabs on the Burwood Hospital Campus was likely due to the liquefaction of underlying soil layers. It is believed that excessive pore water pressures have been dissipated and that further settlement is not expected to occur, unless another significant event was to occur.

While the total settlement experienced by the building is unknown, it has been estimated that settlement on the order of 110-200mm has occurred at other locations on the hospital campus. At the Outpatient / BSU Hostel building differential settlement on the order of 45mm has been observed. It appears as though the high point is centred over a service tunnel below which is founded in deeper soils than the exterior concrete sub-floor walls.

Based upon the geotechnical report provided by Tonkin & Taylor [6] the potential for future total and differential settlements at the Burwood Hospital Campus varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

3.5 LEVEL SURVEY

A detailed survey of the ground floor levels in the Orthopaedic Outpatient / BSU Hostel building was conducted by Fox & Associates and issued on the 18th April, 2012 [7]. The survey indicates a differential settlement of approximately 30-45mm between the lower external perimeter of the building and a central higher portion of the building. The worst case resulting slope is approximately 1:200 over a 4m length. This is typically within the acceptable range for standard occupancy buildings, however given the nature of the patient group occupying the building, CDHB may wish to pursue re-levelling of the building. A discussion on how this could be achieved has been included in Section 4.2.

For the extent of the differential settlement noted see the level survey included in Appendix C.

3.6 SUMMARY OF BUILDING DAMAGE

The following is a summary of the observations made for the building, and our conclusions as to their condition and seismic load resisting capacity. These observations do not specifically distinguish between damage caused by the Darfield Earthquake, the Lyttelton Earthquake or any significant aftershocks, such as those that occurred on the 13th June 2011, the 23rd December 2011 or the 2nd January 2012. Despite not being able to specifically distinguish when individual damage observed occurred, it is believed that the majority of the damaged can be linked to the February 22nd event.

The Orthopaedic Outpatient / BSU Hostel building appears to have performed relatively well considering the age of the building and the seismic actions experienced at the site. Unlike other areas of the hospital campus the differential settlement observed at the Orthopaedic Outpatient / BSU Hostel building is also more limited than many other locations on site and thus the associated damage to the concrete sub-floor walls and the timber framed superstructure above is much more limited.

Our observations suggest that the building would have undergone a limited number of full cycles of primarily elastic deformation. The short duration of the strong ground motion recorded and the damaged observed would support this hypothesis. A summary of the building damage observed can be typified as follows:

- **Differential Ground Settlement** – As previously noted differential settlement on the order of 45mm have been observed. It appears as though the high point is centred over a service tunnel below which is founded in deeper soils than the exterior concrete sub-floor walls. The worst case resulting slope is approximately 1:300 over a 14m length.

- **Distress to Wall and Ceiling Finishes** – Cracking, warping and general distress has been noted to the wall and ceiling linings throughout. The cracking in the gypsum board wall and ceiling linings has typically occurred at the junction of wall to wall and wall to ceiling linings. The warping of the finishes has typically occurred at wall intersections.

- **External Masonry Veneer and Concrete Masonry Walls** – Minor cracking has been observed along the mortar lines to the 190mm thick reinforced concrete masonry wing walls and reinforced masonry block veneer in isolated locations.

- **Elevated Precast Floor -** At the North end of the building some post-earthquake 'bounce' has been noted in the elevated pre-cast floor assembly. This is potentially due to minor cracking between the floor and the walls reducing the fixity at the supports.

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. Secondary elements, such as windows and fittings, have not generally been reviewed.

3.7 ADDITIONAL INVESTIGATIONS REQUIRED

Several assumptions were made in the completion of the Pre-Earthquake (undamaged state) and Post-Earthquake (damaged state) Structural Assessments. Destructive exploration is required in a number of locations in order to verify these assumptions.

3.7.1 Investigations Required For Further Assessment

The areas requiring further investigation to finalize the assessments are as follows:

- Based upon the damage observed further investigations of the exterior block façade is warranted. This includes a summary of the veneers general condition, investigation of damaged mortar joints and a review of the fixings to the exterior timber stud walls. This work should be completed by a qualified Mason and should include any repair recommendations the Mason may have.

  An inspection of the concrete block veneer and wing walls has been completed by S A Thelning Brick & Block Layer. Their report is dated 10th September 2012. A copy of the report has been given in Appendix D. The brick ties have been found to be in overall very good condition at 900mm horizontally and 400mm vertically. For a summary of the recommended block work repairs see Section 4.

- Based upon staff feedback, and observations made on site, it appears as though there are locations where the elevated precast floor assembly has become 'bouncier' post-earthquake. At these locations further investigations are required to determine if delamination has indeed occurred between the precast units and the mesh reinforced topping slab.

The floor in the areas that have been identified as 'bouncier' are constructed with Unispan flooring with spiral loops into the concrete topping. Due to this construction type, delamination is unlikely to be the cause of the perceived 'bounciness'. A small amount of extra flexibility in the floor may be due to minor cracking at the supports reducing the fixity at the support.

3.7.2 Investigations to be Completed During Building Repairs

- Re-inspection of building will be required upon completion of any re-levelling works, to determine if any additional damage has occurred.

- Check existing timber stud wall framing and fixings to concrete slabs below where new and/or repaired wall linings are to be installed.

  Investigations have shown that the bottom plate in this building are generally only 25mm think. The fixings are also insufficient with M10 bolts at approximately 1200mm crs. The estimate of the lateral capacity of the structure in Section 2.3 has been updated to include the findings of these investigation. A strengthening scheme to improve this has been included in Section 5.

- Check existing timber ceiling framing and fixings to timber bracing walls below where new and/or repaired ceiling linings are to be installed.

- Check existing collector elements and connection to bracing walls where new and/or repaired wall linings are to be installed.

3.8 POST-EARTHQUAKE BUILDING CAPACITY

Based upon our investigations to date, we do not consider the Orthopaedic Outpatient / BSU Hostel building to have any significant reduction to the overall gravity load resistance of the structure.

Generally the modest damage observed to the gypsum board linings of the bracing walls will have resulted in a minimal reduction in lateral load capacity, although the actual reduction in strength is difficult to quantify. While there has been some reduction in strength, according to the Department of Building and Housings, *Revised Guidance on Repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence* [14], the primary result of the damage noted will be a reduction in the stiffness of the wall bracing. Based upon the movement observed in the building a similar reduction in stiffness can be expected to the sections of gypsum clad ceiling linings. The reduction in stiffness will cause some ongoing concerns in regards to the buildings performance, primarily to contents and non-structural elements.

The differential settlement observed in the building will also have resulted in a reduction in the overall lateral load resisting capacity of the building but again this is hard to quantify. This includes a reduction in the future differential settlement the building could absorb before severe distress to the structure occurs.

The damage observed will require repair to restore the strength, stiffness, durability and performance of the individual structural components. The repair work is outlined in Section 4. Following the recommended repair of the structural damage, the lateral load resisting performance of the structure will be restored to approximately pre-earthquakes levels (see Section 2.4).

Post-earthquake strengthening has been implemented to the south end of the building to bring the assessed capacity of the external wall of the two extensions to above 33% DBE with the installation of two new plywood lined bracing walls. Due to the insufficient bottom plate and fixings however, the capacity of the building is currently limited to 25% DBE (IL3 - 20%DBE). Recommendations for strengthening to improve seismic performance and bring the building to above 67% DBE are included in Section 5.

4. OBSERVED DAMAGE & REPAIRS REQUIRED

4.1 PRIMARY DAMAGE OBSERVED AND REPAIRS REQUIRED

This section covers the damaged noted during our detailed assessment of the building. Note that our observations have been restricted to structural aspects of the building 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.

Table 4-1 provides a photographic summary of the observed damage and typical repairs required for the Orthopaedic Outpatient / BSU Hostel. Table 4-1 should be read in conjunction with Appendix A – Record of Observation and Appendix B – Reference Plans. The Repair Specification [2] referred to in Table 4-1 have been issued separately.

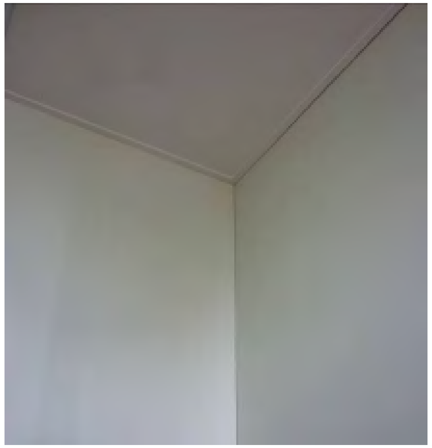
In general, the aim of the repair work indicated in this section is to restore the structure to its pre-earthquake state, as far as practicable, while maintaining the utility of the building. The repairs presented attempt to address the loss of strength, stiffness and durability of the structural elements due to the damage noted.

It should be noted that more damage may be identified during the repair works and (if required) additional repair details will be specified accordingly.

Further recommendations for improvement to the buildings seismic performance, and to achieve a minimum capacity of 67% DBE have been included in Section 5.

Table 4-1: Photographs of Observed Damage and Repairs Required

Damaged Item & Location	Damage	Recommendations	Example Photograph
1. Concrete service tunnels, sub-floor walls and foundations			
1.1. Differential ground settlement	Differential ground settlement of approximately 45mm resulting in a worst case slope in the ground floor slab of approximately 1:300	Based on serviceability concerns re-levelling may be required using either mechanical jacking pressure injected grout techniques. Refer to discussion on re-levelling in Section 4-2 for additional information. (Note: All re-levelling is to occur prior to any other structural or cosmetic repairs).	

Damaged Item & Location	Damage	Recommendations	Example Photograph
2. Timber Framed Structure			
2.1. Interior Wall Linings	Separation of wall linings at existing joint locations. Typical Throughout.	Replace damaged wall boards with new gypsum board sheets. All wall boards to remain are to be re-fixed as per Section 4.3. For additional strengthening options, see Section 5.	
2.2. Ceiling Linings	Cracking noted along ceiling at interface of wall and ceiling linings. Typical throughout.	Replace damaged ceiling boards with new gypsum board sheets. All ceiling boards to remain are to be re-fixed as per Section 4.4. For additional strengthening options, see Section 5.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
3. External Masonry Block Walls			
3.1 External Masonry Block 'Wing' Walls	Diagonal cracking along mortar lines (not transferred through the concrete masonry block).	Mortar beds to be raked out and re-pointed as per Repair Specification.	
3.2 Intersection of Block Veneer and 190mm Block Wall	Vertical crack at intersection of block veneer and 190mm block wall to room G40.	Repair of seal to be specified by others.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
4. Exterior Block Veneer			
4.1. Exterior Block Veneer at Orthopaedic extension.	Horizontal Hairline Cracking along mortar lines noted in block veneer below windows.	Mortar beds to be raked out and re-pointed as per Repair Specification.	n/a

4.2 DISCUSSION ON BUILDING RE-LEVELLING

The detailed survey of the ground floor levels in the Orthopaedic Outpatient / BSU Hostel building, conducted by Fox & Associates, indicates a differential settlement of approximately 45mm between the lower external perimeter of the building and a central higher portion of the building centred over a concrete service tunnel below (see Appendix C for complete level survey). The worst case resulting slope is approximately 1:300 over a 14m length. While the resulting slopes in the ground floor slab are within the typical acceptable range for standard occupancy buildings, CDHB may wish to pursue re-levelling of the building due to the nature of the patient group occupying the building, and ongoing serviceability concerns.

As noted previously, there has also been some reduction in the capacity of the building as a result of the differential settlements noted, along with a reduction in the future differential settlement the building could absorb before severe distress to the structure occurs.

The two primary re-levelling options available include the use of mechanical jacking or the use of either underpinning grout or engineered resin. There are pros and cons of each solution which extend beyond structural performance which will need to be considered by CDHB. These include continuity of operation, degree of re-levelling accuracy and the willingness of the re-levelling sub-contractor to provide a producer statement, amongst other items.

From a structural standpoint, either option is acceptable provided the use of underpinning grout or engineered resin does not create any "hard points" under the building. If "hard points" are created during the re-levelling process the potential for future differential settlements can be increased. If this were to occur it would reduce the capacity of the building going forward.

An evaluation of the suitability of underpinning grout to re-level this specific building, and avoid the "hard points" noted above is still required by Tonkin & Taylor.

The building also lends itself nicely to the use of mechanical jacking due an elevated ground floor slab and the relatively good shape of the exterior and interior concrete sub-floor walls in this area. The exterior sub-floor walls are believed to be similar to the sub-floor walls of the Spinal Injuries Unit which are roughly 1 meter in depth, heavy reinforced and well detailed, and should easily span between jacking locations placed under the sub-floor walls. *This would need to be verified prior to any re-levelling*

The global suitability of re-levelling the building through the use of either mechanical jacking or underpinning grout (or engineered resin) at this specific site will need to be verified by qualified sub-contractors in conjunction with the geotechnical consultant. During the re-levelling process there is also the risk that addition damage could occur to the buildings linings, exterior block veneer, etc. and appropriate contingencies should be provided. Both options would also likely require raising all the exterior and interior footings to the internal high point in the slab located over the service tunnel below.

It should be noted that both options discussed above are not expected to increase the seismic performance of the building or reduce the potential for future differential settlements. Instead the options presented are designed to re-level the building without making the future performance of the building any worse than it was prior to the earthquakes. Based upon the geotechnical report provided by Tonkin & Taylor [6] the potential for future total and differential settlements at the building site would remain between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

To improve the future performance of the building, and reduce the potential for future differential settlements, would likely require the entire footprint of the building to be either piled or the ground under all the sub-floor wall footings, service tunnels and the partial basement improved. *Further geotechnical investigations would be required into the type and depth of ground improvement required.*

4.3 REPAIR OF TIMBER FRAMED BRACING WALLS

The wall linings to the interior and exterior bracing walls have been damaged in locations and require repair. Based upon the movement observed it is also believed the wall lining fixings have been damaged throughout. We believe this has resulted in a reduction to the ongoing strength and stiffness of all the bracing walls. In order to reinstate the pre-earthquake strength and stiffness to the bracing walls, the repair recommendation is to remove all cracked or damaged sections of the wall linings and replace them with new gypsum board sheathing. The new gypsum board sheathing is to be fixed in accordance with GIB Ezybrace system GS2-N specifications (or equivalent) to internal walls and GIB Ezybrace system GS1-N to the external walls (except where the strengthening works, as specified in Section 5.1, calls for an alternative bracing system). Existing internal wall linings, as indicated in Figure 4-1, to remain are to be re-fixed to the existing studs in a similar manner. Any non-gypsum wall boards will need to be replaced in conjunction with these repairs. A new finish is then to be applied to all interior walls.

All repairs to wall bracing are to be completed after any re-levelling to the building has been completed.

Further investigations into the fixings of the bracing walls into the sub-floor concrete walls have shown that the bottom plates are generally only 25mm thick with M10 bolts at approximately 1200mm crs. These fixings will need to be upgraded to enable the bracing walls to reach capacity. A methodology for this is outlined in Section 5.

Figure 4-1: Ground Floor Plan – Damage Repairs

4.4 REPAIR OF GYPSUM BOARD CEILINGS

Similarly to the wall linings, a portion of the existing sections of gypsum clad ceiling diaphragms have been damaged and require repair. In addition to the repairing of the wall linings, the ceilings will need to be re-fixed throughout in order to reinstate their pre-earthquake strength and stiffness of the diaphragms.

The repair recommendation is to remove any cracked or damaged sections of gypsum board ceiling lining and replace them with new gypsum board sheathing fixed in accordance with GIB specifications. All existing ceiling linings that to remain are to be re-fixed to existing ceiling joists in a similar manner. A new finish is then to be applied to all ceilings.

All repairs to wall bracing are to be completed after any re-levelling to the building has been completed.

Figure 4-2: Reflected Ceiling Plan – Damage Repairs

4.5 REPAIR OF CONCRETE BLOCK WORK

An inspection of the concrete block veneer and wing walls has been completed by S A Thelning Brick & Block Layer. Their report is dated 10th September 2012. A copy of the report is given in Appendix D. The report noted that "the blockwork is still in very good condition and seems structurally sound". There are several cracked blocks which require epoxy injection, and some mortar joint cracking which requires grinding out and repointing. Straight internal mortar joints are also cracked and should be ground out for sealant installation.

5. STRENGTHENING RECOMMENDED

The primary lateral force resisting system of the Orthopaedic Outpatient / BSU Hostel superstructure consists of timber framed roof, floor and ceiling diaphragms, which transfer lateral loads to sheet clad timber bracing walls. The sub-structure consists of a rigid precast floor diaphragm, with a reinforced insitu topping, over concrete sub-floor walls and footings below.

As noted in Section 2, Pre-Earthquake Building Condition, and Section 3, Post-Earthquake Building Condition, the lateral load resisting capacity of the majority of the building (as a percentage of the loads imposed by the Design Basis Earthquake) has been assessed at a pre- and post-earthquake capacity of approximately 25% DBE. The sub-structure below the ground floor has been assessed at a capacity above 100% DBE.

The minimum strengthening required to achieve 67% DBE has been included in Section 5.1 below.

5.1 STRENGTHENING WORKS TO ACHIEVE 67% DBE

The initial strengthening scheme for the BSU Hostel, included in Revision 2 of this report, involved upgrading the bracing capacity of a small number of walls to bring the overall bracing of the system up to 67% DBE. This scheme was based on the assumption that the bottom plate fixings to the concrete floor were adequate. However, further investigations into the suitability of the bottom plate connections have revealed that not only are the fixings inadequate (M10s at approx. 1200mm spacing), but the bottom plate itself is only 25mm thick, meaning the initial strengthening scheme is no longer appropriate.

One option to bring the BSU Hostel up to 67% DBE is to increase the thickness of the bottom plate and upgrade the connections in every wall in the building. This may be an unnecessary amount of work however, and perhaps a more feasible option is to improve the bracing quality of some of the longer walls such that upgrading the bottom plate connection is required only in these walls. This would mean that the nominated walls would be considered responsible for the full bracing of the building. Though some bracing capacity would remain with the under designed original walls, it is difficult to quantify the probable performance and is unlikely they would provide much redundancy.

To keep the required work to a minimum, GIB BLP-H lining (GIB braceline one side, plywood the other) is recommended for the internal walls. External walls only required plywood to the inside face. A mark-up of the suggested walls to strengthen is shown in Figure 5.1. An attempt has been made to choose the walls that appear to be easiest to access, though there is some flexibility to change the specific walls chosen if needed.

While the heavy plaster ceiling tile assembly has not been identified thus far as being damage (based upon the observations to date), the tiles assembly has been identified as the primary risk to building occupants as they could shake loose and dislodge during a significant earthquake. It was originally recommended, in Revision 2 of this report, that the ceiling tiles through the corridor were replaced with a GIB ceiling diaphragm as shown in Figure 5.2; this would now be a requirement for the nominated wall system to be effective.

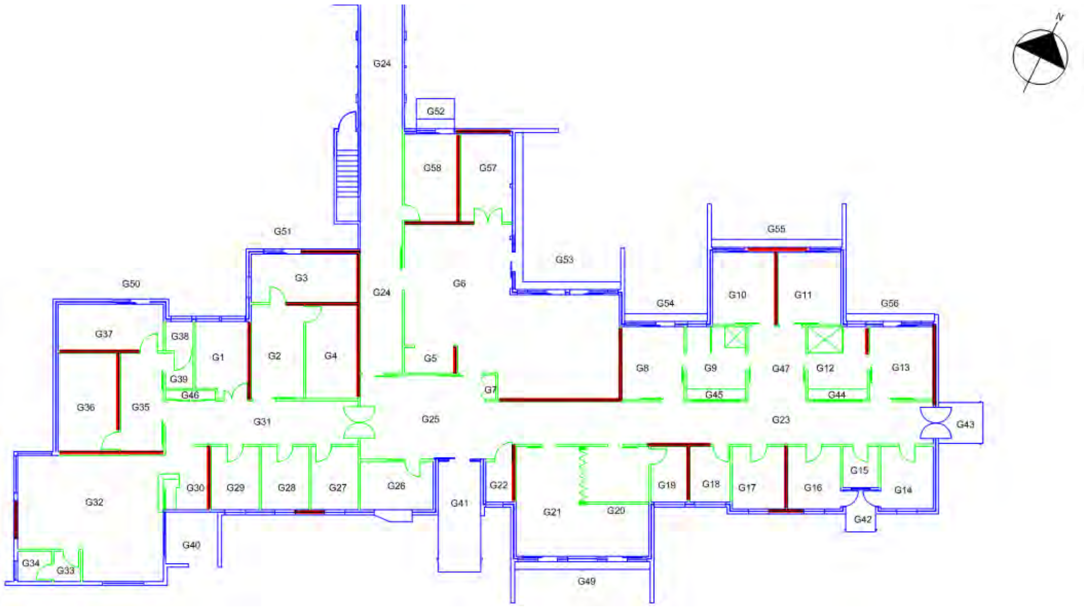


Figure 5-1: Proposed Strengthening Scheme – Plywood Lined Walls

Figure 5-2: Ceiling Plan – Proposed Strengthening

6. REFERENCES

1. *Burwood Hospital – Detailed Seismic Assessment Report – Base Report*, Holmes Consulting Group, November 2011.

2. *Burwood Hospital – Detailed Seismic Assessment Report – Earthquake Repair Specification*, Holmes Consulting Group, July 2011.

3. *Burwood Hospital Spinal Hostel, Ground Floor Master Floor Plan,* Canterbury District Health Board, Revised 15 June 2009.

4. *Partial Ground Floor Plan,* Original architectural drawing, date and author unknown.

5. *Burwood Hospital - Spinal Injuries Unit Hostel, Waiting Room / Reception Additions & Alterations,* Independent Design.

6 *Burwood Hospital Post Earthquake Geotechnical Assessment*, Tonkin and Taylor Ltd., June 2011.

7 *Burwood Elevation Survey – Revision F*, Fox & Associates, April 2012

8 *Compliance Document for New Zealand Building Code - Clause B1 – Structure, Amendment 10 (Canterbury)*, Department of Building and Housing, Wellington, 19 May 2011.

9 *Structural Design Actions Part 5: Earthquake Actions – New Zealand, NZS 1170.5:2004*, Standards New Zealand, 2004.

10 *New Zealand Standard Model Building Bylaw – Chapter 8 Basic Design Loads*, NZSS1900:1965, New Zealand Standards Institute, 1965

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

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

13 *Seismic Rehabilitation of Existing Buildings,* ASCE 41-06, American Society of Civil Engineers, 2007

14 *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury – Part 2 Evaluation Procedure*, Engineering Advisory Group, July 2011

15 *Revised Guidance on Repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence*, Department of Building and Housing, November 2011

16 *Practice Note – Design of Conventional Structural Systems Following the Canterbury Earthquakes*, SESOC, December 2011

17 *CDHB – Burwood Hospital Campus – Rapid Visual Inspection: 106186.03 Site Report 1,* Holmes Consulting Group, February 2011

18 *CDHB – Burwood Hospital – Rapid Seismic Assessment Post June 13th Earthquakes: 106186.03 Site Report 4,* Holmes Consulting Group, 14 June 2011

19 *CDHB – Burwood Hospital – Rapid Seismic Assessment Post June 13th Earthquakes: 106186.03 Site Report 5,* Holmes Consulting Group, 15 June 2011

20 *CDHB – Burwood Hospital – Post Earthquake Rapid Structural Assessment following 5.8 and 6.0 Magnitude Earthquakes, 106186.03 Site Report* 8, Holmes Consulting Group, 24 December 2011

21 *CDHB – Burwood Hospital – Post Earthquake Rapid Structural Assessment following 2nd January 5.5 Magnitude Earthquakes, 106186.03 Site Report 9*, Holmes Consulting Group, 9 January 2012

22 *Burwood Hospital – Paraplegic Hostel,* Original Structural Drawings, Frederick Sheppard and Partners Consulting Engineers, June1978.

Appendix A

Record of Observations

APPENDIX A – RECORD OF OBSERVATIONS & REPAIRS

Inspection date: 14 March 2012

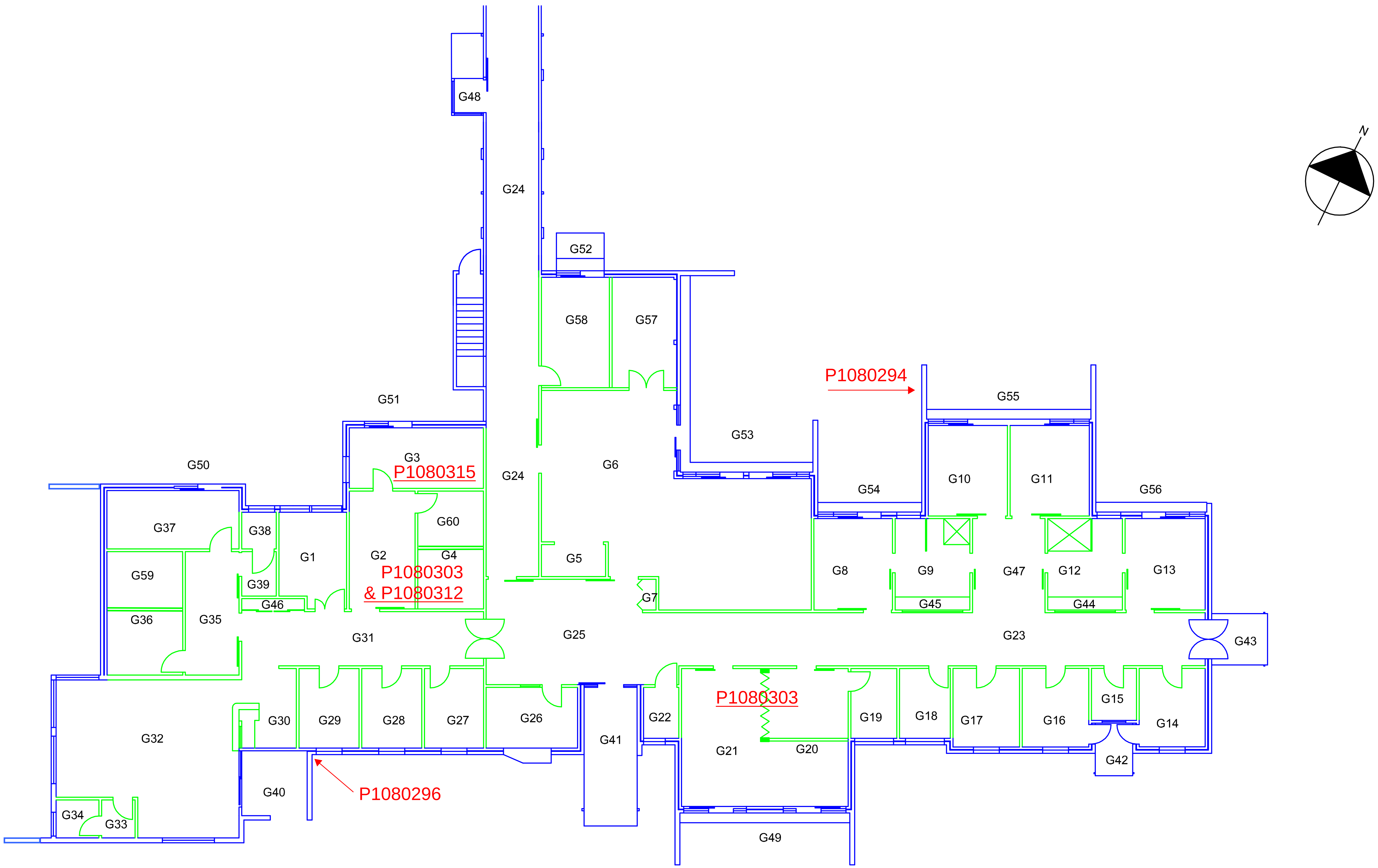
KEY	
N	No repair required
Y	Repair required
F	Further investigation required
C	Repair complete

Level	Building Element	Location	Observations	Repair Required	Repair	Photo Reference
G	External masonry block 'wing' wall	G10	Diagonal cracking along mortar lines	**Y**	Mortar beds to be re-pointes as per HCG specification	P1080294
G	External masonry block veneer	G40	Seperation of 190mm block wall from masonry veneer	**Y**	Epoxy inject crack in accordance with HCG specification	P1080296
G		G32	Minor settlement induced horizontal cracking along mortar lines	**Y**	Epoxy inject crack in accordance with HCG specification	n/a
G	Internal Linings	G20 & G21	Cracking of linings at junction of walls to walls and walls to ceilings	**Y**	All cracked and damamged wall and ceiling boards are to be removed and re-lined with gypsum board sheeting. All existing gypsum boards to remain are to be re-fixed to the timber wall studs and ceiling joists.	P1080303
G		G04 & G60	Cracking of linings at junction of walls to walls and walls to ceilings	**Y**	All cracked and damamged wall and ceiling boards are to be removed and re-lined with gypsum board sheeting. All existing gypsum boards to remain are to be re-fixed to the timber wall studs and ceiling joists.	P1080311 P1080312

Level	Building Element	Location	Observations	Repair Required	Repair	Photo Reference
G		G03	Cracking of linings at junction of walls to walls and walls to ceilings	**Y**	All cracked and damamged wall and ceiling boards are to be removed and re-lined with gypsum board sheeting. All existing gypsum boards to remain are to be re-fixed to the timber wall studs and ceiling joists.	P1080315
G		G29	Ceiling on apparent lean	**Y**	Survey	n/a

Appendix B

Reference Plan



REFERENCE PLAN

AMENDMENT	DRAWN	CHECKED	DATE
MASTER DRAWING CREATED FROM 04000003 AND CDHB BORDER ADDED	BJT		15-06-09

106186.78 Burwood Hospital
BSU Hostel Interim Report
Rev 3_24Sep13

BURWOOD HOSPITAL
SPINAL HOSTEL, GROUND FLOOR
MASTER FLOOR PLAN

Canterbury
District Health Board
Te Poari Hauora ō Waitaha

DESIGNED:
DRAWN: BJT
CHECKED:
DATE: 15-06-09

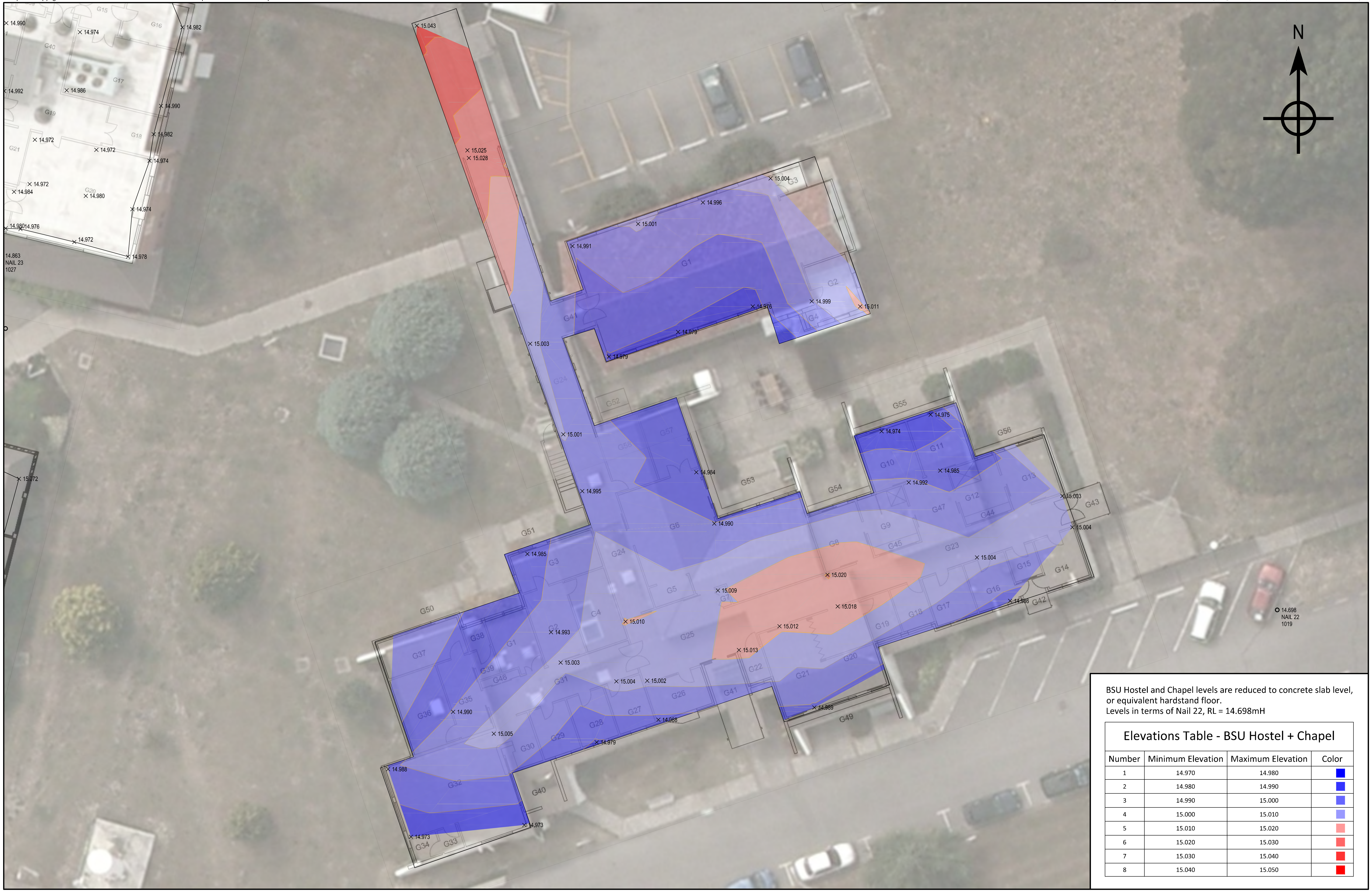
DWG No. 04032060
SCALE 1:100 AT A1
REVISED 15-06-09

XREFED DRAWING/S:

Maintenance and Engineering Department - Christchurch Hospital

Appendix C

Level/Elevation Survey



BSU Hostel and Chapel levels are reduced to concrete slab level, or equivalent hardstand floor.
Levels in terms of Nail 22, RL = 14.698mH

Elevations Table - BSU Hostel + Chapel

Number	Minimum Elevation	Maximum Elevation	Color
1	14.970	14.980	
2	14.980	14.990	
3	14.990	15.000	
4	15.000	15.010	
5	15.010	15.020	
6	15.020	15.030	
7	15.030	15.040	
8	15.040	15.050	

No.	Revision	Approved	Date
G			
F	Beacon House, BSU Hostel, Chapel, Tapper and Milner added	MJM	18/04/2012
E	Audit observations post Dec 23 2011 (not included in this Revision)	MJM	01/02/2012
D	Wards 7 & 8, Surgical Block, Melrose Chairs & Corridor added	MJM	02/12/2011
C	Phys Med, Parafed, Food Services, BIRS, Boilerhouse, EDC added	MJM	24/11/2011
B	Nurses Hostel & Champion Centre, SOU elevations added	MJM	18/11/2011
A	Administration and Allan Bean Centre elevations added	MJM	31/10/2011

032005

Scale		1:125
Reduced	A3	1:250
Designed		----
Drawn		DHP
Checked		MJM
Date		18/04/2012

106186.78 Burwood Hospital
BSU Hostel Interim Report
Rev 3_24Sep13

Burwood Elevations Survey

BSU Hostel + Chapel

SURVEYORS
URBAN DESIGNERS
Fox & Associates
CONSULTANTS
LAND DEVELOPMENT
www.foxsurvey.co.nz
0800 FOX SURVEY
P.O.Box 13-390
CHRISTCHURCH

Job No. 2462C
Sheet No. 16 of 19
Rev. F

Appendix D

Block Veneer Report

S A Thelning Brick&Blocklayer

4 Chamberlain Lane

Rangiora

10/09/2012

Re:Burwood Hospital,Earthquake Damage Report

To whom it may concern,I have been asked to carry out this report on behalf of Naylor Love.Please find below my findings on each of the buildings inspected.

CI # 0144 - 1)BSU Hostel:The blockwork is still in very good condition and seems structurally sound.There is several cracked blocks(I suggest epoxy injectionof these).There is quite a bit of mortar joint cracking(requires grinding out &repointing).Straight internal motar joints are also cracked(Recommend grinding out &sealant installed).

CI # 0147 - 2)Spinal Unit Façade:The blockwork is still in very good condition and seems structurally sound.There has been movement around the windows(Need to be resealed with sealant).The block window sills have moved(Grind out &repoint).The internal vertical & control joints have cracked(Grind out &install sealants).Mortar joints have cracked throughout(Grind out &repoint).East side sills missing(To be supplied &installed by others)

CI # 0149 - 3)Admin Building:The brick veneer is generally still in good condition however there has been moderate movement above the glass foyer causing separation of the bricks(needs to be replaced).The brickwork to the internal walls of the Café area is sound & in good condition.There have been adequate ties installed to all areas.The mortar joints are all in good condition.

S.R No: 5

Any further information on this report contact Simon on 0272427902

Yours Sincerely

S.A.Thelning

HolmesConsultingGroup

DETAILED SEISMIC ASSESSMENT REPORT

STRUCTURAL AND CIVIL ENGINEERS

BURWOOD HOSPITAL CAMPUS

REPORT 1 – ADMINISTRATION BUILDING

PREPARED FOR

CANTERBURY DISTRICT HEALTH BOARD

106186.27

INTERIM REPORT REV 3 – 29 OCTOBER 2014

BURWOOD HOSPITAL CAMPUS – INTERIM DETAILED SEISMIC ASSESSMENT REPORT

REPORT 1 – ADMINISTRATION BUILDING

Prepared For:
CANTERBURY DISTRICT HEALTH BOARD

Date:	29 October 2014
Project No:	106186.27
Revision No:	3

Prepared By:

Eric McDonnell
SENIOR PROJECT ENGINEER

Reviewed By:

Jenny Ovens
PROJECT DIRECTOR

Holmes Consulting Group LP
Christchurch Office

REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
07/02/12	1	Interim results of quantitative assessment (Phase 3) for discussion (some on site investigations still to be completed)
06/08/12	2	Update repair recommendations and assessment of entry canopy / architectural precast wall
29/10/14	3	Update to include repair for northern corridor foundation separation and reference to 100% IL2 and 100% IL3 Preliminary Strengthening Concepts.

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EXECUTIVE SUMMARY

Holmes Consulting Group has been engaged by Canterbury District Health Board (CDHB) to complete a full structural review of the Burwood Campus following the Lyttelton Earthquake. A series of reports have been compiled as part of this process. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The specific building reports, like this one for the Administration Building, should be read in conjunction with the base report, and refer to the repair specification.

This report identifies the structural damage sustained by the Administration Building as a result of the series of earthquakes, including the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The report summarises the effects of the damage on the lateral load capacity of the building, and provides structural repairs for the damage identified. The general form of the building, along with its capacity relative to current code levels, has been included for the buildings pre-earthquake undamaged state and post-earthquake state. Recommendations for improving the seismic performance of the building have also been identified, along with a discussion on the building's likely performance under Maximum Considered Earthquake (MCE).

The Administration Building is a single story structure designed in 2001 and constructed in the period thereafter. The main portion of the building consists of two bays of steel portal frames in the north-south direction, with a timber framed atrium space in between. In the east-west direction the primary structural walls are constructed of reinforced concrete block and are clad in a brick veneer. The roof assembly consists of profiled metal roofing over a combination of timber and cold-formed steel purlins. At the rear of the Administration Building (north end) there is a corridor that links the Administration Building to the Food Services Building, the Orthopaedic Rehabilitation Unit and the Surgical Orthopaedic Unit.

At the entrance to the building there is a large (mostly independent) canopy structure and precast architecture wall feature. A small glass entry is suspended from the architecture precast wall lintel on one end and connected back to the main structure on the other.

The information available for the review included: the original 2001 structural drawings [3], a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [4], along with a level survey of the building completed by Fox & Associates [5].

The majority of the Administration Building appears to have performed relatively well, with the bulk of damage related to differential ground settlement of up to 90mm and/or lateral stretching. In particular, the service tunnel under the main entry to the building has not settled to the same degree as the surrounding spread footings, creating distress in this area. This has resulted in the damage to the glass sliding doors at the entry, cracks to adjacent partition walls and ceiling finishes, cracking to adjacent concrete block walls and damage to the exterior brick veneer near the main entrance.

The lateral stretching noted above is most evident in the main corridor of the building where gaps of up to 10mm have formed at the construction joints in the architecturally exposed concrete slab on grade.

Additional typical damage includes cracking to partition wall and ceiling finishes, along with localized cracking to the concrete block walls. At the roof level, several precast parapet capping stones have also been loosened and/or dislodged.

Damage has also been noted to the entry canopy structure and architectural precast wall. This includes hairline cracking at the base of the canopy columns, hairline cracking in the precast wall elements, visible movement at the base of the precast wall, along with a permanent measured lean of the wall of approximately 1% and in the columns of up to 2%. There has also been damage noted at the hanging glass entry to precast wall connection.

It is believed that the majority of the damage observed, including the onset of damage, occurred as a result of the 22nd February event. Further observations of the earthquake damage observed have been included in the body of this report.

Based upon a review of the drawings available, and the site investigations completed, the primary lateral force resisting elements of the Administration Building were assessed in their pre-earthquake undamaged state. The assessed capacity of the building, relative to the demand imposed by the current loading code Design Basis Earthquake (DBE), is approximately 80% DBE in the north-south direction and approximately 100% DBE in the east-west direction. The limiting factor in the north-south direction is the bracing provided to the steel portal frame beams to prevent buckling under seismic loading.

The entry canopy and architectural precast wall has been assessed at 40% DBE based upon the capacity of the screw piles under the architectural precast wall.

For the purposes of this assessment the Administration Building has been considered to be Importance Level 2 buildings (IL2, R=1.0). If the buildings were to be assessed for an increased importance factor, IL3, the seismic demand would increase by 30% (R=1.3) and as such the assessed capacities would be reduced proportionally.

Based upon an assessment of the damage observed, we do not believe the overall capacity of the building has been significantly altered by the damage noted. The cracking to the block walls, along with differential settlement noted, will have resulted in some reduction in capacity, but the exact percentage is hard to quantify. The primary concern from a structural standpoint would be a reduced ability of the structure to withstand future differential settlements prior to the onset of more severe damage.

The majority of the repairs required are serviceability related rather than structural in nature. This includes re-levelling of the building, which based upon the geotechnical recommendations provided by Tonkin & Taylor, can be achieved with the use of either underpinning grout or mechanical jacking techniques. It is likely the existing ground floor slab will be required to be demolished and replaced in conjunction with the re-levelling process. Any interior fit out supported by the slab would also be required to be demolished and replaced. Further discussion on the re-levelling options is included in Section 4.2.

At the entry canopy and architectural precast wall, the likely repair for the permanent lean in the wall and columns will be demolition and replacement. It may be possible to re-level the footing under the precast wall, but there is a risk the permanent lean in the canopy columns will remain. If re-levelling of the wall and canopy is attempted the steel reinforcing at the base of the wall will need to be further investigation to ensure strain hardening of the bars has not occurred.

The minimum repairs required to reinstate the building, and the entry canopy/wall, to their pre-earthquake undamaged condition, has been included in Section 4. In addition to the minimum repairs, recommended strengthening concepts to increase the seismic performance of the building have been included in section 5.

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. Secondary elements, such as windows and fittings, have not generally been reviewed.

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 and/or strengthening of the building have been completed.

1. INTRODUCTION

Holmes Consulting Group has been engaged by Canterbury District Health Board to complete a full structural review of the Food Services Block at Burwood Hospital following the Lyttelton Earthquake. A series of reports have been completed as part of this. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The individual building reports, like this one, should be read in conjunction with the base report and refer to the repair specification.

The Burwood Hospital base report covers the purpose and scope of the structural review. The current statutory requirements relevant to earthquake damaged buildings are outlined and the level of shaking experienced at the site estimated. Earthquake induced ground settlement damage across the campus is also discussed. The repair specification has been prepared to include repair details for typical damage observed in buildings on the Burwood Hospital and is referred to as required in the specific building reports.

1.1 SCOPE OF WORK

This report is on the Administration Building located on the Burwood Hospital Campus at 255 Mairehau Road, Burwood, Christchurch. The report identifies the general form of the structure, along with the gravity and lateral load resisting systems. Each component of the structural system was reviewed based upon the information available and any potential Critical Structural Weaknesses (CSW's) were noted.

The report also identifies the structural damage observed to date as a result of the series of Earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake, in particular, subjected the building to strong ground motions which significantly exceeded the current code loading demand for buildings of this nature.

The capacity of the Administration Building has been assessed relative to current code loading in the buildings pre-earthquake undamaged state and in its post-earthquake damaged state. The post-earthquake assessment summarises the effects of the damage identified on both the gravity and lateral load resisting elements. Repair options to restore the buildings capacity to pre-earthquake levels for strength, durability and stiffness have been included. The repair options aim to maintain the buildings utility. Where required, strengthening options have also been provided.

1.2 LIMITATIONS

Findings presented as a part of this project are for the sole use of the Canterbury District Health Board, 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. Because all of the structure has not been available for detailed inspection or evaluation, this report is limited to those elements available and engineering judgement as to the likely condition of unseen elements. 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. PRE-EARTHQUAKE BUILDING CONDITION

This section discusses the form and capacity of the building prior to the Darfield Earthquake

2.1 BUILDING FORM

The Administration Building at the Burwood Hospital campus was designed in 2001 and constructed in the period there after. The structural design for the building was provided by Powell Fenwick Consultants Ltd.

The building is a single level structure used for administration purposes. The structure comprises of an entry canopy structure and architectural precast wall feature, the main administration block, and a corridor at the north end of the building which provides access to adjacent buildings.

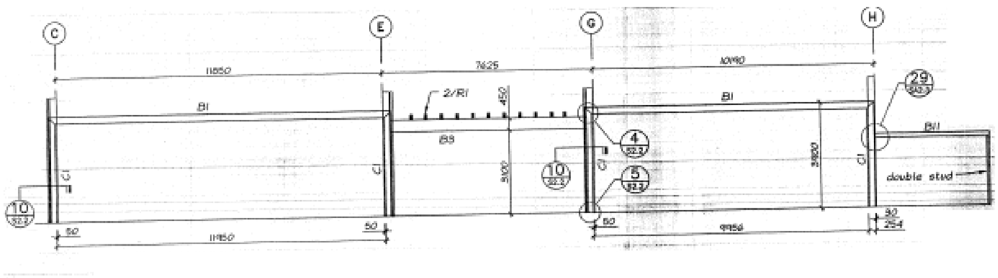
The information available for the review included: the original structural drawings[3], a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor[4], along with a level and verticality survey of the building, completed by Fox & Associates[5].

Figure 2-1: Administration Building – Main Entry

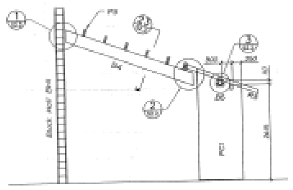
Main Administration Block: The main portion of the Administration Building consists of two bays of steel portal frames in the north-south direction, with a timber framed atrium space in between. The spaces are divided in the perpendicular direction by reinforced concrete block walls. The gravity system of the main block consists of light-weight profiled metal deck roofing on a combination of cold-formed steel and timber roof purlins. The roof purlins are supported by the concrete block walls or glulam beams which span between the block walls.

The lateral load resisting system for the main block consists of the steel portal frames in the north-south direction and the reinforced concrete block walls in the east-west direction. The steel portal frame columns are cast into the block walls. The building has no apparent roof diaphragm to transfer lateral loads to the walls and portal frames below. The forces are therefore transferred to the lateral load resisting elements directly through the roof framing elements (purlins and rafters), and through face loading of the block walls between the portal frames. The block walls and portal frame columns are primarily founded on shallow continuous reinforced concrete footings.

In general there are tie elements connecting the footings, except across the central corridor of the building, running in the north-south direction. The ground floor slab consists of a wire mesh reinforced slab on grade with underfloor heating. There is also a service tunnel which runs under the length of the building (see Figure 2-3).



SECTION THROUGH MAIN BLOCK



SECTION THROUGH CORRIDOR

Figure 2-2: Building Sections

Corridor: The gravity system of the corridor consists of light-weight metal deck roofing on a combination of cold-formed steel and timber purlins. The roof purlins are supported by glulam beams which frame in to precast concrete blade columns at one end and either concrete block walls or steel columns at the other end. In the north-south direction, lateral loads are resisted by the precast blade columns and in the east-west direction lateral loads are resisted by a combination of steel braced frames and gypsum board lined timber bracing walls. The corridor is entirely founded on shallow continuous reinforced concrete footings, which are connected by periodic tie beams. The slab on grade of the corridor is reinforced with wire mesh.

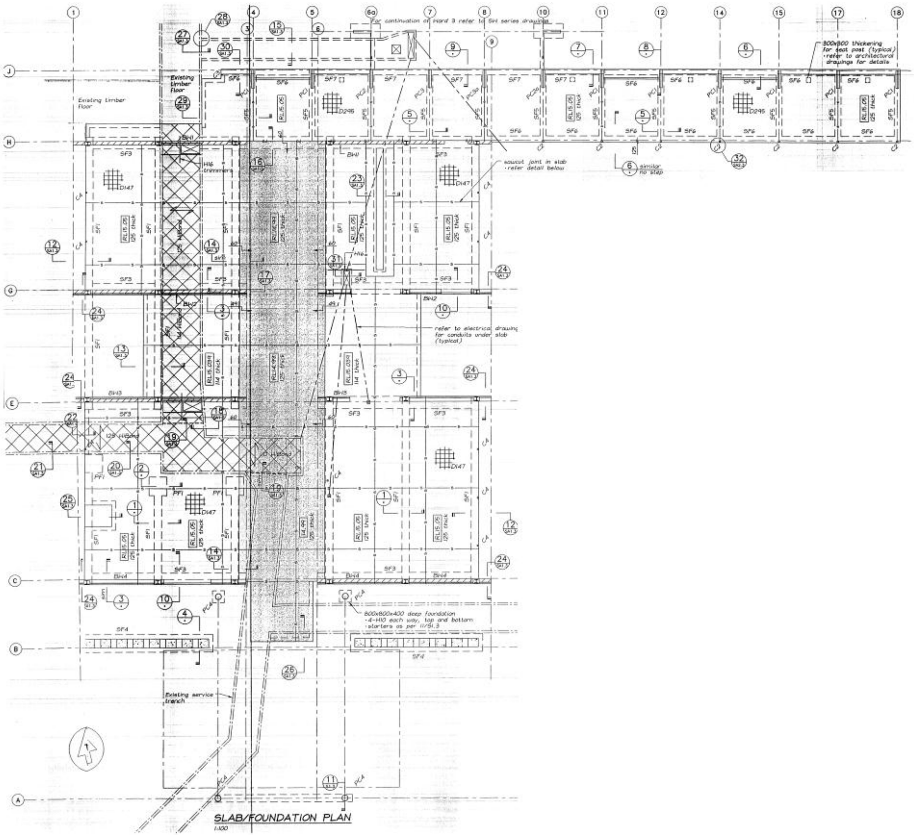


Figure 2-3: Ground Floor / Foundation Plan

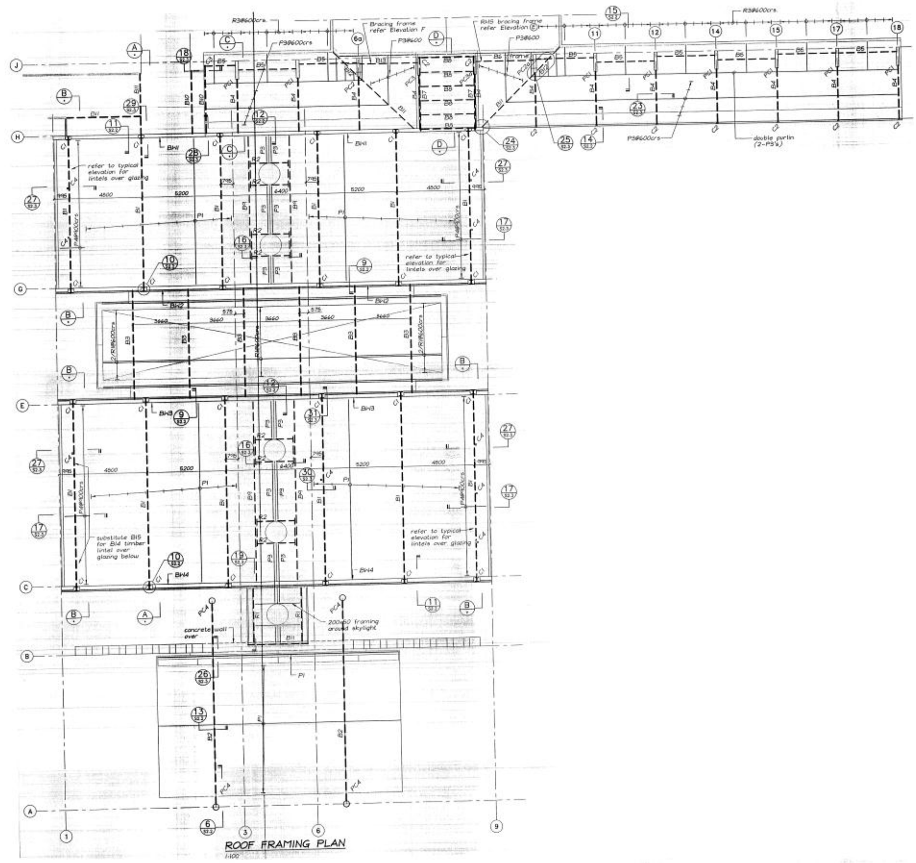


Figure 2-4: Roof Framing Plan

Entry Canopy and Architectural Precast Wall: The canopy structure consists of a light weight profiled metal roof over cold-formed steel purlins which span between steel UB beams. The steel beams are supported at either end by precast concrete columns (4 in total). At the south end of the canopy, the columns are supported by a continuous reinforced concrete footing. On the north end, the columns are supported by isolated spread footings. The continuous footing at the south end provides fixity at the base of the columns which provides the lateral support for this end of the canopy in the east-west direction. At the north end of the canopy, seismic resistance in the east-west direction is provided by the architectural precast concrete wall. In the north-south direction, lateral loads are resisted by cantilever action of the precast wall. Fixity is provided at the base of the wall by a moment couple created by a series of steel screw piles cast into a continuous reinforced concrete footing. For the as-built locations of the screw piles see Figure 2-5 below.

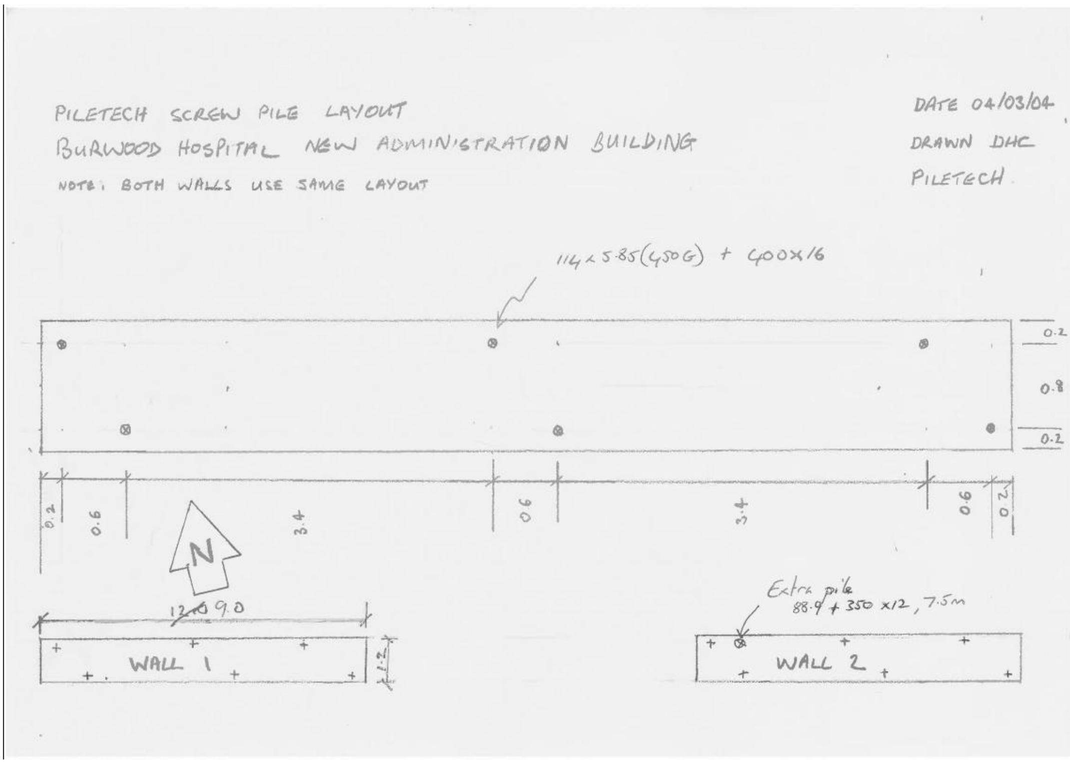


Figure 2-5: Architectural Precast Wall – Screw Pile Locations

The architectural precast wall is constructed of a series of reinforced precast concrete blocks connected by continuous H28 reinforcing bars in grout filled drossbachs ducts. The precast blocks are typically 1800mm long x 600mm tall x 400mm thick. Inside each block is a 200mm thick layer of polystyrene.

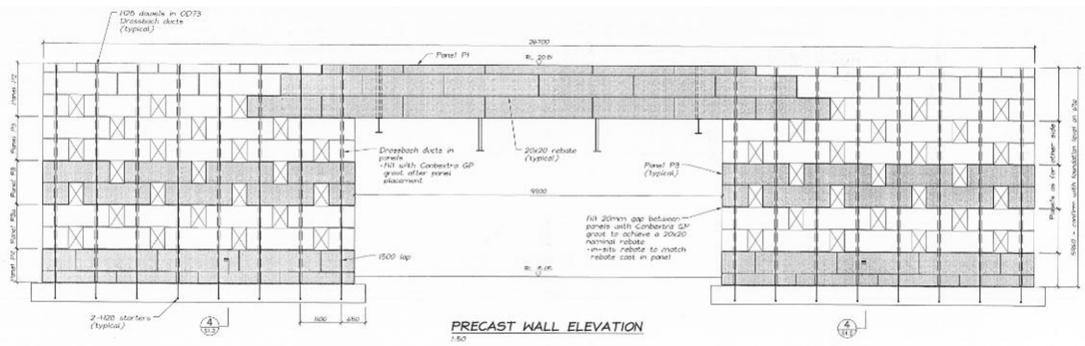


Figure 2-6: Architectural Precast Wall Elevation

A glass framed entry links the main building block to the entry canopy. On the south end the glass entry hangs from the precast wall lintel by two SHS steel posts. Otherwise, the canopy structure and the precast wall are independent structures (see Figure 3-3).

2.2 PRE-EARTHQUAKE BUILDING CAPACITY – DIRECT CODE COMPARISON

The building capacity under earthquake actions discussed in this section is compared to the capacity that a similar building would be designed to today. A new building would be designed to the *Structural Design Actions Standard, Part 5: Earthquake Actions – New Zealand*, NZS 1170.5:2004[9] and incorporating the amendments made to this standard as a result of the Lyttelton Earthquake as outlined in the Amendment 10 of the Building Code [8]. The implications of the recent amendments are discussed more in-depth in the Burwood Hospital Campus Base Report however, for a building of this type the amendments essentially result in an increase to the design loads of 36 % when compared to pre-earthquake design levels.

When the building was designed in 2001 the current loading standard at the time was the Code of Practice for General Structural Design and Design Loadings for Buildings, NZS 4203:1992 [10].

The original structural drawings are available, but the structural calculations and specifications were not, so the exact design and loading assumptions originally made are unknown. For the purposes of this report seismic loading assumptions have been made based on a detailed review of the drawings available and physical observations of the building.

A new building is required to be designed for an earthquake known as the Design Basis Earthquake (DBE), which is based upon the buildings physical location, local soil conditions, building type, fundamental period and importance level. The Administration Building is classified as an Importance Level 2 building in accordance with NZS 1170:2004 [9]. The associated return period of the DBE is 500 years, with a risk factor for design of R = 1.0. The sub soil for the site is taken as Soil Type D, which is consistent with the findings of a post-earthquake geotechnical investigation [4].

Based upon the period of construction, and the detailing of the lateral load resisting elements, the system as a whole in each direction has been concluded to have nominal ductility. As such both the portal frames in the north-south direction and the reinforced concrete blocks in the east-west direction have been assigned a ductility factor of μ=1.25.

A comparison of the Design Basis Earthquake of NZS 4203:1992 [10] and NZS 1170:2004[9] for the site is plotted below. Based upon a fundamental building period below 0.50 seconds, the seismic demands required by the loading code have increased by approximately 40% since 2001. As a result a building designed to 100% of the DBE at the time of construction would currently have a capacity to resist approximately 70% of the demands imposed by the current code level DBE.

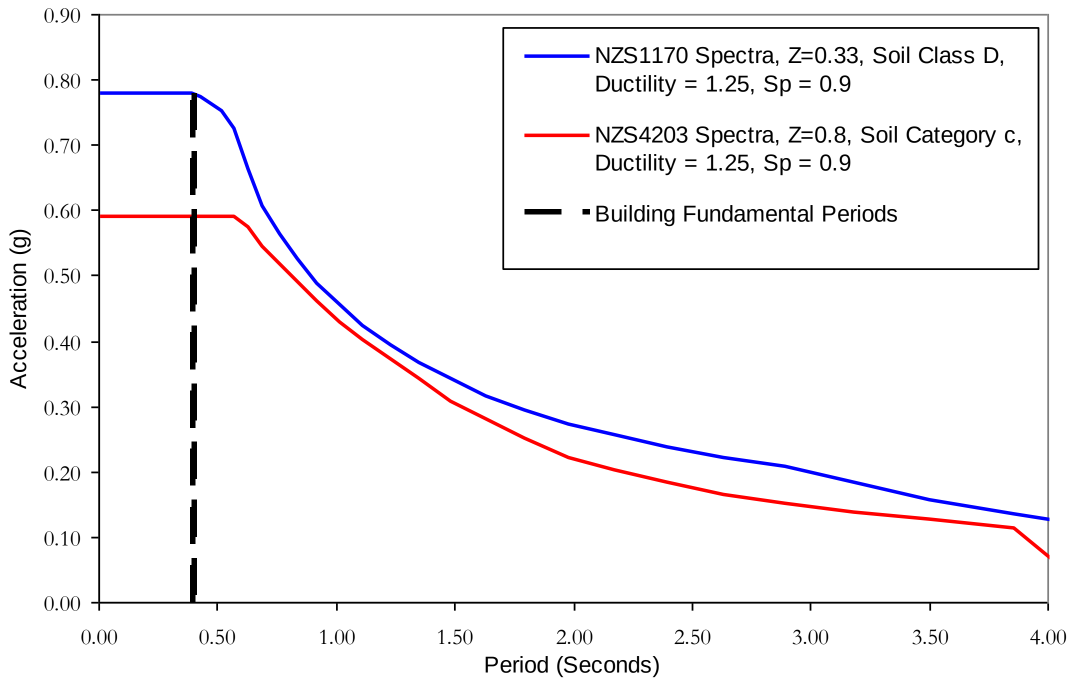


Figure 2-7: Comparison of Design Codes

2.3 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 estimated capacity when compared to current loading standards. The equivalent static analysis was carried out based upon the original structural drawings, and incorporation of on site measurements and as built observations.

Following the Lyttelton earthquake, a geotechnical report was conducted by Tonkin & Taylor, titled "Burwood Hospital Post Earthquake Geotechnical Assessment", dated June 2011 [5]. This report has been used to aid in the evaluation of the site conditions and the likely effect of the ground on the buildings past and future performance. The soil parameters described in the geotechnical report completed by Tonkin and Taylor has been used for the evaluation of the buildings existing foundation system.

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 [17]. 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 DBE 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. These are described in more detail in the Burwood Hospital Base report [1] and include strength governed elements such as short columns and deflection governed elements such as floor and stair elements with inadequate support seating.

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 (DBE), and have been expressed as a %DBE. This includes checks for both the strength and deflection requirements.

Because neither the original structural calculations, specifications nor the general notes were available some assumptions had to be made in regards to the existing material properties of building elements in order to complete the seismic assessment. For example a compressive strength of 4 MPa has been assumed for the reinforced concrete block walls. The foundations have been assessed at an ultimate bearing capacity of 150kPa, for factored loads, as per recommendations provided by Tonkin and Taylor.

In the direction of the steel portal frames (north-south direction), the main administration block of the building has been assessed at approximately 80% DBE in its pre-earthquake undamaged state. This could be improved to 100% DBE with additional restraint to the portal frame beams to prevent buckling under high-seismic loads. In the east-west direction, the 250mm thick reinforced concrete block walls have been assessed at approximately 100% DBE.

At the corridor structure, lateral loads acting in the north–south direction are resisted by precast concrete blade columns, which have been assessed at approximately 100% DBE. In the east-west direction, lateral loads are resisted by two steel braced frames on the northern side of the corridor and by a combination of concrete block walls and gypsum board shear walls on the southern side of the corridor. The limiting factor in the east-west direction is the connection of the RHS steel collector beam to precast blade column connections. These connections have been assessed at approximately 90% DBE.

As previously noted, the main block and corridor of the Administration Building have no apparent roof diaphragm to transfer seismic loads to the lateral load resisting elements below. Thus lateral loads are transferred directly through the roof framing elements (purlins and rafters) on a tributary area basis, and through face loading of the block walls between the portal frames.

In the north-south direction there is also a vertical and horizontal offset in the load path at the low roof linking the two bays of steel portal frames. At this location there are six glulam beams which act to tie the two sections of steel portal frames together. The glulam beams do not line up with the steel portal frames and thus the concrete block walls are required to transfer lateral load between the two elements through weak axis bending. Combined with the flexible nature of the portal frames this makes the brick veneer near these connections susceptible to localized damage.

The architectural precast concrete wall at the main entry to the building has sufficient capacity to provide 100% DBE when ground shaking occurs in the east–west direction. In the north–south direction, lateral loads are resisted by cantilever action of the precast wall. Fixity is provided at the base of the wall by a moment couple created by a series of steel screw piles cast into a continuous reinforced concrete footing. Provided the screw piles are able to achieve the design loads noted on the existing drawings the system would be assessed at approximately 65% DBE.

With that said the stability of the precast wall is reliant on the screw piles which are founded in the medium-dense sand layer directly below the surface. The geotechnical assessment of the site [3] suggests that this layer of sand has a high liquefaction potential. An assessment of the screw pile capacities under the architectural precast wall, completed by Tonkin & Taylor, has determined the capacities used in the original design were likely un-conservative given the local soil conditions. This has lowered the assessed capacity of the entry canopy and precast wall feature from 65% DBE to 40% DBE.

A summary of the %DBE for each primary element has been noted in Table 2-1.

Building Element	%DBE IL2	%DBE IL3	Comments
Steel Portal Frames – N-S Direction	80%	60%	Governed by bracing of portal frame beam
Block Shear Walls – E-W Direction	100%	100%	
Corridor Precast Blade Columns – N-S Direction	100%	100%	
Corridor Steel Braced Frames – E-W Direction	90%	70%	Governed by RHS collector beam connections to precast blade columns
Entry Canopy and Architectural Precast Wall	40%	30%	Governed by capacity of screw piles under precast wall

Table 2-1: Seismic Assessment %DBE

In addition to the primary structural elements noted above the precast parapet capping was identified for evaluation due to the failure of similar capping at the Surgical Services Building. The precast capping was grouted atop the concrete block wall parapets. An assessment of the connection suggested a capacity of approximately 40% DBE. *The precast capping has been removed and replaced with a light metal capping.*

A review of the drawings available and site observations revealed no obvious critical structural weaknesses (CSW's) that could lead to premature collapse of the building.

3. POST-EARTHQUAKE BUILDING CONDITION

This section covers the structural damage sustained by the Administration Building at Burwood Hospital Campus as a result of the series of earthquakes that includes the Darfield Earthquake that struck at 4:36am on 4th September, 2010 and the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011, the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake subjected the building to strong ground motions which likely exceeded the full design earthquake load for buildings of this nature and appears to have caused the bulk of the earthquake damage observed after the initial Darfield event.

3.1 THE LYTTELTON EARTHQUAKE

The fundamental period of the building is estimated to be approximately 0.4 seconds. 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 strong motion data presented in the Base Report, it appears the Lyttelton Earthquake produced ground shaking intensities of approximately 60-120% of the current Ultimate Limit State design spectra for an Importance Level 2 building.

It should be noted that the Lyttelton Earthquake was very short in terms of strong shaking produced with the strong motion only lasting for duration of 5-7 seconds. Rupture of the Alpine Fault is expected to contain 50 to 60 seconds of strong motion.

3.2 PRELIMINARY INVESTIGATIONS

Preliminary investigations were carried out to identify 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
- review of available structural engineering construction documentation
- damage observed during 'Rapid' Structural Assessments following the Lyttelton Earthquake, June 13th aftershocks and subsequent events

In conjunction with a review of the structural drawings for the building the following areas were identified for potential damage:

- movement or damage to structure associated with ground movement and/or settlement
- cracking and joint failure of ground slabs and foundations

- general distress to the steel portal frame beams
- damage to portal frame beam-column joint welds
- damage to concrete block walls at embedded steel column locations – particularly at areas of high differential ground settlement
- connection of glulam beams to concrete block walls in atrium space of the main block
- corridor RHS steel collector beam connections to precast concrete blade columns
- connections of roof framing to exterior wall panels
- roof framing at interface between main building block and corridor
- signs of permanent deformation to architectural precast wall
- distress to original service tunnel and 2001 extension

Rapid Level 2 assessments were carried out on the 24th February 2011[15] and on the 14th June 2011 [16] following the June 13th earthquakes. An additional Rapid Visual Structural Assessment was conducted on 5th January 2012, following the 23rd December 2011 and 2nd January 2012 events. These structural observations involved a complete walk around the exterior and throughout the interior of the building. The following primary areas of damage were identified from the damage assessments:

- spreading of concrete floor slabs at locations of shrinkage control joints
- cracking in ceilings and interior partition walls
- isolated cracking in reinforced concrete block walls
- diagonal "stepped" cracking in exterior brick veneer
- cracking at glass entry hung connection to precast wall spandrel above
- extensive differential ground settlement, particularly at main entry and southeast end of the main administration block

A review of the above information on the building type and preliminary observations highlighted this building as requiring a more detailed inspection. The aim of the detailed inspections was to determine the full extent of the damage caused to the building, particularly those elements identified for potential damage above. These areas were targeted to identify if damage had occurred, and to what extent the damage had reduce the capacity of the buildings lateral load resisting system to resist future seismic events.

3.3 DETAILED STRUCTURAL OBSERVATIONS

Further detailed inspections and structural explorations (including removal of finishes) have been carried out following the initial assessments to ascertain the full extent of structural damage. The detailed structural observations were completed on the 19th October 2011. A full record of these observations can be found in Appendix A, with reference plans describing the location labelling used found in Appendix B. A full photographic record of the observations is available electronically on request. The detailed structural observations, including the removal of linings in isolated locations, did not identify any additional significant damage in the main administration block from that observed during the rapid structural assessments. Some distress was noted in the original service tunnel, although it appeared to be limited to the further opening up of existing cracks.

Additional damage noted to the entry canopy and architectural precast wall is as follows:

- hairline cracking at base precast canopy columns
- cracking of precast wall elements

3.4 GEOTECHNICAL REVIEW

A review of the ground damage and conditions was carried out by Tonkin & Taylor for the Burwood Hospital Campus. A subsequent report titled *Burwood Hospital Post Earthquake Geotechnical Assessment*, was issued in June 2011 [11]. The geotechnical review concluded that the settlement and damage to building foundations and slabs on the Burwood Hospital Campus was likely due to the liquefaction of underlying soil layers. It is believed that excessive pore water pressures have been dissipated and that further settlement is not expected to occur, unless another significant event were to occur.

The majority of the damage to the Administration Building is related to ground settlement and spreading of the founding soils. It is estimated that the ground floor slab has settled a total of 110mm – 200mm overall with a differential settlement of approximately 90mm across the slab. Damage noted to the concrete slab on grade is in the form of cracking in the slab and separation at existing construction joints.

The differential settlement is particularly noticeable at the main entrance to the building were the original service tunnel (founded in deeper soils) has settled less than the surrounding shallow spread footings.

Based up the geotechnical report provided by Tonkin & Taylor [11] the potential for future total and differential settlements at the building site varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

Further recommendations provided by Tonkin & Taylor have indicated that the soil profile under the building is conducive to re-levelling through either through the use of underpinning grout or engineered resin.

3.5 LEVEL SURVEY & VERTICALITY STUDY

A detailed survey of the ground floor levels in the Administration Building was conducted by Fox & Associates and issued on 31st October, 2011 [6]. The survey indicates a differential settlement of approximately 90mm over the footprint of the building, with the most significant differential settlements occurring at the main entrance and southeast corner of the building.

The worst case permanent slope in the slab on grade, based upon this survey, is a drop of approximately 60mm over 3.5 meter length of the main administration block (1.7% or 1:60). This slope, and other slopes noted in the ground floor slab, are outside the typical acceptable range and require repair. Re-levelling options have been included in Section 4.2. For the extent of the differential settlement see the level survey included in Appendix C.

A verticality survey was also completed for the architectural precast wall and for the precast columns of the canopy structure. The survey has indicated a permanent lean to the north of up to approximately 2% in the columns and 1% in the wall. The full verticality survey is also included in Appendix C.

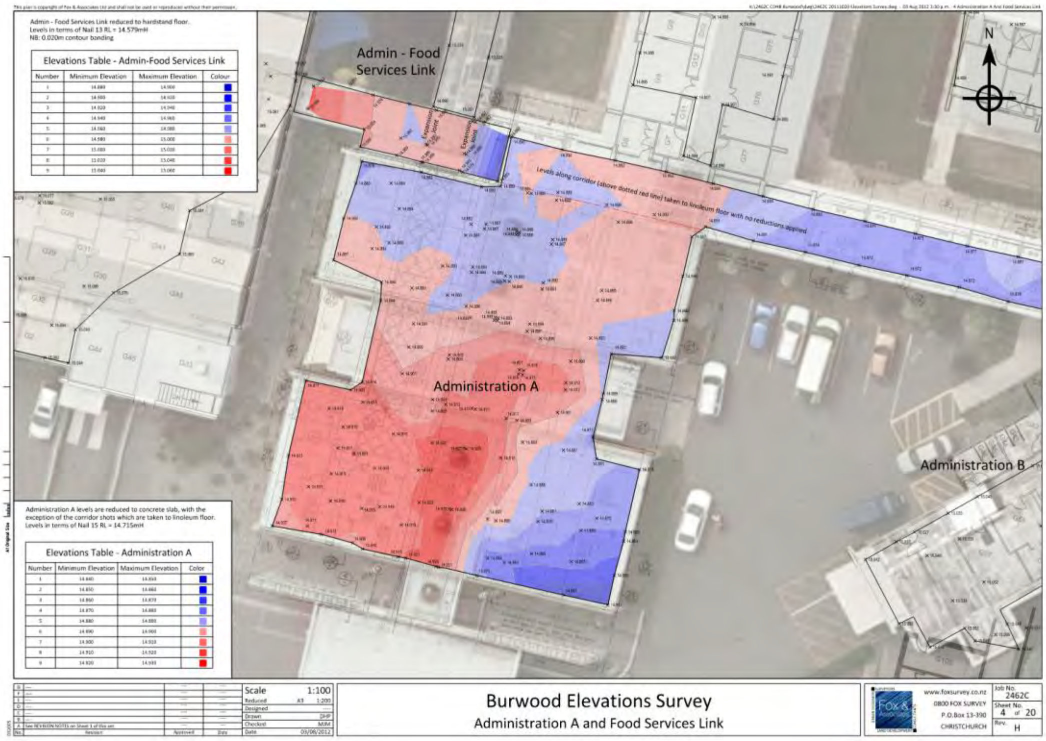


Figure 3-1: Level Survey

Figure 3-2: Verticality Survey

3.6 SUMMARY OF BUILDING DAMAGE

The following is a summary of the observations made for the building, and our conclusions as to their condition and seismic load resisting capacity. These observations do not specifically distinguish between damage caused by the Darfield Earthquake, the Lyttelton Earthquake or any significant aftershocks, such as those that occurred on the 13th June 2011, .the 23rd December 2011 or the 2nd January 2012. Despite not being able to specifically distinguish when individual building damage observed occurred, it is believed that the majority of the damaged, or at least the onset of damage, can be linked to the February 22nd event.

The majority of the Administration Building appears to have performed relatively well with the majority of the damage related to earthquake induced differential ground settlement and lateral stretching. The majority of the damage has been limited to ground floor slabs, partition walls and ceiling finishes and can be considered non-structural in nature. Our observations suggest that the building would have undergone a limited number of full cycles of primarily elastic deformation. The short duration of the strong ground motion recorded and the damaged observed would support this hypothesis. A summary of the building damage observed can be typified as follows:

- **Differential Ground Settlement** – As previously noted the majority of the damage noted to date appears to be due to earthquake induced differential ground settlement and lateral stretching of the ground floor slab. The majority of the differential settlement has been concentrated at the main entry and the southeast corner of the main building block. The large differential settlements at the entry caused distress to the glass entry doors, including the shattering of a glass door mullion in the 23rd December 2011 event. For the extent of the differential settlement see the level survey included in Appendix C.

- **Spreading of concrete floor slabs** – Extensive spreading and cracking was noted in the concrete slab on grade, which are typically located at existing control joints (up to 10mm in width). The majority of the cracks and separation of the joints in the slab were noted in the main hallway and atrium space, which is primarily composed of architecturally exposed slabs with under floor heating.

  The shrinkage control joints are detailed so that the wire mesh reinforcing is stopped short on either side of the joint, and thus no reinforcing is present to tie the sections of the slab on grade together. Only minor differential vertical displacements (1-3mm) was noted in isolated locations across the slab joints.

  Where the concrete floor slab on grid line 11 in the northern corridor has separated, there are ~10mm cracks through the perimeter foundations beams in this location. Therefore, there is likely a discontinuity in the longitudinal reinforcing in these foundations beams at this location.

  In general, there are reinforced concrete tie beams below the slab which tie the exterior footings together. The tie beams have been detailed in such a way that the double layer of DPM below the slab continues over the top of the tie beam and there is no hard connection between the two elements. See Figure 3-3.

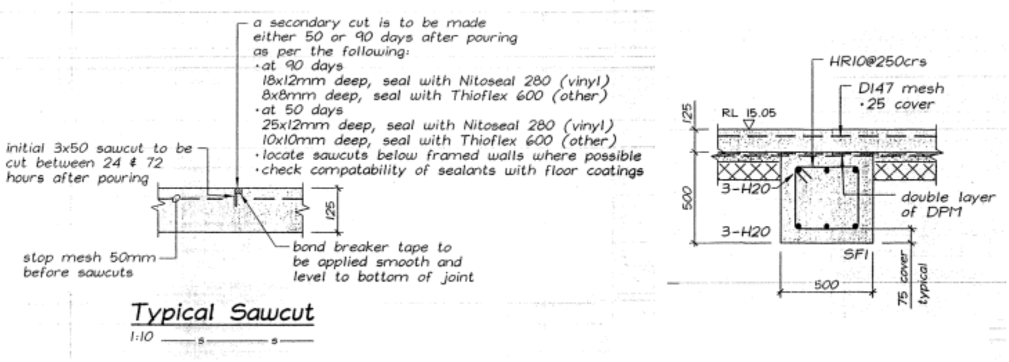


Figure 3-3: S.O.G. Construction Joint and Tie Beam Details

For the extent of the cracking to the slab on grade see the summary included in Figure 3-4 below and the crack map included in Section 4.

Figure 3-4: Observed Damage to Ground Floor Slab

- **Localized Cracking to Concrete Block Walls** – Minor cracking has been noted in localized areas of the concrete block walls. In particular, this was noted in the area of greatest differential settlement, near the intersection of Gridlines 6 and C. At this location it appears as though part of the face shell of the concrete block has cracked were the embedded steel portal frame column occurs. The cracks in the block walls (0.2mm – 0.5mm) are not believed to significant enough to significantly decrease the capacity of the building.

- **Distress to Original Service Tunnel** – Additional opening up of existing cracks has been noted.

- **Diagonal Stepped cracking to Brick Veneer** – "Stepped" diagonal cracking was noted to the exterior brick veneer. In particular this was noted above and adjacent to the main entry. It is believed that this has been caused by the differential ground settlements noted in this area.

- **Precast Parapet Capping** – Damage has been noticed to the grouted connections at several of the precast parapet wall capping stones. It should be noted that almost identical precast capping stones were shed from the top of the Surgical Services Unit in the 22nd of February event. *The precast capping has now been removed and replaced with a light weight metal capping.*

- **Cracking to Interior Partition Walls** – Minor cracking to the interior partition walls was noted throughout. Larger cracks were noted adjacent to the cracks in the slab on grade. The majority of the cracks observed have occurred at the corners of door and window openings, at existing wall board joints and below the glulam beam connections of the corridor.

- **Cracking to Ceilings & other Finishes** - Minor cracking has been observed at wall and ceiling interfaces indicated racking of the ceiling.

- **Canopy Structure and Architectural Precast Wall** – The free standing architectural precast wall and canopy at the main entry of the building have some residual deformation as a result of the earthquakes. A verticality survey of the canopy columns and precast wall indicates that the wall is leaning to the north at approximately 1% (1:100). The columns supporting the entry canopy structure exhibit a larger residual deformation of up to approximately 2% (1:50).

  In general, the canopy structure and precast entry wall are seismically isolated from the main administration block. The one exception is the glass entry roof connection which is hung from the precast wall spandrel above (see Figure 3-5 below). At this location cracking has been noted to the underside of the spandrel due to the main block and the precast wall acted out of phase from each other.

  It appears as though the residual deformation noted in the canopy structure and precast entry wall is a result of earthquake induced settlement/liquefaction in the vicinity of the ground on which the columns are founded. However, it is also possible that the deformation was caused by the main administration block pulling on the canopy/wall at the hanging glass entry connection.

  In addition to the deformation noted, minor cracking has been noted at the base of the precast columns and to the precast elements of the architectural wall feature.

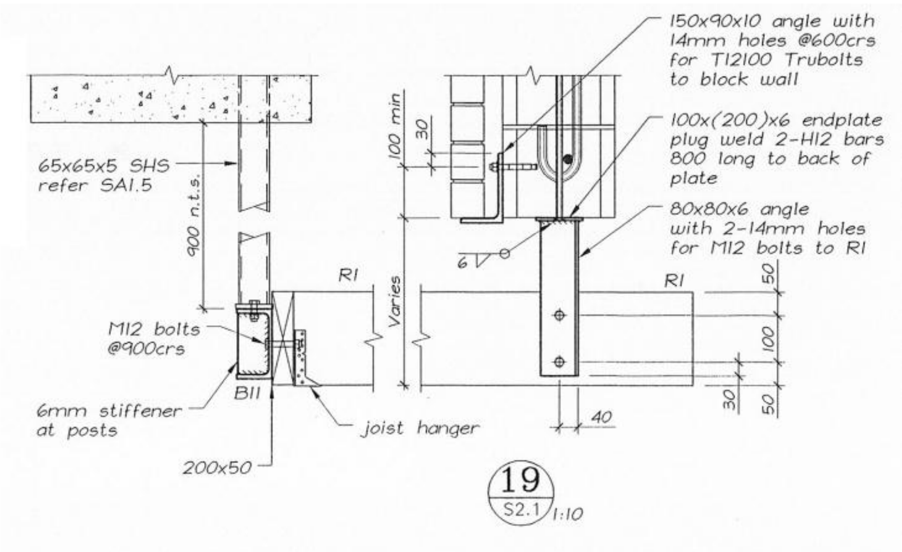


Figure 3-5: Hung Entry Detail

Table 4-1 provides a photographic summary of the typical damage observed. A full record of our detailed observations and repairs required can be found in Appendix A.

3.7 ADDITIONAL INVESTIGATIONS REQUIRED

3.7.1 Investigations Required For Further Assessment

Several assumptions were made in the completion of the pre-earthquake (undamaged state) and post-damaged (damaged state) structural assessments. Destructive exploration is required in a number of locations in order to verify these assumptions. The areas requiring further investigation to finalise the assessments are as follows:

- Based upon the damage observed further investigations of the exterior brick façade is required. This includes a summary of its general condition and the fixings to the exterior concrete block walls. This work should be completed by a qualified Mason and may require the local removal of brick veneer.

- At the damage to the existing slab, it is recommended that further investigations be completed by a qualified waterproofing contractor to determine the integrity of the existing waterproof membrane.

- Along the south side of the corridor (along Gridline H), verify the connection at the base of the gypsum lined timber bracing walls to the concrete foundation elements below.

- If the architectural precast wall is to be re-levelled further investigations are required at the base of the wall to ensure strain hardening of the reinforcement has not occurred.

3.7.2 Investigations to be Completed During Building Repair

- If the architecturally exposed ground floor slab is to be demolished and replaced, prior to its replacement, investigate the concrete tie-beams for evidence of lateral stretching and repair as required.

- Check each glulam timber roof purlin to steel cleat connection for damage. A number of connections are either concealed or too high to easily access.

- Re-inspection of the building will be required upon completion of any re-levelling works to determine if any additional damage has occurred.

3.8 POST-EARTHQUAKE BUILDING CAPACITY

Based upon our investigations to date, we do not consider the Administration Building to have any notable reduction to the overall gravity load resistance of the structure. While damage to the lateral load resisting system has occurred, the actual percentage reduction in the capacity of the building is hard to quantify.

The movement noted in the slab on grade is not believed to have significantly affected the existing capacity of the building as there was no reinforcing present across the control joints prior to the earthquake. This building also has interior tie beams which connect the continuous exterior and interior concrete footings together (unlike at the Orthopaedic Rehabilitation Unit constructed at the same time). The impact of the lateral stretching on the tie beams themselves is unknown, although it is likely the cracking is spread over a longer length, as opposed to being concentrated in one location. We also believe the roof framing is flexible enough to have absorbed the lateral stretching observed without imposing undue stress on the steel portal frames.

The building deformation due to the differential settlement and lateral stretching will have resulted in some reduction in capacity, but again this is difficult to quantify. The primary concern will be a reduced ability of the buildings to absorb future differential settlements prior to the onset of more severe damage to the foundations and superstructure of the buildings.

The damage observed will require repair to restore the strength, stiffness durability and performance of the individual structural components. The differential settlement noted will also require re-levelling to restore the serviceability of the building. The repair work is outlined in Section 4. Following the recommended repair of the structural damage, the lateral load resisting performance of the structure will be restored to approximately pre-earthquakes levels (see Section 2.3).

4. OBSERVED DAMAGE AND REQUIRED REPAIRS

4.1 PRIMARY OBSERVED DAMAGE AND REPAIRS REQUIRED

This section covers the damaged noted during our detailed assessment of the building. Note that our observations have been restricted to structural aspects of the building 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.

Table 4-1 provides a photographic summary of the observed damage and typical repairs required for the Administration Building. Table 4-1 should be read in conjunction with Appendix A – Record of Observation. The Repair Specification [2] referred to in Table 4-1 has been issued separately.

In general, the aim of the repair work indicated in this section is to restore the structure to its pre-earthquake state, as far as practicable, while maintaining the utility of the building. The repairs presented attempt to address the loss of strength, stiffness and durability of the structural elements due to the damage noted.

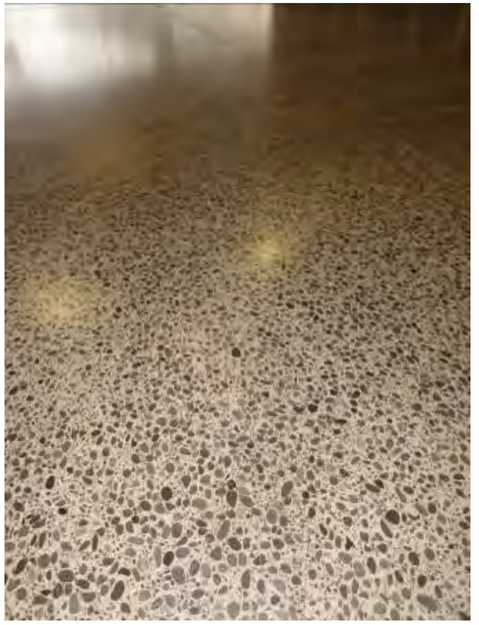
It should be noted that more damage may be identified during the repair works and (if required) additional repair details will be specified accordingly.

Please note that if building is to be re-levelled, all repair works are to be completed after the building has been re-levelled to a satisfactory condition as further damage to the wall and ceiling linings can be expected during the re-levelling process.

Further recommendations for improvement to the buildings seismic performance have been included in Section 5.

Table 4-1: Photographs of observed damage and repairs required

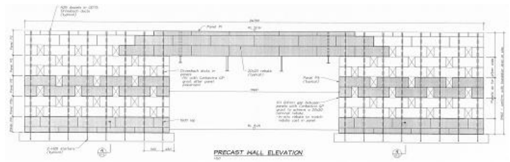
Damaged Item & Location	Damage	Recommendations	Example Photograph
1. Foundations and Ground Floor Slabs			
1.1. Differential Ground Settlement	Differential Ground Settlement of approximately 90 mm	Re-levelling of the building is required to restore the utility of the building. Refer to discussion on re-levelling in Section 4.2 for additional information. (Note: All re-levelling is to occur prior to any other structural or cosmetic repairs).	

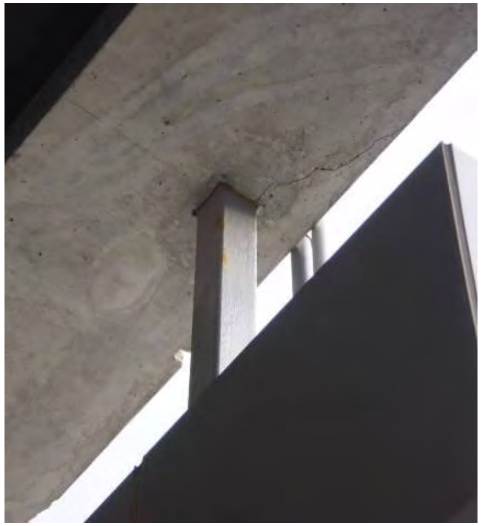
Damaged Item & Location	Damage	Recommendations	Example Photograph
1.2. Foyers and atrium Slab on Grade	Cracking (0.4mm) of concrete slab on grade (excluding cracks in existing construction joints).	Re-level building as noted in item 1.1. It is likely the existing slab on grade will be required to be demolished and replaced in conjunction with the re-levelling process due to access requirements, exposed architectural slabs and under floor heating. If the slab on grade is to remain, epoxy inject all field cracks in the slab between 0.2 & 1mm per the HCG specification, once the re-levelling process has been completed. If cracks of greater than 1mm are observed in the slab advice HCG for addition inspection. Check and reinstate DPM membrane as required. Replace existing floor finishes as required. *Waterproofing and aesthetic repair specification by others.*	
1.3. Foyers and atrium	Differential settlement and stretching at existing construction joints (up to 10mm)	Re-level building as noted in item 1.1. If slabs are to remain, replace the existing grout in the damaged joint, once the re-levelling process has been completed. Check and reinstate DPM membrane as required. Replace existing floor finishes as required. *Waterproofing and aesthetic repair specification by others.*	

Damaged Item & Location	Damage	Recommendations	Example Photograph
1.4. Foyers and atrium	Differential settlement and stretching at existing construction joints (up to 10mm)	See repair item 1.3	
1.5. Northern Corridor	Foundation beams on either side of the corridor at grid 11 separated by ~10mm.	Chases in the slab are to be broken out across the separation in the slab for reinforcing to be lapped across it. Sections of the reinforced concrete perimeter beams on either side of the slab are to be broken out either side of the separations. New reinforcing is to be drill and epoxied into the exposed foundation beams and recast. Repair sketch to be provided prior to construction.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
1.6. Original Service Tunnels	Distress noted at existing cracks in original service tunnel.	Epoxy inject all cracks in concrete >0.2mm and < 1.0mm as per the HCG Repair Specification [3]. For cracks greater than 1.0mm, advice HCG for further investigation in order to confirm integrity of existing reinforcing steel	
1.7. 2001 Service Tunnels	Minor cracking noted to concrete slabs and side walls	See repair item 1.6.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
2. Concrete Block Walls			
2.1. South foyer / Atrium	Crack (approx 0.2mm) propagating from lintel.	Epoxy inject all cracks in concrete >0.2mm and < 1.0mm as per the HCG Repair Specification [3]. For cracks greater than 1.0mm, advice HCG for further investigation in order to confirm integrity of existing reinforcing steel. At cracks in existing mortar joints route out and re-point joints.	
2.2. Northern foyer	Diagonal crack near base of wall (approx 0.5mm wide).	See repair item 2.1.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
3. Entry Canopy and Architectural Precast Wall			
3.1. Free standing wall	Wall has a residual lean of approximately 1% (0.5 degrees).	Demolish and replace entry wall feature, or attempt re-levelling of existing footing. See Section 4.3 for additional information.	
3.2. Canopy columns	Concrete columns have residual lean of approximately 2% (1.0 degree).	Demolish and replace entry canopy structure, or attempt re-levelling of existing footings and columns. See Section 4.3 for additional information.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
3.3. Soffit of entry wall lintel	Cracking of cover concrete at connection of glass entry roof to precast wall spandrel above.	Provide a new connection that allows for movement between the main administration block and the precast entry wall. Alternately, disconnect the front of the glass entry from the precast wall spandrel above and provide alternate gravity and lateral support. In addition, see recommendations included in Section 5.	
4. Miscellaneous			
4.1. Exterior Brick Veneer	Diagonal Stepped Cracks in Exterior Brick Veneer	Repair existing brick veneer as required and check existing veneer ties for distress and damage. *Further investigation of the exterior veneer by a qualified mason is recommended.*	

Damaged Item & Location	Damage	Recommendations	Example Photograph
4.2. Capping of walls	Capping stones have shaken loose	Remove existing precast capping stones and replace with a light weight alternative. *Now completed*. See recommendations included in Section 5.	
4.3. Interior Partition Wall and Ceiling Finishes	Cracks noted to partition walls and ceiling finishes.	In general provide aesthetic repairs to wall board and ceiling finishes. *Repair specification to be provided by others*. At the gypsum board lined bracing walls along the south side of the corridor (Gridline H) replace any damage wall board. Any linings to remain along this length of wall are to be re-fixed to the timber framing beyond.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
4.2. Exterior Site Work	Differential ground settlement damage to surrounding site works, including at service tunnel under drive and at sidewalk.	*Repair specification by others.*	 

4.2 DISCUSSION ON RE-LEVELLING OF MAIN ADMINISTRATION BLOCK

The level survey, completed by Fox & Associates has indicated differential ground settlement of the main administration block of approximately 90mm. It is believed the total earthquake induced ground settlement experienced by the building is on the order of 110mm – 200mm. While the differential settlement has been noted throughout (see Appendix C for complete level survey) the worst of the differential settlement noted has occurred at the main entrance and southeast corner of the building. The worst case permanent slope in the slab on grade, based upon this survey is approximately 1.7% (1:60). This slope, and other permanent slopes in the ground floor slab, are outside the typical acceptable range and require re-levelling.

At the main entrance there is a service tunnel below, founded in deeper soils which have not settled to the same degree as the surrounding continuous concrete spread footings. This has resulted in a "high point" relative to the surrounding slab on grade, with a differential settlement of approximately 60mm over a 3.5 meter length. Any re-levelling of the existing footings and ground floor slabs should be lifted to this high point or as close as practical. If this option is chosen care will need to be taken at all interfaces between buildings and hallways.

Alternatively, the top of the tunnels could be lowered in combination with the lifting process so that the existing footings have to be lifted to a lesser degree. This would require the main entrance slab over the service tunnels to be broken out and replaced to a logical point. Which ever option is chosen we would suggest that a detailed serviceability, accessibility and circulation study be completed for the hospital as a whole, including the potential effects of any re-levelling proposed for the campus.

The two primary re-levelling options available for the Administration building include the use of either underpinning grout or mechanical jacking techniques.

Based upon the information provided by Tonkin & Taylor the soil profile under the Administration Building (medium dense sand overlying dense sand) lends itself to localized lifting through the use of underpinning grout. As most of the building requires re-levelling it would be recommended that all the footings under the main administration block have grout installed under the foundations to reduce the risk of creating hard points.

The building could also be re-levelled through the use of mechanical jacking under the existing foundations. In this scenario the existing foundations would be jacked up to level, with the void created under the footings filled with cementicious grout.

With either option it is likely the existing slab on grade will be required to be demolished and replaced in conjunction with the re-levelling of the existing foundations. This is due to access requirements to the underside of the foundations, and the presence of architecturally exposed slabs and under floor heating. If the slabs are required to be demolished and replaced the interior fit out will likely be required to be demolished and replaced as well.

There are advantages and disadvantages for each re-levelling solution proposed which extend beyond structural performance which will need to be considered by CDHB. These include continuity of operation, degree of re-levelling accuracy, risk of damage existing foundation system and/or superstructure, and the willingness of the re-levelling sub-contractor to provide a producer statement, amongst other items. From a structural standpoint, either option is acceptable provided the use of underpinning grout does not create any detrimental "hard points" under the building.

It should be noted that neither of the re-levelling options discussed above is expected to increase the seismic performance of the building or reduce the potential for future differential settlements. Instead the option presented are intended to re-level the building without making the future performance of the building any worse than it was prior to the earthquakes. To improve the future performance of the building, and reduce the potential for future differential settlements, would likely require the entire footprint of the building to be either piled or the ground under all the existing footings improved to the appropriate depth. Further geotechnical investigations would be required into the type and depth of ground improvement required.

Appropriate contingencies should be provided to account for the risks to the building foundations and superstructure during any re-levelling process.

4.3 DISCUSSION ON RE-LEVELLING OF ENTRY CANOPY

Based upon visual observations, and a verticality study Fox & Associates (see Section 3.5), the entry canopy and architectural precast entry wall is on a permanent lean between 1-2%. In order to straighten the canopy and the wall, the footing under the wall would be required to be re-levelled. This would likely require the existing screw piles under the wall to be disconnected, the footing re-levelled and new screw piles installed and attached to either side of the existing footing. The re-levelling of the footing could be achieved through the use of underpinning grout or mechanical jacking techniques.

While it is likely the wall could be brought back to vertical, there is a risk the lean in the canopy columns would remain.

4.4 GROUND FLOOR SLAB

As previously noted, extensive spreading and cracking was noted in the concrete slab on grade, which were typically located at existing control joints (up to 10mm in width). The majority of the cracks and separation of the joints in the slab were noted in the main hallway and atrium space, which is primarily composed of architecturally exposed slabs with under floor heating.

The shrinkage control joints are detailed so that the wire mesh reinforcing is stopped short on either side of the joint, and thus no reinforcing is present to tie the sections of the slab on grade together. Only minor differential vertical displacements (1-3mm) was noted in isolated locations across the slab joints. At the Orthopaedic Rehabilitation Unit the recommendation was to repair the slab on grade with reinforced stitch joints across the existing slab on grade. A primary reason for this was the absence of concrete ties beams to tie the exterior strip footings together and limit future lateral stretching of the slab, which could in turn destabilize the building.

In general, the Administration Building has reinforced concrete tie beams below the slab which tie the exterior footings together. The exception to this is across the main hallway, running in the north-south direction, through the main administration block. For this reason we are recommending the repair of the slab include a new tie across the hallway along Gridlines C, E and G. As the slab is likely to be required to be demolished and replaced, new tie beams under the slab can be cast in conjunction with the new slab. (Note: If the existing slab is to remain the tie can be created with the installation of reinforcing bars placed in chases cut in the existing slab on grade and filled with grout).

4.5 PRECAST PARAPET CAPPING

Removal of the concrete capping blocks to the top of the concrete masonry walls is recommended. Movement of the capping blocks has been noted, with some of them visibly loose and/or displaced. Similar cappings have fallen off the adjacent Surgical Orthopaedic Unit. While the capping does not affect the structural system, the heavy concrete elements are at risk of falling on building occupants during an earthquake. *The precast blocking has been removed and replaced with a light weight metal capping.*

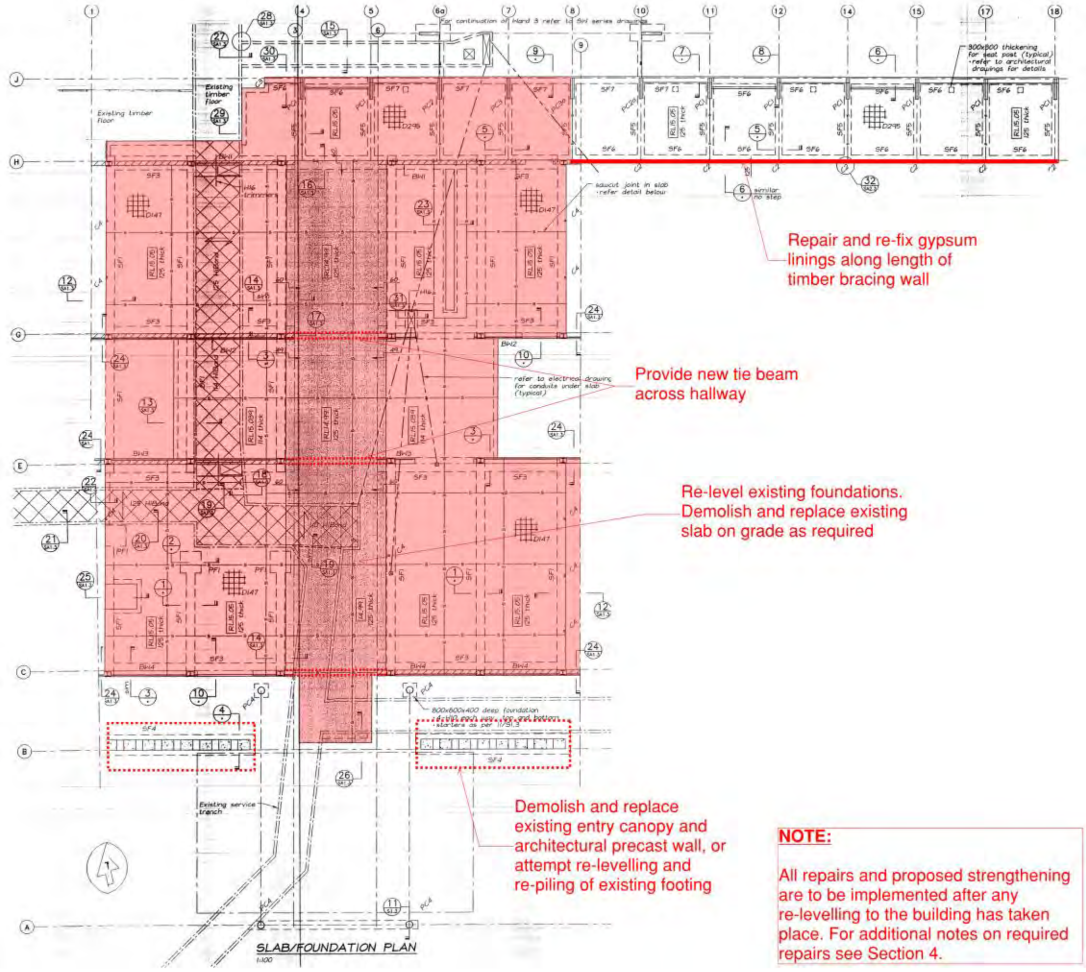


Figure 4-1: Foundation / Ground Floor Plan - Required Repairs

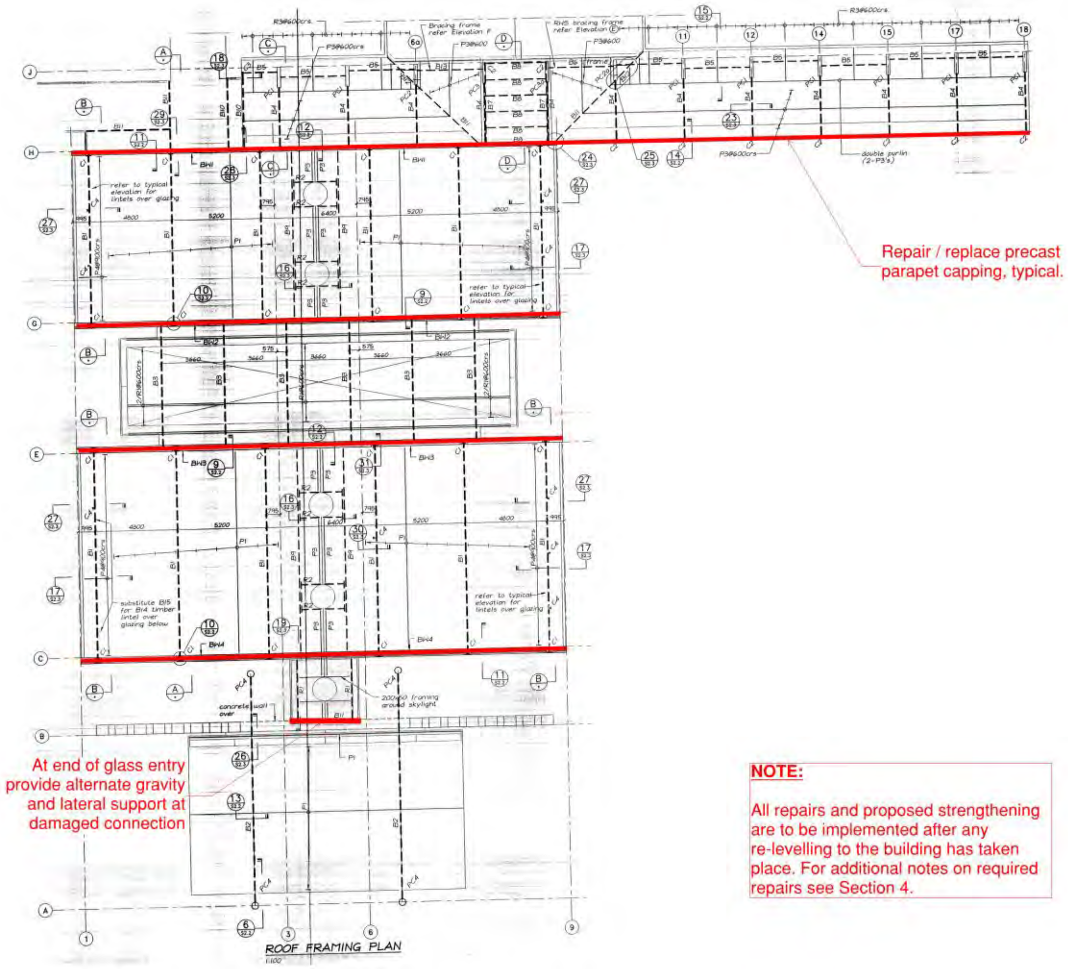


Figure 4-2: Roof Plan - Required Repairs.

5. STRENGTHENING RECOMMENDED

As noted in Section 2, Pre-Earthquake Building Condition, and Section 3, Post-Earthquake Building Condition, the lateral load resisting capacity of each section of the building has been assessed as a percentage of the loads imposed by the Design Basis Earthquake (DBE).

In general, the main lateral force resisting elements of the Administration Building & corridor structure have an assessed capacity is in excess of 67% DBE. There were also no Critical Structural Weaknesses (CSW's) noted in the main administration block or the corridor.

The accessed capacity of the entry canopy and architectural precast wall has been assessed at approximately 40% DBE based upon the capacity of the screw piles under the footing of the architectural wall feature.

Provided the permanent repairs works noted in Section 4 are completed, the assessed capacity of the main administration block and corridor will be reinstated to approximately pre-earthquake levels. If re-levelling and re-piling of the architectural precast wall is attempted, the assessed capacity of the canopy and wall would be increased to approximately 65%, provided there is no strain hardening of the reinforcing bars at the base of the wall.

Strengthening works to improve the future seismic performance of the building under serviceability level, ultimate and Maximum Considered Earthquake's (MCE) has been included below. This includes recommendations for associated site structures, such as the main entrance canopy and architectural precast wall, along with recommendations for non-structural items that may pose an identified risk to the building occupants. These recommendations are as follows:

5.1 STEEL PORTAL FRAMES

The steel portal frames, which form the primary lateral force resisting elements in the north-south direction, have been assessed at 80% DBE. Once this capacity has been exceeded, lateral buckling of the portal beam is expected to occur, potentially leading to loss of stability of the concrete masonry walls. The performance of this system structural system could be improved as a whole) by providing additional bracing to the portal beams along their length. We believe the addition of these bracing elements would provide a high benefit for a relatively low cost.

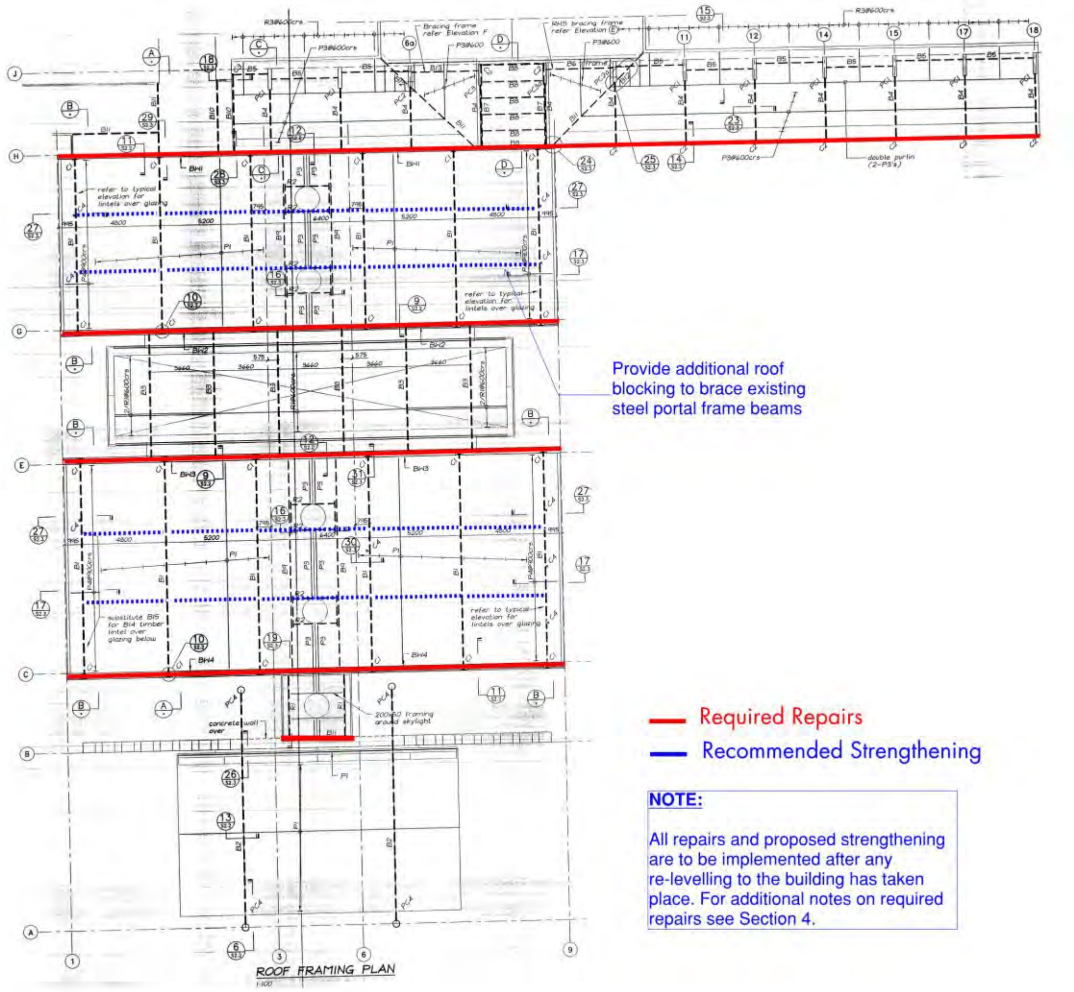


Figure 5-1: Roof Framing Plan - Recommended Strengthening

5.2 IL2 & IL3 100% PRELIMINARY STRENGTHENING CONCEPTS

Preliminary Strengthening Concepts have been developed for the Administration Block and Northern Corridor for 100% IL2 and 100% IL3. Please refer to the Preliminary Strengthening Concepts Report.

The schemes are shown on mark-ups of the original structural drawings:

– BURW.AD.140825_HCG_Preliminary Strengthening Concept 100% IL2_Rev1
– BURW.AD.140825_HCG_Preliminary Strengthening Concept 100% IL3_Rev1

Both of these schemes assume the entrance canopy and feature wall have been replaced in order to bring the overall capacity of the building to 100% IL2 or IL3.

6. MAXIMUM CONSIDERED EARTHQUAKE PERFORMANCE

The maximum considered earthquake (MCE) is generally accepted as an earthquake with a 2% annual probability of being exceeded in 50 years, or a return period of 2500 years. This is a larger earthquake than new buildings are specifically designed for; however, buildings built to the current code are expected to be detailed in such a way that they are not likely to collapse in a MCE. Buildings sustaining damage following MCE are expected to suffer significant structural damage with the potential of being rendered uneconomic to repair and unable to be re-entered.

Under an MCE, the level and duration of strong ground motion will be higher, and significantly longer than that of the Lyttelton Earthquake. As such, more severe cracking can be expected, and at additional locations in the building, to absorb the energy that the ground movements will impart on the building.

Under an MCE event, the steel Administration Building, would likely experience extensive damage to partitions, non-structural elements and building attachments such as the dislodging of the precast parapet capping stones and damage to exterior brick veneer. The buckling of the steel portal frame beams running in the north south direction is also likely. This would cause extensive distress to the existing metal deck roof. The RHS steel beam to precast blade columns are also likely to see extensive damage although the risk of collapse of this area would be low.

The entrance canopy and precast architecture wall would also likely see extensive damage in an MCE event. If the surrounding soils were to liquefy the structures could become unstable and collapse. The hung connection glass entry connection could also see extensive damage leading to a collapse of the entry roof.

Based upon the information provided by Tonkin & Taylor, ground settlement in the order of 160 to 250mm can be expected at the site for an ULS event. The settlement at an MCE event would be expected to be at least this amount if not greater. If large differential settlement under the building footprint does occur, significant damage can be expected to foundation elements, slabs on grade and service tunnels. Associated distress to the superstructure could also be expected.

A low probability of collapse and preservation of life safety are the main assessment criteria considered for the MCE. The Administration Building has been designed and detailed to relatively recent codes and should have a limited amount of ductile behaviour. The main risk to life safety identified for the buildings current configuration are entrance canopy and architectural precast walls, the glass entry and the precast parapet capping. As long as the minimum repairs proposed in Section 4 are implemented and the recommendations in Section 5 are implemented, the likelihood of the building experiencing a full or partial collapse during an MCE event is considered to be low.

7. REFERENCES

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3. *Burwood Hospital – New Administration Building.* Original structural drawings, Powell Fenwick Consultants LTD, 18 May 2001

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5. *CDHB – Burwood Field Survey – Revision E*, Fox & Associates, Jan 2012

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8. Department of Building and Housing, *Compliance Document for New Zealand Building Code - Clause B1 – Structure, Amendment 10 (Canterbury)*, Department of Building and Housing, Wellington, 19 May 2011

9. *Structural Design Actions Part 5: Earthquake Actions – New Zealand, NZS 1170.5:2004*, Standards New Zealand, 2004

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

12. *Concrete Structures Standard, NZS 3101:2006*, Standards New Zealand, 2006

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

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

15. *Practice Note – Design of Conventional Structural Systems Following the Canterbury Earthquakes*, SESOC, December 2011

16. *CDHB – Burwood Hospital Campus – Rapid Visual Inspection: 106186.03SR1,* Holmes Consulting Group, February 2011

17. *CDHB – Burwood Hospital – Rapid Seismic Assessment Post June 13th Earthquakes: 106186.03SR4* Holmes Consulting Group, 14 June 2011

18. *CDHB – Burwood Hospital – Post Earthquake Rapid Structural Assessment following 5.8 and 6.0 Magnitude Earthquakes, 106186.03 SR*8 Holmes Consulting Group, 24 December 2011

19. *CDHB – Burwood Hospital – Post Earthquake Rapid Structural Assessment following 2nd January 5.5 Magnitude Earthquakes, 106186.03 SR9*, Holmes Consulting Group, 9 January 2012

20. CDHB - *Burwood Hospital Campus, Orthopaedic Rehabilitation Unit – Report 9, Interim Detailed Seismic Assessment Report,* Holmes Consulting Group, December 2011

21. CDHB - *Burwood Hospital – Surgical Orthopaedic Unit– Site Report 02 – Surgical Orthopaedic Unit – Precast Parapet Capping Repair,* Holmes Consulting Group, 28 November 2011

APPENDIX A

Record of Observations

APPENDIX A – RECORD OF OBSERVATIONS & REPAIRS

Inspection date: 19 October 2011

KEY	
N	No repair required
Y	Repair required
F	Further investigation required
C	Repair complete

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G28	Wall between windows	Horizontal crack in wall partition	Y	Aesthetic Repair to finishes, repair specification by others	836
G	G27	Wall at door	Vertical crack in bulkhead	Y	Aesthetic Repair to finishes, repair specification by others	837
G	G38	Wall at door	Horizontal and vertical crack to partition at door	Y	Aesthetic Repair to finishes, repair specification by others	838, 839
G	G37	Wall at door	Vertical crack to partition at door	Y	Aesthetic Repair to finishes, repair specification by others	840
G	G35	Wall at window	Horizontal and vertical crack to partition at window	Y	Aesthetic Repair to finishes, repair specification by others	841
G	G35	Wall at door	Vertical crack to partition at bulkhead	Y	Aesthetic Repair to finishes, repair specification by others	842
G	G9	Wall at door	Vertical crack to partition at door bulkhead	Y	Aesthetic Repair to finishes, repair specification by others	843
G	G7	Wall	Horizontal and vertical cracks to partition at door and window	Y	Aesthetic Repair to finishes, repair specification by others	844-846

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G1	Ceiling	Horizontal crack to ceiling partitions	Y	Aesthetic Repair to finishes, repair specification by others	847
G	G1	Wall	Wall panel has 'popped-off' at the glued connection to the steel column. Appears to be due to differential settlement between timber corridor and concrete slabs.	F	Aesthetic Repair to finishes, repair specification by others. May require repair or replacement of adjacent corridor	848, 851
G	G1	Floor	Approximately 50mm differential settlement between timber corridor floor and reinforced concrete floor slabs. NB services run underneath suspended timber corridor floor.	F	May require repair or replacement of adjacent corridor	849-852
G	G1	Ceiling	Crack in ceiling partitions at interface of different buildings	Y	Aesthetic Repair to finishes, repair specification by others	853
G	G1	Floor	Horizontal and vertical movement at joint in slabs	Y	Re-level or replace existing slab on grade in conjunction with re-levelling of building foundations. Remove and replace damaged grout at existing construction joints	854
G	G1	Wall at door	Vertical crack in partition at door bulkhead	Y	Aesthetic Repair to finishes, repair specification by others	855
G	G1	Wall	Horizontal and vertical cracks between wall, bulkhead and beam	Y	Aesthetic Repair to finishes, repair specification by others	856
G	G1	Wall	Diagonal crack (approx. 5mm wide) to partition, cladding screw has stripped through approximately 10mm from red cladding element.	Y	Aesthetic Repair to finishes, repair specification by others	857

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G1	Wall and slab – building joint	Vertical and horizontal differential movements between wall and slab elements at construction joint. Vertical step in slab <10mm. Minor building to cladding on outside of building continuous over joint	Y	Re-level or replace existing slab on grade in conjunction with re-levelling of building foundations. Remove and replace damaged grout at existing construction joints	858-861
G	G1	Wall at door	Vertical cracks at both sides of door bulkhead in partition	Y	Aesthetic Repair to finishes, repair specification by others	862
G	G1	Wall	Vertical crack in partition at beam connection location. Typical to corridor.	Y	Aesthetic Repair to finishes, repair specification by others	863
G	G2	Floor	Spreading of slab at concrete joint	Y	Re-level or replace existing slab on grade in conjunction with re-levelling of building foundations. Remove and replace damaged grout at existing construction joints	864, 865
G	G2	Floor	Hairline crack to slab offset from slab joint	Y	Re-level or replace existing slab on grade in conjunction with re-levelling of building foundations. Remove and replace damaged grout at existing construction joints	866
G	G1	Floor	Vertical differential settlements between slabs <10mm. Crack concealed by vinyl floor covering	Y	Re-level or replace existing slab on grade in conjunction with re-levelling of building foundations. Remove and replace damaged grout at existing construction joints	867
G	G2	Concrete Block Wall	0.5mm tapered diagonal crack in concrete masonry wall. Crack does not appear to have propagated past reference mark on wall.	Y	Epoxy inject crack in accordance with HCG specification	868
G	G13	Ceiling	Cracking between wall and ceiling partitions	Y	Aesthetic Repair to finishes, repair specification by others	869, 870

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G2	Concrete Block Wall	0.2mm tapered diagonal crack propagating from lintel corner	**Y**	Epoxy inject crack in accordance with HCG specification	871, 872
G	G3	Floor Slab	8mm crack in flooring adjacent to slab joint	**Y**	Re-level or replace existing slab on grade in conjunction with re-levelling of building foundations. Remove and replace damaged grout at existing construction joints and overlaying floor finishes	873, 874
G	G3	Floor Slab	Spreading at joint central to atrium, approximately 10mm	**Y**	Re-level or replace existing slab on grade in conjunction with re-levelling of building foundations. Remove and replace damaged grout at existing construction joints and overlaying floor finishes	875-877
G	G3	Floor Slab	Hairline crack offset from joints in slab. Diagonal crack propagating southwards from RCM wall.	**Y**	Re-level or replace existing slab on grade in conjunction with re-levelling of building foundations. Once re-levelling is complete epoxy inject cracks in slab on grade per HCG specifications	878
G	G4	Floor Slab	Spreading along slab joint, approximately 5mm	**Y**	Re-level or replace existing slab on grade in conjunction with re-levelling of building foundations. Remove and replace damaged grout at existing construction joints	879
G	G4	Floor Slab	0.2-0.4mm diagonal crack propagating perpendicular to slab joints. Diagonal cracks link between RCM wall (see Photo 878 and N-S sawn joint Photo884). Cracks appear to be located over walls of service tunnel underneath.	**Y**	Re-level or replace existing slab on grade in conjunction with re-levelling of building foundations. Once re-levelling is complete epoxy inject cracks in slab on grade per HCG specifications	880, 882-884

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G4	Floor Slab	Spreading in slab joint, approximately 5mm	Y	Re-level or replace existing slab on grade in conjunction with re-levelling of building foundations. Remove and replace damaged grout at existing construction joints	881
G	G4	Floor Slab	Eastern side (left of pictures) of construction joint appears to have settled significantly more than the Western side. Cracking to construction joint. Concrete slab on Western side of joint form the lid of a reinforced concrete service tunnel running underneath the building and appears to be bearing on the walls of this tunnel. Staff reported that glass doors had 'blown-out' during the 22nd Febuarary 2011 earthquake. Staff have since chased out a gap for the doors to run in. Further damage was noted in the 23rd December 2011 event.	Y	Re-level or replace existing slab on grade in conjunction with re-levelling of building foundations. Remove and replace damaged grout at existing construction joints	884, 885, 888
G	G4	Floor Slab	Vertical movement between concrete slabs at joint, damage noted in slab and adjacent window joinery	Y	Re-level or replace existing slab on grade in conjunction with re-levelling of building foundations. Remove and replace damaged grout at existing construction joints	886, 887
G	G4	Concrete Block Wall	Vertical crack in RCM wall propagating from lintel	Y	Epoxy inject crack in accordance with HCG specification	889
G	G26	Ceiling	Movement between ceiling and sprinkler causing damage to ceiling around sprinkler head.	Y	Aesthetic Repair to finishes, repair specification by others	890
G	G26	Wall at door	Horizontal and vertical cracking to partitions at bulkhead of door.	Y	Aesthetic Repair to finishes, repair specification by others	891

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G3	Floor	Spreading of slab joint, approximately 10mm	Y	Re-level or replace existing slab on grade in conjunction with re-levelling of building foundations. Remove and replace damaged grout at existing construction joints	892, 893
G	North Elevation	External Paving	'Hump' in car park paving over the top of a service tunnel due differential settlement between service tunnel and subgrade either side. Movement and diagonal crack in paving slab and kerb adjacent to service tunnel and hump in car park.	Y	Repair specification by others	976, 977
G	North Elevation	Brick Cladding	Cracking to base of brick veneer units. Due to movement of either; Masonry moving off foundation see Photo 980 or differential settlements of path see Photo 977	F	Replacement of damaged bricks required. Further investigation into integrity of existing brick ties is required.	978
G	North Elevation	Parapet	Capping over clay brick veneer has been dislodged	Y	Replace with light weight capping. See repair sketches included in Appendix D.	979
G	North Elevation	Brick veneer wall	Masonry and grout appear as though they may have moved off foundation. May also be as constructed.	F	Replacement of damaged bricks required. Further investigation into integrity of existing brick ties is required.	980-982
G	North Elevation	Awning	Diagonal cracks in soffit of reinforced concrete beam where hangers for lightweight awning below are anchored into the beam.	F	See Section 4	983, 986
G	North Elevation	Reinforced concrete beam	Horizontal crack in soffit of reinforced concrete beam at location of chamfered sections of beam.	F	See Section 4	984, 985

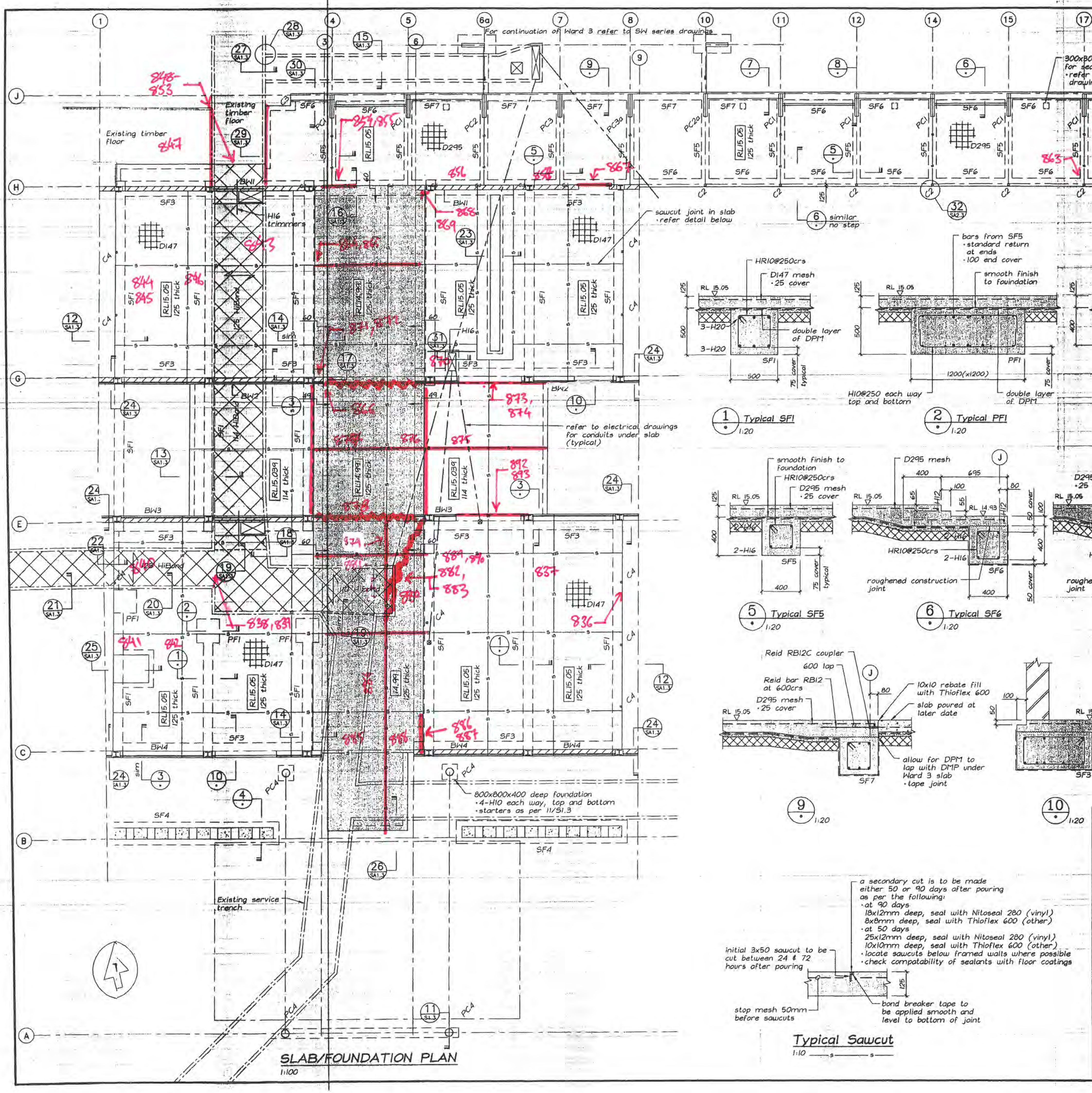
Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	North Elevation	Brick Veneer Wall	Stepped/Diagonal crack to brick veneer over the lightweight entry awning	**Y**	Check existing brick ties and replace as required. Repoint damaged all damaged mortor joints	987
G	G30	Awning	Differential settlement between external slab and entry awning slab. 10mm step has developed at the Eastern end of the door.	**Y**	Repair specification of site slab by others	989

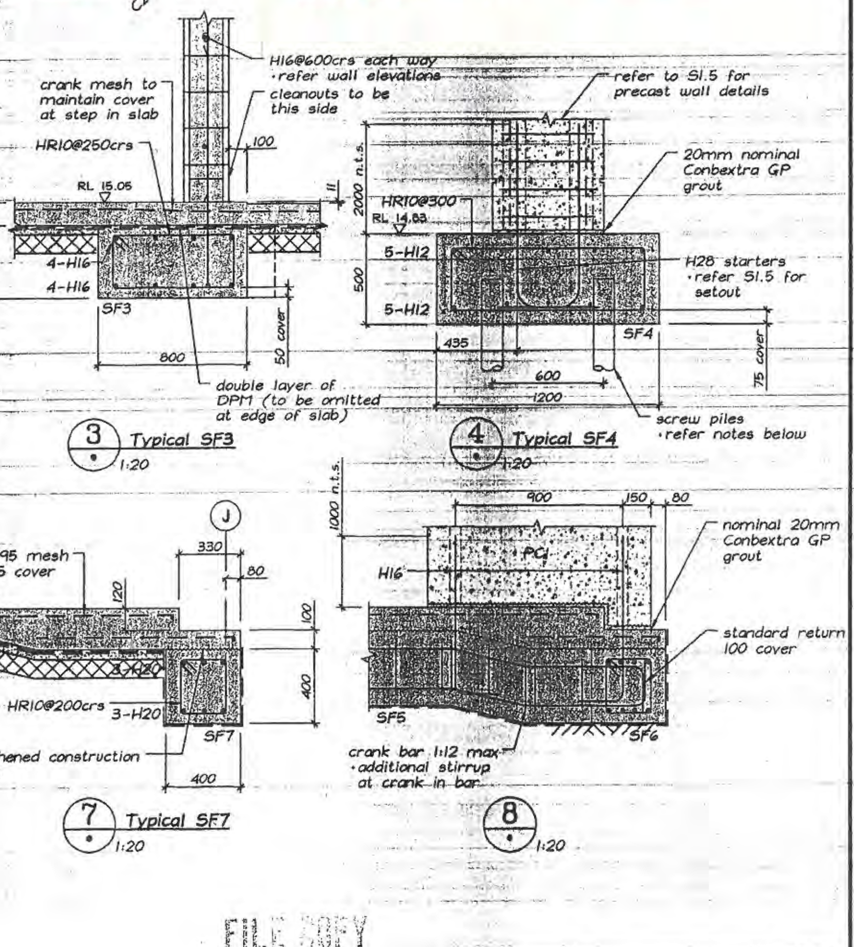
APPENDIX B

Reference / Key Plans

CDHB BURWOOD
ADMIN EQ REPORT
HCG # 106186.27

CRACK MAP +
PHOTO REFERENCE





FILE COPY

CHRISTCHURCH CITY COUNCIL
CONSENT DOCUMENT
26 JUN 2001
All building work shall comply with the New Zealand Building Code notwithstanding any inconsistencies which may occur in the drawings and specifications.

NOTES:

- 125 thick slab on ground reinforced with D147 mesh unless noted otherwise, 25 top cover slab to be poured on 2 layers of DPM, on 50mm layer of no fines granular fill on compacted hardfill
- Refer to architects drawings for external slabs on ground, paving and landscaping details
- Refer to architects drawings for all setout dimensions
- All sheet references on this sheet are to SA series drawings unless noted otherwise
- Form vinyl joints over slab control joints where possible

Screw Piling
- Screw piles are to be used under the foundation for the main precast feature wall (SF4) - refer Section 4. Piles shall be designed for:
  23 tonnes/m compression
  14 tonnes/m tension
- The contractor is to provide the Engineer with the proposed pile layout
- The piles are to extend below the existing service tunnels
- Reinforcing details above piles to be confirmed upon receipt of the pile layout

ISSUE	DATE	AMENDMENT	CHECKED
A	18.05.01	Construction Issue	

RECEIVED
16 MAY 2005
HOLMES CONSULTING GROUP LIMITED CHRISTCHURCH

POWELL FENWICK CONSULTANTS LTD
Consulting Engineers, Structural, Civil, Acoustic, Fire, Electrical, Mechanical, Heating and Ventilation

Canterbury HEALTH

BURWOOD HOSPITAL
NEW ADMINISTRATION BUILDING

GROUND FLOOR/FOUNDATION PLAN

CONTRACTOR MUST VERIFY ALL DIMENSIONS ON SITE

SCALES:	DESIGNED	SMW	01/01
1:100 1:20 1:10	DRAWN	PPN	01/01
	CHECKED		05/01

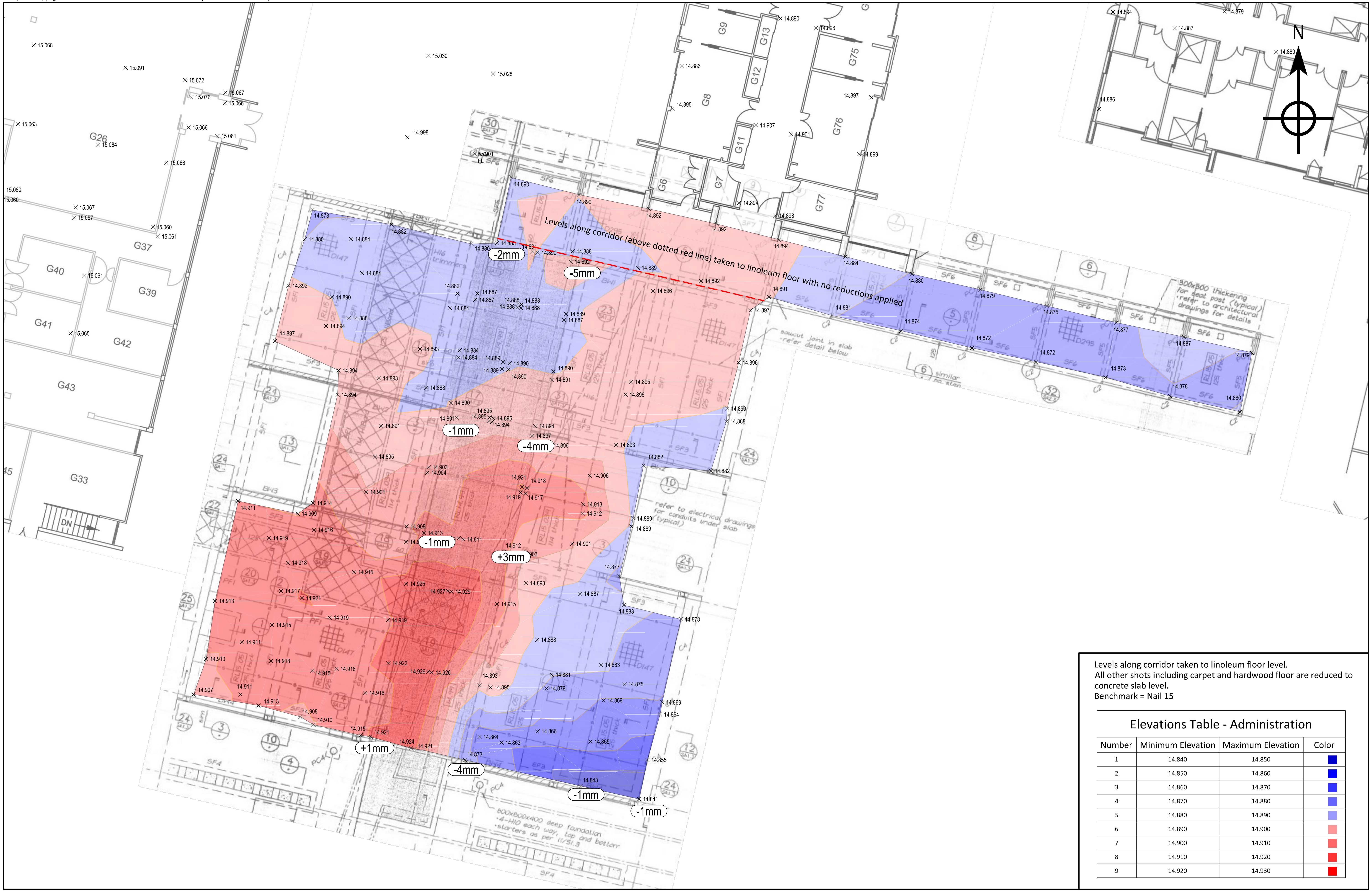
SET	JOB NO.	SHEET NO.	ISSUE
	991032	SA1.2 OF - SHEETS	A

DATE PRINTED: 18 MAY 2001

APPENDIX C

Survey of Levels & Verticality Study

K:\2462C CDHB Burwood\dwg\2462C 20120124 Elevations Survey Jan 2012.dwg : 01 Feb 2012 12:49 p.m. : 4 Administration



Levels along corridor taken to linoleum floor level.
All other shots including carpet and hardwood floor are reduced to concrete slab level.
Benchmark = Nail 15

Elevations Table - Administration

Number	Minimum Elevation	Maximum Elevation	Color
1	14.840	14.850	
2	14.850	14.860	
3	14.860	14.870	
4	14.870	14.880	
5	14.880	14.890	
6	14.890	14.900	
7	14.900	14.910	
8	14.910	14.920	
9	14.920	14.930	

No.	Revision	Approved	Date
G			
F			
E	Resurvey and audit observations post Dec 23 2011	MJM	01/02/2012
D	Wards 7 & 8, Surgical Block, Melrose Chairs & Corridor added	MJM	02/12/2011
C	Phys Med, Parafed, Food Services, BIRS, Boilerhouse, EDC added	MJM	24/11/2011
B	Nurses Hostel & Champion Centre, SOU elevations added	MJM	18/11/2011
A	Administration and Allan Bean Centre elevations added	MJM	31/10/2011

Scale		1:100
Reduced	A3	1:200
Designed		----
Drawn		DHP
Checked		MJM
Date		02/12/2011

Burwood Elevations Survey

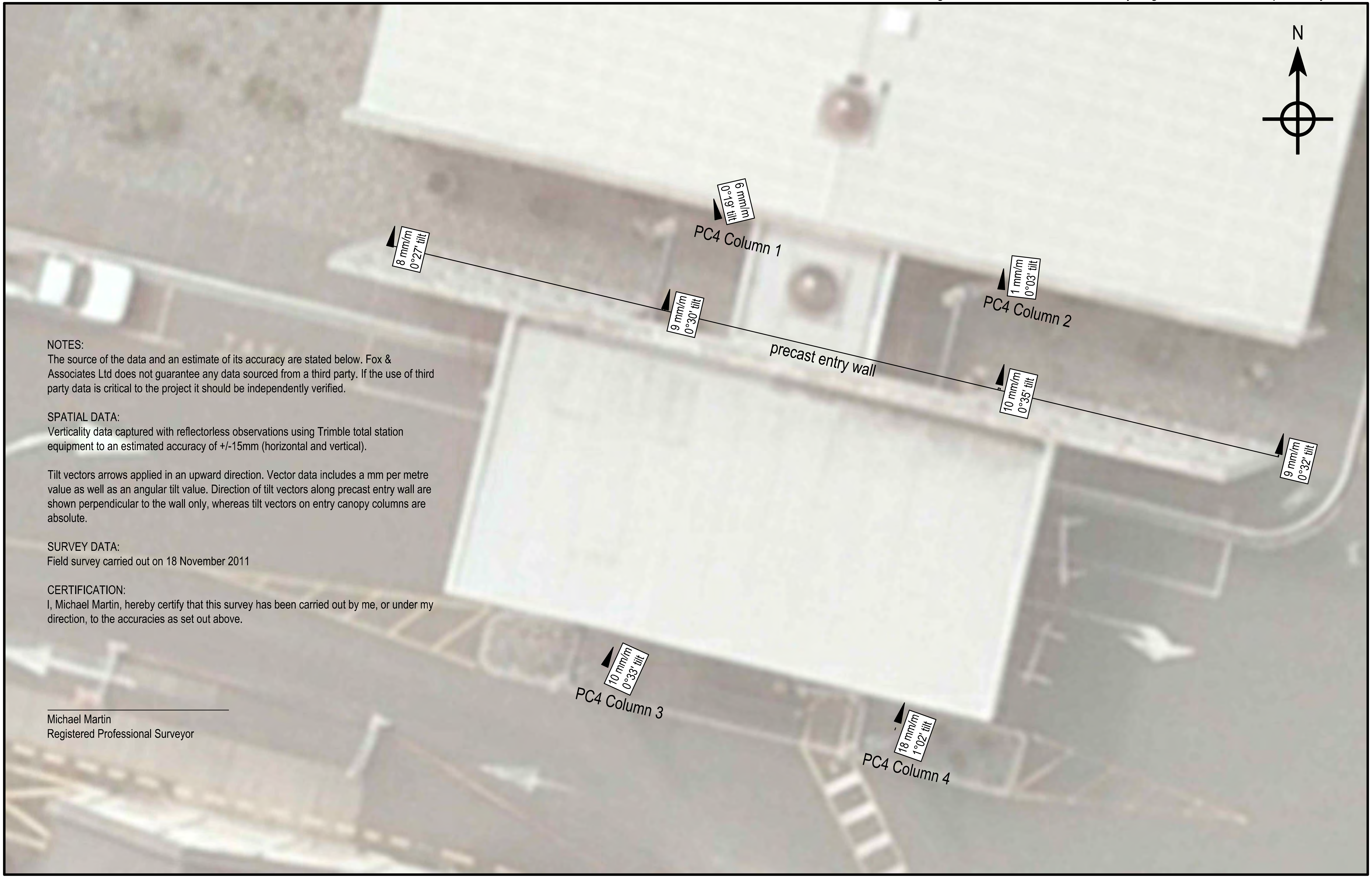
Administration

SURVEYORS
Fox & Associates
URBAN DESIGNERS
CONSULTANTS
LAND DEVELOPMENT
www.foxsurvey.co.nz
0800 FOX SURVEY
P.O.Box 13-390
CHRISTCHURCH

Job No. 2462C
Sheet No. 4 of 15
Rev. E

A1 Original Size

032005



NOTES:
The source of the data and an estimate of its accuracy are stated below. Fox & Associates Ltd does not guarantee any data sourced from a third party. If the use of third party data is critical to the project it should be independently verified.

SPATIAL DATA:
Verticality data captured with reflectorless observations using Trimble total station equipment to an estimated accuracy of +/-15mm (horizontal and vertical).

Tilt vectors arrows applied in an upward direction. Vector data includes a mm per metre value as well as an angular tilt value. Direction of tilt vectors along precast entry wall are shown perpendicular to the wall only, whereas tilt vectors on entry canopy columns are absolute.

SURVEY DATA:
Field survey carried out on 18 November 2011

CERTIFICATION:
I, Michael Martin, hereby certify that this survey has been carried out by me, or under my direction, to the accuracies as set out above.

Michael Martin
Registered Professional Surveyor

A3 Original Size

No.	Revision	Appr.	Date
D			
C			
B			
A			

Scale 1:100
Designed
Drawn DHP
Date 18/11/2011

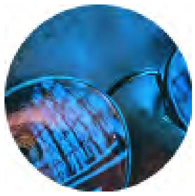
CDHB - Burwood Administration
Verticality survey of entry wall

SURVEYORS URBAN DESIGNERS Fox & Associates CONSULTANTS LAND DEVELOPMENT

www.foxsurvey.co.nz
0800 FOX SURVEY
P.O.Box 13-390
CHRISTCHURCH

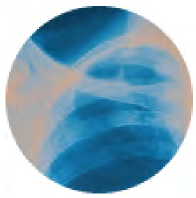
Job No. 2462C
Rev.

032005



DETAILED SEISMIC ASESSMENT REPORT

STRUCTURAL AND CIVIL ENGINEERS



BURWOOD HOSPITAL CAMPUS

REPORT 12 - BOILER HOUSE AND

SITE MAINTENANCE BUILDING

PREPARED FOR

CANTERBURY DISTRICT HEALTH BOARD

106186.25

INTERIM REPORT REV 1 – AUGUST 2012

BURWOOD HOSPITAL CAMPUS – INTERIM DETAILED SEISMIC ASSESSMENT REPORT

REPORT 12 – BOILER HOUSE AND SITE MAINTENANCE BUILDING

Prepared For:
CANTERBURY DISTRICT HEALTH BOARD

Date: 22 August 2012
Project No: 106186.25
Revision No: 1

Prepared By:

Craig O'Neill
PROJECT ENGINEER

Reviewed By:

Didier Pettinga
PROJECT ENGINEER

Holmes Consulting Group LP
Christchurch Office

REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
22/08/2012	1	Interim Issue for release

CONTENTS

APPENDICES

TABLES Page

FIGURES Page

EXECUTIVE SUMMARY

Holmes Consulting Group has been engaged by Canterbury District Health Board (CDHB) to complete a full structural review of the Burwood Campus following the Lyttelton Earthquake. A series of reports have been compiled as part of this process. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The specific building reports, like this one for the Maintenance and Boiler House, should be read in conjunction with the base report, and refer to the repair specification.

This report identifies the structural damage sustained by the Maintenance and Boiler House as a result of the series of Earthquakes, including the Darfield Earthquake that struck at 4:36am on the 4th September 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February 2011; the June Earthquake that struck at 2.20pm on the 13th of June 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The report summarises the effects of the damage on the lateral load capacity of the building, and provides structural repairs for the damage identified. The general form of the building, along with its capacity relative to current code levels, has been included for the buildings pre!earthquake undamaged state and post!earthquake state. Recommendations to increase the strength of the building to greater than 67% current code capacity have also been summarized.

The Boiler house and Site Maintenance Building were designed in 1965 by Mawson, Seward and Stanton Registered Architects and constructed in the period there after. The building consists of 3 connected structures; the boiler house, the maintenance building and the central plant/substation room. There is also a small lean!to garbage enclosure sharing the eastern wall of the Boiler house.

The Boiler house is predominately a single storey structure with a reinforced concrete roof. The walls are framed by concrete beams and columns with a lightly reinforced masonry infill and a masonry veneer. The building houses three boiler units and supports three large coal bunkers at roof level. Coal bunkers are constructed of reinforced concrete and are supported by a steel frame structure. Above the bunkers is a conveyor room constructed of a light!weight timber 'cut' roof supported on a perimeter concrete beam on concrete columns. There is a lightly reinforced masonry infill. The lean to garbage enclosure also has a reinforced concrete roof. It is supported on concrete perimeter beams and columns. Internal and external walls are infill unreinforced masonry. There are also three large steel chimneys connected to the southern elevation.

The Maintenance building is a two!storey concrete framed building with a light!weight steel trussed roof. It contains workshops on the ground floor with offices and staff facilities on the first floor. Interior walls on the first floor are timber stud, lined with plasterboard and ground floor is a combination of reinforced concrete and partially reinforced masonry. External walls are reinforced concrete piers and walls with partially reinforced masonry piers, spandrels, and infill panels. The roof is braced by steel cross bracing welded between trusses.

The Plantroom is a central single!storey link between the Maintenance building and Boiler house. It has a light!weight roof supported on steel portal frames. The steel columns are concrete encased. The North and South ends of the roof are connected to the Maintenance building and Boiler house walls. There is a central substation with reinforced concrete walls and lid. Internal walls are partially reinforced masonry with external walls being a combination of reinforced concrete columns and partially reinforced masonry.

The information available for review included; a sample of the original 1962 architectural drawings [3], a post!earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [4], along with a level survey of the building completed by Fox & Associates [5]. Although no structural drawings were available for review, a reinforcement survey was carried out by Hilti on 23rd May 2012 in order to confirm reinforcement quantities in walls and concrete elements.

The majority of the builder appears to have performed well considering the construction, age of the building and the seismic action experienced at the site. The damage has predominately been to the concrete and masonry elements of the building in the way of vertical and diagonal cracking. The most sever case of cracking is in the internal masonry wall to the plantroom which has opened up approximately 5mm. There are hairline to 1mm cracks located throughout the Maintenance building, Boiler house and Plantroom.

There is approximately a 10mm separation between the Plantroom wall in the North!East corner and adjacent Maintenance building wall. There is also damage to the plantroom roof connection in this area.

Liquefaction has caused settlement to the external pavements around the South!West corner of the Boiler house entry. There was also around 150mm settlement to the coal store ramp in the same area. The crib wall had become damage and rectifications had been put in place after the February 2011 event, but some damage and re!settlement has since re!occurred.

While some amount of damage has likely occurred in all the significant events noted, it is believed that the onset, and majority, of damage occurred as a result of the 22nd February event.

Further observations of the earthquake damage observed have been included in the body of this report.

Based upon a review of the construction documentation available and the site investigations completed, the primary lateral force!resisting elements of the building have been assessed in their pre!earthquake undamaged state.

Considering the building use and the services it provides, the Plantroom and Boiler House have been classified as an Importance Level 3 (IL3) building in accordance with NZS 1170:2004 [9]. While the Maintenance building is not an essential facility, collapse or damage may result in the Boiler House and Plantroom being deemed inaccessible so has also been included in this category.

The first floor of the Maintenance Building has been assessed to have a pre!earthquake capacity to resist approximately 45% of the demand required by the current code Design Basis Earthquake (DBE) in both the north!south and east!west directions. The ground floor has been assessed to have 55% and 36% DBE in the north!south and east!west directions respectively. The limiting factor for the DBE% was the shear capacity of the shear walls to handle the required lateral loads.

The Boiler House has been assessed to have a pre!earthquake capacity to resist approximately 37% DBE at a nominally ductile level in east!west direction, and 58% in the north!south direction. These capacities assume that the masonry infill panels have sufficient integrity to participate in the lateral force resistance. It was found the structure above the roof level (conveyor room and coal bunkers) would be able to resist and transfer 75% and 100% DBE in the north!south and east!west directions respectively. The limiting factor for the DBE% in the main structure was the capacity of the shear walls to handle the required lateral loads.

The ground floor masonry walls in the boiler house have been assessed to have 100% capacity under face loading in the ground floor walls when they are bounded by the first floor concrete beams. The upper half!height walls beneath the windows are assessed to have out!of!plane capacities around 30%, however this is under the assumption that there is no bond!beam at the underside of the window frame. During a design level seismic event is it likely that these walls will sustain considerable damage and potentially pose a falling object hazard.

The lean to garbage enclosure to the east of the Boiler House was found to have approximately 100% and 70% DBE in the north!south and east!west directions respectively based on the capacity for shear walls to support to required lateral loads. The chimneys structures have a capacity of 100% DBE although the connection of the smaller chimney to the east is questionable. Nevertheless we understand this chimney is scheduled to be demolished.

The Plantroom steel portal frames have the capacity to resist approximately 35% DBE in the east!west direction and 40% in the north!south direction with a limited ductile load level. The deflections of the portals will be excessive and likely damage windows and external masonry infills.

From the damage we have observed, we conclude that the lateral capacity of the building has not been severely reduced. The majority of the damage is cracking in walls and linings, much of which requires repair for functional reasons rather than for seismic force resistance. In some focussed locations more significant repairs will be required to re!instate the seismic capacity and resistance of the structure.

The minimum repairs required to reinstate the building to its pre!earthquake undamaged condition have been included in Section 4. This includes epoxy injecting cracks in various locations, reconstruction of pavements and local repairs to the plantroom roof. One location in the plantroom will require reconstruction of the concrete block wall in order to reinstate the seismic strength of this portion of the building.

In addition to the repairs, recommended strengthening concepts to increase the seismic performance of the building to attain 67% DBE, have been included in section 5.

With regards to Critical Structural Weaknesses, the Boiler house structures can be considered to have a CSW due to the rigidly connected cast in!situ stairs in the Maintenance Building. This CSW does not affect the global performance of the building as the masonry infill walls are still the governing elements. However it is an issue that can readily be resolved with an appropriate cut and release of the stairs, along with re!seating to allow relative lateral movements to occur between the ground and first floors.

The Maintenance building and Boiler House have been assessed above 33% DBE in both principal directions for in!plane wall panel response. The assessed east!west capacities of 36% and 37% indicate that, respectively, these buildings are an earthquake risk. It is recommended that these be seismically upgraded to 67% DBE.

1. INTRODUCTION

Holmes Consulting Group has been engaged by Canterbury District Health Board to complete a full structural review of the Burwood Hospital Campus following the Lyttelton Earthquake. A series of reports have been completed as part of this. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The individual building reports, like this one, should be read in conjunction with the base report and refer to the repair specification.

The Burwood Hospital Campus base report covers the purpose and scope of the structural review. The current statutory requirements relevant to earthquake damaged buildings are outlined and the level of shaking experienced at the site estimated. Earthquake induced ground settlement damage across the campus is also discussed. The repair specification has been prepared to include repair details for typical damage observed in buildings on the Burwood Hospital Campus and is referred to as required in the specific building reports.

1.1 SCOPE OF WORK

This report is on the Boiler House and Site Maintenance Building, at Burwood Hospital, Mairehau Rd, Christchurch. The report identifies the general form of the structure, along with the gravity and lateral load resisting systems. Each component of the structural system was reviewed based upon the information available and any potential Critical Structural Weaknesses (CSW's) were noted.

The report also identifies the structural damage observed to date as a result of the series of Earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake, in particular, subjected the building to strong ground motions which significantly exceeded the current code loading demand for buildings of this nature.

The capacity of the Boiler House and Site Maintenance Building have been assessed relative to current code loading in the buildings pre-earthquake undamaged state and in its post-earthquake damaged state. The post-earthquake assessment summarizes the effects of the damage identified on both the gravity and lateral load resisting elements. Repair options to restore the buildings capacity to pre-earthquake levels for strength, durability and stiffness have been included. The repair options aim to maintain the buildings utility. Where required, strengthening options have also been provided.

1.2 LIMITATIONS

Findings presented as a part of this project are for the sole use of the Canterbury District Health Board, 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. Because all of the structure has not been available for detailed inspection or evaluation, this report is limited to those elements available and engineering judgement as to the likely condition of unseen elements. 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. PRE-EARTHQUAKE BUILDING CONDITION

This section discusses the form and capacity of the building prior to the Darfield Earthquake

2.1 BUILDING FORM

The Boiler house and Site Maintenance Building were designed in 1965 by Manson Seward and Stanton Registered Architects and constructed in the period there after. The building consists of three connected structures; the Boiler House, the Maintenance Building and a central plant/substation room. There is also a small lean-to garage enclosure sharing the eastern wall of the Boiler house. The west elevation of the buildings is shown in Figure 2-1 with the different areas indicated.

Figure 2-1: L-R Maintenance Building, Plantroom and Boiler House – West Elevation

The information available for review included; a sample of the original 1962 Architectural drawings [3], a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [4], along with a level survey of the building completed by Fox & Associates [5]. Although no structural drawings were available for review, a reinforcement survey was carried out by Hilti on 23rd May 2012 in order to confirm reinforcement quantities in walls and concrete elements.

Due to the different construction types for the

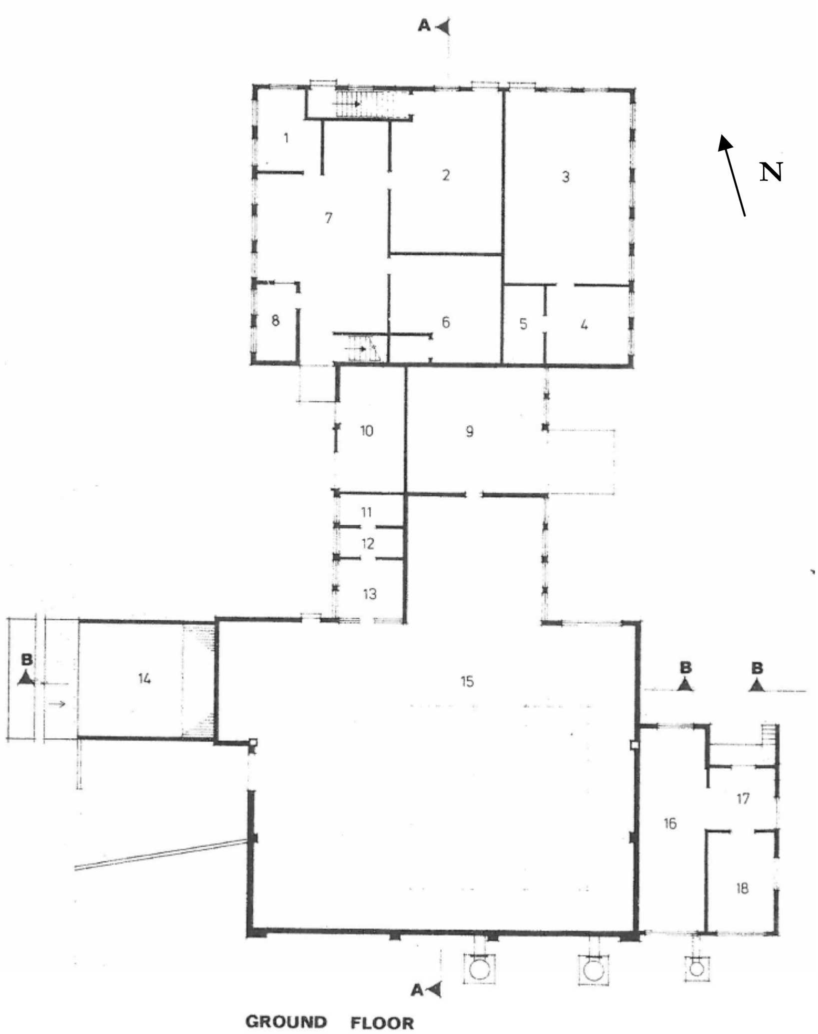


Figure 2-2: Boiler House, Plantroom and Maintenance Building – Ground Floor Plan

The Boiler house is predominately a single storey structure with a reinforced concrete roof. The walls are framed by concrete beams and columns with a lightly reinforced masonry infill and a masonry veneer. The building houses three boiler units and supports three large coal bunkers at roof level. Coal bunkers are constructed of reinforced concrete and are supported by a steel frame structure. Above the bunkers is a conveyor room constructed of a light-weight timber 'cut' roof supported on a perimeter concrete beam on concrete columns. There is a lightly reinforced masonry infill. The lean to garbage enclosure also has a reinforced concrete roof. It is supported on concrete perimeter beams and columns. Internal and external walls are infill unreinforced masonry. There are also three large steel chimneys connected to the southern elevation.

The Maintenance building is a two-storey concrete framed building with a light-weight steel trussed roof. It contains workshops on the ground floor with offices and staff facilities on the first floor. Interior walls on the first floor are timber stud, lined with plasterboard and ground floor is a combination of reinforced concrete and partially reinforced masonry. External walls are reinforced concrete piers and walls with partially reinforced masonry piers, spandrels, and infill panels. The roof is braced by steel cross bracing welded between trusses.

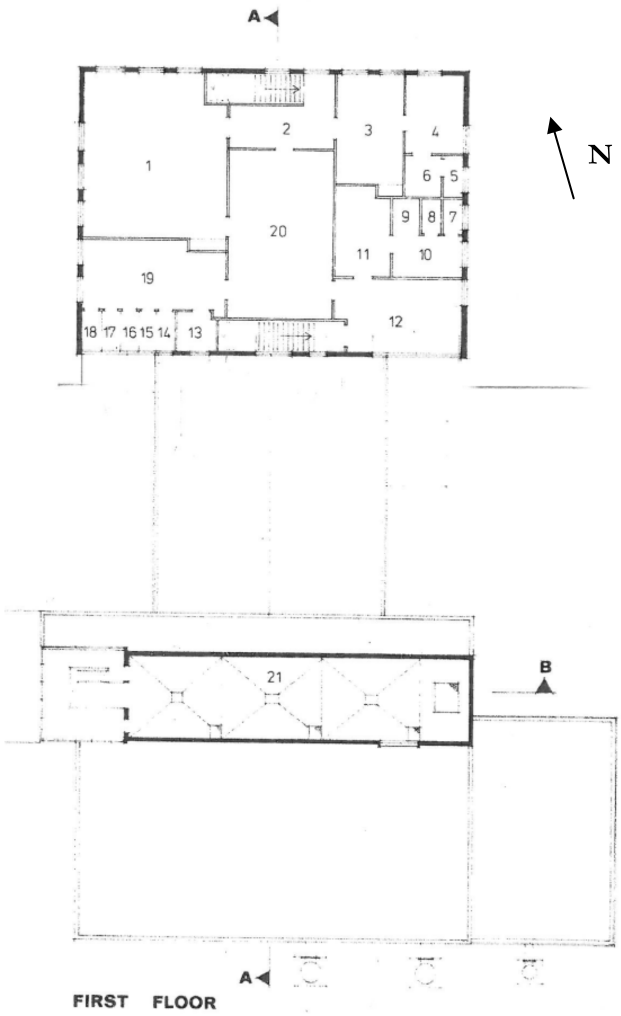


Figure 2-3: Boiler House, Maintenance Building – First Floor Plan

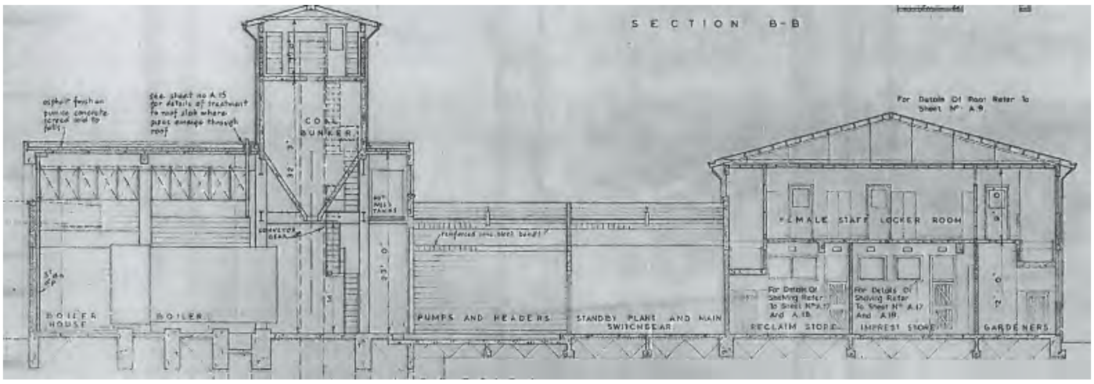


Figure 2-4: Building Section – Left to Right; Boiler House, Plantroom, Maintenance Building

The Plantroom is a central single-storey link between the Maintenance building and Boiler house. It has a light-weight roof supported on steel portal frames. The steel columns are concrete encased. The North and South ends of the roof are connected to the Maintenance building and Boiler house walls. There is a central substation with reinforced concrete walls and lid. Internal walls are partially reinforced masonry with external walls being a combination of reinforced concrete columns and partially reinforced masonry.

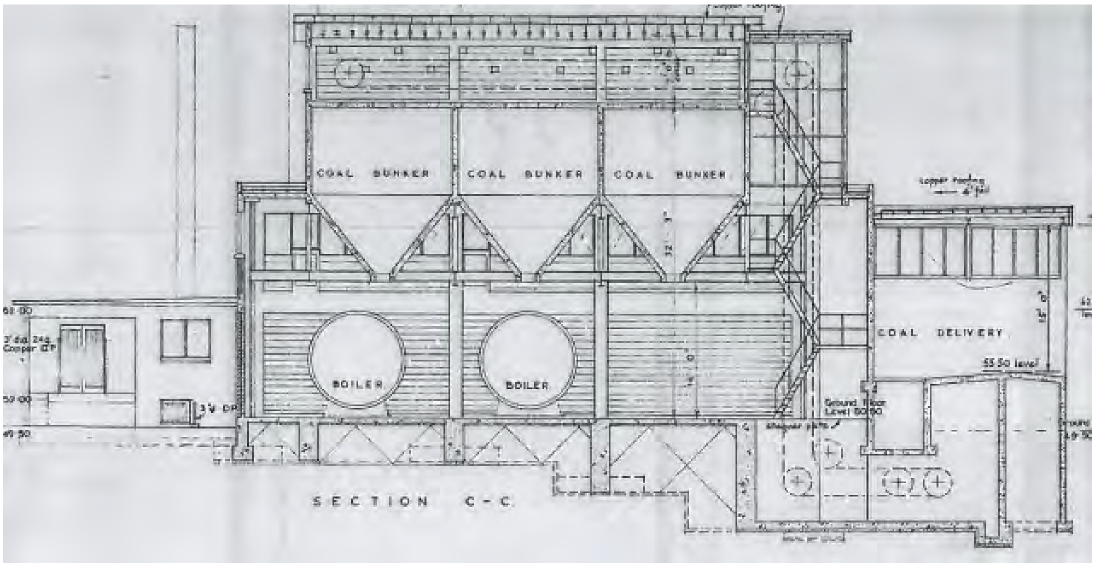


Figure 2-5: Building Section –Boiler House

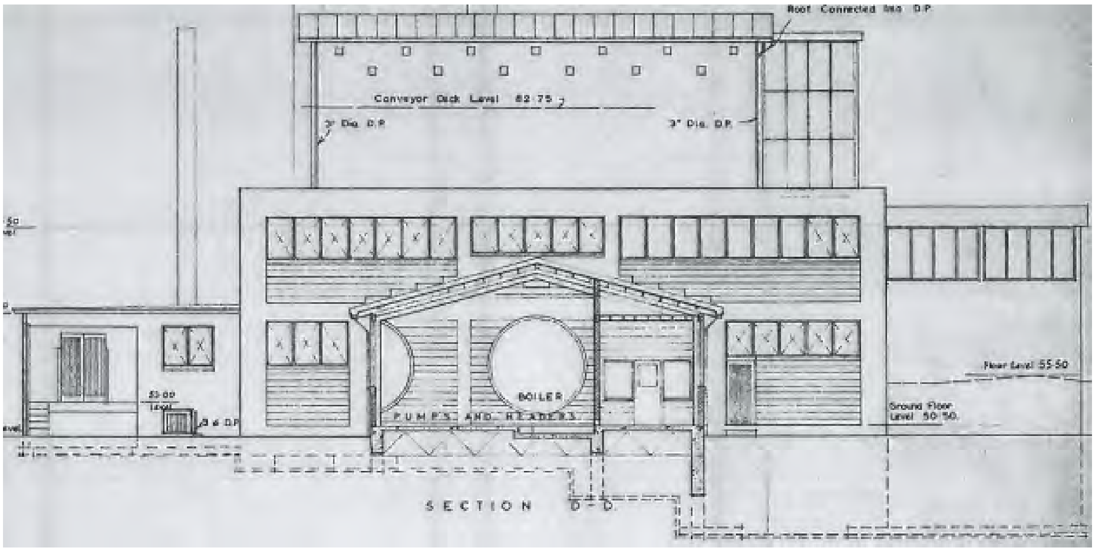


Figure 2-6: Building Section – Through Plantroom, Boiler House beyond

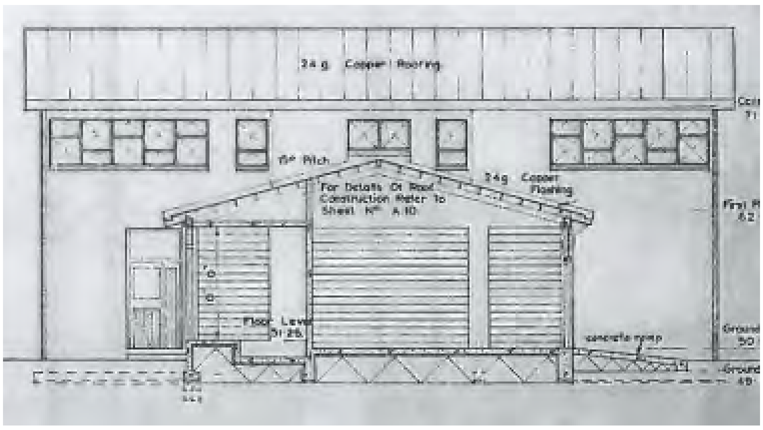


Figure 2-7: Building Section – Through Plantroom, Maintenance Building beyond

2.2 LATERAL LOAD RESISTING SYSTEMS

Boiler House: The lateral load resisting system for the Boiler house consists of concrete and concrete masonry walls. The lateral loads at the west end of the conveyor room at the access stair are resisted by a steel frame. The lateral loads in the NS and EW direction at the top of the coal bunkers are transferred to the boiler room roof diaphragm by the concrete walls of the coal bunkers. The concrete roof diaphragm transfers the lateral loads at the level of the boiler room to the concrete frames and walls at the exterior of the building. It is assumed the steel frame below the coal bunkers support gravity loads only and do not contribute resisting lateral loads as they have been determined to be very flexible.

The garbage enclosure lateral loads are resisted by internal and external concrete walls. The loads are transferred to the walls through the concrete roof diaphragm.

The adjoining chimneys are cantilevered steel circular sections laterally supported by bolted strut connections to the Boiler house and garbage enclosure.

Maintenance Building: The lateral load resisting system for the Maintenance building consists of a cross braced steel trussed roof on the first floor transferring loads into the external concrete frames and walls. The concrete first floor distributes the lateral loads at the lower level to internal and external concrete frames and walls.

Plantroom: The lateral load resisting system for the Plantroom is comprised of unreinforced masonry walls and concrete columns in the NS direction, and steel portal frames in the EW direction. The roof is constructed of timber purlins with a straight timber sheathing diaphragm.

2.3 PRE-EARTHQUAKE BUILDING CAPACITY – DIRECT CODE COMPARISON

The building capacity under earthquake actions discussed in this section is compared to the capacity that a similar building would be designed to today. A new building would be designed to the *Structural Design Actions Standard, Part 5: Earthquake Actions – New Zealand*, NZS 1170.5:2004[9] and incorporating the amendments made to this standard as a result of the Lyttelton Earthquake as outlined in the Amendment 10 of the Building Code [8]. The implications of the recent amendments are discussed more fully in the Burwood Hospital Campus Base Report however, for a building of this type the amendments essentially result in an increase to the design loads of 36 % when compared to pre-earthquake design levels.

When the building was originally designed in 1965, the loading standard at the time was the *New Zealand Standard Model Building Bylaw – Chapter 8, Basic Design Loads*, NZSS 1900:1965 [11]. When these By-Laws were written, neither the seismology of the different areas within New Zealand, or the impact this could have on buildings was as well understood as it is today. Along with an increase in the seismic demands required by the change in the loading code over this period, the seismic detailing requirements have also progressed significantly resulting in more ductile and better performing buildings.

The current code requires a new building to be designed for an earthquake, known as the Design Basis Earthquake (DBE), which is based upon the buildings physical location, local soil conditions, building type, fundamental period and importance level.

The original structural drawings for the building are not available. For the purposes of this report seismic loading assumptions have been made based on a detailed review of the drawings available and physical observations of the building.

Based on the building use and the services it provides, the Plantroom and Boiler House have been classified as an Importance level 3 building in accordance with NZS 1170:2004 [9]. While the Maintenance building is not an essential facility, collapse or damage may result in the Boiler House and Plantroom being deemed inaccessible so has also been included in this category. The associated return period of the DBE is 1,000 years, with a risk factor for design of R = 1.3. The sub soil for the site is taken as Soil Type D, which is consistent with the findings of a post-earthquake geotechnical investigation [4].

Based upon the period of construction, and the detailing of the lateral load resisting elements, the concrete portion of the building has been concluded to have nominal ductility, and as such the reinforced concrete walls have been assigned a ductility factor of **µ**=1.25. The steel framed portion of the building is believed to have limited ductility and has been assigned a ductility factor of **µ**=2.00. Unreinforced masonry portions have been assigned a ductility factor of **µ**= 1.00.

A comparison between the Design Basis Earthquake of NZSS 1900:1965 and NZS 1170:2004 for the site is plotted below. Based upon a fundamental building period below 0.50 seconds, the seismic demands required by the loading code have increased on the concrete and steel portion of the structure by approximately 560% and 300% respectively since 1975.

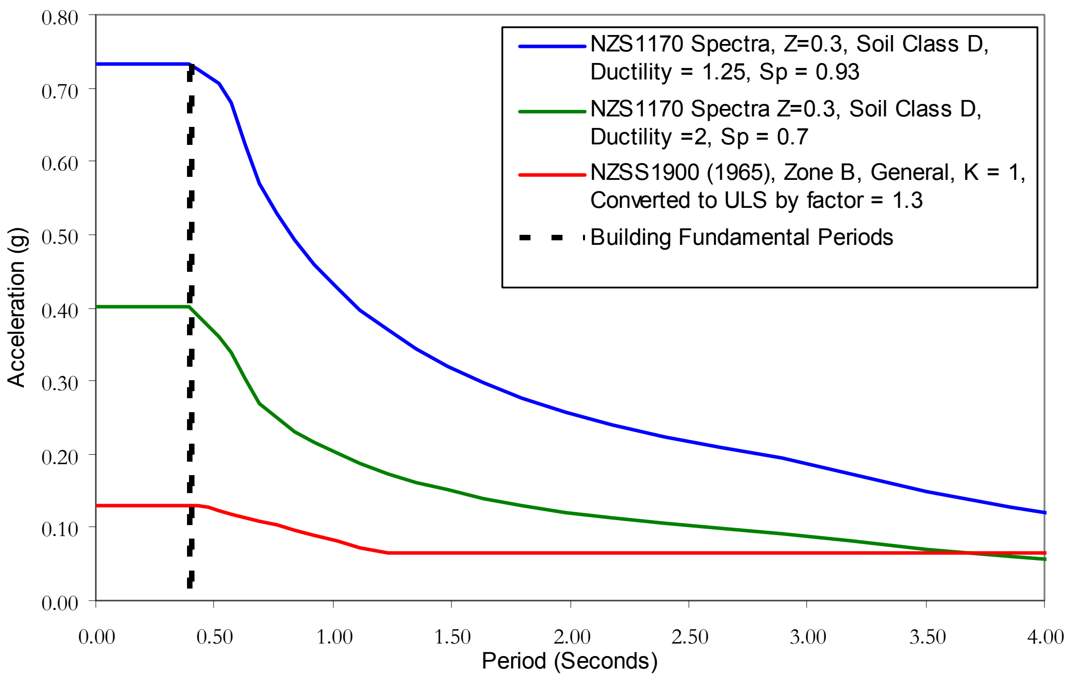


Figure 2-8: Comparison of Design Codes

2.4 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 estimated 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.

Following the Lyttelton earthquake, a geotechnical report was conducted by Tonkin & Taylor, titled "Burwood Hospital Post Earthquake Geotechnical Assessment", dated June 2011 [4]. This report has been used to aid in the evaluation of the site conditions and the likely effect of the ground on the buildings past and future performance. The soil parameters described in the geotechnical report complete by Tonkin and Taylor have been used for the evaluation of the buildings existing foundation system.

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 [14]. 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. These are described in more detail in the Burwood Hospital Base report [1] and include strength governed elements such as short columns and deflection governed elements such as floor and stair elements with inadequate support seating, as well as out-of-plane masonry wall failure.

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 the strength and deflection requirements.

As previously noted, the three sections of the building, the Maintenance Building, Plantroom and Boiler House have been treated as independent structural elements.

The limiting factor for the capacity of the Maintenance Building is the ability of the end walls to absorb the required shear forces. In cases where there is sufficient shear capacity, the lightly reinforced piers are unable to handle the bending moments induced at their restraints, and would essentially have a brittle failure mechanism.

The Plantroom portal frames do not have the required capacity to absorb the bending induced during a design seismic event in order to brace the building. Excessive deflections are also predicted in the frames.

The Boiler House is limited in capacity by the ability for end walls to absorb shear forces induced during a design seismic event. The lightly reinforced infill masonry walls do not contribute to bracing as they are below the required strength and are likely to fail in a brittle manner and collapse under face loadings.

For all concrete elements, we have assumed a 20MPa concrete strength and a 300MPa reinforcing steel strength. Other structural steel components have been assumed to have a yield strength of 250MPa

The foundations have been assessed at an ultimate bearing capacity of 150kPa, for factored loads, as per recommendations provided by Tonkin and Taylor.

A summary of the %DBE for each section of the building has been noted in Table 2-1 below.

Table 2-1: Summary of Seismic Assessment %DBE

Building Element	%DBE (IL3)	Comments
Maintenance building		
N-S	55%	Limited by concrete and in-fill wall shear capacity
E-W	36%	Limited by concrete and in-fill wall shear capacity
Plantroom		
N-S	40%	Limited by ceiling diaphragm capacity
E-W	35%	Limited by portal frame capacity
Boiler House		
N-S	58%	Limited by concrete and in-fill wall shear capacity
E-W	37%	Limited by concrete and in-fill wall shear capacity

A review of the drawings available indicates that the cast in-situ staircases in the Maintenance Building may be considered a Critical Structural Weakness. This is a relatively simple CSW to remediate, for which indication is provided in Section 5.

Methodologies to improve the seismic performance of the buildings by providing strengthening to enhance up to 67% DBE, have been included in Section 5.

3. POST-EARTHQUAKE BUILDING CONDITION

This section covers the structural damage sustained by the Boiler House and Maintenance Building, and its effect on the buildings capacity to resist seismic loads, as a result of the series of earthquakes which includes the Darfield Earthquake that struck at 4:36am on 4th September, 2010, the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011, the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake subjected the building to strong ground motions which significantly exceed the full design earthquake load for buildings of this nature and appears to have caused the majority of the earthquake damage observed, after the Darfield event.

3.1 THE LYTTELTON EARTHQUAKE

The fundamental period of the buildings is estimated to be less than 0.5 seconds. Due to the highly variable ground conditions around Christchurch, it is not possible to determine what the actual shaking experienced at the site was. However, based on the strong motion data presented in the Base Report, it appears the Lyttelton Earthquake produced ground shaking intensities of between 45 90% of the current Ultimate Limit State design spectra for an Importance Level 3 building.

It should be noted that the Lyttelton Earthquake was very short in terms of strong shaking produced with the strong motion only lasting for duration of 5 7 seconds. Rupture of the Alpine Fault is expected to contain 50 to 60 seconds of strong motion.

3.2 PRELIMINARY INVESTIGATIONS

Preliminary investigations were carried out to identify 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
- review of available construction documentation
- damage observed during 'Rapid' Structural Assessments following the Lyttelton Earthquake, June 13th aftershocks and subsequent events

In conjunction with a review of the structural drawings for the building the following areas were identified for potential damage:

- **Maintenance Building**
    - movement or damage to structure associated with ground movement and/or settlement
    - cracking and joint failure of concrete beams, columns, shear walls, floor diaphragm and foundations
    - cracking in reinforced and unreinforced masonry shear/infill walls
    - general distress in steel roof trusses and diagonal cross bracing
- **Substation Room**
    - movement or damage to structure associated with ground movement and/or settlement
    - cracking and joint failure of concrete walls, and foundations
    - general distress to the steel portal frames, including beam column joint welds
    - cracking in concrete and masonry shear walls
    - distress to roof diaphragm and connections into adjacent buildings
- **Boiler House (including adjacent garbage waste room)**
    - movement or damage to structure associated with ground movement and/or settlement
    - cracking and joint failure of concrete beams, columns, shear walls, floor and roof diaphragm and foundations
    - cracking in reinforced and unreinforced masonry shear walls
    - general distress to steel framing supporting the hopper including bolted and welded connections
    - distress and cracking to infill masonry walls
    - signs of distress at interfaces between different sections of the building
    - damage to the concrete encase of the steel column supporting the hopper frame on the east wall

Rapid Level 2 assessments were carried out on the 24th and 28th February 2011[19] and on the 16th June 2011 [20] following the June 13th earthquakes. An additional Rapid Visual Structural Assessment was conducted on 6th January 2012 [21], following the 23rd December 2011 and 4th January 2012 events. These structural observations involved a complete walk around the exterior and throughout the interior of the building. The following primary areas of damage were identified from the damage assessments:

Maintenance Building

- cracking to southern stair walls
- cracking to northern stair walls
- cracking out from window corners
- minor interior damage to linings

Boiler House

- cracking to eastern wall and concrete piers, level 2
- minor cracking to infill panels
- movement of chimney pad bases
- damage to crib walls at coal store entry

Plantroom

- 10mm gap between plantroom and Maintenance Building
- Step cracking to concrete blockwork

A review of the above information on the building type and preliminary observations highlighted this building as requiring a more detailed inspection. The aim of the detailed inspections was to determine the full extent of the damage caused to the building, particularly those elements identified for potential damage above. These areas were targeted to identify if damage had occurred, and to what extent the damage had reduce the capacity of the buildings lateral load resisting system to resist future seismic events.

3.3 DETAILED STRUCTUAL OBSERVATIONS

Further detailed inspections and structural explorations have been carried out following the initial assessments to ascertain the full extent of structural damage. The majority of the detailed structural observations were completed on the 20th October 2011, with additional trips made to the site on 31st January 2012, 30th March 2012 and 1st May 2012.

A full record of these observations can be found in Appendix A, with reference plans describing the location labelling used found in Appendix B. A full photographic record of the observations is available electronically on request. The detailed structural observation identified the following additional damage to those items noted in the initial rapid assessments:

- cracking to north and south stair walls in the Maintenance Building
- cracking to concrete and masonry elements in the Boiler House that may represent a critical structural weakness due to out of plane failure of the masonry walls
- liquefaction and external ground settlement around the SW corner of the Boiler House

- large cracks to the north south masonry wall in the plantroom
- separation between the Plantroom and Maintenance Building structure

3.4 GEOTECHNICAL REVIEW

A review of the ground damage and conditions was carried out by Tonkin & Taylor for the Burwood Hospital Campus. A subsequent report titled *Burwood Hospital Post Earthquake Geotechnical Assessment* was issued in June 2011 [4]. The geotechnical review concluded that the settlement and damage to building foundations and slabs on the Burwood Hospital Campus was likely due to the liquefaction of underlying soil layers. It is believed that excessive pore water pressures have been dissipated and that further settlement is not expected to occur, unless another significant earthquake event was to occur.

It is estimated that parts of the building have settled a total of 80 100mm overall with a differential settlement of approximately 83mm noted across the elevated ground floor slab. The most severe settlement has occurred at the north west corner of the building although there is no visible evidence of this besides the survey. The settlement is shown below in Figure 3 1, a more detailed plan can be found in Appendix C.

Figure 3-1: Survey of building settlement

Based up the geotechnical report provided by Tonkin & Taylor [4] the potential for future total and differential settlements at the building site varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

3.5 LEVEL SURVEY & VERTICALITY STUDY

A detailed survey of the ground floor levels at the Site Maintenance Block was conducted by Fox & Associates and issued on 31stth October, 2011 [5]. The survey indicates a differential settlement of approximately 82mm across the Site Maintenance building, with the most significant differential settlements occurring at the north west corner of the building. The worst case permanent slope in the slab on grade, based upon this survey, is a drop of approximately 82mm over a 17 metre length resulting in a slope in the ground floor slab of approximately 0.5% or 1:200 which exceeds the typical acceptable range for non residential concrete or concrete masonry buildings.

The differential settlement noted for the rest of the building is minimal and inside the acceptable range for standard occupancy buildings.

Figure 3-2: Evidence of Ground Fissures and Liquefaction

Settlement of the external pavement and coal store ramp has occurred along the western wall of the Boiler House due to ground fissures and liquefaction. The building does not appear to have settled in this area. This could be attributed to the deep foundations due to the presence of the undercroft. The settlement in the pavement is approximately 150mm from its original location.

The coal store crib wall was damage during the February 22nd event. Cribs were damage and the ramp had settled up to 140mm as per the Earthquake damage assessment report issued on the 27th of April [23]. Details were issued for rectification on the 4th of May 2011 [24] and 20th June 2011 [25]. These crib wall repairs were carried out along with re grading of the ramp to remove the step

3.6 SUMMARY OF BUILDING DAMAGE

The following is a summary of the observations made for the building, and our conclusions as to their condition and seismic load resisting capacity. These observations do not specifically distinguish between damage caused by the Darfield Earthquake, the Lyttelton Earthquake or any significant aftershocks, such as those that occurred on the 13th June 2011, .the 23rd December 2011 or the 2nd January 2012. Despite not being able to specifically distinguish when individual damage observed occurred, it is believed that the majority of the damaged can be linked to the February 22nd event.

The majority of the builder appears to have performed well considering the construction, age of the building and the seismic action experienced at the site. The damage has predominately been to the concrete and masonry elements of the building in the way of vertical and diagonal cracking. The most sever case of a cracking is in the concrete wall to the substation which has opened up approximately 5mm. There are hairline to 1mm cracks located throughout the Maintenance building, Boiler house and Plantroom.

There is approximately a 10 15mm separation between the top of the Plantroom wall in the North East corner and adjacent Maintenance building wall. There is also damage to the plantroom roof connection in this area.

The coal store crib wall has been damaged, with spalling of concrete present on the southern wall at the building/concrete infill junction. The steel dowels are now exposed although there does not appear to be any additional settlement.

The observed damage sustained by the building as a whole would be considered minor as the reduction in lateral capacity of the building caused by cracking in the concrete and masonry walls is not significant.

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. Secondary elements, such as windows and fittings, have not generally been reviewed.

3.7 ADDITIONAL INVESTIGATIONS REQUIRED

Further investigations are required in order to understand the full extent of damage to the Maintenance and Boiler house. An exhaustive survey of the full extent of cracking to the building has yet to be completed due to access and height issues, as well as ceiling and wall finishes. The additional investigations have been divided into investigations that should be completed as a priority for further assessment and investigations that can take place as the repairs are undertaken.

3.7.1 Investigations Required For Further Assessment

- A complete investigation of cracking damage to concrete elements and masonry panels is required. In particular confirmation of crack widths in the upper levels of the building exterior is necessary. It crack widths are greater than 0.5 mm it may be necessary to complete material tests on the reinforcement to determine residual capacities.

- The extent or presence of reinforcement in the masonry infill panels needs to be confirmed for the Boiler House. This should be carried out on all panels on the east, west and south elevation to both the interior and exterior faces. Both the lower and upper panels require scanning.

- Damage to the walls of both north and south stair cases in the Maintenance building requires further inspection. Wall linings need to be removed to expose the structural materials and extent of cracking.

3.7.2 Investigations to be Completed During Building Repair

- Where significant cracks in the concrete vertical elements of the Boiler house Plantroom have been noted near the infill panel interface, investigation is needed to confirm if reinforcement crosses the crack

- Roof diaphragm connection of the Plantroom to the Boiler house and Maintenance building needs to be confirmed via additional inspection.

3.8 POST-EARTHQUAKE BUILDING CAPACITY

Based upon our investigations to date, we do not consider the Maintenance Building, Boiler House or Plantroom to have any significant reduction to the overall gravity load resistance of the structure. Nor does the damage noted to date appear to have any significant reduction to the lateral load capacity of the concrete and steel portions of the building (Sections 2 & 3).

The damage observed will require repair to restore the strength, stiffness, durability and performance of the individual structural components. The repair work is outlined in Section 4. Following the recommended repair of the structural damage, the lateral load resisting performance of the structure will be restored to approximately pre earthquakes levels (see Section 2.4).

In its pre earthquake and post earthquake condition, areas of the Maintenance Building and Boiler House have been assessed at 36% and 37% of the load imposed by the current loading standards DBE.

The low DBE% is concerning as the concrete masonry portions may fail in a brittle manner. The primary risk to the building occupants is the collapse of the masonry walls in the building. As the assessed capacity is very close to the arbitrary 33% definition of earthquake prone buildings, consideration may be given to temporarily strengthening to raise the building capacity, as outlined in section 5.

4. OBSERVED DAMAGE AND REQUIRED REPAIRS

This section covers the damage noted during our detailed assessment of the building. Note that our observations have been restricted to structural aspects of the building 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.

Table 4 1 provides a photographic summary of the observed damage and typical repairs required for the Boiler House and Maintenance building. Table 3 1 should be read in conjunction with Appendix A – Record of Observation and Appendix B – Reference Plans. The Repair Specification [2] referred to in Table 4 1 has been issued separately.

In general, the aim of the repair work indicated in this section is to restore the structure to its pre earthquake state, as far as practicable, while maintaining the utility of the building. The repairs presented attempt to address the loss of strength, stiffness and durability of the structural elements due to the damage noted.

It should be noted that more damage may be identified during the repair works and (if required) additional repair details will be specified accordingly.

Further recommendations for improvement to the buildings seismic performance, and to achieve improvements in capacity beyond 67% DBE have been included in Section 5.

Table 4-1: Photographs of observed damage and repairs required

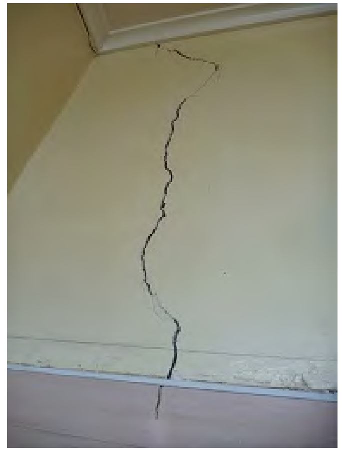
Damaged Item & Location	Damage	Recommendations	Example Photograph
1.0 Boiler House			
1.01. Concrete Pier, East Elevation	Cracking up to 5mm in concrete pier below high level windows on East elevation. Roughly 1800mm from corner.	Further investigation is required to determine if there is reinforcement crossing the crack, and if so how much remaining capacity the reinforcement has. The cracked/damaged corner concrete should be chipped off to expose the reinforcement for testing. Replacement concrete patch and repair to make good after.	
1.02. Concrete Pier, North Elevation	Cracking up to 2mm in concrete pier below high level windows on North elevation. Roughly 1000mm from corner.	Further investigation is required to determine if there is reinforcement crossing the crack, and if so how much remaining capacity the reinforcement has. The cracked/damaged corner concrete should be chipped off to expose the reinforcement for testing. Replacement concrete patch and repair to make good after.	
1.03. External Pavement, South Elevation	Localised settlement of bitumen pavement around chimney concrete footing, up to 100mm.	Demolish pavement and reinstate	

Damaged Item & Location	Damage	Recommendations	Example Photograph
1.04. External Pavement, West Elevation	Localised settlement of bitumen pavement near west elevation entry, south of coal store, up to 150mm.	Demolish pavement and reinstate	
1.05 Coal Store, South Crib Wall	Concrete repair to ramp crib wall has been damaged and concrete has spalled at dowel connection to wall.	Patch repair concrete with primer and suitable material.	
1.06. Coal Store, North Wall	Hairline diagonal cracking to wall	No action required	
1.07. Coal Store, South Wall	Cracking up to 3mm (estimated) on south wall of coal store, near joint to Boiler house	Further investigation is required to determine if there is reinforcement crossing the crack, and if so how much remaining capacity the reinforcement has. The cracked/damaged corner concrete should be chipped off to expose the reinforcement for testing. Replacement concrete patch and repair to make good after.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
1.08. South Elevation, Concrete Beam	Cracking at column/beam junction in top right hand corner of building.	Further investigation is recommended to confirm crack widths. If crack widths exceed 0.5mm reinforcement testing should be carried out to determine remaining capacity. If crack widths are less than 0.5mm then epoxy fill.	
1.09. Garbage Enclosure, South Wall	Cracking near internal floor level in bottom left hand corner	Further investigation is recommended to confirm crack widths. If crack widths exceed 0.5mm reinforcement testing should be carried out to determine remaining capacity. If crack widths are less than 0.5mm then epoxy fill.	
1.10. Garbage Enclosure, North Wall	Vertical crack below window on north wall, approximately 0.5mm, extends through sill down to ground floor level.	Cracks should be epoxy filled.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
1.11. Column, East Wall internal	Cracking and potential for spalling of concrete encased steel column near hopper support frame beam.	Further investigation required (ref. S.R. 002 13/7/12)	
1.12. Concrete block walls internal and external	Confirmation of reinforcement content in walls	In order to confirm the construction of the walls and repair/upgrade further reinforcement scans recommended on both inside and outside faces of walls on east/west/south elevations. Both lower (ground level) and upper wall panels to be scanned.	
2.0 Maintenance Building			

Damaged Item & Location	Damage	Recommendations	Example Photograph
2.01. North Stair Concrete Walls	Diagonal, vertical and horizontal cracking throughout internal stairwell walls, up to 1mm	Further investigation required. Remove wall linings to expose concrete and confirm crack extent. Reinforcement capacity testing likely required to determine remaining capacity.	
2.02. South Stair Concrete Walls	Diagonal, vertical and horizontal cracking throughout internal stairwell walls, up to 1mm	Further investigation required. Remove wall linings to expose concrete and confirm crack extent. Reinforcement capacity testing likely required to determine remaining capacity.	
3.0 Plantroom			

Damaged Item & Location	Damage	Recommendations	Example Photograph
3.01. Internal masonry wall	Large crack, up to 10mm in internal masonry wall running in NS direction. Cracking is located towards the substation and is reflected through both sides of the wall.	Damage is excessive to repair locally. Recommend removal & replacement of this portion of the wall and replacement with doweled in block or concrete wall.	
3.02. Exterior wall, North East Corner	Separation between NE corner and Maintenance building, gap up to 15mm at top of wall. Damage also sustained to the roof/building connection.	Further investigation required. Connection of roof diaphragm needs to be assessed for sustained damage and if adequate seating of roof present. Seismic separation may be necessary.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
			
3.03. Concrete Pier, East Elevation	Vertical crack to concrete pier #2 on East elevation, up to 2mm in width	Chip off damaged cover concrete and make good with patch concrete repair.	
3.04. Masonry Wall, West Elevation	Horizontal crack between the brickwork and concrete substation junction on the West wall, < 0.2mm	No Action required	

4.1 DISCUSSION ON DIFFERENTIAL SETTLEMENT REMEDIATION

The level survey, completed by Fox & Associates, has indicated differential ground settlement of approximately 83mm across the length of the ground floor slab in the Maintenance Building. The worst differential settlement is concentrated at the east end of the building (see Appendix C for complete level assessment) with a grade of 1:200 which is outside the typical acceptable range and will need to be addressed to restore the function of the building. This can be achieved by re levelling.

If re levelling is to occur, the north west corner of the building would be proposed to be lifted up to the highest point of the building which is located roughly in the south east corner of the building. For the extent of the proposed re levelling see Figure 4 1 below. This would address the slope in the ground floor slab of the building as shown in Appendix C.

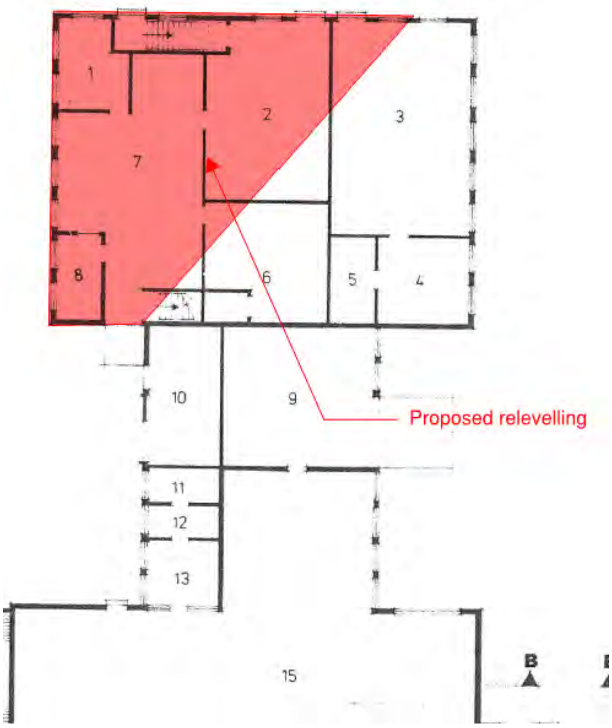


Figure 4-1: Foundation Plan – Damage Repairs

It needs to be ensured that re levelling works by use of underpinning grout or engineered resin do not create any "hard points" under the building. If "hard points" are created during the re levelling process the potential for future differential settlements can be increased. If this were to occur it would reduce the capacity of the building going forward.

Based upon the information provided by Tonkin & Taylor, the soil profile under the Site Maintenance Building (medium dense sand overlying dense sand) lends itself to localized lifting through underpinning grout or engineered resin techniques, and should not create any undesirable "hard points" as described above.

While indications are that the building may be suitably re levelled as described above, this will need to be verified by qualified sub contractors in conjunction with the geotechnical consultant.

It should be noted that re levelling of the building should not be expected to increase the seismic performance of the building or reduce the potential for future differential settlements. Instead the options presented are designed to re level the building without making the future performance of the building any worse than it was prior to the earthquakes. To improve the future performance of the building, and reduce the potential for future differential settlements, would likely require the entire footprint of the building to be either piled or the ground under all the sub floor wall footings, service tunnels and the partial basement improved. Further geotechnical investigations would be required into the type and depth of ground if improvement is required.

Based up the geotechnical report provided by Tonkin & Taylor [11] the potential for future total and differential settlements at the building site varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

During the re levelling process there is also the risk that addition damage could occur to the building linings, exterior block veneer, etc. Appropriate contingencies should be provided

5. STRENGTHENING RECOMMENDED

As noted in Section 2, Pre-Earthquake Building Condition, and Section 3, Post-Earthquake Building Condition, the lateral load resisting capacity of each section of the building has been assessed as a percentage of the loads imposed by the Design Basis Earthquake (DBE).

The Maintenance Building ground floor lateral load-resisting system in both pre and post-earthquake conditions achieved 55% and 36% DBE rating in the North-South and East-West directions respectively. The Boiler House achieved 58% in the North-South and 37% East-West directions in both pre and post-earthquake conditions. The limiting factors have been the ability for the concrete columns and walls to resist the shear and bending loads.

At present a Critical Structural Weakness in the Maintenance building has been identified due to the stair cases being rigidly connected to both the ground and first floors. The presence of the stair flights may act as a compression strut between the floors as the move relative one-another. If the movement applied by the first floor is excessive the stairs may be damaged, thus preventing occupants from evacuating safely.

Further to the repairs noted in section 4, additional recommended strengthening to extend the capacity of the buildings towards, or above, 67% DBE have been included below.

5.1 REMEDIATION OF CRITICAL STRUCTURAL WEAKNESSES

Maintenance Building – The cast in-situ stairs on the north and south walls should be modified to allow the relative movement of the first floor to the ground floor. This could be achieved by saw-cutting a slot in the intermediate landings and introducing a new upstand seating support wall under the landing.

Boiler House – At present our inspections indicate that the masonry infill panels may have a minimal amount of reinforcement. Further reinforcement scans of the internal and external faces of the east, west and south infill panel elevations, is recommended. These scans should be carried out on the lower and upper panels. The results from this may negate the need for remediation works for the out-of-plane failure of the infills. However if the reinforcement is not adequate then restraint beams attached to the inside and outside faces, at the mid-height of the lower panels and top row of blocks on the upper panels will need to be installed. Once scan results are received, instruction for this works will be provided.

5.2 STRENGTHENING WORKS TO ACHIEVE 67% DBE

Maintenance Building – In order to bring the Maintenance building to 67% capacity, it is proposed that additional shear walls are provided to both ground and first floors in the north-south and east-west directions.

Figures 5-3 and 5-4 represent indicative layouts for the new shear walls. These walls would be 200mm thick reinforced concrete, tied into adjacent beams, columns and footings with epoxied reinforcement dowels.

Boiler House – In order to achieve a minimum 67% lateral load capacity in the Boiler house, it is proposed additional shear walls be installed to all four sides of the building. Masonry in-fills and windows should be removed from the proposed areas as shown in Figure 5-5. 200mm thick in-situ reinforced concrete walls should be poured in place of the removed masonry walls. The concrete walls will be tied into the adjacent beams, columns and footings with epoxied reinforcement dowels.

With these installations in place, most masonry in-fills will be replaced with concrete. The remaining in-fills should be demolished and replaced with concrete walls or alternatively light-weight construction.

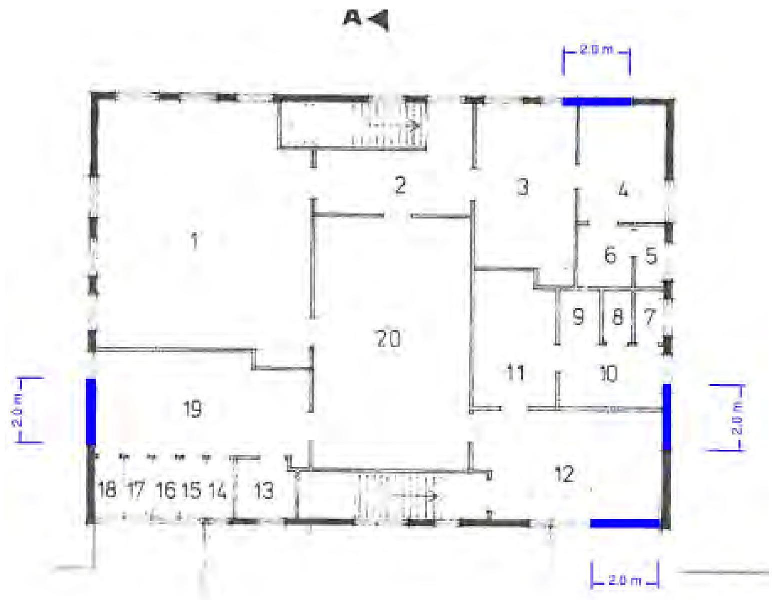


Figure 5-1: Maintenance Building First Floor Plan – Proposed 67% Strengthening

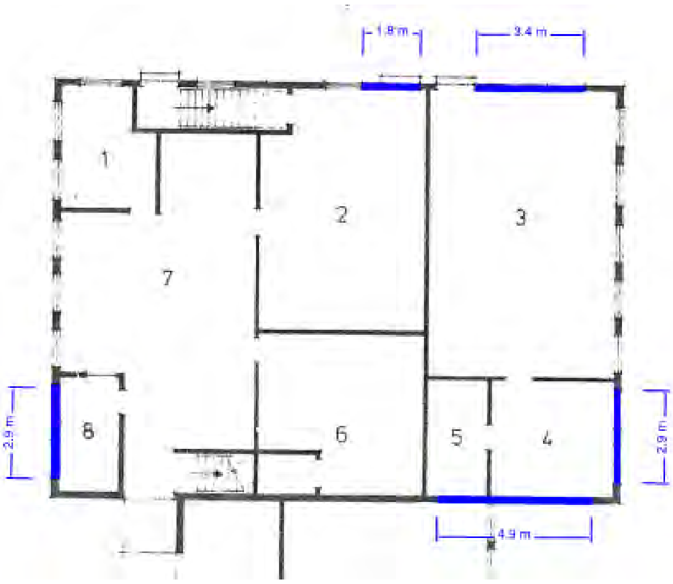


Figure 5-2: Maintenance Building Ground Floor Plan – Proposed 67% Strengthening

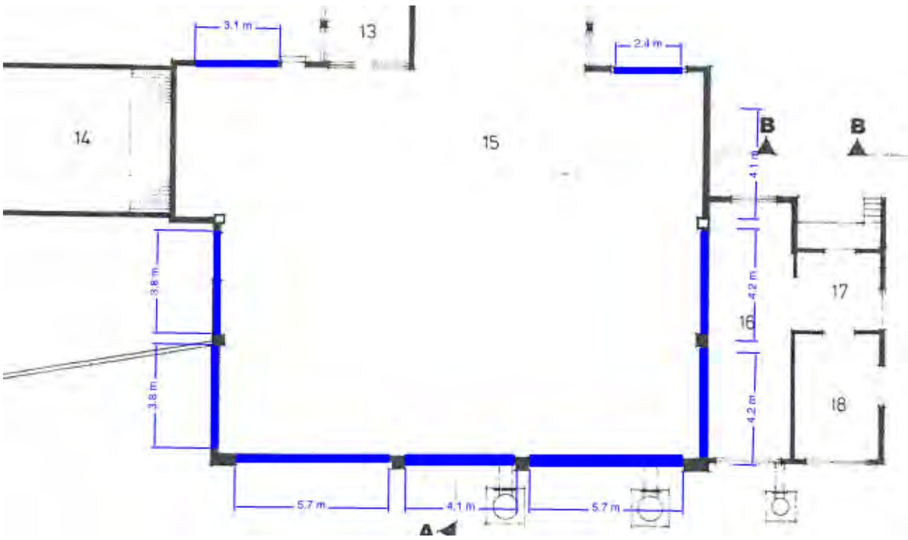


Figure 5-3: Boiler House – Proposed 67% Strengthening

Plantroom – The deflections which would be experienced during the design seismic event have been determined to be excessive and similarly the resulting strength demands on the portal frames. Is it likely that external finishes such as windows and walls may collapse due to these deflections, but it is unlikely that the roof would collapse. In order to reduce deflections and limit the demand on the portral frames, roof and wall bracing may be installed as shown in Figure 5-7. This concept may also be adopted in order to strengthen the structure in lieu of the fly bracing installations shown above, as it will tie the roof into the common concrete walls of the Maintenance Building and Boiler House.

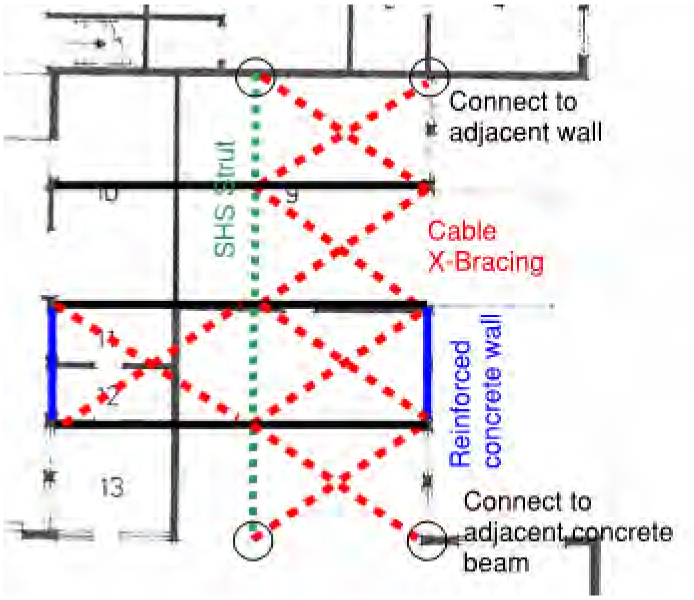


Figure 5-4: Plantroom – 67% Strengthening Recommended

6. REFERENCES

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21. *CDHB – Burwood Hospital – Post Earthquake Rapid Structural Assessment following 5.8 and 6.0 Magnitude Earthquakes, 106186.03 Site Report* 8, Holmes Consulting Group, 24 December 2011

22. *CDHB – Burwood Hospital – Post Earthquake Rapid Structural Assessment following 2nd January 5.5 Magnitude Earthquakes, 106186.03 Site Report 9*, Holmes Consulting Group, 9 January 2012

23. *Earthquake repair assessment, 106186.25 Site Report 1*, Holmes Consulting Group, 27 April 2011

24. *Burwood Hospital Boiler Ramp repair, 106186.25*, Holmes Consulting Group, 4 May 2011

25. *Boiler Ramp Northside Concrete Capping Detail, 106186.25 Site Report 2*, Holmes Consulting Group, 20 June 2011

APPENDIX A

Record of Observations

APPENDIX A – RECORD OF OBSERVATIONS & REPAIRS
SITE MAINTENANCE, BOILER HOUSE AND PLANTROOM
Inspection date: May 2012

KEY	
N	No repair required
Y	Repair required
F	Further investigation required
C	Repair complete

G - GROUND FLOOR
F - FIRST FLOOR
R - ROOF

Level	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
MAINTENANCE BUILDING						
G, F	North Stairwell	Concrete Wall	Horizontal, Vertical and diagonal cracking throughout stairwell. Up to 0.5mm	Y	Epoxy inject cracks	121-124
G, F	South Stairwell	Concrete Wall	Horizontal, Vertical and diagonal cracking throughout stairwell. Up to 1.0mm	Y	Epoxy inject cracks	125-130
BOILER HOUSE						
G	East Elevation, external	Concrete Pier	Vertical crack in concrete pier up to 5mm (estimate as out of reach to measure). Cracking is approximately 1800mm in from the North elevation and half way up the wall below the high-level windows. Looks as though spalling will occur	Y	Epoxy inject	106-108

Level	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	North Elevation, external	Concrete Pier	Vertcal crack in concrete pier up to 2mm (estimate as out of reach to measure). Cracking is approximately 1000mm in from east elevation and half way up wall below high level windows.	**Y**	Epoxy inject	404-405
G	West Elevation, external	Bitumen Pavement	Liquifaction has caused pavement to settle up to 150mm in areas. Large step between pavement and western door entry.	**Y**	Pavement should be demolished and relaid with similar. Dowel connections to building or support on building footings bay prevent future settlement	216-217
G	Coal Store, South	Concrete Crib Wall	Previous repairs to concrete crib wall have been damaged. Concrete has spalled where dowelled into coal store wall.	**Y**	Face of concrete should be primed and new concrete infill should be reset	218-219
G	South Elevation, external	Pavement and chimney pad	Settlement of pavement and possible movement of chimney pad footing due to liquifaction. Pavement has crtacked around chimney pad	**N**		114
G	North Elevation, external	Wall, Coal Store	Hairline diagonal cracks throughout wall	**N**		227
G	South Elevation, external	Wall, Coal Store	Vertical crack in masonry wall up to 3mm (estimate as out of reach to measure). Approximately 4000mm up the wall near the coal store/boiler house joint.	**Y**	Epoxy inject	422-423
R	South Elevation, external	Concrete beam	High-level concrete beam cracking around column location in SE corner. Cracking up to 0.5mm (estimated as out of reach to measure).	**F**	Closer inspection should be carried out. Possible epoxy fill may be required.	424

Level	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	South Elevation, external	Concrete wall, garbage enclosure	Diagonal cracks at bottom corner of window, up to 0.3mm.	**Y**	Epoxy inject	425
G	North Elevation, external	Block wall, garbage enclosure	Vertical crack up to 1mm stemming from bottom of window, through sill and down to floor level.	**Y**	Epoxy inject	407-410
G	North Elevation, external	Block wall, garbage enclosure	Hairline diagonal cracks under window. Has caused paint and render to crack and dislodge	**N**		411
PLANTROOM						
G	East Elevation, external	Blockwall	NE corner of plantroom has moved and caused up to 15mm separation between itself and adjacent Maintenance building wall.	**Y**		414-415
G	East Elevation, external	Roof	Where separation has occure,d the roof framing has also been damaged and separated from wall.	**F**	Further inspection required. Roof should be removed locally and connection into maintenance building inspected for damage	413
G	East Elevation, external	Wall	General separation between walls and window/door framings	**N**		
G	East Elevation, external	Concrete pier	Vertical crack to second concrete pier. Crack up to 2mm stemming from bottom of window corner.	**Y**	Epoxy inject	412
G	West Elevation, external	Wall	Horizontal hairline crack along wall above substation garage door. Likealy a result of the junction between the concrete substation and infill blockwork.	**N**		417

Level	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	Internal wall	Wall	Infill north-south wall has large vertical/diagonal crack up to 10mm. Visible in both sides of the wall, close to substation concrete wall	**Y**	Epoxy fill crack	201, 202, 210, 211

APPENDIX A – RECORD OF OBSERVATIONS & REPAIRS
SITE MAINTENANCE, BOILER HOUSE AND PLANTROOM
Inspection date: May 2012

KEY	
N	No repair required
Y	Repair required
F	Further investigation required
C	Repair complete

G - GROUND FLOOR
F - FIRST FLOOR
R - ROOF

Level	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
MAINTENANCE BUILDING						
G, F	North Stairwell	Concrete Wall	Horizontal, Vertical and diagonal cracking throughout stairwell. Up to 1.0mm	**F**	Remove wall linings to expose concrete and confirm cracking. Reinforcement testing likely required	121-124
G, F	South Stairwell	Concrete Wall	Horizontal, Vertical and diagonal cracking throughout stairwell. Up to 1.0mm	**F**	Remove wall linings to expose concrete and confirm cracking. Reinforcement testing likely required	125-130
BOILER HOUSE						
G	East Elevation, external	Concrete Pier	Vertical crack in concrete pier up to 5mm (estimate as out of reach to measure). Cracking is approximately 1800mm in from the North elevation and half way up the wall below the high-level windows. Looks as though spalling will occur	**Y**	Chip off damaged concrete edge/corner. Confirm if reinforcement crosses crack - if so reinforcement testing required. Otherwise patch repair and make good.	106-108

Level	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	North Elevation, external	Concrete Pier	Vertcal crack in concrete pier up to 2mm (estimate as out of reach to measure). Cracking is approximately 1000mm in from east elevation and half way up wall below high level windows.	**Y**	Chip off damaged concrete edge/corner. Confirm if reinforcement crosses crack - if so reinforcement testing required. Otherwise patch repair and make good.	404-405
G	West Elevation, external	Bitumen Pavement	Liquifaction has caused pavement to settle up to 150mm in areas. Large step between pavement and western door entry.	**Y**	Pavement should be demolished and relaid with similar. Dowel connections to building or support on building footings bay to prevent future settlement	216-217
G	Coal Store, South	Concrete Crib Wall	Previous repairs to concrete crib wall have been damaged. Concrete has spalled where dowelled into coal store wall.	**Y**	Face of concrete should be primed and new concrete infill should be reset	218-219
G	South Elevation, external	Pavement and chimney pad	Settlement of pavement and possible movement of chimney pad footing due to liquifaction. Pavement has crtacked around chimney pad	**N**		114
G	North Elevation, external	Wall, Coal Store	Hairline diagonal cracks throughout wall	**N**		227
G	South Elevation, external	Wall, Coal Store	Vertical crack in masonry wall up to 3mm (estimate as out of reach to measure). Approximately 4000mm up the wall near the coal store/boiler house joint.	**F**	Confirm if reinforcement crosses crack - if so, reinforcement testing required. Patch repair and make good and/or epoxy inject.	422-423
R	South Elevation, external	Concrete beam	High-level concrete beam cracking around column location in SE corner. Cracking up to 0.5mm (estimated as out of reach to measure).	**F**	Confirm if cracks 0.5mm or greater - if so, reinforcement testing may be required. Patch repair and make good and/or epoxy inject.	424

Level	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	South Elevation, external	Concrete wall, garbage enclosure	Diagonal cracks at bottom corner of window, up to 0.3mm.	**Y**	Epoxy inject	425
G	North Elevation, external	Block wall, garbage enclosure	Vertical crack up to 1mm stemming from bottom of window, through sill and down to floor level.	**Y**	Epoxy inject	407-410
G	North Elevation, external	Block wall, garbage enclosure	Hairline diagonal cracks under window. Has caused paint and render to crack and dislodge	**Y**	Epoxy inject	411
G	East elevation, internal	Concrete encasement of steel column	Damage to concrete encasement of steel column has been noted at northern column adjacent to hopper framing	**F**	Ref. S.R. 002 issued 13/7/12 recommending removal of encasing concrete to inspect steel framing	501-511
PLANTROOM						
G	East Elevation, external	Blockwall	NE corner of plantroom has moved and caused up to 15mm separation between itself and adjacent Maintenance building wall.			414-415
G	East Elevation, external	Roof	Where separation has occurred the roof framing has also been damaged and separated from wall.	**F**	Further inspection required. Roof should be removed locally and connection into maintenance building inspected for damage.	413
G	East Elevation, external	Wall	General separation between walls and window/door framings	**N**		
G	East Elevation, external	Concrete pier	Vertical crack to second concrete pier. Crack up to 2mm stemming from bottom of window corner.	**Y**	Chip off damaged cover concrete and make-good with patch concrete repair	412

Level	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	West Elevation, external	Wall	Horizontal hairline crack along wall above substation garage door. Likealy a result of the junction between the concrete substation and infill blockwork.	**N**		417
G	Internal wall	Wall	Infill north-south wall has large vertical/diagonal crack up to 10mm. Visible in both sides of the wall, close to substation concrete wall	**Y**	Remove this length of wall and replace with reinforced block or concrete wall doweled into the existing wall edge.	201, 202, 210, 211

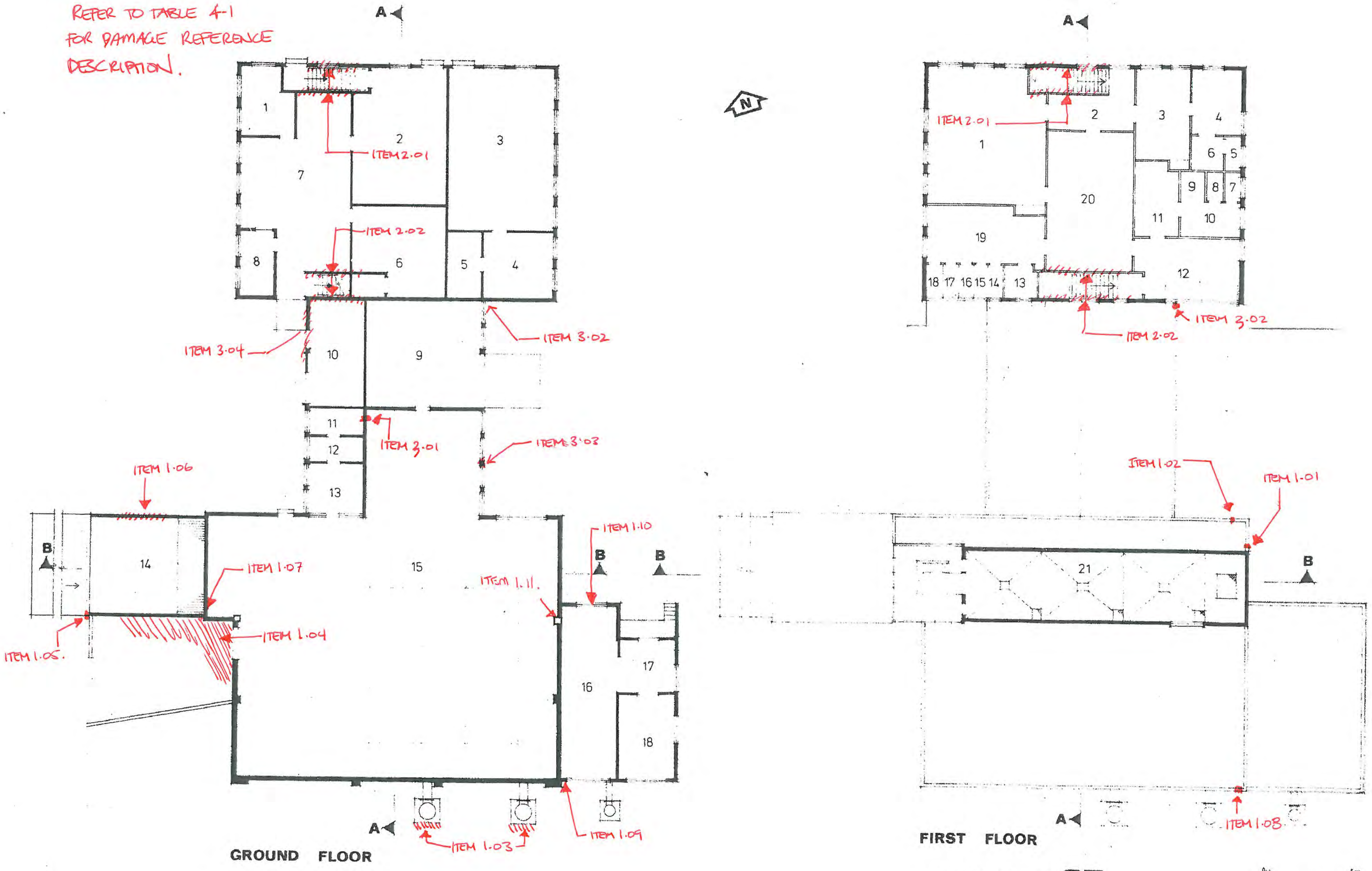
APPENDIX B

Location Plans

106186.25. SITE MAINTENANCE AND BOILER HOUSE.

DAMAGE MAP.

REFER TO TABLE 4-1 FOR DAMAGE REFERENCE DESCRIPTION.



BUILDING № 27

BOILERHOUSE

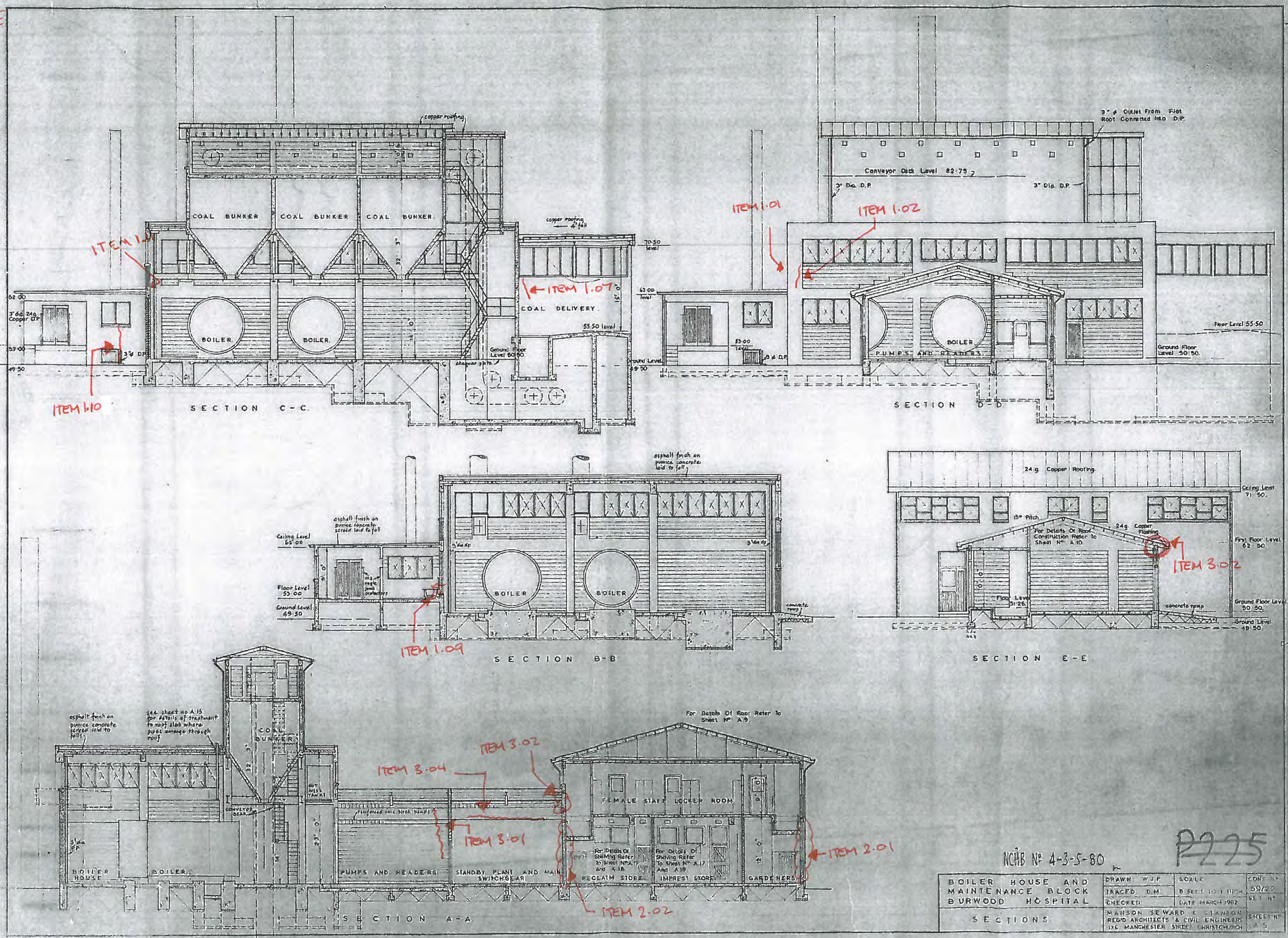
10686.25.

SITE MAINTENANCE AND BOILER HOUSE.

DAMAGE MAP.

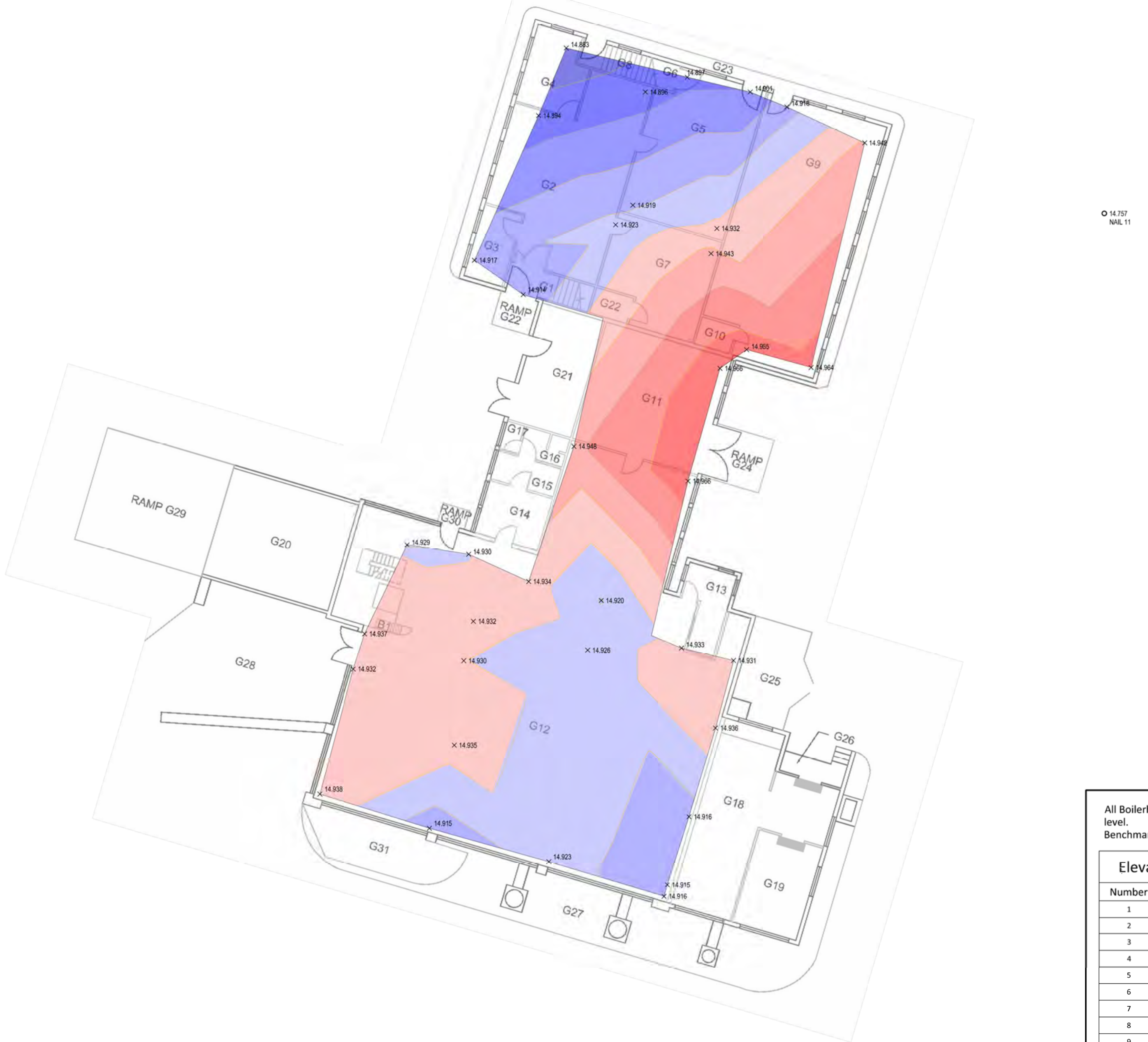
REFER TO TABLE 4-1 FOR DAMAGE REFERENCE DESCRIPTION

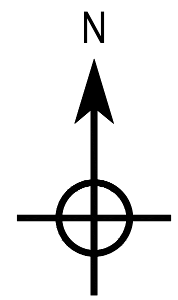
Burwood Hospital - ~~Nurses~~ Hostel -Misc



APPENDIX C

Level/Elevation Survey





All Boilerhouse and Site Maintenance level observations taken to concrete slab level.
Benchmark = Nail 8

Elevations Table - Boilerhouse and Site Maintenance

Number	Minimum Elevation	Maximum Elevation	Color
1	14.880	14.890	
2	14.890	14.900	
3	14.900	14.910	
4	14.910	14.920	
5	14.920	14.930	
6	14.930	14.940	
7	14.940	14.950	
8	14.950	14.960	
9	14.960	14.970	

A1 Original Size

No.	Revision	Approved	Date
G			
F			
E			
D	Wards 7 & 8, Surgical Block, Melrose Chairs & Corridor added	MJM	02/12/2011
C	Phys Med, Parafed, Food Services, BIRS, Boilerhouse, EDC added	MJM	24/11/2011
B	Nurses Hostel & Champion Centre, SOU elevations added	MJM	18/11/2011
A	Administration and Allan Bean Centre elevations added	MJM	31/10/2011

032005

Scale		1:100
Reduced	A3	1:200
Designed		----
Drawn		DHP
Checked		MJM
Date		02/12/2011

Burwood Elevations Survey

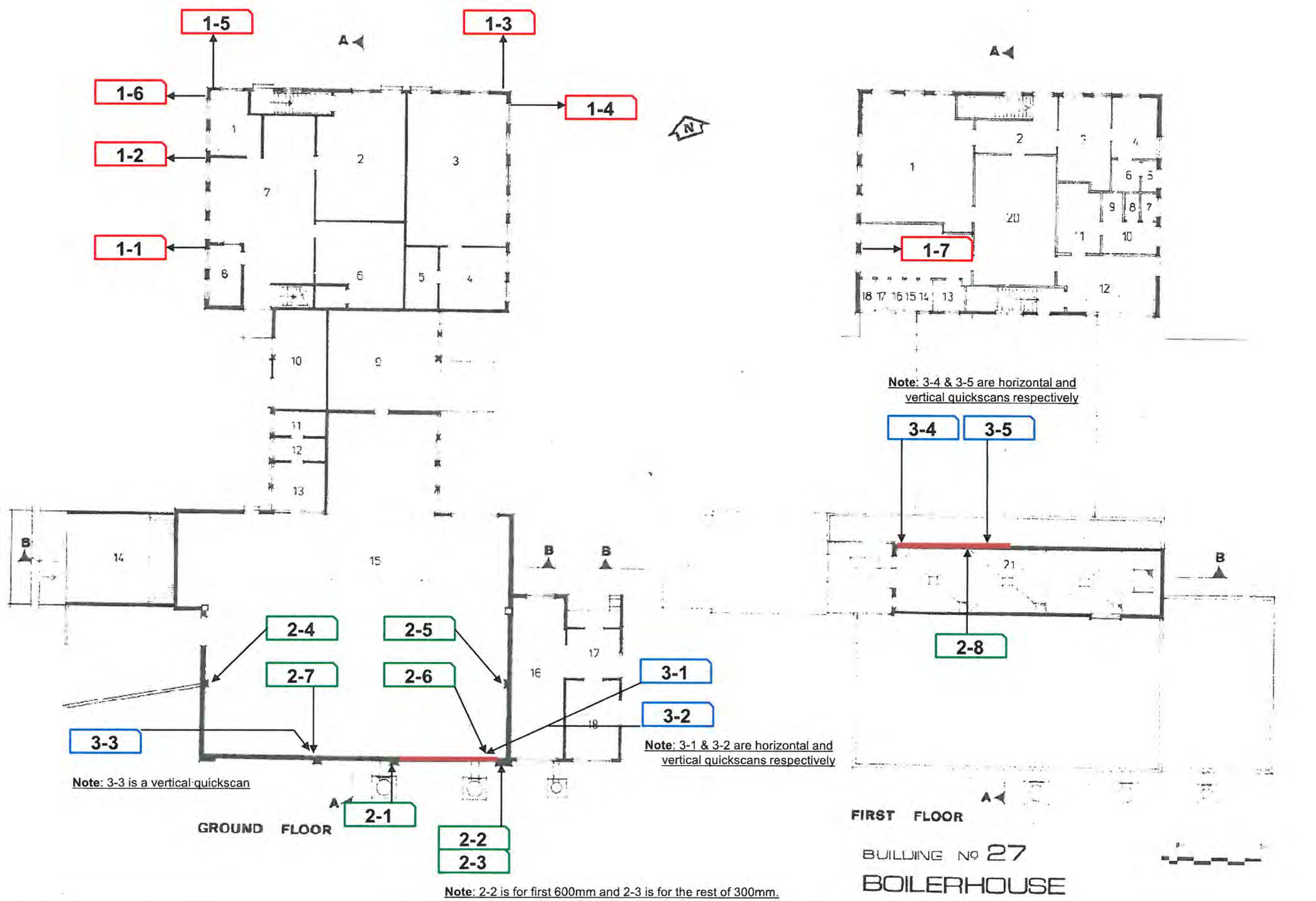
Boilerhouse and Site Maintenance

SURVEYORS
Fox & Associates
URBAN DESIGNERS
CONSULTANTS
LAND DEVELOPMENT
www.foxsurvey.co.nz
0800 FOX SURVEY
P.O.Box 13-390
CHRISTCHURCH

Job No. 2462C
Sheet No. 10 of 14
Rev. D

APPENDIX D

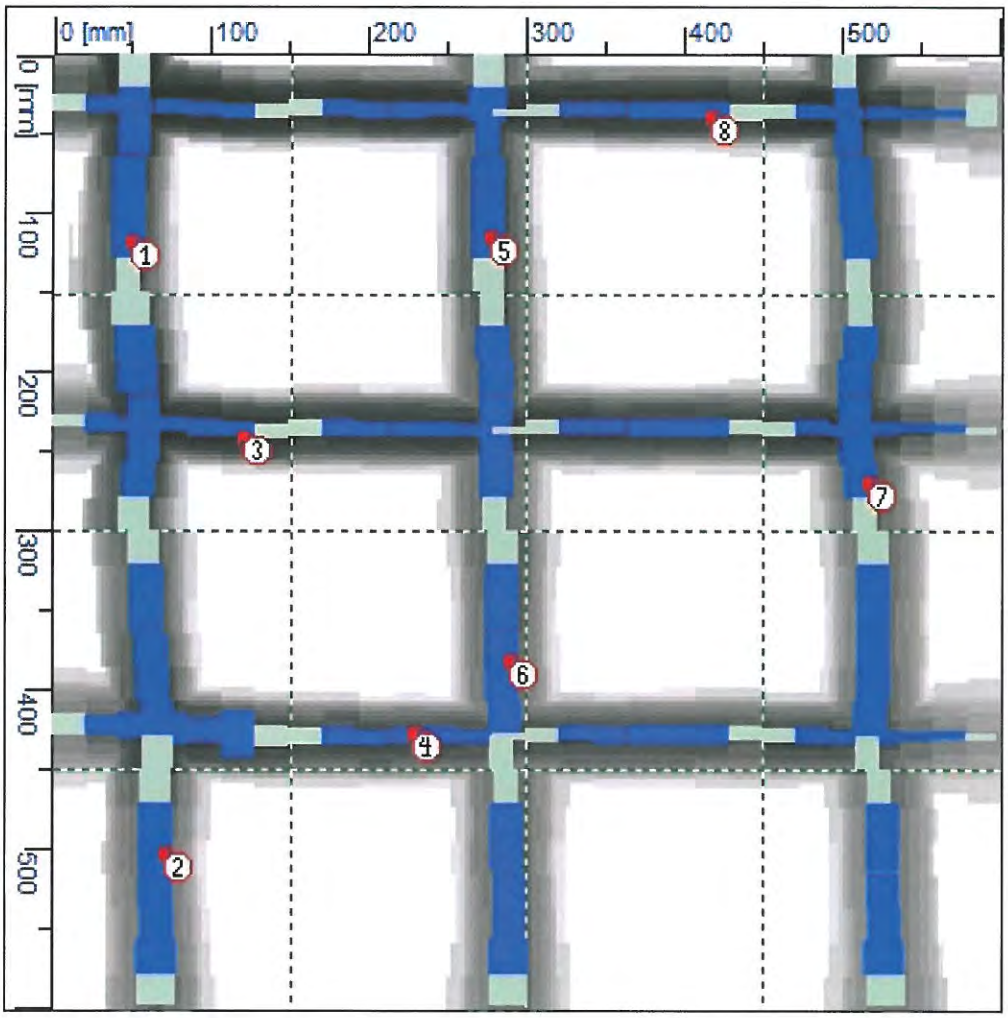
Reinforcement Survey


1-5
1-3
1-6
1-4
1-2
1-1
2-4
2-5
2-7
2-6
3-1
3-2
3-3
2-1
2-2
2-3
Note: 3-3 is a vertical quickscan
GROUND FLOOR
Note: 3-1 & 3-2 are horizontal and vertical quickscans respectively
Note: 2-2 is for first 600mm and 2-3 is for the rest of 300mm.
1-7
Note: 3-4 & 3-5 are horizontal and vertical quickscans respectively
3-4
3-5
2-8
FIRST FLOOR
BUILDING Nº 27
BOILERHOUSE

Imagescan: *FS001548.XFF*

1-1

Date / Time: 2012-05-23 11:39:09 SSN: 04806010 [mm]



Customer: Craig - Holmes Consulting Group

Location: 298 Burwood Road, Burwood, Christchurch

Operator: Frank Kang

Comment:

1-1

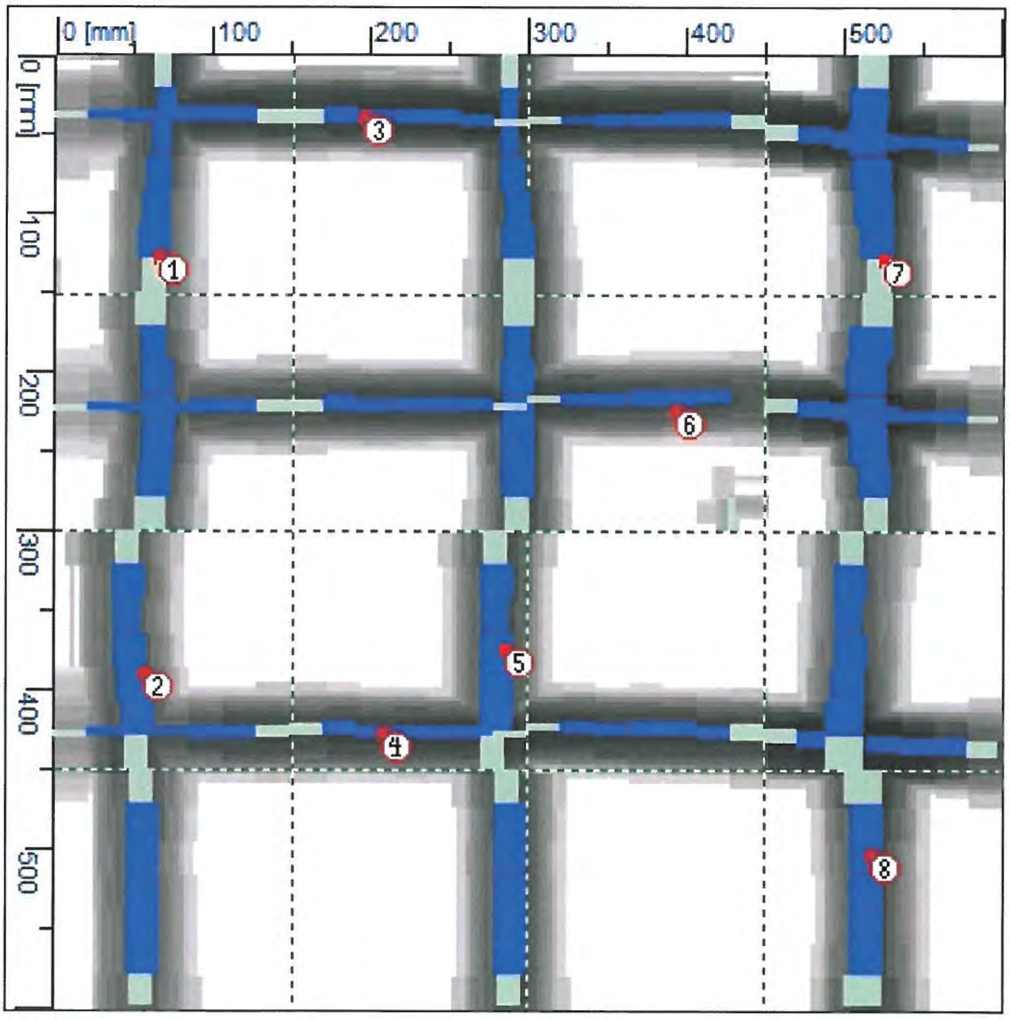
Marker	x: [mm]	y: [mm]	Comment:
1	47	115	Concrete cover = 38mm, Bar size = 20mm
2	67	501	Concrete cover = 46mm, Bar size = 20mm
3	118	237	Concrete cover = 25mm, Bar size = 10mm
4	225	425	Concrete cover = 31mm, Bar size = 10mm
5	274	111	Concrete cover = 37mm, Bar size = 20mm
6	286	379	Concrete cover = 42mm, Bar size = 20mm
7	514	266	Concrete cover = 43mm, Bar size = 20mm
8	415	36	Concrete cover = 27mm, Bar size = 12mm

Project: Burwood Hospital - Maintenance building

Imagescan: *FS001549.XFF*

1-2

Date / Time: 2012-05-23 11:42:46 SSN: 04806010 [mm]



Customer: Craig - Holmes Consulting Group

Location: 298 Burwood Road, Burwood, Christchurch

Operator: Frank Kang

Comment:

1-2

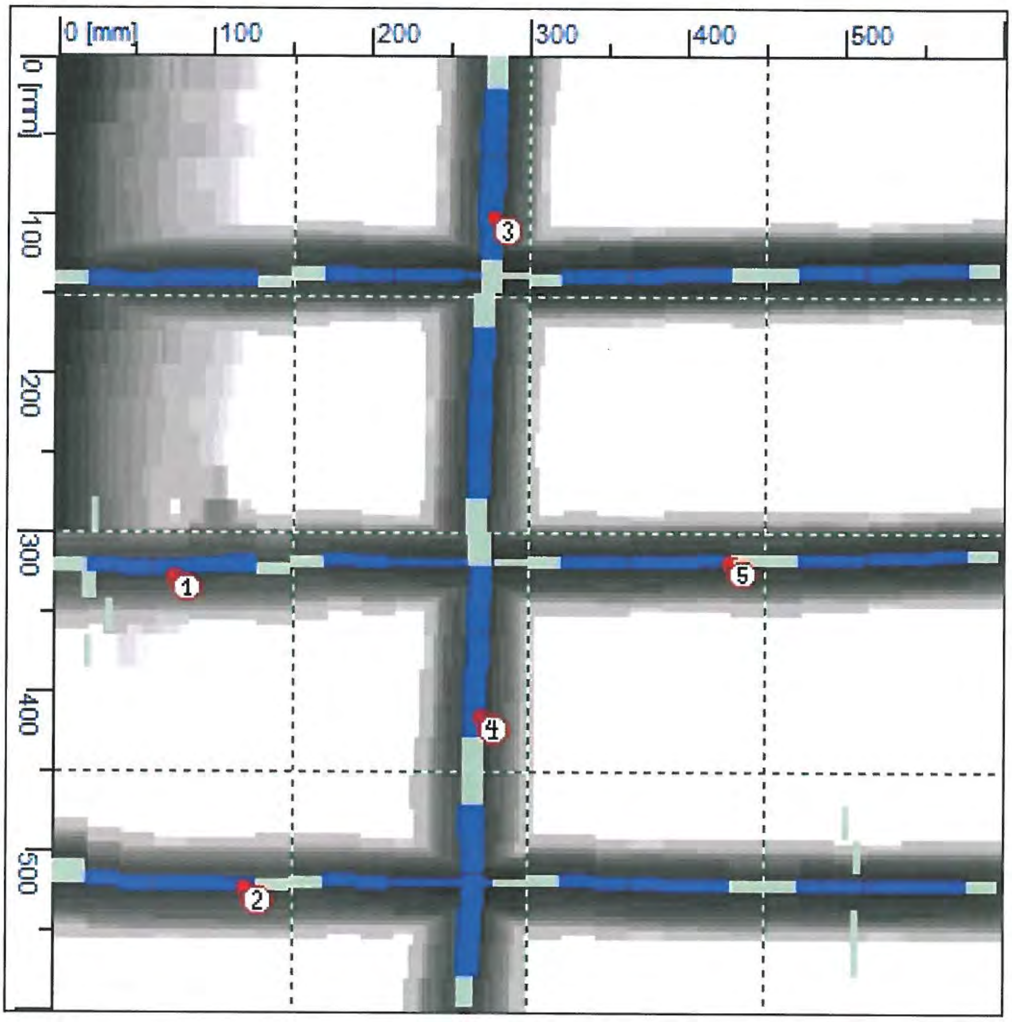
Marker	x: [mm]	y: [mm]	Comment:
1	63	123	Concrete cover = 48mm, Bar size = 20mm
2	53	388	Concrete cover = 45mm, Bar size = 20mm
3	193	36	Concrete cover = 33mm, Bar size = 12mm
4	205	425	Concrete cover = 33mm, Bar size = 8mm
5	282	371	Concrete cover = 45mm, Bar size = 20mm
6	392	221	Concrete cover = 37mm, Bar size = 10mm
7	523	125	Concrete cover = 44mm, Bar size = 20mm
8	516	500	

Project: Burwood Hospital - Maintenance building

Imagescan: *FS001550.XFF*

1-3

Date / Time: 2012-05-23 11:50:39 SSN: 04806010 [mm]



Customer: Craig - Holmes Consulting Group

Location: 298 Burwood Road, Burwood, Christchurch

Operator: Frank Kang

Comment:

1-3

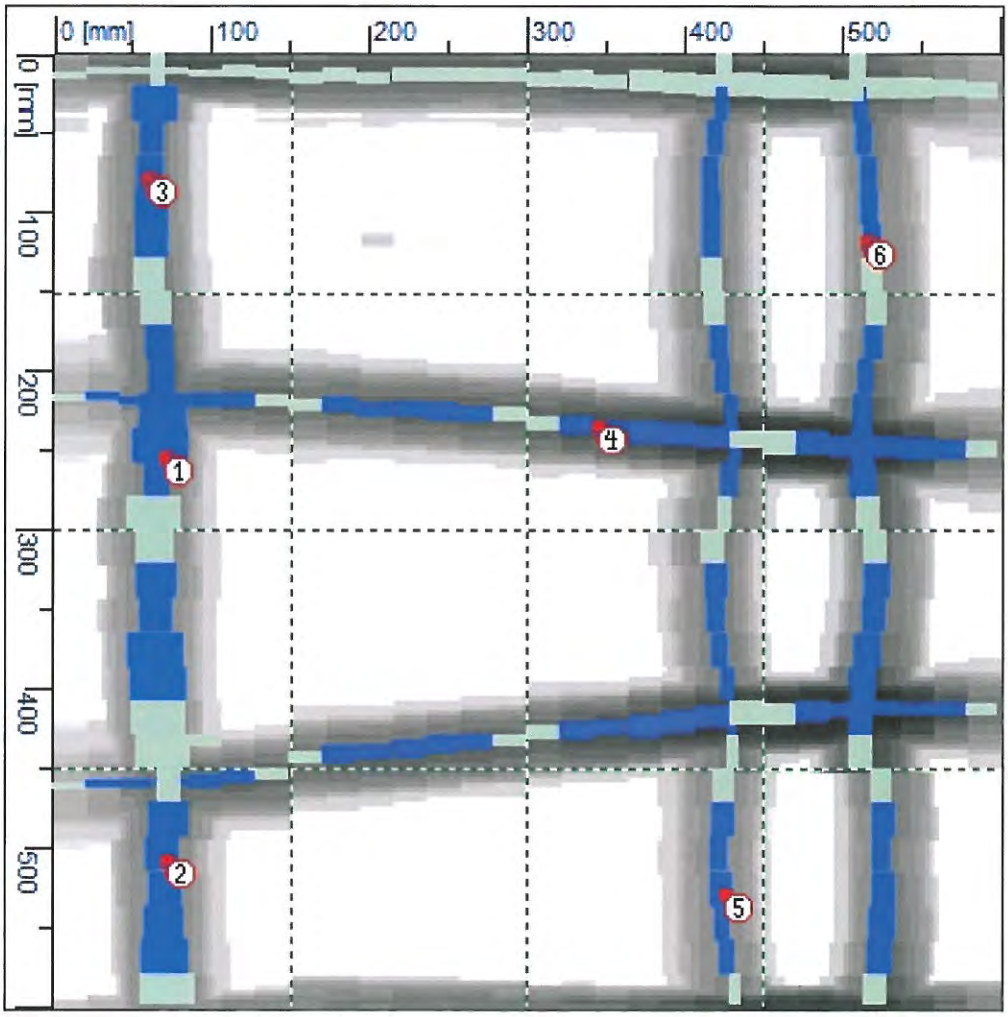
Marker	x: [mm]	y: [mm]	Comment:
1	71	323	Concrete cover = 43mm, Bar size = 10mm
2	118	521	Concrete cover = 41mm, Bar size = 8mm
3	274	97	Concrete cover = 46mm, Bar size = 16mm
4	266	412	Concrete cover = 45mm, Bar size = 14mm
5	425	315	

Project: Burwood Hospital - Maintenance building

Imagescan: *FS001551.XFF*

1-4

Date / Time: 2012-05-23 11:52:56 SSN: 04806010 [mm]



Customer: Craig - Holmes Consulting Group

Location: 298 Burwood Road, Burwood, Christchurch

Operator: Frank Kang

Comment:

1-4

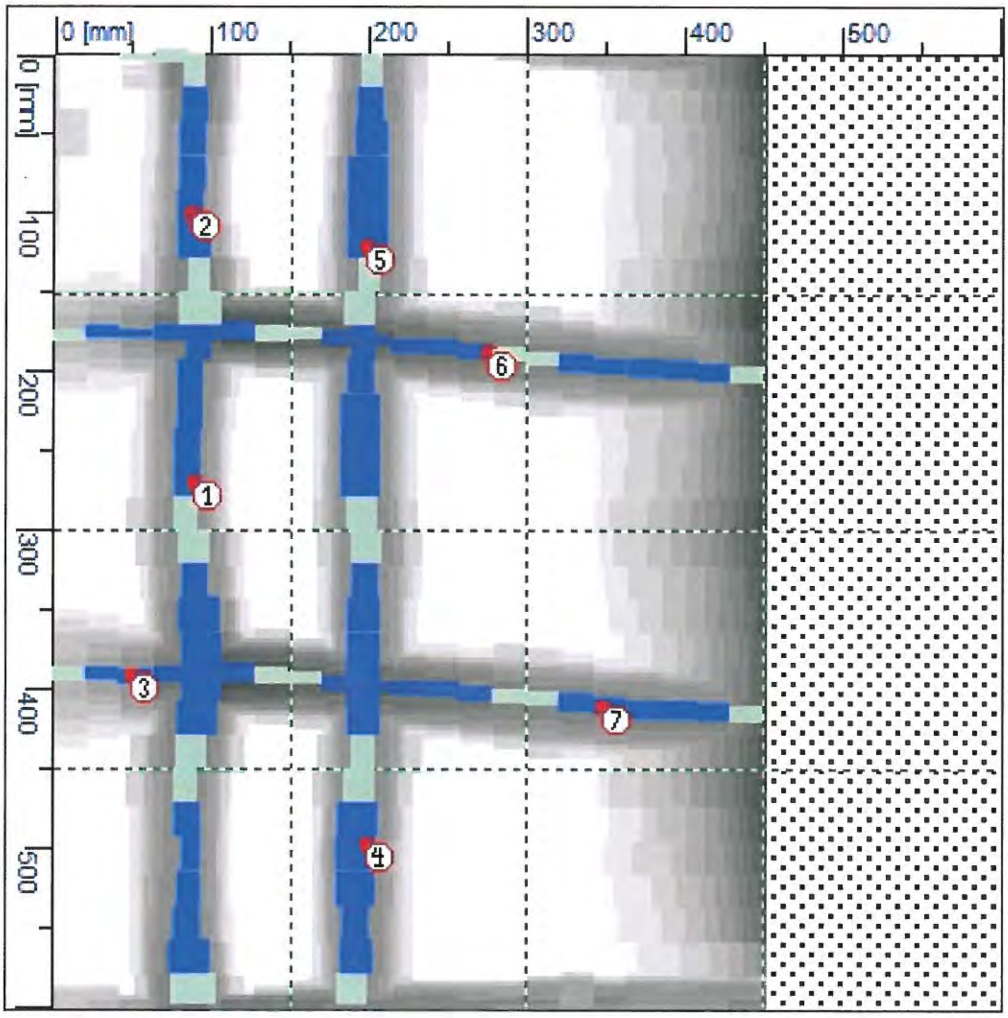
Marker	x: [mm]	y: [mm]	Comment:
1	67	251	Concrete cover = 50mm, Bar size = 16mm
2	68	505	Concrete cover = 55mm, Bar size = 20mm
3	58	75	Concrete cover = 53mm, Bar size = 20mm
4	342	232	Concrete cover = 37mm, Bar size = 12mm
5	423	526	Concrete cover = 53mm, Bar size = 14mm
6	512	115	

Project: Burwood Hospital - Maintenance building

Imagescan: *FS001552.XFF*

1-5

Date / Time: 2012-05-23 11:57:03 SSN: 04806010 [mm]



Customer: Craig - Holmes Consulting Group

Location: 298 Burwood Road, Burwood, Christchurch

Operator: Frank Kang

Comment:

1-5

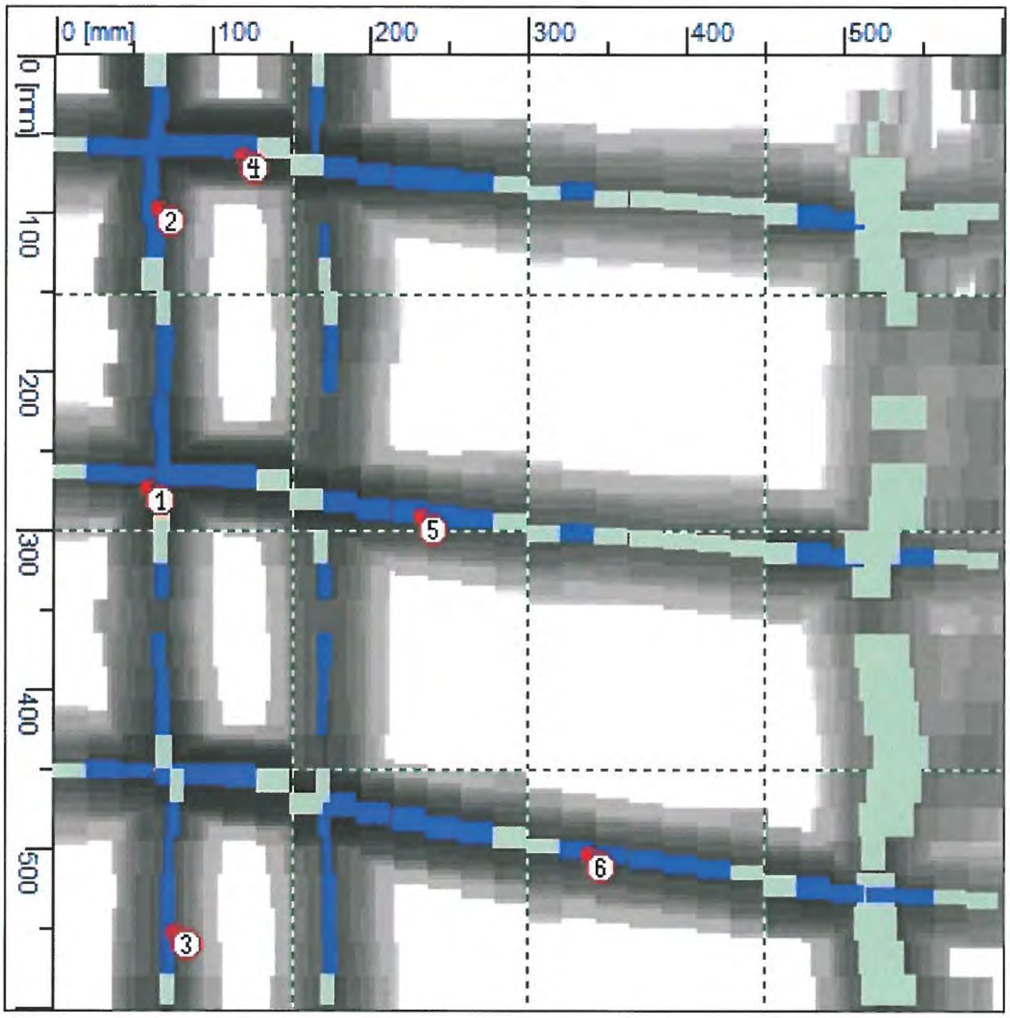
Marker	x: [mm]	y: [mm]	Comment:
1	85	266	Concrete cover = 45mm, Bar size = 16mm
2	84	96	Concrete cover = 47mm, Bar size = 16mm
3	47	389	Concrete cover = 43mm, Bar size = 10mm
4	195	495	Concrete cover = 47mm, Bar size = 25mm
5	195	118	Concrete cover = 47mm, Bar size = 25mm
6	273	184	Concrete cover = 36mm, Bar size = 10mm
7	347	408	

Project: Burwood Hospital - Maintenance building

Imagescan: *FS001553.XFF*

1-6

Date / Time: 2012-05-23 12:00:22 SSN: 04806010 [mm]



Customer: Craig - Holmes Consulting Group

Location: 298 Burwood Road, Burwood, Christchurch

Operator: Frank Kang

Comment:

1-6

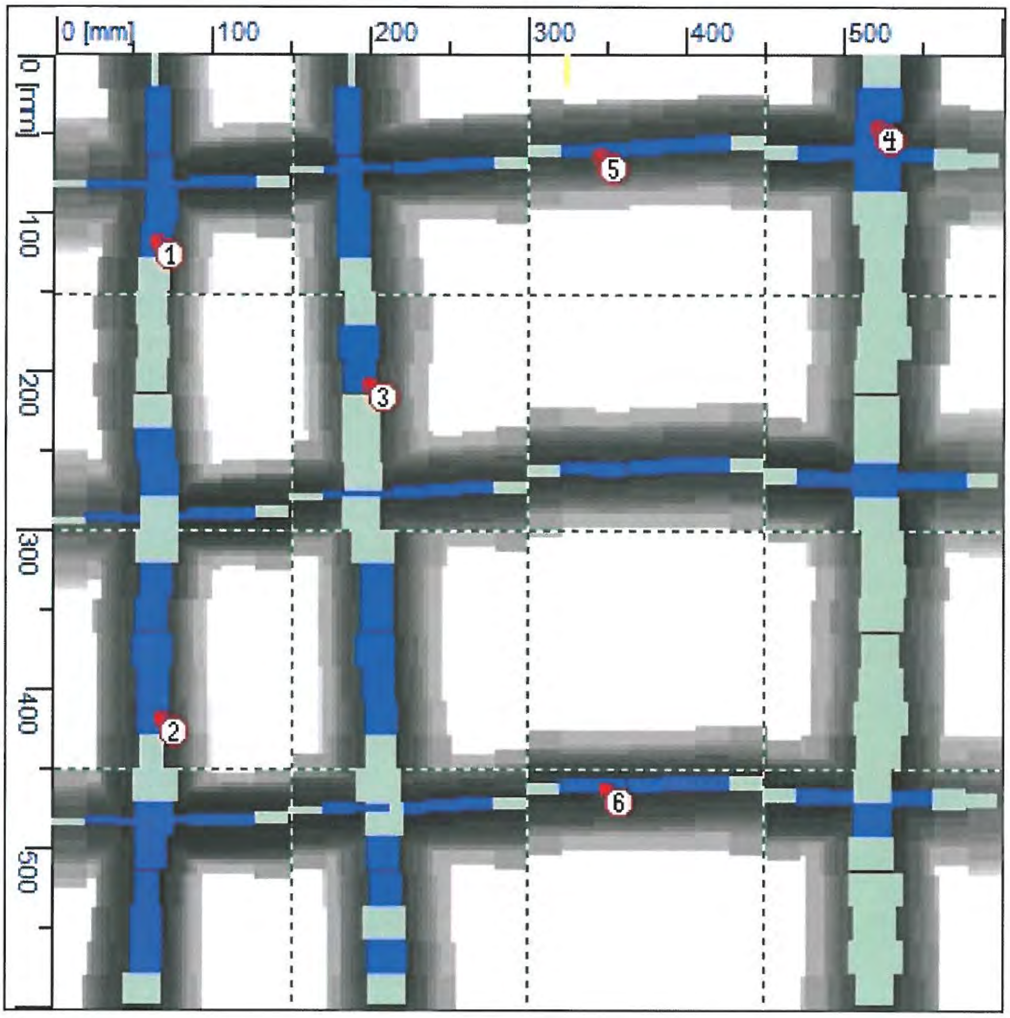
Marker	x: [mm]	y: [mm]	Comment:
1	55	268	Concrete cover = 50mm, Bar size = 12mm
2	62	93	Concrete cover = 55mm, Bar size = 12mm
3	73	549	Concrete cover = 54mm, Bar size = 8mm
4	114	60	Concrete cover = 46mm, Bar size = 16mm
5	227	288	Concrete cover = 54mm, Bar size = 14mm
6	336	501	

Project: Burwood Hospital - Maintenance building

Imagescan: *FS001554.XFF*

1-7

Date / Time: 2012-05-23 12:04:52 SSN: 04806010 [mm]



Customer: Craig - Holmes Consulting Group

Location: 298 Burwood Road, Burwood, Christchurch

Operator: Frank Kang

Comment:

1-7

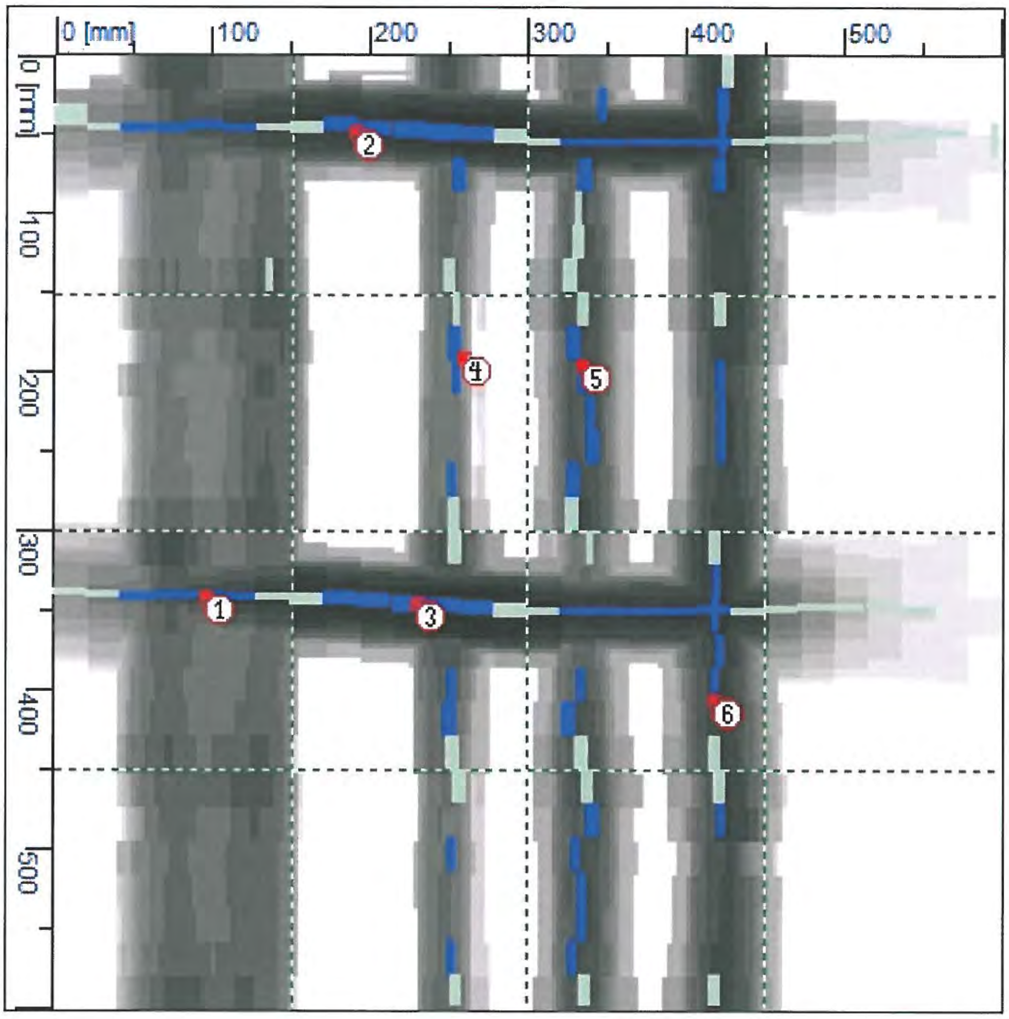
Marker	x: [mm]	y: [mm]	Comment:
1	62	114	Concrete cover = 58mm, Bar size = 20mm
2	64	416	Concrete cover = 57mm, Bar size = 20mm
3	196	204	Concrete cover = 59mm, Bar size = 20mm
4	519	41	Concrete cover = 59mm, Bar size = 30mm
5	342	60	Concrete cover = 49mm, Bar size = 8mm
6	348	460	Concrete cover = 48mm, Bar size = 10mm

Project: Burwood Hospital - Maintenance building

Imagescan: *FS001555.XFF*

2-1

Date / Time: 2012-05-23 12:20:29 SSN: 04806010 [mm]



Customer: Craig - Holmes Consulting Group

Location: 298 Burwood Road, Burwood, Christchurch

Operator: Frank Kang

Comment:

2-1

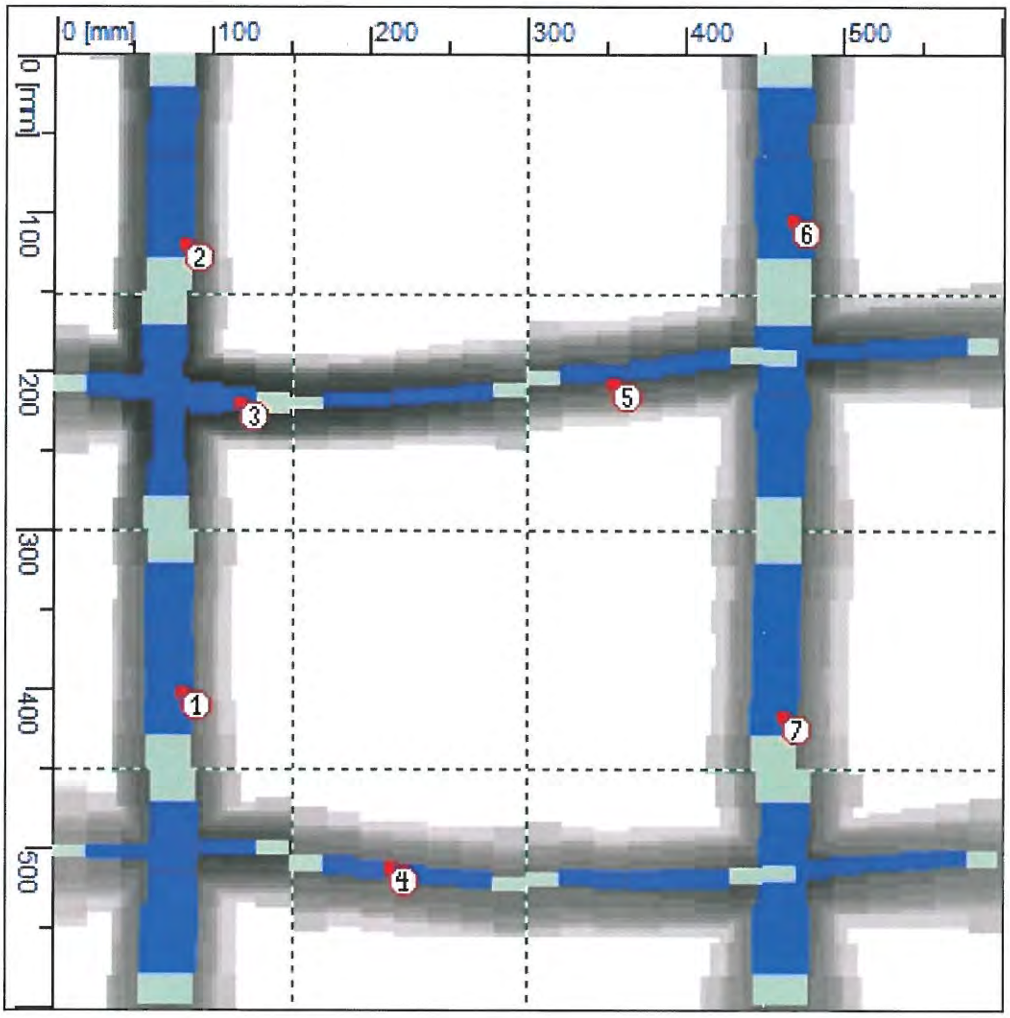
Marker	x: [mm]	y: [mm]	Comment:
1	93	338	Concrete cover = 48mm, Bar size = 6mm
2	188	45	Concrete cover = 46mm, Bar size = 10mm
3	226	344	Concrete cover = 45mm, Bar size = 12mm
4	255	188	Concrete cover = 50mm, Bar size = 8mm
5	333	192	Concrete cover = 50mm, Bar size = 8mm
6	416	404	

Project: Burwood Hospital - Maintenance building

Imagescan: *FS001556.XFF*

2-2

Date / Time: 2012-05-23 12:25:39 SSN: 04806010 [mm]



Customer: Craig - Holmes Consulting Group

Location: 298 Burwood Road, Burwood, Christchurch

Operator: Frank Kang

Comment:

2-2

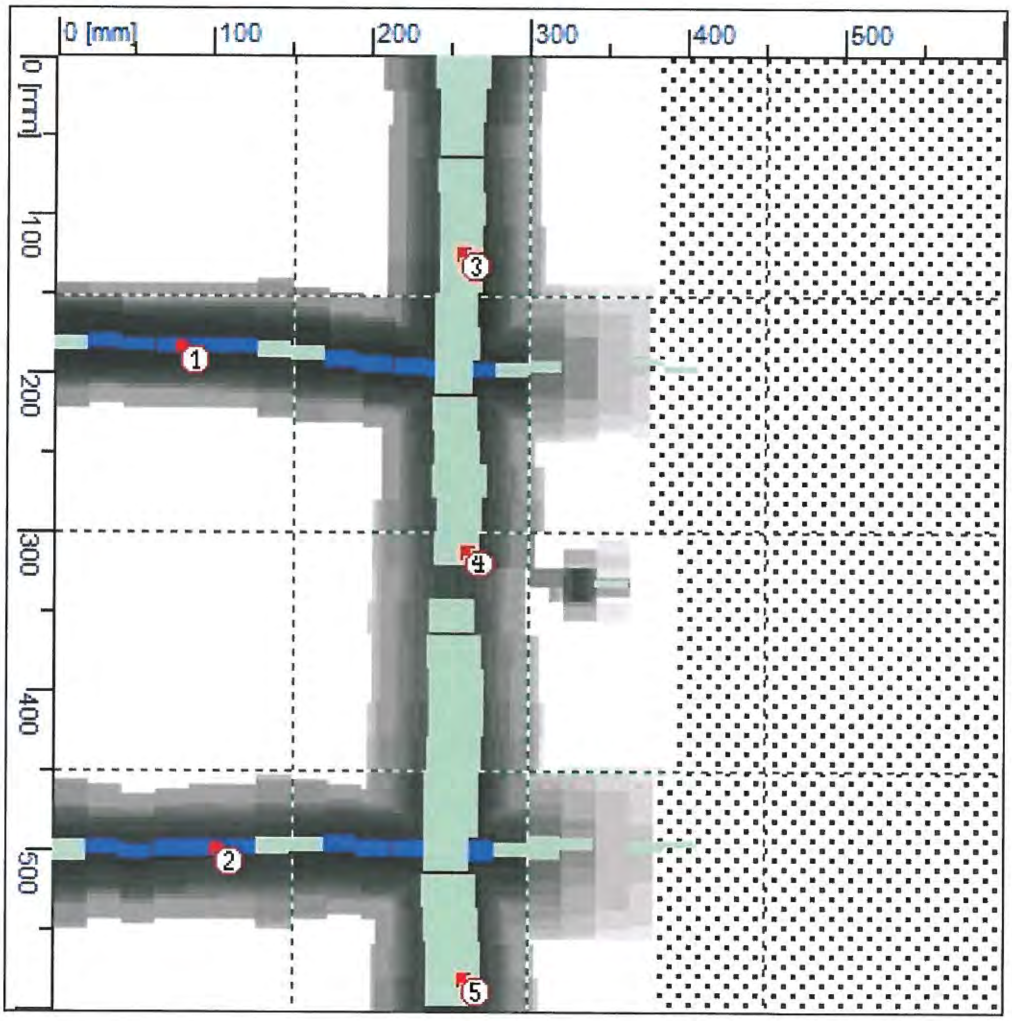
Marker	x: [mm]	y: [mm]	Comment:
1	78	399	Concrete cover = 53mm, Bar size = 30mm
2	79	116	Concrete cover = 45mm, Bar size = 30mm
3	115	216	Concrete cover = 32mm, Bar size = 20mm
4	210	510	Concrete cover = 46mm, Bar size = 10mm
5	352	204	Concrete cover = 42mm, Bar size = 10mm
6	466	101	Concrete cover = 52mm, Bar size = 36mm
7	460	415	Concrete cover = 56mm, Bar size = 30mm

Project: Burwood Hospital - Maintenance building

Imagescan: *FS001557.XFF*

2-3

Date / Time: 2012-05-23 12:27:34 SSN: 04806010 [mm]



Customer: Craig - Holmes Consulting Group

Location: 298 Burwood Road, Burwood, Christchurch

Operator: Frank Kang

Comment:

2-3

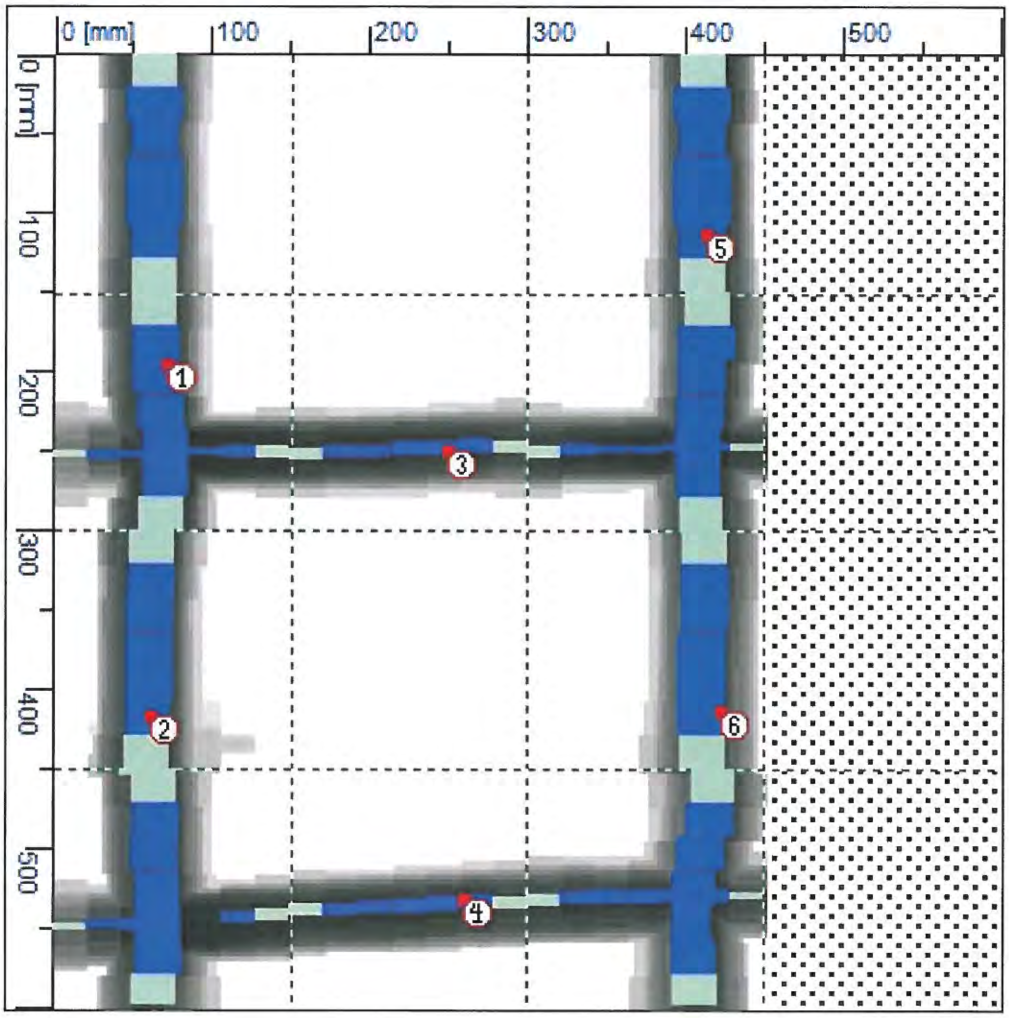
Marker	x: [mm]	y: [mm]	Comment:
1	77	179	Concrete cover = 51mm, Bar size = 10mm
2	99	496	Concrete cover = 59mm, Bar size = 12mm
3	253	121	Concrete cover = 69mm, Bar size = 28mm
4	256	308	Concrete cover = 75mm, Bar size = 30mm
5	255	578	

Project: Burwood Hospital - Maintenance building

Imagescan: *FS001558.XFF*

2-4

Date / Time: 2012-05-23 12:37:04 SSN: 04806010 [mm]



Customer: Craig - Holmes Consulting Group

Location: 298 Burwood Road, Burwood, Christchurch

Operator: Frank Kang

Comment:

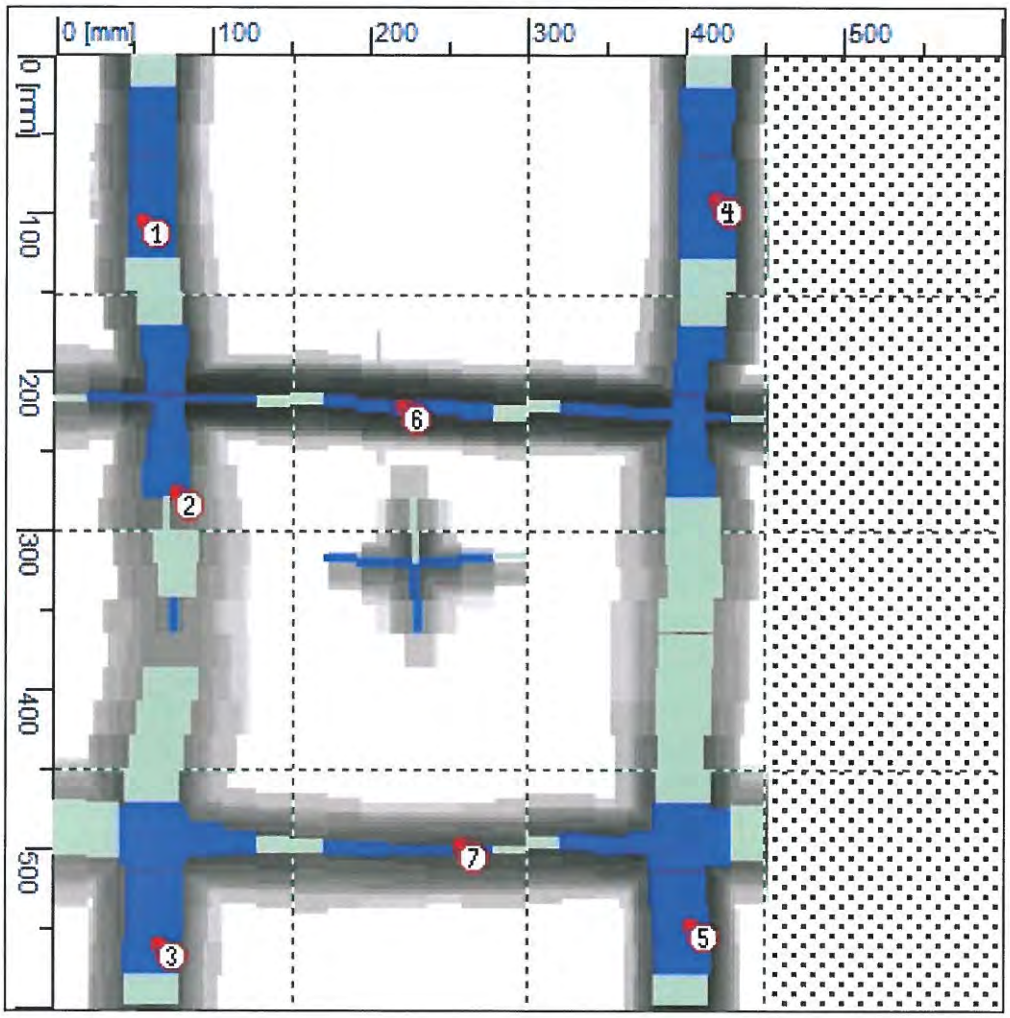
2-4

Marker	x: [mm]	y: [mm]	Comment:
1	68	192	Concrete cover = 40mm, Bar size = 30mm
2	59	414	Concrete cover = 41mm, Bar size = 28mm
3	247	247	Concrete cover = 32mm, Bar size = 10mm
4	256	530	Concrete cover = 31mm, Bar size = 10mm
5	411	110	Concrete cover = 42mm, Bar size = 30mm
6	421	411	

Project: Burwood Hospital - Maintenance building

Imagescan: ***FS001559.XFF*** 2-5

Date / Time: 2012-05-23 12:41:44 SSN: 04806010 [mm]



Customer: Craig - Holmes Consulting Group

Location: 298 Burwood Road, Burwood, Christchurch

Operator: Frank Kang

Comment:

2-5·

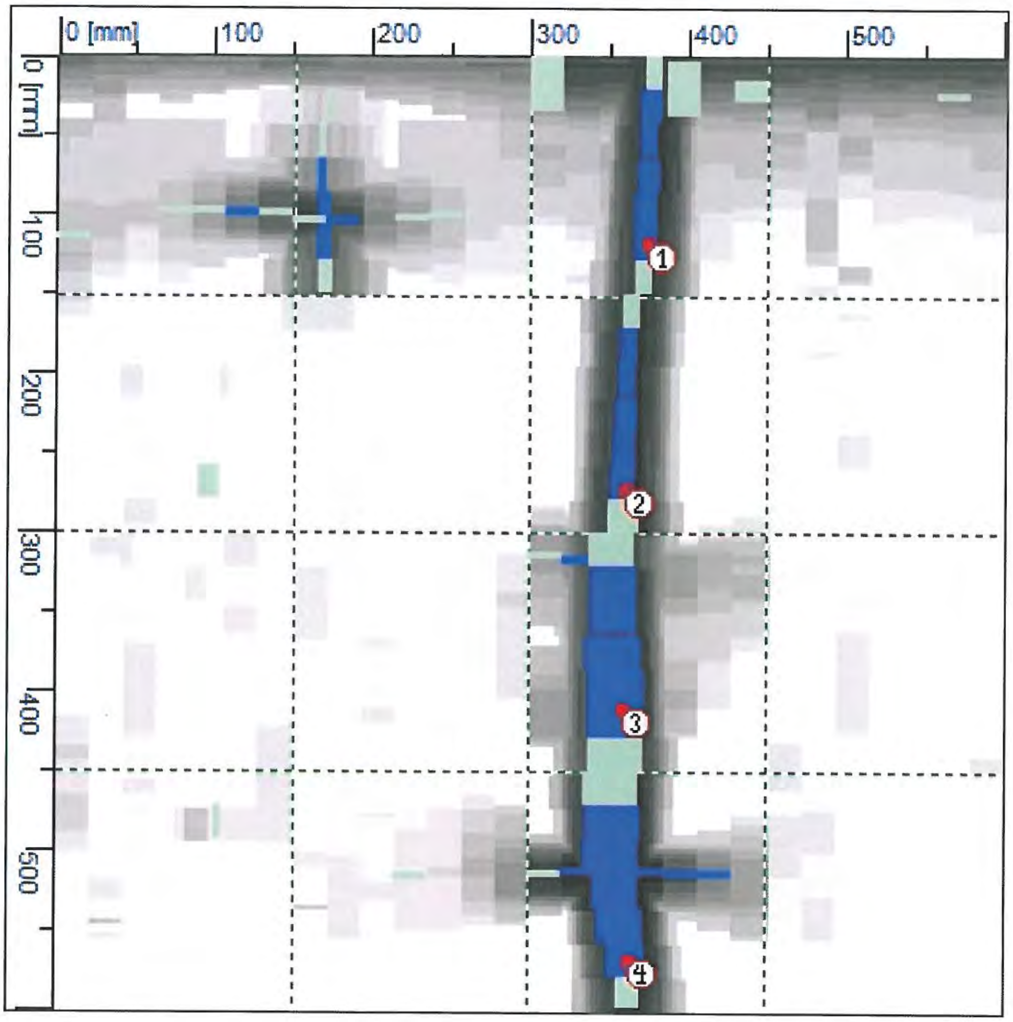
Marker	x: [mm]	y: [mm]	Comment:
1	52	101	Concrete cover = 49mm, Bar size = 30mm
2	74	273	Concrete cover = 58mm, Bar size = 30mm
3	63	556	Concrete cover = 53mm, Bar size = 36mm
4	416	88	Concrete cover = 47mm, Bar size = 36mm
5	401	545	Concrete cover = 44mm, Bar size = 36mm
6	218	218	Concrete cover = 32mm, Bar size = 8mm
7	253	495	

Project: Burwood Hospital - Maintenance building

Imagescan: *FS001562.XFF*

2-6

Date / Time: 2012-05-23 12:52:20 SSN: 04806010 [mm]



Customer: Craig - Holmes Consulting Group

Location: 298 Burwood Road, Burwood, Christchurch

Operator: Frank Kang

Comment:

2-6

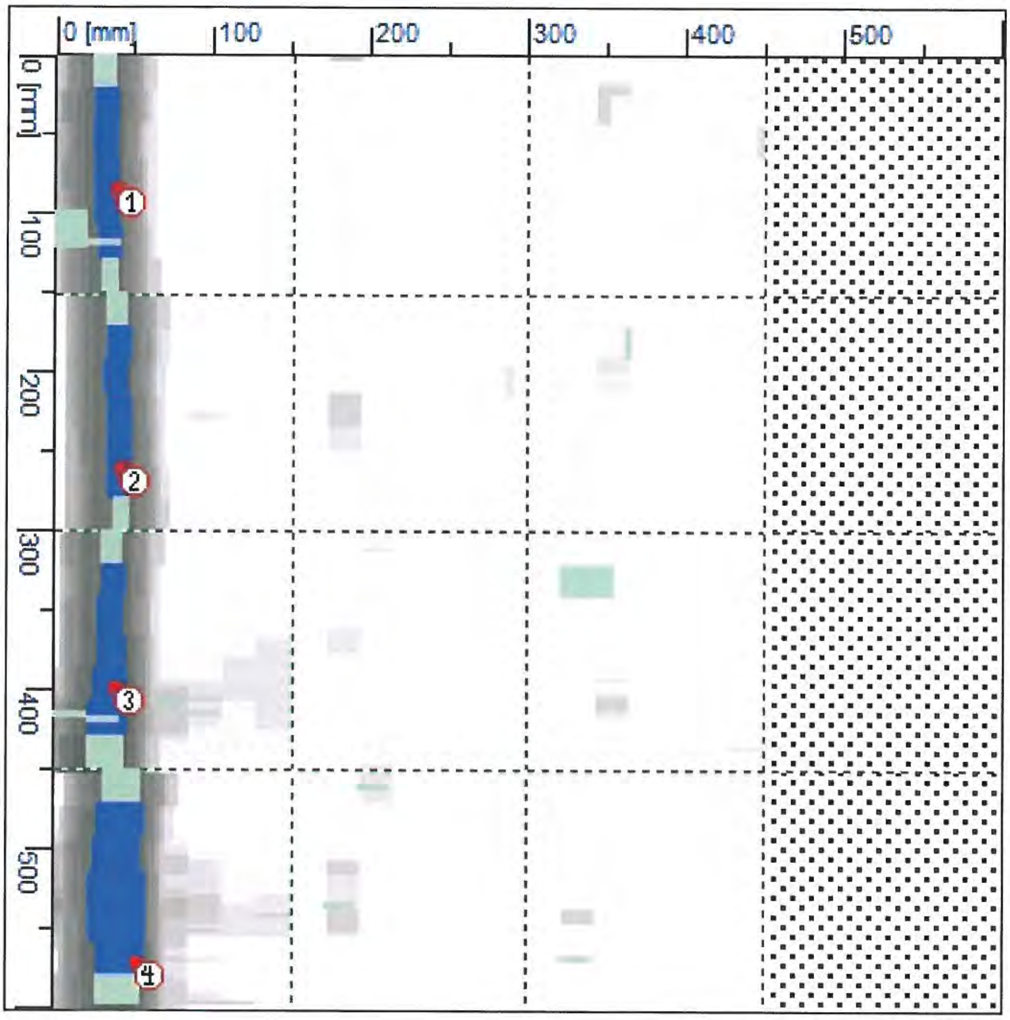
Marker	x: [mm]	y: [mm]	Comment:
1	373	115	Concrete cover = 40mm, Bar size = 14mm
2	359	268	Concrete cover = 39mm, Bar size = 16mm
3	358	408	Concrete cover = 43mm, Bar size = 36mm
4	362	566	Concrete cover = 41mm, Bar size = 25mm

Project: Burwood Hospital - Maintenance building

2-7

Imagescan: ***FS001563.XFF***

Date / Time: 2012-05-23 13:02:33 SSN: 04806010 [mm]



Customer: Craig - Holmes Consulting Group

Location: 298 Burwood Road, Burwood, Christchurch

Operator: Frank Kang

Comment:

2-7

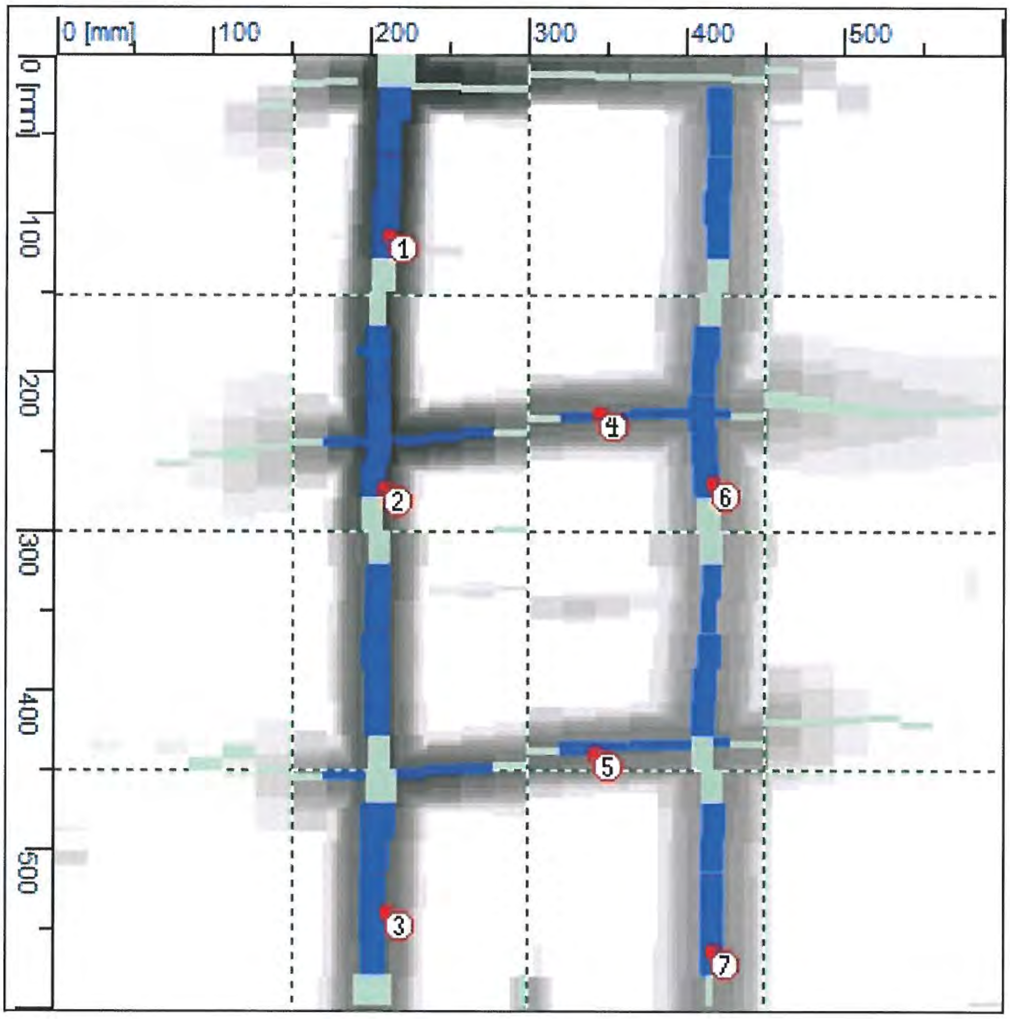
Marker	x: [mm]	y: [mm]	Comment:
1	36	81	Concrete cover = 40mm, Bar size = 14mm
2	38	258	Concrete cover = 38mm, Bar size = 14mm
3	36	396	Concrete cover = 38mm, Bar size = 20mm
4	49	568	

Project: Burwood Hospital - Maintenance building

Imagescan: *FS001567.XFF*

2-8.

Date / Time: 2012-05-23 13:14:15 SSN: 04806010 [mm]



Customer: Craig - Holmes Consulting Group

Location: 298 Burwood Road, Burwood, Christchurch

Operator: Frank Kang

Comment:

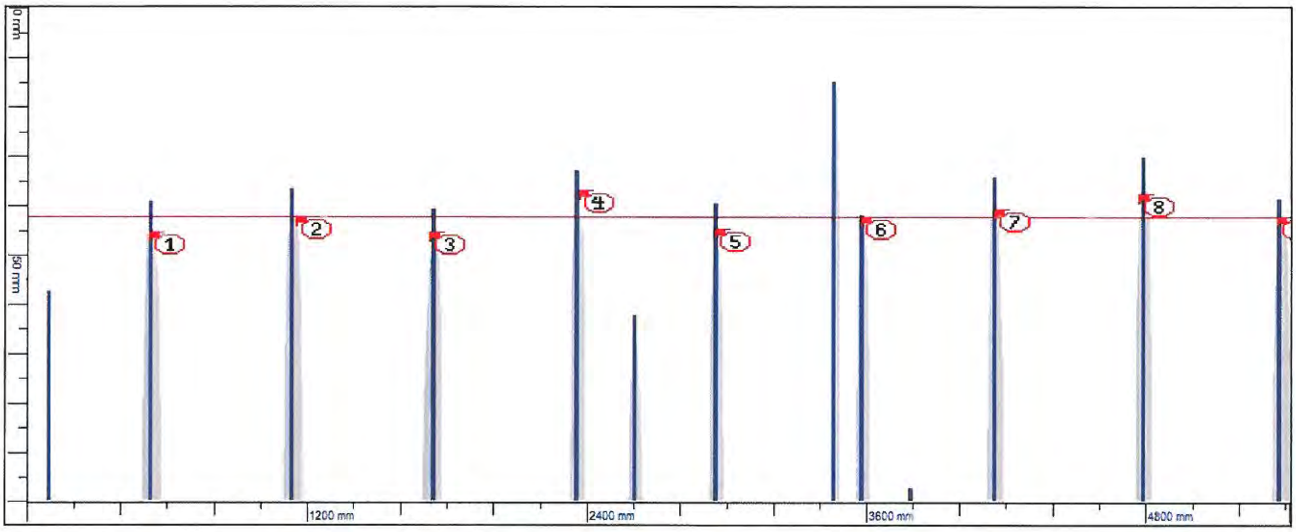
2-8

Marker	x: [mm]	y: [mm]	Comment:
1	208	110	Concrete cover = 25mm, Bar size = 16mm
2	205	268	Concrete cover = 27mm, Bar size = 16mm
3	207	537	Concrete cover = 35mm, Bar size = 16mm
4	344	222	Concrete cover = 28mm, Bar size = 6mm
5	340	437	Concrete cover = 32mm, Bar size = 8mm
6	415	266	Concrete cover = 46mm, Bar size = 16mm
7	415	562	Concrete cover = 47mm, Bar size = 14mm

Project: Burwood Hospital - Maintenance building

Quickscan: ***FQ001560.XFF*** 3-1

Date / Time: 2012-05-23 12:45:54 Bar: 16mm SSN: 04806010



0 mm 5415 mm

Quickscan Statistic:

Minimum Coverage:	15 mm	T1:	42 mm
Maximum Coverage:	97 mm	#Bars at T1:	10
Mean Coverage:	43 mm	T2:	100 mm
Standard Deviation:	20 mm	#Bars at T2:	13
Cut-Off:	100 mm	T3:	100 mm
#Bars at Cut-Off:	13	#Bars at T3:	13

Customer: Craig - Holmes Consulting Group

Location: 298 Burwood Road, Burwood, Christchurch

Operator: Frank Kang

Comment:

- Measurement results

Max coverage : 92mm
Min coverage : 17mm
Mean coverage : 45mm
Standard deviation : 18mm

Vertical rebars are quite evenly spaced

3-1

Marker	x: [mm]	z: [mm]	Comment:
1	522	45	Concrete cover = 45mm, Distance from left column = 522mm
2	1150	43	Concrete cover = 43mm, Distance from left column = 1150mm
3	1733	45	Concrete cover = 44mm, Distance from left column = 1733mm
4	2372	37	Concrete cover = 36mm, Distance from left column = 2372mm
5	2951	45	Concrete cover = 44mm, Distance from left column = 2951mm
6	3577	43	Concrete cover = 42mm, Distance from left column = 3577mm
7	4150	41	Concrete cover = 40mm, Distance from left column = 4150mm
8	4776	38	Concrete cover = 35mm, Distance from left column = 4776mm
9	5369	43	Concrete cover = 42mm, Distance from left column = 5369mm

Project: Burwood Hospital - Maintenance building

Quickscan: *FQ001561.XFF*

3-2

Date / Time: 2012-05-23 12:46:30 Bar: 16mm SSN: 04806010

0 mm 1651 mm

Quickscan Statistic:

Minimum Coverage:	81 mm	T1:	100 mm
Maximum Coverage:	81 mm	#Bars at T1:	1
Mean Coverage:	81 mm	T2:	100 mm
Standard Deviation:	0 mm	#Bars at T2:	1
Cut-Off:	100 mm	T3:	100 mm
#Bars at Cut-Off:	1	#Bars at T3:	1

Customer: Craig - Holmes Consulting Group

Location: 298 Burwood Road, Burwood, Christchurch

Operator: Frank Kang

Comment:

- Measurement results

Max coverage : 81mm
Min coverage : 71mm
Mean coverage : 71mm
Standard deviation : 14mm

3-2

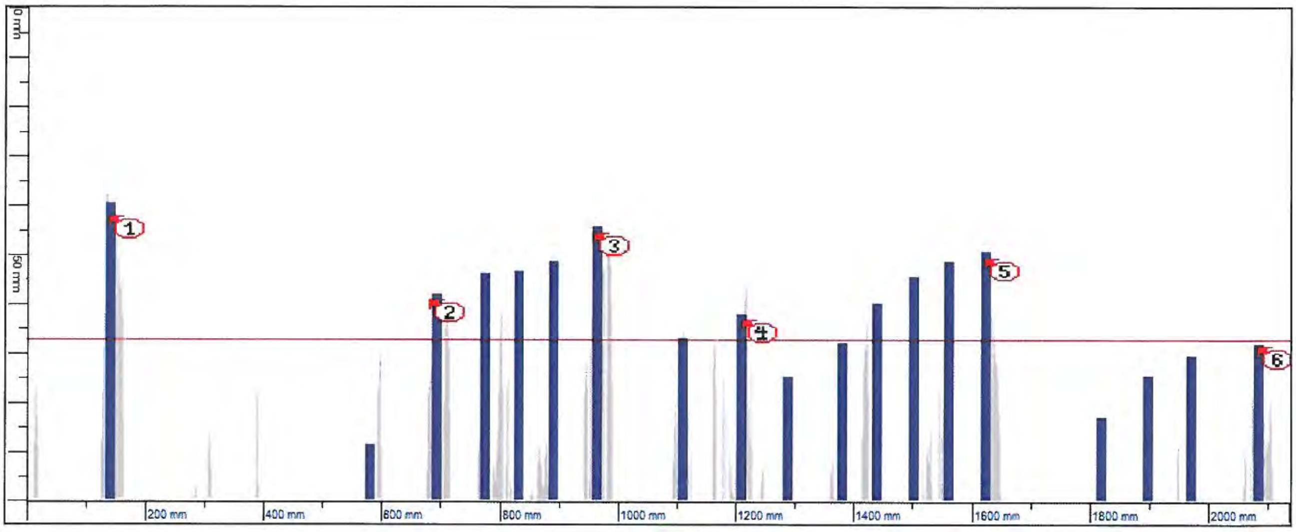
Marker	x: [mm]	z: [mm]	Comment:
1	4	57	Concrete cover = 58mm, Distance from 1.65m high from the ground = 4mm
2	96	81	Concrete cover = 81mm, Distance from 1.65m high from the ground = 96mm

Project: Burwood Hospital - Maintenance building

Quickscan: *FQ001564.XFF*

3-3

Date / Time: 2012-05-23 13:05:28 Bar: 16mm SSN: 04806010



0 mm 2136 mm

Quickscan Statistic:

Minimum Coverage: 40 mm	T1: 67 mm
Maximum Coverage: 89 mm	#Bars at T1: 12
Mean Coverage: 61 mm	T2: 100 mm
Standard Deviation: 13 mm	#Bars at T2: 19
Cut-Off: 100 mm	T3: 100 mm
#Bars at Cut-Off: 19	#Bars at T3: 19

Customer: Craig - Holmes Consulting Group

Location: 298 Burwood Road, Burwood, Christchurch

Operator: Frank Kang

Comment:

- Measurement results

Max coverage : 89mm
Min coverage : 40mm
Mean coverage : 61mm
Standard deviation : 13mm

Possibly ties from masonry wall to into concrete. Scanned vertically right next to concrete column.

3-3

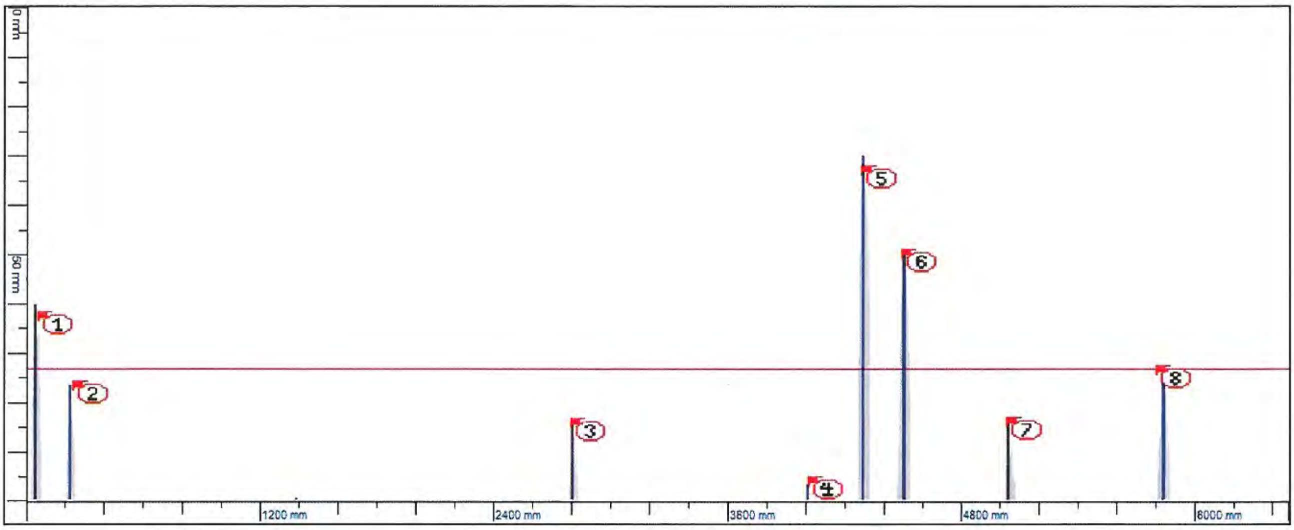
Marker	x: [mm]	z: [mm]	Comment:
1	138	43	Concrete cover = 40mm, Distance from 2.13m high from the ground = 138mm
2	683	60	Concrete cover = 58mm, Distance from 2.13m high from the ground = 683mm
3	959	46	Concrete cover = 44mm, Distance from 2.13m high from the ground = 959mm
4	1211	63	Concrete cover = 61mm, Distance from 2.13m high from the ground = 1211mm
5	1621	51	Concrete cover = 49mm, Distance from 2.13m high from the ground = 1621mm
6	2081	69	Concrete cover = 67mm, Distance from 2.13m high from the ground = 2081mm

Project: Burwood Hospital - Maintenance building

Quickscan: ***FQ001565.XFF***

Date / Time: 2012-05-23 13:10:41 Bar: 16mm SSN: 04806010

3-4



0 mm 6483 mm

Quickscan Statistic:

Minimum Coverage:	30 mm	T1:	73 mm
Maximum Coverage:	100 mm	#Bars at T1:	4
Mean Coverage:	75 mm	T2:	100 mm
Standard Deviation:	23 mm	#Bars at T2:	8
Cut-Off:	100 mm	T3:	100 mm
#Bars at Cut-Off:	10	#Bars at T3:	8

Customer: Craig - Holmes Consulting Group

Location: 298 Burwood Road, Burwood, Christchurch

Operator: Frank Kang

Comment:

- Measurement results

Max coverage : 100mm
Min coverage : 30mm
Mean coverage : 75mm
Standard deviation : 23mm

No vertical rebars are detected. Number 1,2,5 & 6 are vertical rebars installed in concrete column.

3-4

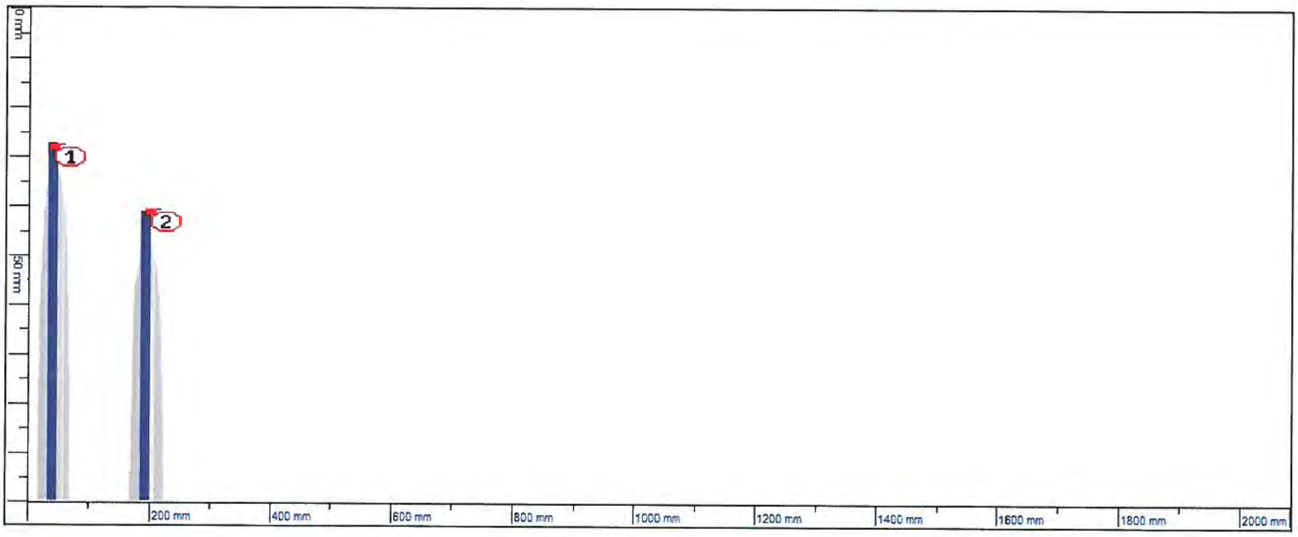
Marker	x: [mm]	z: [mm]	Comment:
1	55	62	Concrete cover = 62mm, Distance from left column = 55mm
2	244	76	Concrete cover = 76mm, Distance from left column = 244mm
3	2800	84	Concrete cover = 84mm, Distance from left column = 2800mm
4	4014	95	Concrete cover = 95mm, Distance from left column = 4014mm
5	4290	32	Concrete cover = 32mm, Distance from left column = 4290mm
6	4488	49	Concrete cover = 49mm, Distance from left column = 4488mm
7	5040	83	Concrete cover = 83mm, Distance from left column = 5040mm
8	5805	73	Concrete cover = 73mm, Distance from left column = 5805mm

Project: Burwood Hospital - Maintenance building

Quickscan: *FQ001566.XFF*

Date / Time: 2012-05-23 13:11:19 Bar: 16mm SSN: 04806010

3-5



0 mm 2083 mm

Quickscan Statistic:

Minimum Coverage:	27 mm	T1:	100 mm
Maximum Coverage:	41 mm	#Bars at T1:	2
Mean Coverage:	34 mm	T2:	100 mm
Standard Deviation:	10 mm	#Bars at T2:	2
Cut-Off:	100 mm	T3:	100 mm
#Bars at Cut-Off:	2	#Bars at T3:	2

Customer: Craig - Holmes Consulting Group
Location: 298 Burwood Road, Burwood, Christchurch
Operator: Frank Kang

Comment:

- Measurement results

Max coverage : 41mm
Min coverage : 27mm
Mean coverage : 34mm
Standard deviation : 10mm

Possibly two horizontal rebars at 2.04m & 1.88m high from the ground.

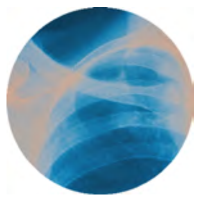
3-5

Marker	x: [mm]	z: [mm]	Comment:
1	36	28	Concrete cover = 28mm, Distance from 2.08m high from the ground = 36mm
2	193	41	Concrete cover = 41mm, Distance from 2.08m high from the ground = 193mm

Project: Burwood Hospital - Maintenance building

DETAILED SEISMIC ASSESSMENT REPORT

STRUCTURAL AND CIVIL ENGINEERS



BURWOOD HOSPITAL CAMPUS

REPORT 17 – CHAPEL

PREPARED FOR

CANTERBURY DISTRICT HEALTH BOARD

106186.54

INTERIM REPORT REV 3 – 8 SEPTEMBER 2015

BURWOOD HOSPITAL CAMPUS – INTERIM DETAILED SEISMIC ASSESSMENT REPORT

REPORT 27 – CHAPEL

Prepared For:
CANTERBURY DISTRICT HEALTH BOARD

Date: 8 September 2015
Project No: 106186.54
Revision No: 3

Prepared By:

Lachlan Sykes
PROJECT ENGINEER

Reviewed By:

Alan Park
PROJECT DIRECTOR

Updated By:

Scott Davies-Colley
STRUCTURAL ENGINEER

Reviewed By:

Jenny Fisher
PROJECT DIRECTOR

Holmes Consulting Group LP
Christchurch Office

REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
7/5/12	1	Interim Report for Review
8/6/12	2	Interim Report, Revised Assessment of Portal Frames and Updated Repair and Strengthening Recommendations
8/9/15	3	Updated to include strengthening works to 67% DBE (IL2)

CONTENTS

APPENDICES

TABLES Page

FIGURES Page

Holmes Consulting Group has been engaged by Canterbury District Health Board to complete a full structural review of the Burwood Hospital Campus following the Lyttelton Earthquake. A series of reports have been completed as part of this. These consist of a base report, a number of specific building reports and a repair specification. The individual building reports, like this one on the Chapel, should be read in conjunction with the base report and refer to the repair specification.

This report identifies the structural damage sustained by the Chapel building as a result of the series of Earthquakes, including the Darfield Earthquake that struck at 4:36am on the 4th September 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February 2011; the June Earthquake that struck at 2.20pm on the 13th of June 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The report summarises the effects of the damage on the lateral load capacity of the building, and provides structural repairs for the damage identified. The general form of the building, along with its capacity relative to current code levels, has been included for the buildings pre-earthquake undamaged state and post-earthquake state. The strengthening work carried out to increase the capacity of the building to 67% of a Design Basis Earthquake (%DBE) for an importance Level 2 Structure (IL2) is summarised.

The Burwood Hospital Campus Chapel consists of a single story, mostly timber framed structure, originally designed in 1962 and constructed in the period thereafter. The building was relocated to its current site in 2001, at which point a new foundation system was designed and constructed.

The building consists of primarily timber framed walls, clad on the exterior with a combination of vertical weather board and brick veneer, the latter of which was removed as part of the strengthening completed in April-May 2014. There are a series of internal steel portal frames which form the nave and resist lateral forces in the north-south direction. The roof of the Chapel consisted of a clay tile roof over timber battens and purlins which are supported by the interior steel portal frames and exterior bearing walls. The clay tile roofing was replaced with a lightweight metal roof as part of the strengthening completed in April-May 2014. An elevated timber framed ground floor is supported by continuous exterior concrete sub-floor walls and isolated interior concrete piers.

The information available for the review included: a 2001 Plan and Details drawing for the Chapel Relocation by Powel Fenwick Consultants Ltd [1], a 2009 Master Floor Plan provided from the CDHB's Maintenance and Engineering Department [2], a 1976 Survey of the building by Cutter Pickmere Douglas Architects [3], a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [4], and a level survey of the building completed by Fox & Associates [5].

The Chapel building appeared to have performed as would be expected for a building of this type and age. The bulk of structural damage as typified by cracking of the timber framed wall and ceiling linings. Differential ground settlement had been noted within the south-east corner of the building, resulting in a worst case slope in the elevated timber ground floor of 23mm over a 4.3m length (1:190 or 0.53%). Associated minor cracking had been noted in isolated locations to the concrete sub-floor walls and foundations. Additional damage had been noted to the clay roof tile assembly and the exterior brick veneer.

The structural damage sustained by the building as a whole would be categorized as minor to moderate. This is due to the differential ground settlement noted and the reduction in lateral capacity of the building caused by the cracking of the linings to the timber framed walls and ceilings (which provide the primary lateral support to the building in the East-West direction). This Earthquake damage was repaired as part of the strengthening work carried out during April-May 2014, as noted in Sections 4 and 5.

It is believed that the significant damage observed to date occurred during the 22nd February event. Further observations of the earthquake damage have been included in the body of this report. The minimum repairs required have been included in Section 4.

The actual percentage reduction in the lateral bracing capacity of the timber framed walls and the ceiling diaphragm as a result of the damage observed is hard to quantify. Although there was some reduction in strength due to the damage noted, the primarily affect was to the ongoing stiffness of the building. The reduced stiffness would have resulted in larger future displacements during seismic events and consequential damage to interior linings and building contents.

For the purposes of this assessment the CDHB Chapel has been considered to be an IL2 building. Based on our analysis, the primary lateral force resisting elements of the building, in its undamaged state prior to the earthquakes, had the capacity to resist approximately 35% DBE requirements in the North-South direction and 40% DBE in the east-west direction. In the north-south direction the lateral load carrying capacity was limited by the strength capacity of the steel portal frames. In the East-West direction the %DBE was limited by the lateral capacity of the ground floor walls.

Strengthening work to bring the capacity of the Chapel up to 67% DBE (IL2) was carried out during April-May 2014. This work included:

- Improving the condition of the portal frame baseplate connections to the foundation walls
- Installing fly braces on the portal frames
- Relining bracing walls with plywood
- Installing a plywood roof diaphragm

As part of this work, the brick veneer has been removed and replaced with lightweight cladding and the heavy roof tiles replaced with lightweight metal roofing. This has significantly reduced the seismic weight of the building.

If the building were to be assessed as an Importance Level 3 building the capacity of the strengthened building would be approximately 52% DBE (IL3).

1. INTRODUCTION

Holmes Consulting Group has been engaged by Canterbury District Health Board to complete a full structural review of the Burwood Hospital Campus following the Lyttelton Earthquake. A series of reports have been completed as part of this. These consist of a base report, a number of specific building reports and a repair specification. The individual building reports, like this one on the Chapel, should be read in conjunction with the base report and refer to the repair specification.

The Burwood Campus base report covers the purpose and scope of the structural review. The current statutory requirements relevant to earthquake damaged buildings are outlined and the level of shaking experienced at the site estimated. Earthquake induced ground settlement damage across the campus is also discussed. The repair specification has been prepared to include repair details for typical damage observed in buildings on the campus and is referred to as required in the specific building reports.

1.1 SCOPE OF WORK

This report covers the Chapel, at the Burwood Hospital Campus. The report identifies the general form of the structure, along with the gravity and lateral load resisting systems. Each component of the structural system was reviewed based upon the information available and any potential Critical Structural Weaknesses (CSW's) were noted.

The report also identifies the structural damage observed to date as a result of the series of Earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake, in particular, subjected the building to strong ground motions which significantly exceeded the current code loading demand for buildings of this nature.

The capacity of the Chapel has been assessed relative to current code loading in the buildings pre-earthquake undamaged state and in its post-earthquake damaged state. The post-earthquake assessment summarizes the effects of the damage identified on both the gravity and lateral load resisting elements. Repair options to restore the buildings capacity to pre-earthquake levels for strength, durability and stiffness have been included. The repair options aim to maintain the buildings utility. Where required, strengthening options have also been provided.

1.2 LIMITATIONS

Findings presented as a part of this project are for the sole use of the Canterbury District Health Board, 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. Because all of the structure has not been available for detailed inspection or evaluation, this report is limited to those elements available and engineering judgement as to the likely condition of unseen elements. 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. PRE-EARTHQUAKE BUILDING CONDITION

This section discusses the form and capacity of the building prior to the Darfield Earthquake.

2.1 BUILDING FORM

The Chapel (Figure 2-1) is located in the CDHB's Burwood Hospital Campus, approximately 7 km north-east of Christchurch City. The building was originally built in 1962 in the north-east corner of the Burwood Campus, before being moved to its current location, north of the BSU hostel, in 2001.

The building has a single storey open floor plan with interior steel portal frames, spanning in the north-south direction, forming the nave (Figure 2-2). The remainder of the Chapel is timber framed, with an elevated timber ground floor over continuous exterior concrete sub-floor walls and footings, and isolated interior concrete piers. The roof assembly consist of clay tiles on timber battens and roof purlins, which are supported by the interior steel portal frames and exterior timber bearing walls.

Figure 2-1: Chapel - View from the South-East

The information available for the review included: a 2001 Plan and Details drawing for the Chapel Relocation by Powel Fenwick Consultants Ltd [1], a 2009 Master Floor Plan provided from the CDHB's Maintenance and Engineering Department [2], a 1976 Survey of the building by Cutter Pickmere Douglas Architects [3], a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [4], and a level survey of the building completed by Fox & Associates [5].

Most of the Chapel has a clay-tile roof believed to be supported by a grid of timber battens which span over the top of the roof purlins running in the east-west (longitudinal) direction of the building. The roof purlins in turn span over the top of the exposed interior steel portal frames, spaced at approximately 2.9m centres, and the exterior timber framed bearing walls. The interior gypsum board ceiling is inset between the roof purlins, which are clad in timber finishings. See Figure 2-2 below. The roof over the Vestry Section of the building is of lightweight steel construction.

Figure 2-2: Chapel – Interior View of Nave

The majority of the perimeter walls are load bearing timber framed stud walls, lined with gypsum wallboards on the inside face and brick veneer on the exterior face. The brick veneer is believed to be fixed to the studs with screw-ties conforming to the requirements of the New Zealand Standard, Built in Components for Masonry Construction, Part 1: Wall Ties*,* NZS 2699.2 [6]. Any internal walls, such as those at the east end of the building, are lined on both sides with gypsum wallboards. The external walls, on the east end of the building, are lined internally with gypsum wallboard and externally with vertically orientated weatherboard (as seen in Figure 2-1).

In general, the interior and exterior walls bear directly on elevated timber ground floor framing. The ground floor is constructed of straight tongue and groove sheathing boards over raised timber floor joists. The floor joists are supported by reinforced concrete sub-floor walls around the perimeter of the building and by timber bearers on isolated interior concrete piles. The bottom timber plate of the walls is believed to be nailed to the ground floor joists which are in turn nailed to an additional timber plate below which is fixed to the top of the concrete sub-floor walls with steel anchor bolts. See Figure 2-4 below for clarification. The timber bearers rest on the isolated concrete piers and are believed to be tied down with steel wire. The reinforced concrete sub-floor walls are founded below grade, and form the footings for the perimeter load bearing walls.

During the Chapel's relocation in 2001, new reinforced concrete sub-floor walls and footings, along with the isolated internal concrete piles were constructed to support the existing superstructure. New connections between the super structure and the sub-floor system were also added at this time. It is believed the fixings of the timber framed walls and ground floor framing to the exterior concrete sub-floor walls and footings conform to the requirements of New Zealand Standard Code for Timber-Framed Buildings, NZS3604:1999 [7], although the existing floor finishes have not been removed to confirm this assumption.

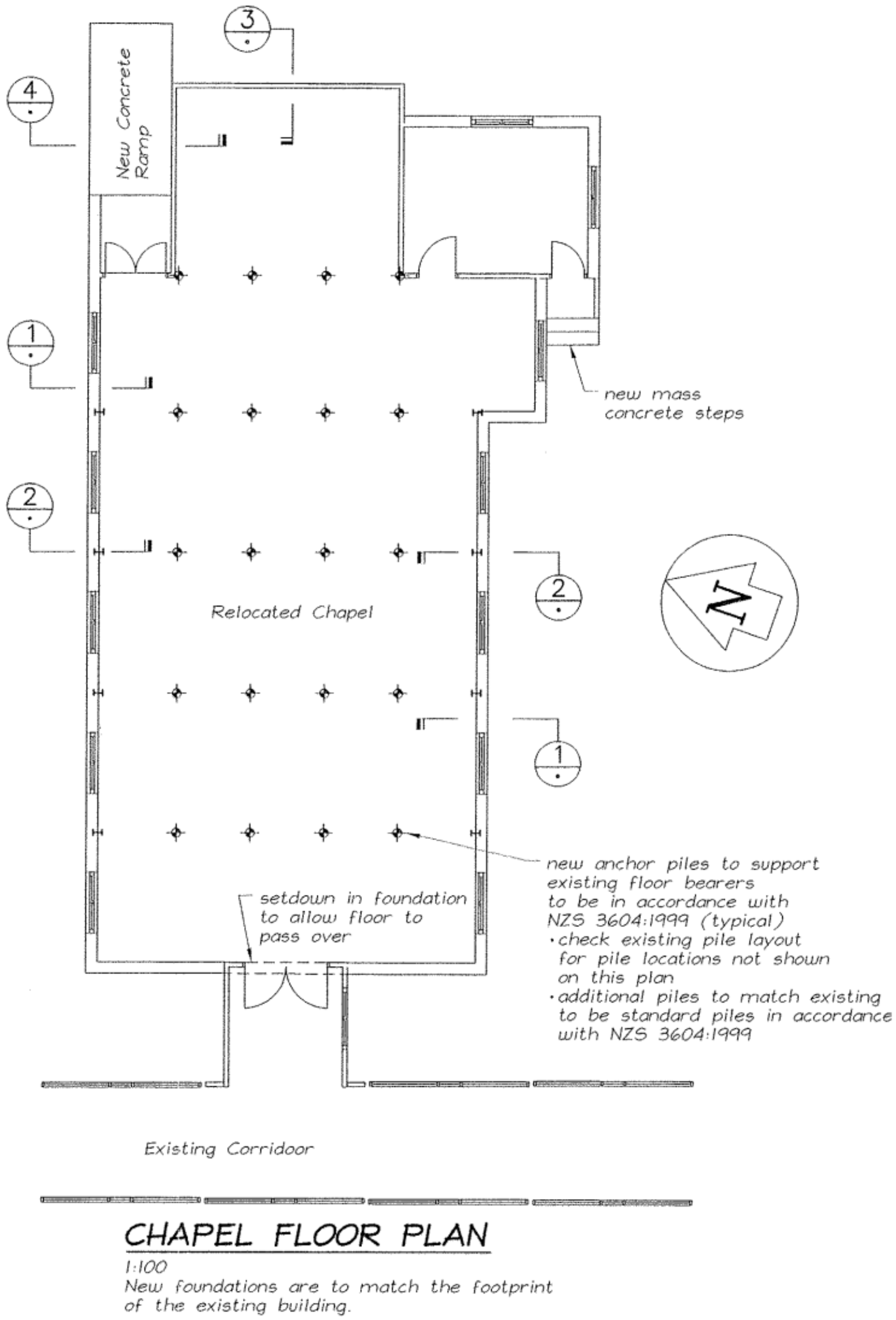


Figure 2-3: Chapel – 2001 Foundation Plan

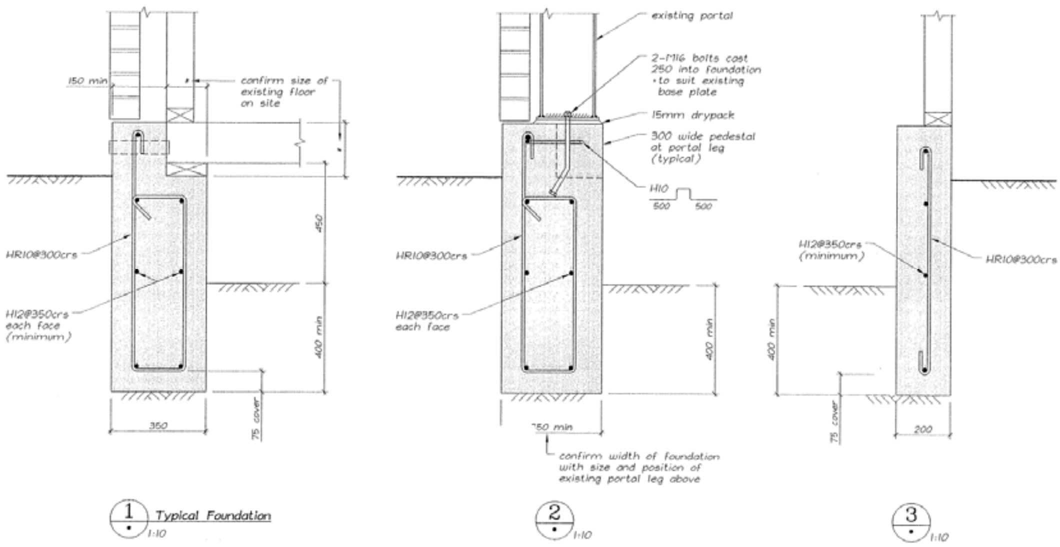


Figure 2-4: Chapel – Typical Foundation Details

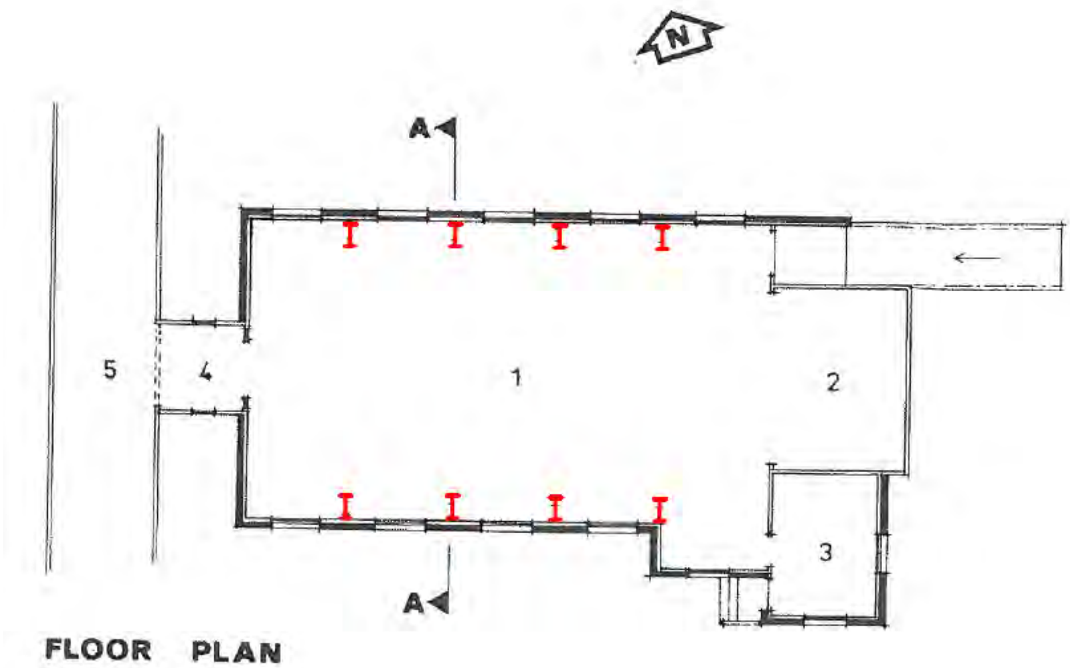


Figure 2-5: 1976 Survey – Ground Floor Plan

2.2 LATERAL LOAD RESISTING SYSTEMS

In the north-south direction of the Chapel, the primary lateral load resisting system for the superstructure is a combination of internal steel portal frames and exterior timber framed gable end bracing walls. Lateral loads are distributed to the steel portal frames and the end walls by the flexible gypsum board ceiling diaphragm which is inset between the timber roof purlins.

In the east-west (transverse) direction of the building, the lateral-load resisting system consists of the flexible ceiling diaphragm which distributes load to the external and interior timber bracing walls below.

At the ground floor level, lateral loads from the superstructure are either transferred directly to the concrete sub-floor walls below or distributed by the timber framed floor diaphragm to the sub-floor walls. The ground floor diaphragm assembly consists of straight tongue and groove timber sheathing over timber floor joists.

2.3 PRE-EARTHQUAKE BUILDING CAPACITY – DIRECT CODE COMPARISON

The building capacity under earthquake actions discussed in this section is compared to the capacity that a similar building would be designed to today. A new building of the form of the Chapel would be designed to the Structural Design Actions Standard, Part 5: Earthquake Actions – New Zealand, NZS 1170.5:2004 [8], incorporating the amendments made to these standards as a result of the Lyttelton Earthquake. These changes are outlined in the Amendment 10 of the Building Code [9]. The implications of the recent amendments are discussed more fully in the Burwood Hospital Campus Base Report. For a building of this type the amendments essentially result in an increase to the design loads of 36% when compared to pre-earthquake NZS 1170.5:2004 [8] design levels.

The original structural drawings for the Chapel were not available for this report; as such the original loading assumptions for the structure are unknown. For the purposes of this report seismic loading assumptions have been made based on a detailed review of the limited drawings available and physical explorations of the building.

When the building was originally designed in 1962, the loading standard at the time was likely the New Zealand Standard Model Building By-Law NZSS95:1939 [10]. When these By-Laws were written, neither the seismology of the different areas within New Zealand, or the impact this could have on buildings was as well understood as it is today. Along with an increase in the seismic demands required by the change in the loading code over this period, the seismic detailing requirements have also progressed significantly resulting in more ductile and better performing buildings. In particular, the bracing requirements of a similar building design and constructed to current code requirements would be several times larger.

The current code requires a new building to be designed for an earthquake, known as the Design Basis Earthquake (DBE), which is based upon the buildings physical location, local soil conditions, building type, fundamental period and importance level.

The structure is not regarded as an essential hospital facility by the CDHB and is therefore classified as an Importance Level 2 building in accordance with NZS 1170:2004, with a design life of 50 years for performance and durability of elements based upon the buildings use. This assumption will need to be verified by CDHB. The associated Risk Factor for design is R = 1.0, with an associated DBE return period of once in 500 years, which is typical for commercial-use type buildings as prescribed in the loadings code (no post-disaster or special function). The sub soil for the site has been taken as Soil Type D, which is consistent with the findings of the post-earthquake geotechnical investigation [4].

Based upon the period of construction and the detailing of the time, the lateral load resisting system of the Chapel can be concluded to have nominal ductility in the north-south direction. The steel portal frames have thus been assessed with an assumed ductility of μ=1.25. The gypsum board ceiling and wall bracing have been assigned a ductility of μ=3.3.

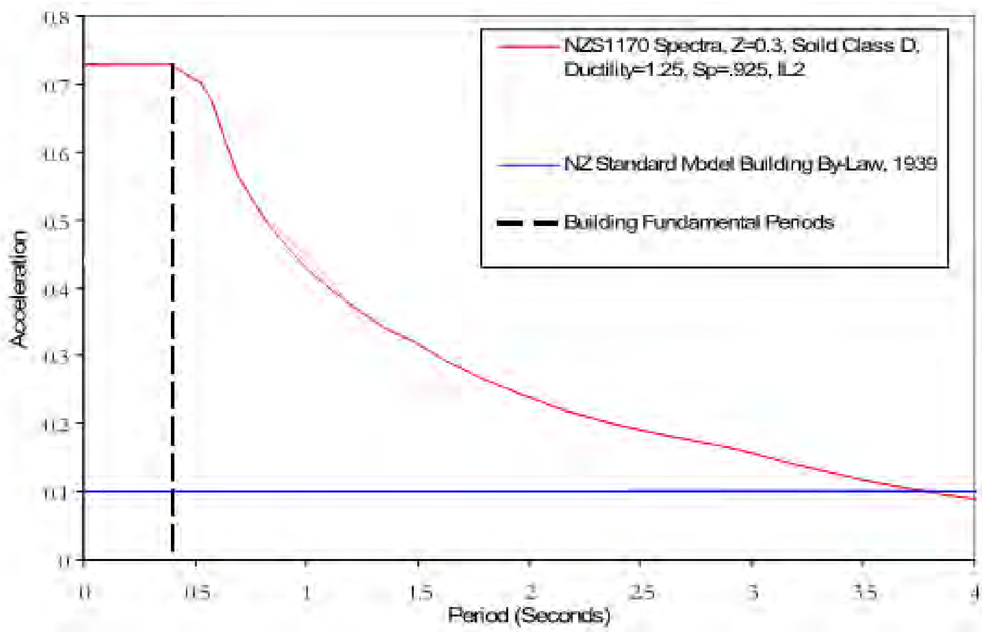


Figure 2-6: Comparison of Design Codes

A comparison of the design load levels for the steel portal frames of the building is plotted in Figure 2-6. The figure shows that, based upon a fundamental building period below 0.5 seconds, if the steel portal frames of the building were designed to 100% of the Design Basis Earthquake (DBE) in 1962, they would currently sit at approximately 15% of the current DBE. Empirical methods to calculate the fundamental building period for the Chapel have been performed which suggests that the Chapel's building period is approximately 0.4 seconds, represented by the dotted line in Figure 2-6.

2.4 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 estimated 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. To provide a comparison for each primary structural component, the relative capacity of the elements has been represented as percentage of the current Design Basis Earthquake (%DBE).

Following the Lyttelton earthquake, a geotechnical report was conducted by Tonkin & Taylor, titled "Burwood Hospital Post Earthquake Geotechnical Assessment", dated June 2011. This reports has been used to aid in the evaluation of the site conditions and the likely effect of the ground on the buildings past and future performance. The soil parameters described in the report have also been used for the evaluation of the buildings existing foundation system.

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 [11] and the requirements of NZS 1170.5:2004 [8]. 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. These are described in more detail in the Burwood Hospital Base report [12].

For the purpose of this evaluation several assumptions also had to be made in regards to the existing building properties. Specifically, the existing diaphragm properties of gypsum board ceiling and straight timber board sheathing, along with the bracing capacity of interior and exterior walls, were of primary concern. The expected strength values for these elements were taken from NZSEE 2006, Assessment and Improvement of the Structural Performance of Buildings in Earthquakes [11] and ASCE 41-06, Seismic Rehabilitation of Existing Buildings [11]. The expected diaphragm strength values presented in NZSEE 2006 for straight board sheathing have been divided by 1.5 to account for built in overstrength. This value is based upon the data from the NEHRP ABK Program for which the data in NZSEE 2006 is based. These values could be further refined through destructive investigations of the existing materials. The assumed diaphragm and shear wall factored expected strength values are as follows:

- Exterior Walls: Unblocked timber framed walls with gypsum wallboard on the inside face and either brick veneer or vertical orientated weatherboard on the exterior face. Expected strength = 1.5 kN/m (30 BU/m) $\mu = 3.3$

- Interior Walls: Unblocked timber framed stud walls with gypsum wallboard or fibrous wall board sheathing and plaster finish on two sides. Expected strength = 3.0 kN/m (60 BU/m) $\mu = 3.3$

- Ceiling Diaphragm: Unblocked, gypsum board clad, timber framed ceiling. Expected strength = 1.5 kN/m (30 BU/m) $\mu = 3.3$

- Ground Floor Diaphragms: 1 inch x 4 inch straight timber board sheathing. Expected strength = 2.8 kN/m (56 BU/M) $\mu = 3.5$

- Reinforced Concrete Subfloor Walls: Expected strength = 11.6 kN/m (233 BU/m)

The foundations have been assessed with an ultimate bearing capacity of 150kPa as per the recommendations provided by Tonkin and Taylor.

A summary of the capacity of each primary lateral element as a percentage of the demand imposed by the Design Basis Earthquake (DBE) have been noted in Table 2-1 below.

Building Element	%DBE	Comments
Ceiling Diaphragm - N-S - E-W	100% 100%	
Steel Portal Frames – N-S	35%	Governed by bracing of both the portal frame beams and columns.
Gypsum Bracing Walls – N-S - E-W	45% 40%	
Ground Floor Diaphragm – N-S - E-W	50% 100%	Limited in N-S direction by the spacing of the concrete sub-floor walls.
Concrete Sub-Floor Walls – N-S - E-W	100% 100%	

Table 2-1: Seismic Assessment %DBE

Earthquake strengthening work carried out in April-May 2014 to bring the chapel capacity up to 67% DBE (IL2) is noted in Section 5.

3. POST-EARTHQUAKE BUILDING CONDITION

This section covers the structural damage sustained by the Chapel as a result of the series of earthquakes, including the Darfield Earthquake that struck at 4:36am on 4th September 2010, the Lyttelton Earthquake that struck at 12:51pm on the 22nd February 2011, the June earthquake that struck at 2.20pm on the 13th June 2011 and the December earthquake that struck at 3.18pm on the 23rd December 2011. This section also covers the resultant reduction in lateral load capacity of the building due to the above mentioned earthquakes. The Lyttelton Earthquake subjected the building to strong ground motions which significantly exceed the full design earthquake load for buildings of this nature and appears to have caused the majority of the earthquake damage observed, after the Darfield event.

3.1 THE LYTTELTON EARTHQUAKE

The fundamental period of the building is estimated to be between 0.2 and 0.4 seconds. 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 strong motion data presented in the Base Report, it appears the Lyttelton Earthquake produced ground shaking intensities of approximately 60-120% of the current Ultimate Limit State design spectra for an Importance Level 2 building of nominal ductility.

It should be noted that the Lyttelton Earthquake was very short in terms of strong shaking produced with the strong motion only lasting for duration of 5-7 seconds. Rupture of an alpine fault is expected to contain 50 to 60 seconds of strong motion.

3.2 PRELIMINARY INVESTIGATIONS

Preliminary investigations were carried out to identify 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.
- Review of available documentation.
- Damage observed during 'Rapid' Structural Assessments following the Lyttelton Earthquake, 13th June 2011, aftershocks, 23rd December 2011, and January 2nd, Earthquakes.

Following the review of the drawings, and previous work associated with this building, the following areas were identified for potential damage:

- Connections of timber framing to foundation supports.
- Damage to roof framing at connections to timber framed walls and steel portal frames.
- Cracking to linings of timber framed walls and ceilings.
- Distress to timber framed floor diaphragms.
- Distress to steel portal frame.
- Cracking in continuous concrete footings due to liquefaction induced differential settlement.
- Displacement of ground around perimeter of building.

Rapid Level 2 Assessments were carried out on the 24th February 2011[13] and on the 14th June 2011 [14]. An additional Visual Structural Assessment [15] was completed on the 5th January, 2012 following the 23rd December 2011 and 2nd Jan 2012 events. These structural observations involved a complete walk around the exterior and throughout the interior of the building. The following primary areas of damage were identified from the damage assessments:

- Cracking to wall linings in chapel office.
- Signs of movement at corners of the nave.
- Cracks in plaster off upper corners of window and door openings.

A review of the above information on the building type and preliminary observations highlighted this building as requiring a detailed inspection. The aim of the detailed inspection was to determine the cause and full extent of damage to the building, particularly the elements identified for potential damage above. These items were targeted to identify if damage had occurred and to what extent the damage had reduced the capacity of the buildings lateral load resisting system to withstand future seismic events.

3.3 DETAILED OBSERVATIONS

Further detailed inspections and structural explorations have been carried out following the initial assessments to ascertain the full extent of structural damage. A detailed structural observation was completed on the 24th April, 2012. A full record of these observations can be found in Appendix A. Reference plans describing the location labelling used can be found in Appendix B. A full photographic record of the observations is available electronically on request. The detailed structural observation identified the following additional damage to those items noted in the initial rapid assessments:

- Additional occurrences of cracking of internal wall and ceiling linings, corners and openings.
- Stepped cracking in brick façade, including associated minor cracking in the reinforced concrete sub-floor walls below.

- Spalling of concrete finish around corners of strip footings.
- Top level bricks on external façade have been dislodged.
- Fracture in bricks underneath window sill.
- Cracking and separation in timber floor panels.
- Visual displacement of clay roof tiles in isolated locations.

3.4 GEOTECHNICAL REVIEW

A review of the ground damage and conditions was carried out by Tonkin & Taylor for the Burwood Hospital Campus. A subsequent report titled Burwood Hospital Post Earthquake Geotechnical Assessment was issued in June 2011. The geotechnical review concluded that the settlement and damage to building foundations on the Burwood Hospital Campus was likely due to the liquefaction of underlying soil layers. It is believed that excessive pore water pressures have been dissipated and that further settlement is not expected to occur, unless another significant event was to occur.

Based on the geotechnical report provided by Tonkin & Taylor the potential for future total and differential settlements at the building site varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

3.5 LEVEL SURVEY

A detailed survey of the ground floor levels in the Chapel was conducted by Fox & Associates and issued on 18th April, 2012. The survey indicates significant differential ground settlement in the south-east corner of the building. The worst differential settlement occurs in the area just west of room G2, where the floor drops 23mm over a distance of approximately 4.3m (1:190 or 0.53%).

The resultant slopes in the ground floor as a result of the differential settlement experienced by the building exceed the typical acceptable range for a building of this type, and thus require remediated. Further discussion on re-levelling is included in Section 4.1.

A discussion on re-levelling on a campus wide basis is also included in the Burwood Hospital Campus base report. This includes a study on the affect of re-levelling individual buildings on the serviceability of the hospital campus as a whole.

For the full extent of differential settlement noted to the building see Appendix C: Survey of Levels.

3.6 SUMMARY OF BUILDING DAMAGE

The following is a summary of the observations made for the building, and our conclusions as to their condition and seismic load resisting capacity. These observations do not specifically distinguish between damage caused by the Darfield Earthquake, the Lyttelton Earthquake, or any significant aftershocks thereafter, such as those that occurred on 13th June 2011, 23rd December 2011 and 2nd January 2012. Despite not being able to specifically distinguish when individual damage observed occurred, it is believed that the majority of the damaged can be linked to the February 22nd event.

The Chapel appears to have performed as would be expected for a building of this type and age. The bulk of structural damage is typified by cracking of the linings on the timber framed walls and ceilings. The structural damage sustained by the building as a whole would be categorized as minor to moderate due to the reduction in lateral capacity of the building caused by the cracking of the linings to the timber walls (and loosening of fixings), which provide the gravity and lateral support to the building. A summary of the typical damage observed is as follows:

- **Differential Ground Settlement** – Differential settlement of up to 23mm over a distance of 4.3m (1:190 or 0.53%) has been noted in the South-East corner of the building.

- **Cracking of Wall Finishes** – Cracking, and general distress has been noted to internal and external wall linings, primarily at corners, openings and along wall board joints. Based upon the movements observed it is believe the wall board fixings have been damaged as well.

- **Cracking of Ceiling Finishes** – Cracking, and general distress has been noted to ceiling linings, primarily at corners, openings and along wall board joints. Base upon the movements observed it is believe the wall board fixings have been damaged as well.

- **Distress to Ground Floor Diaphragm** - Cracking and minor separation has been noted between timber floor boards.

- **Damage to Concrete Sub-floor Walls** - Settlement induced cracking has occurred in the concrete sub-floor walls. In addition, spalling has been noted in concrete finishes of the sub-floor walls.

- **Cracking of Exterior Brick Veneer** - Stepped cracking has been noted in external brickwork around areas of changing geometry (i.e. around corners). The dislodgement of the top layer of bricks has also been noted in various locations in addition to localised cracking of bricks below window sills.

- **Clay Roof Tiles** – The clay roof tiles have become visually dislodged in several places.

- **Damage to Non-structural Elements** - Cracking to non-structural elements such as window reveals, door jambs and finishes.

Table 4-1 provides a photographic summary of the typical damage observed. A full record of our detailed observations and repairs required can be found in Appendix A.

3.7 FURTHER INVESTIGATIONS REQUIRED

3.7.1 Investigations Required For Further Assessment

Several assumptions were made in the completion of the pre-earthquake (undamaged state) and post-earthquake (damaged state) structural assessments. Destructive exploration is required in a number of locations in order to verify these assumptions. The areas requiring further investigation to finalize the assessments are as follows:

- Localised removal of the ceiling finishes is required in the main nave area to determine the existing roof framing assembly. This includes confirming the size, and orientation of the timber roof battens, along with the fixings of the battens to the timber roof purlins. Confirmation is also required for the connection of the inset gypsum board ceiling linings to the timber roof purlins.

The roof has been reconstructed with new battens and 90x45 blocking between rafters for plywood nailing. The ceiling lining has also been replaced during strengthening work in April-May 2014.

- An isolated section of the timber cladding is required to be removed at the roof purlin to steel portal frame beam connection, in order to determine the existing fixing.

The roof purlins are bolted through steel cleats welded to the top of the portal beams. Assessed capacities remain as reported.

- Localised removal of ceiling finishes is required in the nave and the flat ceiling of Vestry (room G2) in order to confirm the connection of the roof framing to the exterior bracing walls.

CF40 connection plates have been installed on all bracing walls during strengthening work in April-May 2014 so the full bracing capacity of the wall linings is utilised.

- Removal of isolated wall finishes and floor finishes is required to confirm that the fixings of the exterior bracing walls and the ground floor framing to the exterior concrete sub-floor walls conforms to the requirements of NZS3604:1999*,* New Zealand Standard Code for Timber-Framed Buildings, as assumed. Likewise the connection between the ground floor timber bearers to the isolated interior concrete piles needs to be confirmed.

During plywood wall relining, extra CF40 connector plates between the wall framing and foundation walls were installed during strengthening work in April-May 2014 to utilise the full capacity of the bracing. The connection between the bearers and piles is made with wire ties.

- Further investigations are required into the post-earthquake condition of the existing clay roof tile assembly by a qualified roofing contractor to determine the full extent of the earthquake damage sustained. The report should also include associated roof repair or replacement recommendations.

The heavy tile roof was removed during strengthening work in April-May 2014.

- Further investigations are required into the post-earthquake condition of the exterior brick veneer by a qualified mason to determine the full extent of the earthquake damage and repairs required.

The brick veneer was removed during strengthening work in April-May 2014.

3.7.2 Investigations Completed During Building Repair

- The brick veneer ties are to be assessed during the repair of the damaged brick veneer to confirm they meet the requirements of the New Zealand Standard, Built in Components for Masonry Construction, Part 1: Wall Ties, NZS 2699.1, as assumed.

The brick veneer was removed during strengthening work in April-May 2014.

3.8 POST-EARTHQUAKE BUILDING CAPACITY

Post-earthquake, based upon our observations to date, we do not consider the Chapel to have any significant reduction in gravity load resistance. The damage observed to the interior wall and ceiling linings will have resulted in some reduction in lateral load capacity, although it is difficult to quantify the percentage reduction in strength. While there has been some reduction in strength, according to the Department of Building and Housings, Revised Guidance on

Repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence [13], the primarily result of the damage noted will be a reduction in the stiffness of the wall bracing. The reduction in stiffness will cause ongoing concerns in regards to the buildings performance, primarily to contents and non-structural elements, including the clay roof tile assembly and exterior brick facade. There will also be some addition reduction in capacity due to the differential ground settlement observed.

The damage observed will require repair to restore the strength, stiffness, durability and performance of lateral bracing system. The repair work required to reinstate the building to pre-earthquake levels is outlined in Section 4. Following the recommended repairs to the structural damage noted, the lateral load capacity of the existing structure will be restored to close to the earthquake levels, which are summarised in Section 2.4.

Repair and strengthening work was carried out in April-May of 2014 to improve the seismic performance and bring the building above 67% DBE as noted in Section 5.

4. OBSERVED DAMAGE AND REQUIRED REPAIRS

4.1 PRIMARY DAMAGE OBSERVED AND REPAIRS REQUIRED

This section covers the damage noted during our detailed assessment of the building. Note that our observations have been restricted to structural aspects of the building 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.

Table 4-1 provides a photographic summary of the observed damage and typical repairs required. The table should be read in conjunction with Appendix A – Record of Observations and Appendix B – Location Reference Plans. The Repair Specification referred to in Table 4-1 has been issued separately.

In general the aim of the repair work indicated is to restore the structure to its pre-earthquake state as close as practicable. However, please note that based upon the extent of the repairs required, and the low % DBE of the building in its pre-earthquake, undamaged state, we would recommend that any repairs be combined with a strengthening scheme to improve the performance of the building.

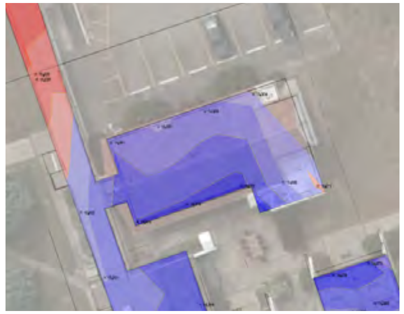
It should be noted that more damage may be identified during the repair works and (if required) additional repair details will be specified accordingly.

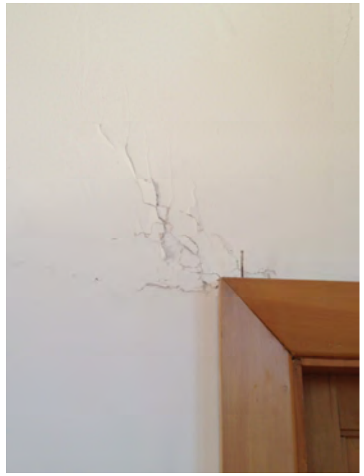
Please note that all repair works are to be completed after any re-levelling work to the building has been completed to a satisfactory condition, as further damage to the wall and ceiling linings can be expected during the re-levelling process.

Further recommendations for improvement to the buildings seismic performance, and to achieve a minimum capacity of 67% DBE have been included in Section 4.

The Earthquake damage repair work carried out in April-May 2014 in conjunction with the strengthening work to 67% DBE (IL2) is noted in Table 4-1 and Section 5.

Table 4-1: Photographic Summary of Primary Damage Observed and Repairs Required

Damaged Item	Photo Ref: Location	Recommended Repair	Example
1.) Areas of differential settlement have resulted in sloping within the ground floor timber framing of up to 0.53% (1:190), in the south-east corner of the building.	Refer: Appendix C - Survey of Levels.	Remediation of floor levels requires localised lifting of the structure. See section 4.2 for additional information. **29.05.14** – Timber flooring packed and re-levelled.	
2.) Cracking to sub-floor concrete walls.	Refer: Appendix A- Record of Observations.	Epoxy inject cracks that are less than 1mm, in accordance with HCG specification. For cracks greater than 1mm, HCG to confirm the integrity of the reinforcement at top and bottom of wall. If reinforcement is damaged, an engineered repair will be required. Refer to HCG specification. **29.05.14** – Sub-floor concrete walls repaired in accordance with HCG specification.	 

Damaged Item	Photo Ref: Location	Recommended Repair	Example
3.) Spalling of concrete finish to reinforced concrete strip footings.	Refer: Appendix A-Record of Observations.	Remove any loose and spalling concrete. Replace finish with new concrete to ensure adequate cover to reinforcement remains. **29.05.14** – Isolated areas of spalling plaster removed and reinstated with plaster to match existing finish.	
4.) Cracking to internal wall and ceiling linings at numerous locations.	Refer: Appendix A-Record of Observations.	Replace all cracked or damaged wall and ceiling boards with new gypsum sheets. All wall and ceiling boards to remain are to be re-fixed as per Section 4.3 and 4.4. For additional strengthening options, see Section 5. **29.05.14** – Wall and ceiling cladding replaced with Plywood.	 

Damaged Item	Photo Ref: Location	Recommended Repair	Example
5) Stepped cracking in mortar, fracture of brickwork and dislodged bricks.	Refer: Appendix A- Record of Observations.	Further investigation is required to determine the full extent of the earthquake damage to the exterior brick façade. See Section 4.6 for additional information. **29.05.14** – Brick veneer removed and replaced with plywood and weatherboard cladding.	  
6) Dislodged Clay Roof Tiles.	Refer: Appendix A- Record of Observations.	Further investigation of the clay roof tile assembly is required to determine the extent of repair or replacement required. See Section 4.5 for additional information. **29.05.14** – Clay tile roof removed and replaced with lightweight metal roofing.	

4.2 DISCUSSION ON BUILDING RE-LEVELLING

The level survey, completed by Fox & Associates has indicated areas of the building which sustained significant earthquake induced differential settlement. Whilst differential settlement has been noted throughout the building (see Appendix C for complete level survey) the worst differential settlement recorded has occurred in the south-east corner of the building, where a drop of 23mm in the floor framing over a distance of 4.3m (1:190 or 0.53%) has been recorded.

Remediation of the floor levels is required in the most affected areas to bring the ground floor framing back to level, and could be achieved through the use of mechanical jacking. If mechanical jacking is pursued it would involve disconnecting the Chapel superstructure from the sub-structure, jacking the ground floor up to a level position, and then reconnecting the floor to the concrete sub-floor walls, internal pier footings.

During the re-levelling process there is a risk that addition damage could occur to the buildings linings, exterior block veneer, etc. and appropriate contingencies should be provided.

A discussion on re-levelling on a campus wide basis is also included in the Burwood Hospital campus base report. This includes a study on the affect of re-levelling individual buildings on the serviceability of the hospital campus as a whole.

For the extent of the work proposed see Figure 4-1 below.

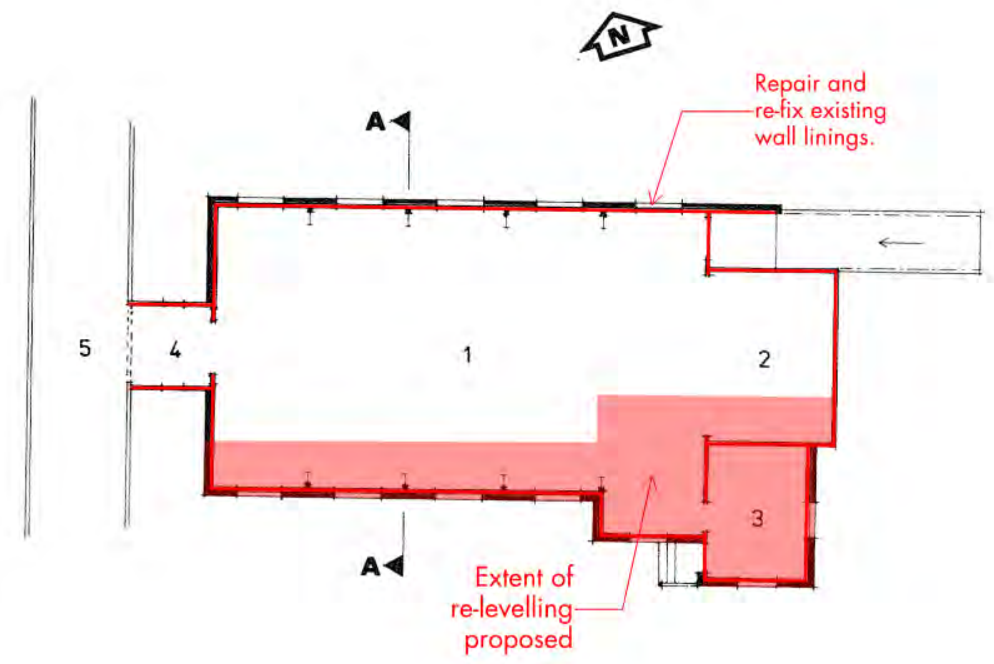


Figure 4-1: Ground Floor Plan – Repairs Required

It should be noted that re-levelling the building through the use of mechanical jacking will not reduce the potential for future differential settlements. The ground conditions under the building will remain roughly as they were prior to the earthquakes. Based up the geotechnical report provided by Tonkin & Taylor [4] the potential for future total and differential settlements at the building site would remain between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

29.05.14 - The South-East corner floor of the building was packed and re-levelled during the strengthening work carried out in April-May 2014.

4.3 REPAIR OF WALL BRACING

The wall linings to the interior and exterior bracing walls have been damaged in locations and require repair. Based upon the movement observed it is also believed the wall lining fixings have been damaged throughout. We believe this has resulted in a reduction to the ongoing strength and stiffness of all the bracing walls. In order to reinstate the pre-earthquake strength and stiffness to the bracing walls, the repair recommendation is to remove all cracked or damaged sections of the wall linings and replace them with new gypsum board sheathing. The new gypsum board sheathing is to be fixed in accordance with GIB 'ezybrace' GS2-N specifications (or equivalent). All existing internal wall linings to remain are to be re-fixed to the existing studs in a similar manner. Any non-gypsum wall boards will need to be replaced in conjunction with these repairs. A new finish is then to be applied to all interior walls.

All repairs to wall bracing are to be completed after the re-levelling and repair of the footings is complete. Refer to figure 4-1 for extent of wall repairs.

29.05.14 - Bracing walls were relined with plywood during the strengthening work carried out in April-May 2014.

4.4 REPAIR OF CEILING DIAPHRAGMS

Similarly to the wall linings, the ceiling diaphragm and its fixings have been damaged and require repair. The repair recommendation is to remove any cracked or damaged sections of ceiling lining and replace with new gypsum wallboard sheathing fixed in accordance with GIB specifications. All existing ceiling linings that are undamaged are to be re-screwed to existing ceiling joists. A new finish is then to be applied to all ceilings.

All repairs to the ceiling diaphragms are to be completed after the re-levelling and repair of the footings. See Figure 4-2 for extent of ceiling repairs.

29.05.14 - A new plywood roof diaphragm was installed during the strengthening work in April-May 2014.

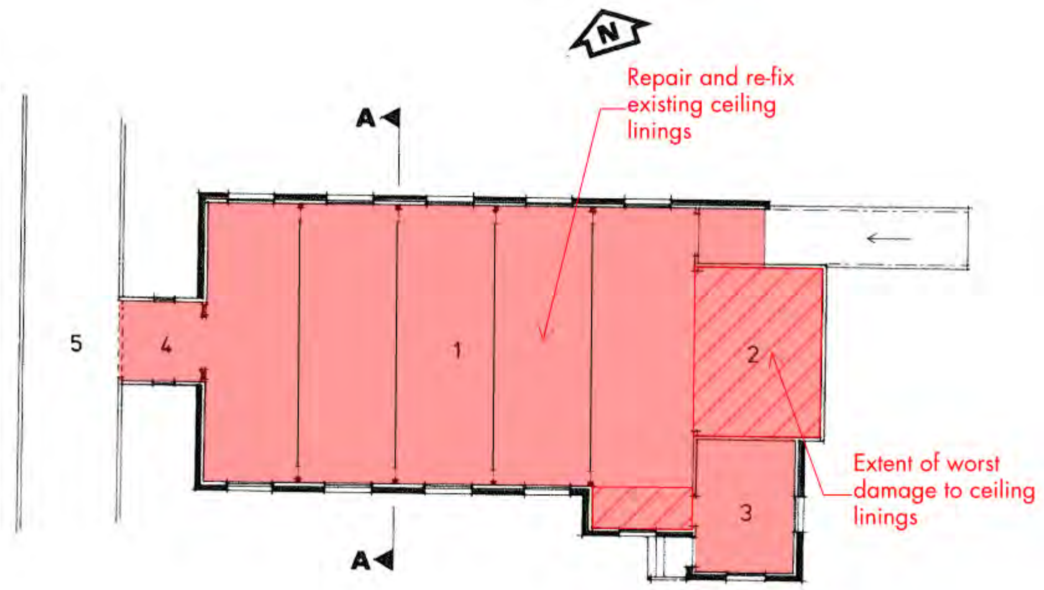


Figure 4-2: Reflected Ceiling Plan – Repairs Required

4.5 REPAIR/INVESTIGATION OF CLAY ROOF TILES

Exterior visual observations appear to indicate that some existing clay roof tiles have dislodged, particularly on the north side of the building. Based upon the visual observations, further investigation into the existing clay is assembly is required by a qualified roofing contractor to determine the full extent of the earthquake damage along with repair or replacement recommendations. As the existing clay roof tiles are believed to be at risk of being dislodged and "shed" during a significant seismic event, we would recommend the additional investigation be completed as soon as is practical.

If the investigation calls for replacement of the existing roof assembly, we would recommend the roof be replaced with a light-weight alternative, such as a standing seem metal roof over a layer of plywood sheathing, in lieu of in kind material. This could reduce the seismic demands by up to 25%, increasing the assessed capacity of the building (in its current state) up to approximately 45% DBE.

29.05.14 - Clay tiles were removed during the strengthening work in April-May 2014.

4.6 REPAIR/INVESTIGATION OF BRICK FACADE

The external brick work which makes up the façade of the building has been damaged and requires repair. Stepped cracking has been noted in external brickwork around areas of changing geometry (i.e. around corners). Dislodgement of the top layer of bricks has also been noted in various locations around the building in addition to localised cracking of bricks below window sills.

Based upon the extent of damage observed, a more detailed investigation of the exterior brick veneer should be completed by a qualified Mason. The Mason should investigate the overall condition of the brick veneer, localized damage to individual bricks and mortar joints, along with investigating the type, size, spacing and condition of the existing brick ties.

29.05.14 - Brick veneer was removed during the strengthening work in April-May 2014.

The main lateral load resisting system of the Chapel is provided by internal steel portal frames, which form the nave and exterior timber framed bracing walls. Lateral loads are distributed to the portal frames and bracing walls by the gypsum board clad ceiling diaphragm. At the ground floor level the straight board sheathed floor diaphragm distributes load to the external concrete sub-floor walls and footings.

As noted in Section 2 and Section 3, the lateral load resisting capacity of the building, in its Pre-Earthquake and Post-Earthquake condition, was assessed (as a percentage of the loads imposed by the Design Basis Earthquake) at approximately 35% DBE. This is just above the limit for which the building would be considered Earthquake Prone.

Strengthening work was carried out during April-May 2014 to improve the buildings seismic performance and bring the capacity of the entire building above 67% DBE (IL2) as outlined in Section 5.1.

5.1 STRENGTHENING WORKS TO ACHIEVE 67% DBE (IL2)

Strengthening work to bring the capacity of the Chapel up to 67% DBE (IL2) was carried out during April-May 2014. This work was completed in accordance with CDHB Burwood Hospital Chapel Strengthening - Construction Issue Drawings [16] and Burwood Hospital Chapel Repairs – Construction Issue Drawings [17]. This work included:

- Improving the condition of the portal frame baseplate connections to the foundation walls
- Installing fly braces on the portal frames
- Relining bracing walls with plywood
- Installing a plywood roof diaphragm

As part of this work, the brick veneer was removed and replaced with lightweight cladding and the heavy roof tiles replaced with lightweight metal roofing. This has significantly reduced the seismic weight of the building.

Additional Ground-Floor Wall Bracing

Additional exterior wall bracing was installed at the ground floor level in order to bring the assessed capacity of the building above 67% DBE. This includes the exterior walls running in the East-West direction along with the gable end walls running in the North-South direction of the building. Foundation fixings have been provided for all walls.

The additional wall bracing as part of the strengthening work during April-May consists of new plywood sheathing applied to the outside face of the exterior walls. The exterior brick veneer has been removed to allow for this.

Concrete Sub-Floor Walls and Ground Floor Diaphragms

As noted in Section 2, the concrete sub-floor walls were assessed at approximately 100% DBE in the North-South and East-West directions. However, the spacing of the sub-floor walls also directly effects the demands imposed on the ground floor diaphragms, which as noted in Section 2, had been assessed below 67% DBE in the North-South direction. The New Zealand Standard Code for Timber-Framed Buildings NZS3604:2011 [18] also notes a maximum spacing of 5 meters for sub-floor bracing walls. As such, new concrete sub-floor walls (or timber bracing between the existing piles) are recommended.

The removal of the brick veneer and clay roof tiles as part of the strengthening work during April-May reduced the weight of the building such that the ground floor diaphragm capacity would be 67% DBE (IL2).

Steel Portal Frame Strengthening N-S

The internal steel portal frames in the North-South direction of the building had been assessed at approximately 35% DBE in their pre-earthquake undamaged state. The assessed percentage of the frames was governed by onset of lateral buckling of the portal frame beams and columns. The earthquake strengthening work included removing the existing heavy roof and replacing it with a light weight alternative. This reduced the load on the steel portal frames which form the primary lateral resisting elements in the north-south direction, increasing their capacity to approximately 45% DBE (IL2).

Fly braces were installed as part of the strengthening work during April-May 2014 to prevent lateral buckling of the steel frames. The capacity is now 80% DBE (IL2).

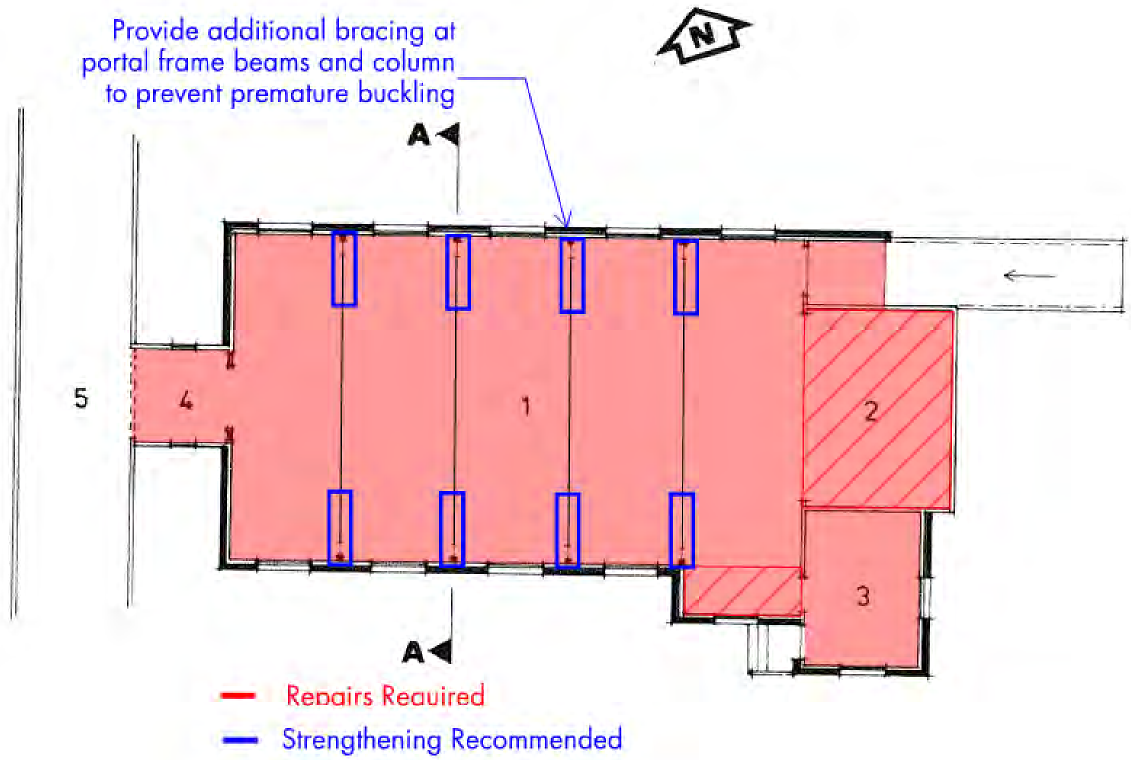


Figure 5-1: Reflected Ceiling Plan – Strengthening with Fly Braces

Wall Bracing Strengthening

The gypsum walls had been assessed at approximately 45% DBE and 40% DBE in the North-South and East-West directions respectively.

By replacing the gypsum walls with plywood walls as part of the strengthening work during April-May 2014 the capacity is now 85% DBE (IL2).

Summary

A summary of the capacity of each primary lateral element as a percentage of the demand imposed by the IL2 Design Basis Earthquake (DBE) following the completion of the strengthening work have been noted in Table 5-1 below.

Building Element	%DBE (IL2)	Comments
Ceiling Diaphragm - N-S - E-W	100% 100%	
Steel Portal Frames – N-S	80%	Steel Portal Frames were strengthened using fly braces.
Bracing Walls – N-S - E-W	85% 85%	Gypsum walls were replaced with plywood bracing walls.
Ground Floor Diaphragm – N-S - E-W	67% >100%	Removal of the heavy tile roof and brick veneer increased the capacity from 50% DBE to 67% DBE.
Concrete Sub-Floor Walls – N-S - E-W	>100% >100%	

Table 5-1: Seismic Assessment %DBE (IL2) – Following the strengthening April-May 2014

If the building were to be assessed for an increased importance factor, IL3, the seismic demand would increase by 30% (R=1.3) and as such the assessed capacities would be reduced proportionally.

6. REFERENCES

1. *CDHB – Burwood Hospital Chapel Relocation: Plan and Details,* Powell Fenwick Consultants, 2001.

2. *CDHB – Chapel Master Floor Plan,* Maintenance and Engineering Department – Christchurch Hospital, 2009.

3. *Burwood Hospital Christchurch – Survey of Existing Building – Chapel,* Cutter Pickmere Douglas Architects, 1976.

4. *Burwood Hospital Post Earthquake Geotechnical Assessment*, Tonkin and Taylor Ltd. June 2011.

5. *CDHB – Chapel Level Survey*, Fox & Associates, April 2012.

6. *NZS2699.2:2000, New Zealand Standard Code For Built in Components for Masonry Construction*

7. *NZS3604:1999, New Zealand Standard Code for Timber-Framed Buildings.*

8. *AS/NZS1170.5:2004, Australian/New Zealand Standards for Structural Design Actions.*

9. Department of Building and Housing, *Compliance Document for New Zealand Building Code - Clause B1 – Structure, Amendment 10 (Canterbury)*, Department of Building and Housing, Wellington, 19 May 2011.

10. *NZSS95:1939, New Zealand Standard Model Building By-Law.*

11. Department of Building and Housing, *Revised guidance on repairing and rebuilding houses affected by the Canterbury earthquake sequence,* Wellington, November 2011.

12. *Burwood Campus – Detailed Seismic Assessment Report – Base Report*, Holmes Consulting Group, November 2011.

13. *CDHB – Burwood Hospital Campus – Rapid Visual Inspection:* 106186.03SR1, Holmes Consulting Group, February 2011.

14. *CDHB – Burwood Hospital Campus – Rapid Seismic Assessment Post June 13th Earthquakes:* 106186.03SR4, Holmes Consulting Group, 14 June 2011.

15. *CDHB – Burwood Hospital Campus – Post Earthquake Rapid Visual Assessment Post June 2nd Jan 2012 5.5M EQ,* Holmes Consulting Group, 9th January 2012.

16. *CDHB Burwood Hospital Chapel Strengthening - Construction Issue Drawings,* Holmes Consulting Group, Issued 21 February 2014.

17. *Burwood Hospital Chapel Repairs – Construction Issue Drawings*, Shepard & Rout Architects LTD, Issued February 2014.

18. *NZS3604:2011, New Zealand Standard Code for Timber-Framed Buildings.*

19. *CHDB – Burwood Campus – Detailed Seismic Assessment Report – Repair Specification*, Holmes Consulting Group, November 2011.

Appendix A

Record of Observations

APPENDIX A – RECORD OF OBSERVATIONS - Chapel

Inspection date: 29/05/2014

KEY	
N	Not Completed
Y	Repair required
F	Further investigation required
C	Repair complete

Note: At the time of the initial inspection the following rooms could not be accessed:
- Vestry (G2)

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G		Entrance	Ceiling	Crack in ceiling panel near the edge of a doorway.	C	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0972
G		Entrance	Ceiling	Crack in ceiling panel near the edge of a doorway.	C	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0973
G	G41	Interior	Wall	Horizontal crack in wall finish panel.	C	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0974
G	G41	Interior	Wall	Horizontal crack in wall finish panel.	C	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0975

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G1	Interior	Wall	Crack in wall lining above corner of doorway to base of window.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0977
G	G1	Interior	Wall	Vertical crack in wall lining full height of wall.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0978
G	G1	Interior	Wall	Vertical crack between vertical interface of plasterboard walls.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0979
G	G1	Interior	Wall	Vertical crack in wall lining above window. Typical for both sides of window and all windows in the building.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0980
G	G1	Interior	Floor	Differential settlement along length of floor boards.	**N**	Floor re-levelling as per Section 4, any damaged floor boards are to be replaced.	0981
G	G1	Interior	Wall and ceiling	Vertical crack between vertical interface of plasterboard walls, as well as seperation between timber cornices, ceiling and walls.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0983
G	G1	Interior	Wall	Horizontal crack in wall lining.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0984

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G1	Interior	Ceiling	Seperation between ceiling panels in false ceiling area.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0985
G	G1	Interior	Wall and ceiling	Vertical crack between vertical interface of plasterboard walls, as well as seperation between timber cornices, ceiling and walls.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0986
G	G1	Interior	Wall	Webbed cracking in wall lining around corner of doorway.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0987
G	G1	Interior	Wall	Various cracks in wall lining.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0988
G	G1	Interior	Wall	Vertical crack in wall lining above an area of additional wall panel finishing.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0989
G	G1	Interior	Wall and ceiling	Crack and minor seperation between wall and ceiling panels.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0990

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G1	Interior	Wall	Horizontal crack in wall lining around a wall return.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0991
G	G1	Interior	Wall and ceiling	Crack in lining where the wall and ceiling panels meet, as well as crack in wall lining at corner of window.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0992
G	G1	Interior	Wall	Horizontal crack in wall lining from bottom corner of window.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0993
G	G1	Interior	Wall	Vertical crack in wall lining underneath middle of window.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0994
G	G1	Interior	Wall	Horizontal crack in wall lining under window to end of wall.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0995
G	G1	Interior	Wall and ceiling	Vertical crack and seperation between vertical interface of wall panels (full height of wall), as well as cracking between ceiling and wall panels.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0996

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G1	Interior	Wall	Vertical crack in wall panel, full height of wall.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0997
G	G1	Interior	Wall	Vertical crack in wall panel, full height of wall.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0998
G	G1	Interior	Wall and ceiling	Cracking and seperation between wall and ceiling panels.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	0999
G	G1	Interior	Ceiling	Crack and minor seperation between ceiling panels, at ridge line.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	1001
G	G1	Interior	Wall and ceiling	Cracking and seperation between wall and ceiling panels.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	1002
G	G1	Interior	Wall	Cracking in wall linings around corners of windows, typical for entire building.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	1003

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G1	Interior	Wall	Vertical crack in wall lining underneath middle of window.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	1004
G	G1	Interior	Wall	Diagonal crack in wall lining out from corner of window, as well as horizontal crack in wall lining beneath window.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	1005
G	G1	Interior	Ceiling	Perpendicular cracks meet in ceiling lining.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	1006
G	G1	Interior	Wall and ceiling	Minor cracks between wall and ceiling panels.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	1007
G	G1	Interior	Wall	Horizontal crack in wall lining out from corner of doorway.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	1008
G	G1	Interior	Wall	Vertical crack in wall lining above archway.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	1009

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G1	Interior	Wall and ceiling	Crack and seperation between wall and ceiling panel connection.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	1010
G	G1	Interior	Wall	Vertical crack between vertical interface of wall panels.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	1011
G	G1	Interior	Floor	Crack in floorboards and seperation between boards due to differential settlement.	**N**	Floor re-levelling as per Section 4, any damaged floor boards are to be replaced.	1012
G	G1	Interior	Floor	Substantial local raising of floor area.	**N**	Floor re-levelling as per Section 4, any damaged floor boards are to be replaced.	1013
G	G1	Interior	Wall	Vertical crack between vertical interface of wall panels.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	1014
G	G1	Interior	Wall and ceiling	Seperation between wall and ceiling panels.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	1015

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G1	Interior	Wall	Crack in wall lining around corner of switch board.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	1016
G	G1	Interior	Wall	Crack in wall lining around corner of doorway.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	1017
G	G1	Interior	Timber window framing	Paint cracking in timber under window.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	1018
G	G1	Interior	Wall	Crack in wall lining above archway of window.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	1019
G	G1	Interior	Wall and timber window framing	Crack in ridge connection in window and in wall lining around connection.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheeting. Any existing boards remaining are to be re-fixed to timber framing. Ridge connection damaged to be examined further and replaced if necessary.	1020
G	G1	Interior	Wall	Crack in wall lining above archway of window.	**C**	All cracked and damaged wall and ceiling boards are to be removed and relined with gypsum board sheathing. Any existing boards remaining are to be re-fixed to timber framing.	1021

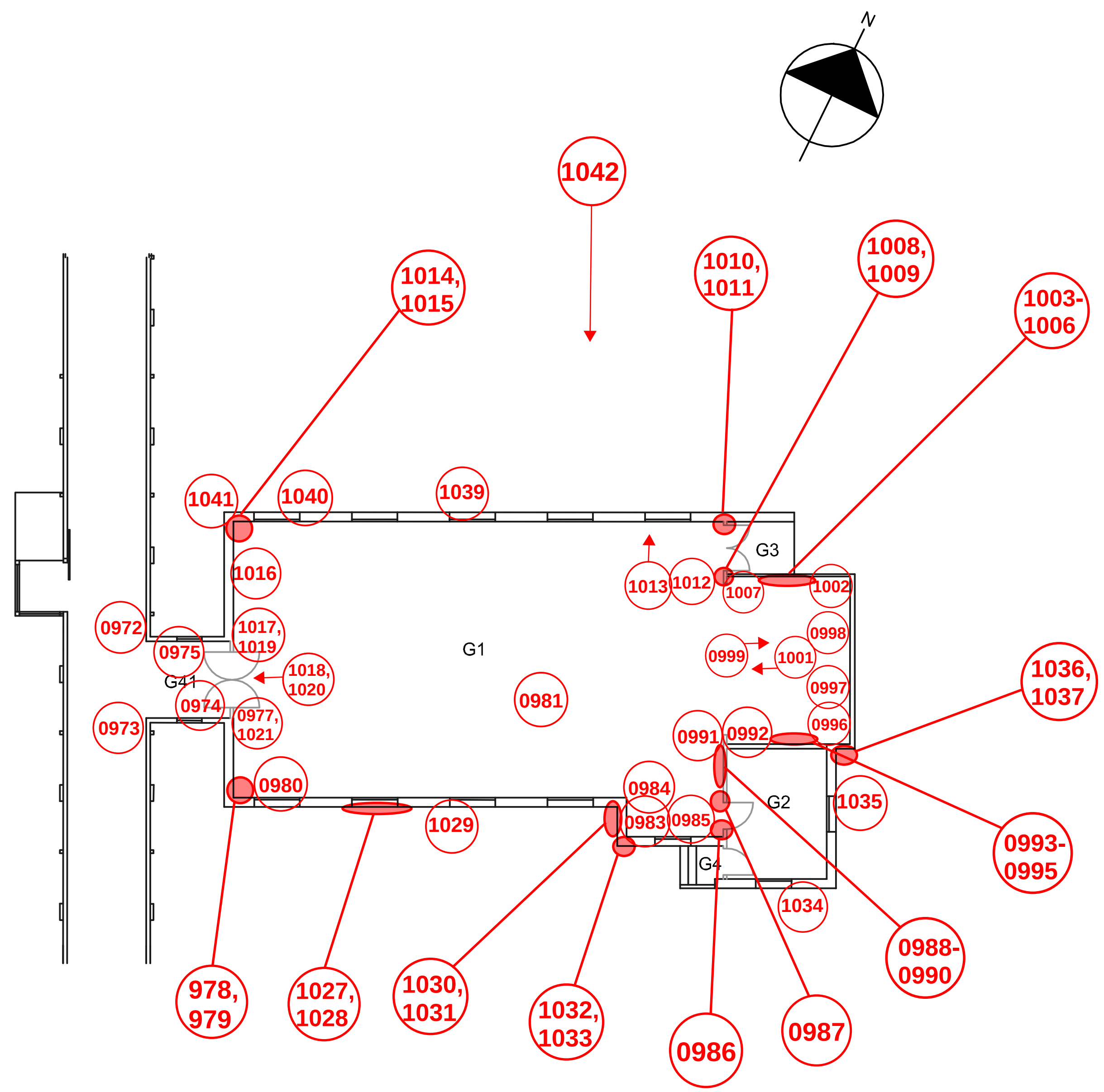
Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G		Exterior	Brick Façade	Fracture to bricks at base of window.	**C**	Ground out and re-point areas of minor cracking in mortar. Fractured or dislodged bricks must be removed and replaced. **29.05.14** - Brick veneer removed and replaced with plywood and weatherboard.	1027
G		Exterior	Brick Façade	Fracture to bricks at base of window.	**C**	Ground out and re-point areas of minor cracking in mortar. Fractured or dislodged bricks must be removed and replaced. **29.05.14** - Brick veneer removed and replaced with plywood and weatherboard.	1028
G		Exterior	Brick Façade	Brick at top corner of window has been dislodged.	**C**	Ground out and re-point areas of minor cracking in mortar. Fractured or dislodged bricks must be removed and replaced. **29.05.14** - Brick veneer removed and replaced with plywood and weatherboard.	1029
G		Exterior	Brick Façade	Stepped cracking in mortar of brickwork.	**C**	Ground out and re-point areas of minor cracking in mortar. Fractured or dislodged bricks must be removed and replaced. **29.05.14** - Brick veneer removed and replaced with plywood and weatherboard.	1030
G		Exterior	Brick Façade	Stepped cracking in mortar of brickwork.	**C**	Ground out and re-point areas of minor cracking in mortar. Fractured or dislodged bricks must be removed and replaced. **29.05.14** - Brick veneer removed and replaced with plywood and weatherboard.	1031

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G		Exterior	Brick Façade	Stepped cracking in mortar of brickwork.	**C**	Ground out and re-point areas of minor cracking in mortar. Fractured or dislodged bricks must be removed and replaced. **29.05.14** - Brick veneer removed and replaced with plywood and weatherboard.	1032
G		Exterior	Strip Footing	Spalling of concrete finish around strip footing.	**C**	Remove any loose and spalling concrete. Replace finish with new concrete to ensure adequate cover to reinforcement remains.	1033
G		Exterior	Brick Façade	Top brick on wall has been dislodged.	**C**	Ground out and re-point areas of minor cracking in mortar. Fractured or dislodged bricks must be removed and replaced. **29.05.14** - Brick veneer removed and replaced with plywood and weatherboard.	1034
G		Exterior	Strip Footing	Minor vertical crack in strip footing.	**C**	For cracks greater than 0.2mm and less than 1mm, epoxy inject cracks in accordance with HCG specification. Cracks>1mm require further investigation to confirm the integrity of the steel reinforcement. Refer to HCG specification.	1035
G		Exterior	Strip Footing	Spalling of concrete finish around strip footing.	**C**	Remove any loose and spalling concrete. Replace finish with new concrete to ensure adequate cover to reinforcement remains.	1036
G		Exterior	External wall panels	Various cracking in wall lining, typical for area.	**C**	Paint finish cracking has not been caused by the series of earthquakes. Any structural cracks in panels will mean that the panels must be replaced and refixed to stud walls, as per Section 4.2.	1037

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G		Exterior	Brick Façade	Top brick on wall has been dislodged.	C	Ground out and re-point areas of minor cracking in mortar. Fractured or dislodged bricks must be removed and replaced. **29.05.14** - Brick veneer removed and replaced with plywood and weatherboard.	1039
G		Exterior	Strip Footing	Horizontal crack in top of footing near connection to external brickwork.	C	For cracks greater than 0.2mm and less than 1mm, epoxy inject cracks in accordance with HCG specification. Cracks>1mm require further investigation to confirm the integrity of the steel reinforcement. Refer to HCG specification.	1040
G		Exterior	Strip Footing	Spalling of concrete finish around strip footing.	C	Remove any loose and spalling concrete. Replace finish with new concrete to ensure adequate cover to reinforcement remains.	-
G		Exterior	Eave	Minor cracking and seperation in corner of eave.	C	Seal gap in eaves with approved sealing compound to maintain weatherproofing.	1041
G		Exterior	Roof	Local area of roof tiles which have had fixings broken.	C	Further investigation of the roof tile fixings is recommended. The investigations should be completed by a qualified roofing contractor. If fixing is damaged then consideration should be made for a new light-weight roof alternative. **29.05.14** - Tiled roof was replaved with a light weight metal roofing.	1042

Appendix B

Reference Plans



AMENDMENT	DRAWN	CHECKED	DATE
MASTER DRAWING CREATED FROM 04000003 AND CDHB BORDER ADDED	BJT		16-06-09

106186.54 Interim DSA Report
Rev3_8September15

BURWOOD HOSPITAL
CHAPEL
MASTER FLOOR PLAN

Canterbury
District Health Board
Te Poari Hauora ō Waitaha

DESIGNED:
DRAWN: BJT
CHECKED:
DATE: 16-06-09

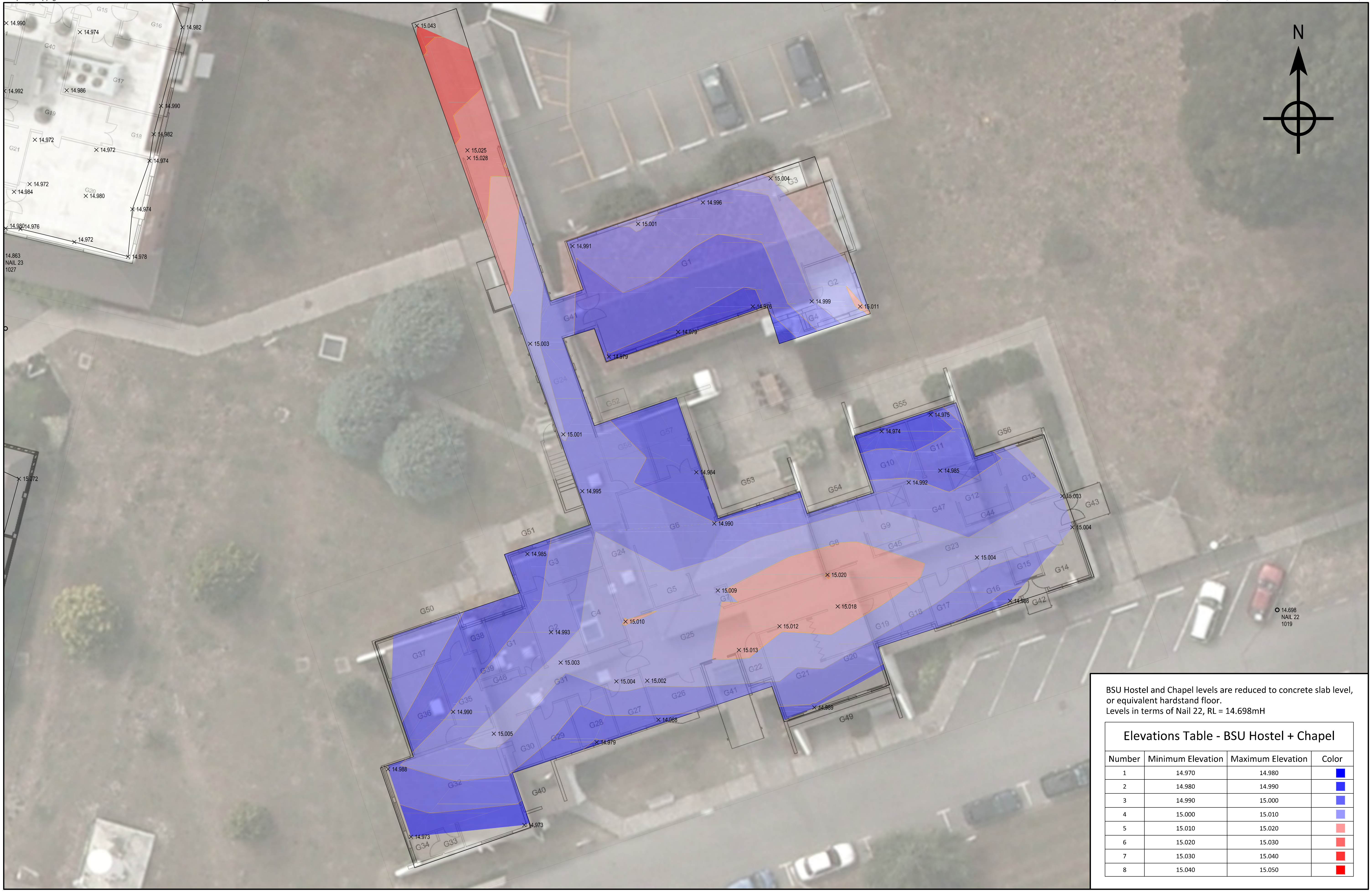
XREFED DRAWING/S:

DWG No. 04082020
SCALE 1:100 AT A1
REVISED 16-06-09

Maintenance and Engineering Department - Christchurch Hospital

Appendix C

Survey of Levels



BSU Hostel and Chapel levels are reduced to concrete slab level, or equivalent hardstand floor.
Levels in terms of Nail 22, RL = 14.698mH

Elevations Table - BSU Hostel + Chapel

Number	Minimum Elevation	Maximum Elevation	Color
1	14.970	14.980	
2	14.980	14.990	
3	14.990	15.000	
4	15.000	15.010	
5	15.010	15.020	
6	15.020	15.030	
7	15.030	15.040	
8	15.040	15.050	

No.	Revision	Approved	Date
G			
F	Beacon House, BSU Hostel, Chapel, Tapper and Milner added	MJM	18/04/2012
E	Audit observations post Dec 23 2011 (not included in this Revision)	MJM	01/02/2012
D	Wards 7 & 8, Surgical Block, Melrose Chairs & Corridor added	MJM	02/12/2011
C	Phys Med, Parafed, Food Services, BIRS, Boilerhouse, EDC added	MJM	24/11/2011
B	Nurses Hostel & Champion Centre, SOU elevations added	MJM	18/11/2011
A	Administration and Allan Bean Centre elevations added	MJM	31/10/2011

032005

Scale		1:125
Reduced	A3	1:250
Designed		----
Drawn		DHP
Checked		MJM
Date		18/04/2012

106186.54 Interim DSA Report
Rev3_8September15

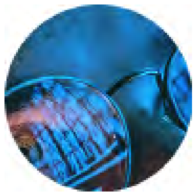
Burwood Elevations Survey

BSU Hostel + Chapel

SURVEYORS
LAND DEVELOPMENT
URBAN DESIGNERS
CONSULTANTS
Fox & Associates
www.foxsurvey.co.nz
0800 FOX SURVEY
P.O.Box 13-390
CHRISTCHURCH

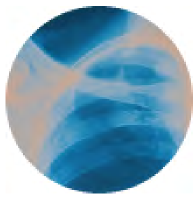
Job No. 2462C
Sheet No. 16 of 19
Rev. F

HolmesConsultingGroup



DETAILED SEISMIC ASSESSMENT REPORT

STRUCTURAL AND CIVIL ENGINEERS



BURWOOD HOSPITAL CAMPUS

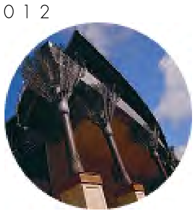
REPORT 21 – ENGINEERING SERVICES BUILIDNG

PREPARED FOR

CANTERBURY DISTRICT HEALTH BOARD

106186.89

INTERIM REPORT REVISION 1 – 14 SEPTEMBER 2012



BURWOOD HOSPITAL CAMPUS– DETAILED SEISMIC ASSESSMENT REPORT

REPORT 21 – ENGINEERING SERVICES BUILDING

Prepared For:
CANTERBURY DISTRICT HEALTH BOARD

Date: 14 September 2012
Project No: 106186.89
Revision No: 1

Prepared By:

Matthew Stewart
PROJECT ENGINEER

Reviewed By:

Jenny Ovens
PROJECT DIRECTOR

Holmes Consulting Group LP
Christchurch Office

REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
14/9/12	1	Interim Report

CONTENTS

APPENDICES

TABLES Page

FIGURES PAGE

EXECUTIVE SUMMARY

Holmes Consulting Group has been engaged by Canterbury District Health Board (CDHB) to complete a full structural review of the Burwood Campus following the Lyttelton Earthquake. A series of reports have been compiled as part of this process. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The specific building reports, like this one for the Engineering Services Building, should be read in conjunction with the base report, and refer to the repair specification.

This report identifies the structural damage sustained by Engineering Services Building as a result of the series of earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The report summarises the effects of the damage on the lateral load capacity of the building and provides structural repairs for the damage identified. The general form of the building, along with its capacity relative to current code levels, has been included for the buildings pre-earthquake undamaged state and post-earthquake state.

The Engineering Services Building was constructed in 1978, and currently contains mechanical equipment in various rooms such as the pump room, switch room, generator room, transformer room, medical gases storage room, and communication network equipment room. Below the north-western side of the building is a subterranean service duct that connects the Spinal Injury Unit to Spinal Injuries Hostel and Ward 7 and 8.

For the purposes of this assessment the Engineering Services building has been considered to be an Importance Level 3 building (IL3).

The primary material used in the building construction is reinforced concrete. This includes insitu concrete floor slabs, along with insitu interior and exterior walls. The northwest portion of the roof is insitu concrete and the southeast portion is timber. The ceiling under the timber roof is lined with GIB board. The roof and floor slabs are two way spanning concrete slabs supported on the concrete walls below. The ground floor slab is supported by a combination of concrete sub-floor and service duct walls, which are founded on shallow strip footings. Interior partition walls in the sprinkler room and communications room are GIB lined timber framing. A 100mm concrete masonry unit veneer on 200mm insitu concrete wall covers the exterior of the building. The exterior wall at the loading dock is constructed of block work. Figures showing building layout including plans, elevations, and sections are in section 2 of this report.

The information available for the review included: the original 1977 structural drawings by Frederick Sheppard and Partners[3], a limited number of architectural drawings by Cutter Pickmere, Douglas [4], a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [5].

The Engineering Services Building has performed relatively well considering the age of construction and the seismic actions experienced at the site. The damage to the building is typified by cracking to the floor slabs and service tunnel wall, and to the exterior block corridor wall. The damage to the service tunnel wall is concentrated around service openings.

Earthquake induced differential settlement have been noted in the structure and also relative to the adjacent corridors and Spinal Injuries Unit. These differential settlements result in slopes in the ground floor of up to 0.45% or 1:220 which are outside the acceptable tolerances specified in NZS3109:1997 [15].

It is believed that the majority of the damage observed, including the onset of damage, occurred as a result of the 22nd February event.

Based upon a review of the drawings available, and the site investigations completed, the primary lateral force resisting elements of the Engineering Services Building were assessed in their pre-earthquake undamaged state. The assessed capacity of the building, relative to the demand imposed by the current loading code Design Basis Earthquake (DBE), is approximately 77% DBE in both the north-south direction and the east-west direction.

The reduction in the lateral capacity of the building due to the earthquake damage observed is hard to quantify. As noted, the primary structure damage to the building is the cracking to the concrete wall and slab elements which will have resulted in some reduction in the capacity of these elements. Upon the repairs recommended in Section 4, these elements will be reinstated to approximately pre-earthquake undamaged levels.

The minimum repairs required to reinstate the building to its pre-earthquake undamaged condition, have been included in Section 4.

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. Secondary elements, such as windows and fittings, have not generally been reviewed.

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 Canterbury District Health Board to complete a full structural review of the Burwood Hospital Campus following the Lyttelton Earthquake. A series of reports have been completed as part of this. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The individual building reports, like this one, should be read in conjunction with the base report and refer to the repair specification.

The Burwood base report covers the purpose and scope of the structural review. The current statutory requirements relevant to earthquake damaged buildings are outlined and the level of shaking experienced at the site estimated. Earthquake induced ground settlement damage across the campus is also discussed. The repair specification has been prepared to include repair details for typical damage observed in buildings on the Burwood Hospital Campus and is referred to as required in the specific building reports.

1.1 SCOPE OF WORK

This report is on the Engineering Services Building, at Burwood Hospital, Mairehau Road, Christchurch. The report identifies the general form of the structure, along with the gravity and lateral load resisting systems. Each component of the structural system was reviewed based upon the information available and any potential Critical Structural Weaknesses (CSW's) were noted.

The report also identifies the structural damage observed to date as a result of the series of Earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the Christchurch Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake, in particular, subjected the building to strong ground motions which were likely to have exceeded the current code loading demand for buildings of this nature.

The capacity of the Engineering Services Building has been assessed relative to current code loading in the buildings pre#earthquake undamaged state and in its post#earthquake damaged state. The post#earthquake assessment summarizes the effects of the damage identified on both the gravity and lateral load resisting elements. Repair options to restore the buildings capacity to pre#earthquake levels for strength, durability and stiffness have been included. The repair options aim to maintain the buildings utility.

1.2 LIMITATIONS

Findings presented as a part of this project are for the sole use of the Canterbury District Health Board, 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. Because all of the structure has not been available for detailed inspection or evaluation, this report is limited to those elements available and engineering judgement as to the likely condition of unseen elements. 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.

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 of the building have been completed.

2. PRE-EARTHQUAKE BUILDING CONDITION

The information available for the review included: the original 1977 structural drawings [3], a limited number of architectural drawings [4], and a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [5].

2.1 BUILDING FORM

The Engineering Services Building is a single story concrete building located at the Canterbury District Health Board (CDHB) Burwood Hospital Campus, approximately 7 km north-east of downtown Christchurch.

The Engineering Services Building was constructed in 1978, and currently houses mechanical equipment in various rooms such as the pump room, switch room, generator room, transformer room, medical gases storage room, and communication network equipment room. On the north-western side of the building, under the corridor, a subterranean service duct connects the Spinal Injury Unit to Spinal Injuries Hostel and Ward 7 and 8.

Figure 2-1 shows a plan view of the building and surrounding area. Figures 2-2 and 2-3 show the original architectural ground floor and roof plans respectively.

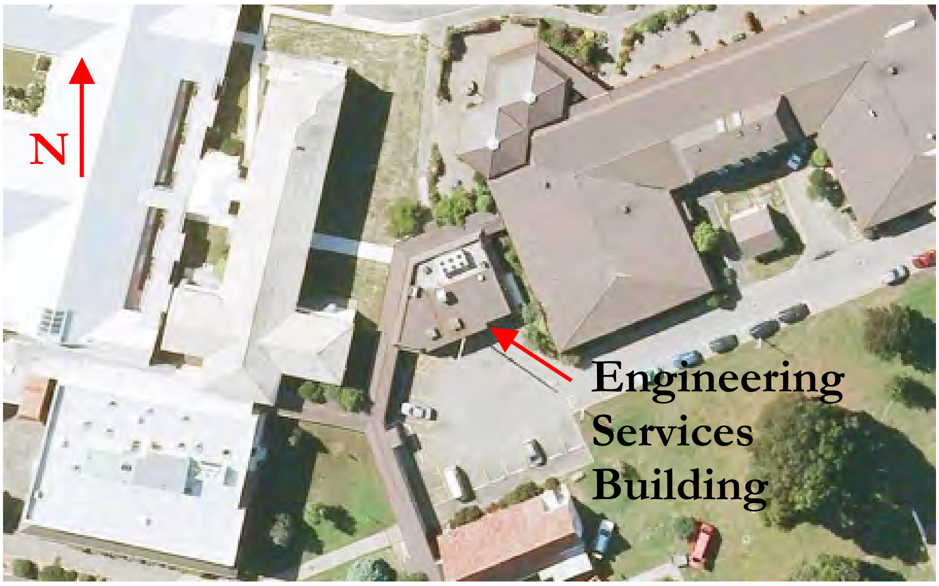


Figure 2-1: Engineering Services – Plan View

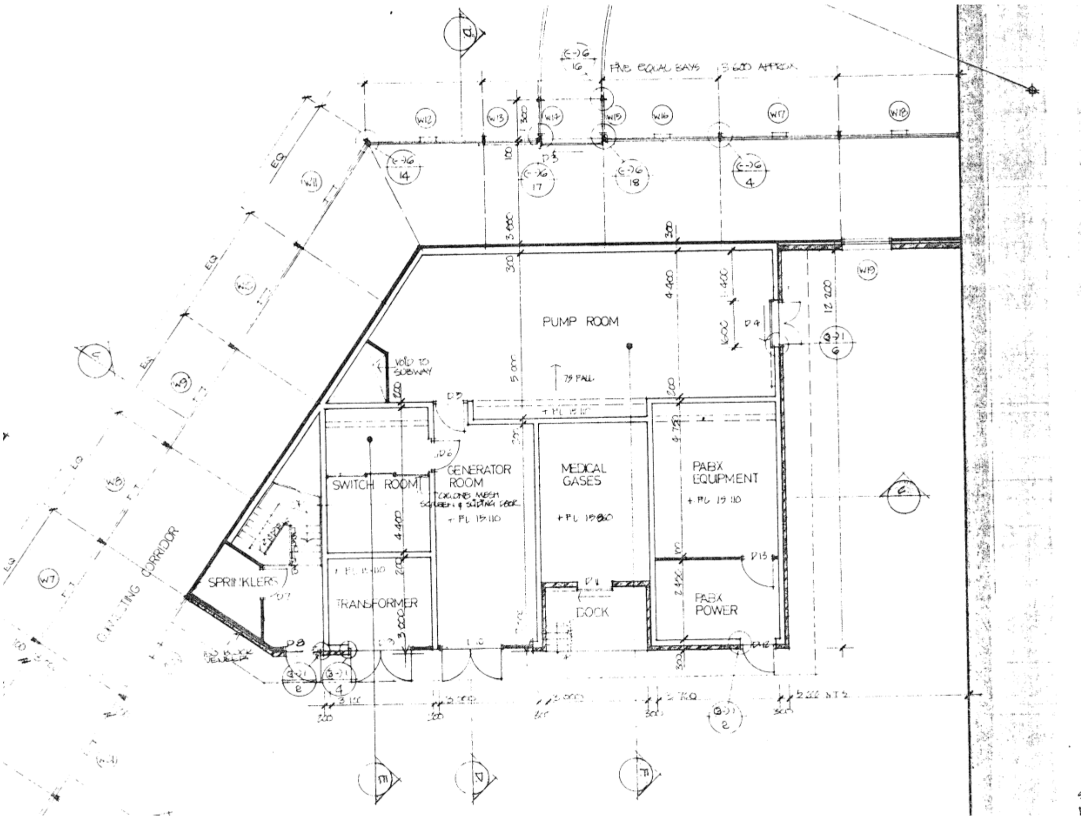


Figure 2-2: Engineering Services – Original Ground Floor Plan

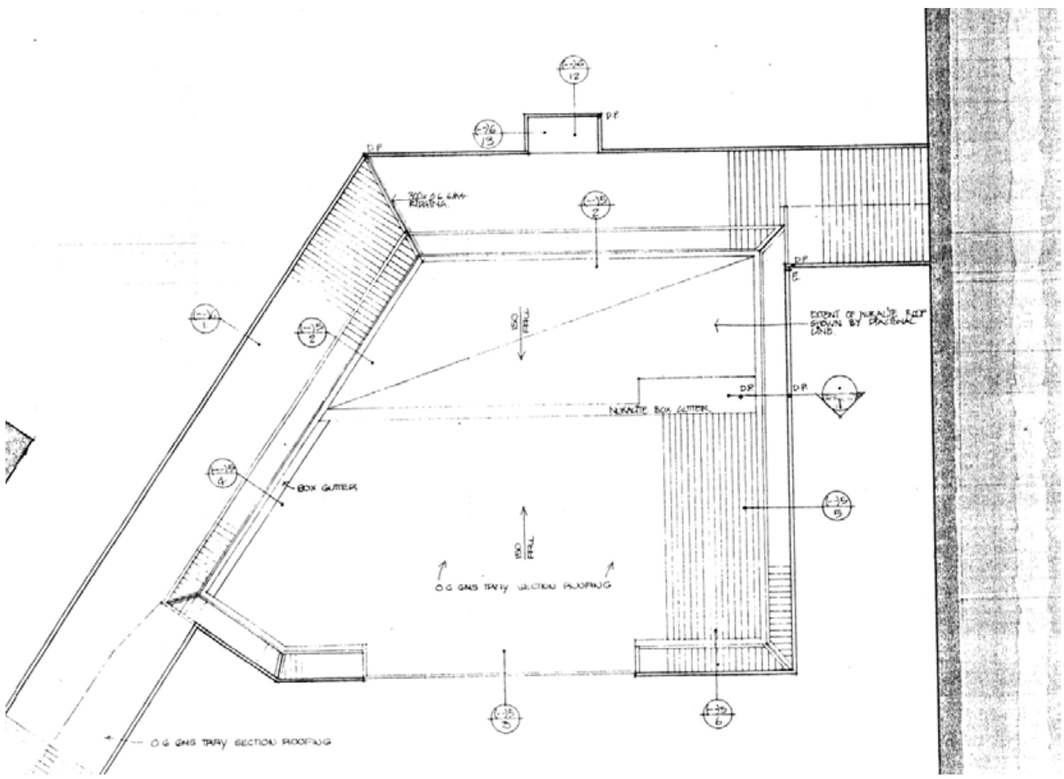


Figure 2-3: Engineering Services – Original Roof Plan

The building is primarily constructed of reinforced concrete. This includes 200mm insitu concrete floor slabs, along with 200mm insitu interior and exterior walls. A northwest portion of the roof is insitu concrete and the southeast portion is timber. The 150mm roof slab supported by the insitu concrete walls below. The 200mm ground floor slab is supported by a combination of concrete sub‐floor, service tunnel and partial basement walls below, which are founded on shallow strip footings. Figure 2‐4 and 2‐5 show the foundation plan and ground floor plan respectively.

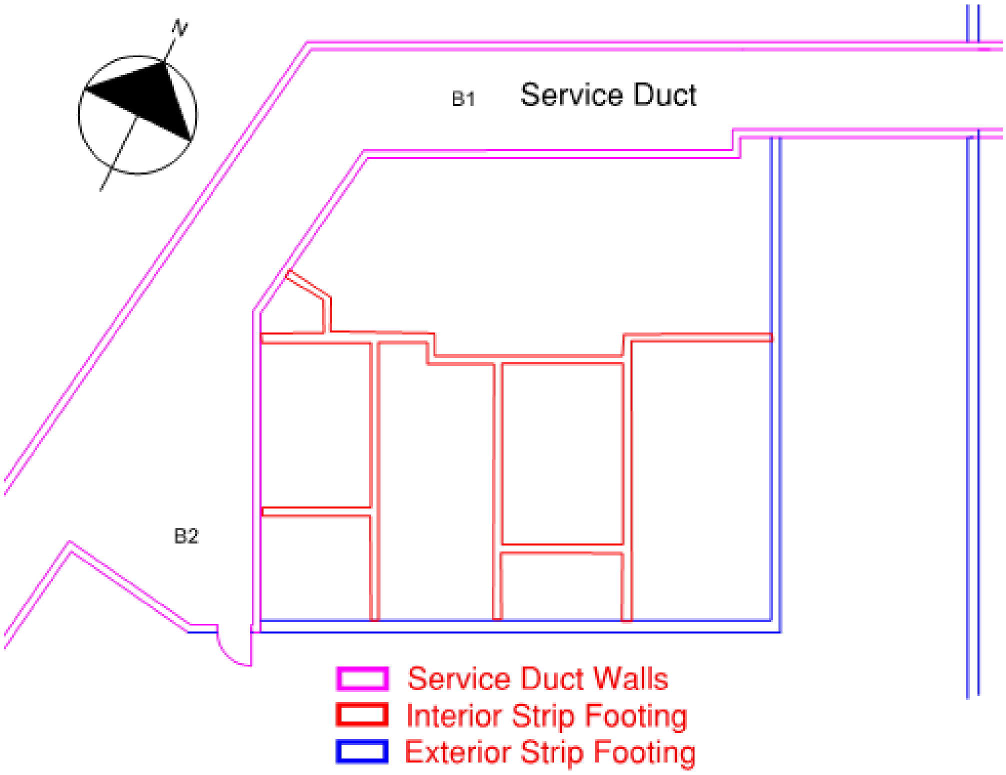


Figure 2-4: Engineering Services – Foundation Plan

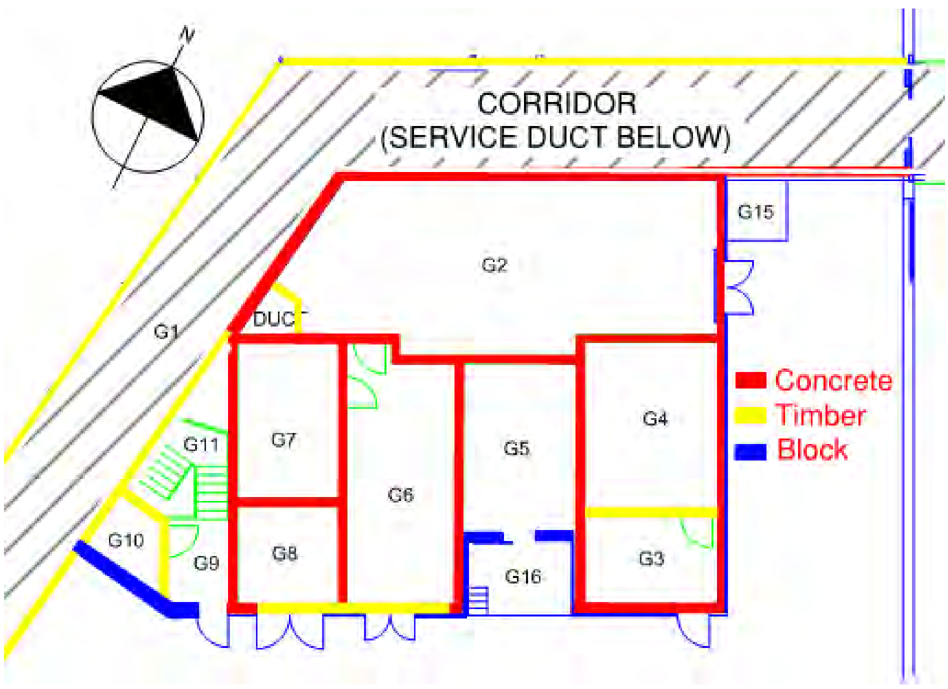


Figure 2-5: Engineering Services – Ground Floor Plan

At the roof level, a 100mm thick reinforced concrete parapet extends above the roof and is approximately 500mm in height above the roof slab at northern extents of the building. The exterior walls of the building are clad in an exterior skin of 100mm thick block veneer. The exact fixing of the block veneer to the exterior concrete walls is unknown.

Interior partition walls in the sprinkler room and communications room are constructed from timber framing lined with GIB board. The exterior wall at the loading dock is block work. On the northern side of the building, Nuralite roofing covers the concrete roof slab. On the southern side of the roof, light weight metal tray deck roofing over timber roof framing covers the timber and concrete roof. The ceiling below the timber roof is lined with GIB board. For the extent of timber and concrete roof, see roof plan, figure 2-6.

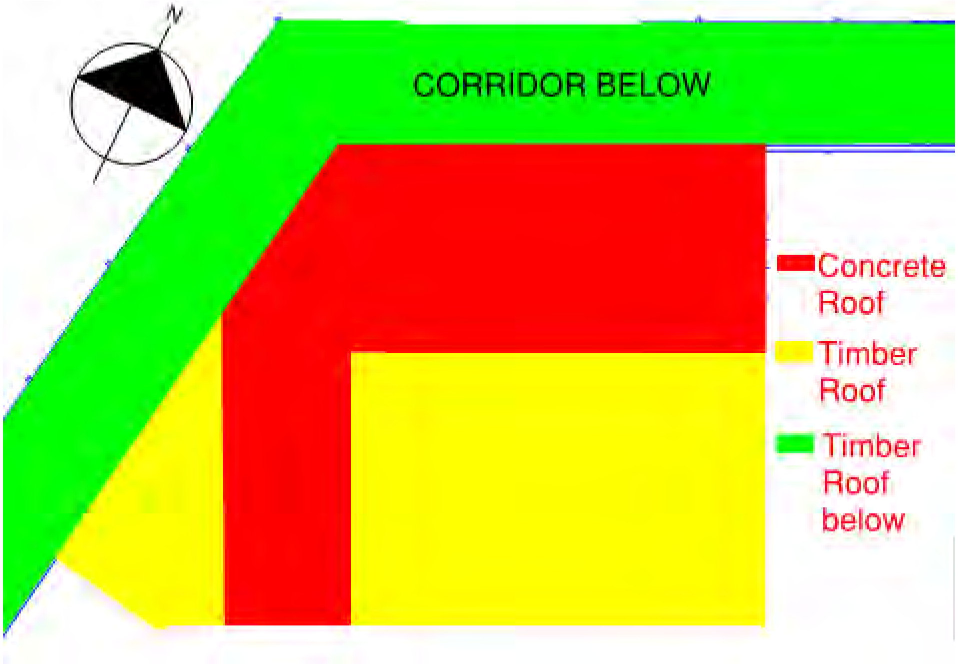


Figure 2-6: Engineering Services – Roof Plan

The service duct is approximately 2600mm in height. A crawl space of approximate 800mm in height exists between ground floor slab and the soil. Building elevations and sections are shown in figures 2-7, 2-8, 2-9, and 2-10.

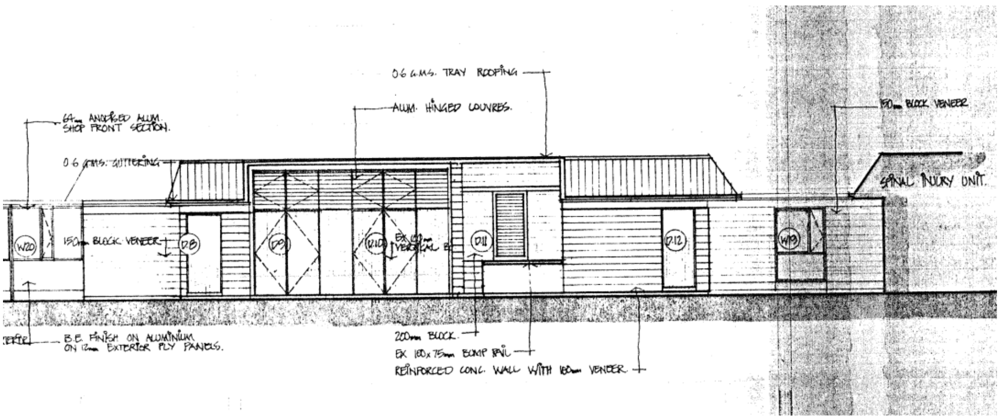


Figure 2-7: Engineering Services – South Elevation

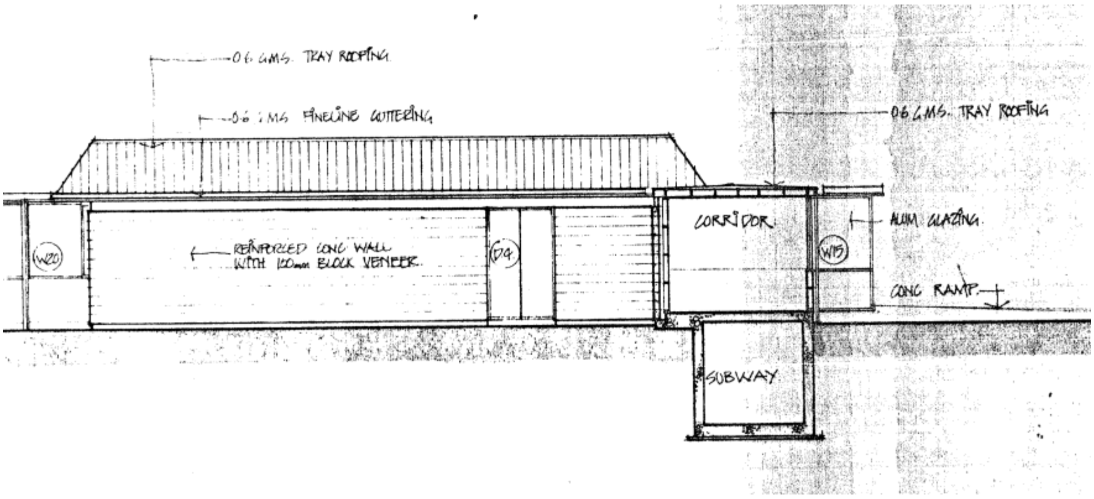


Figure 2-8: Engineering Services – East Elevation

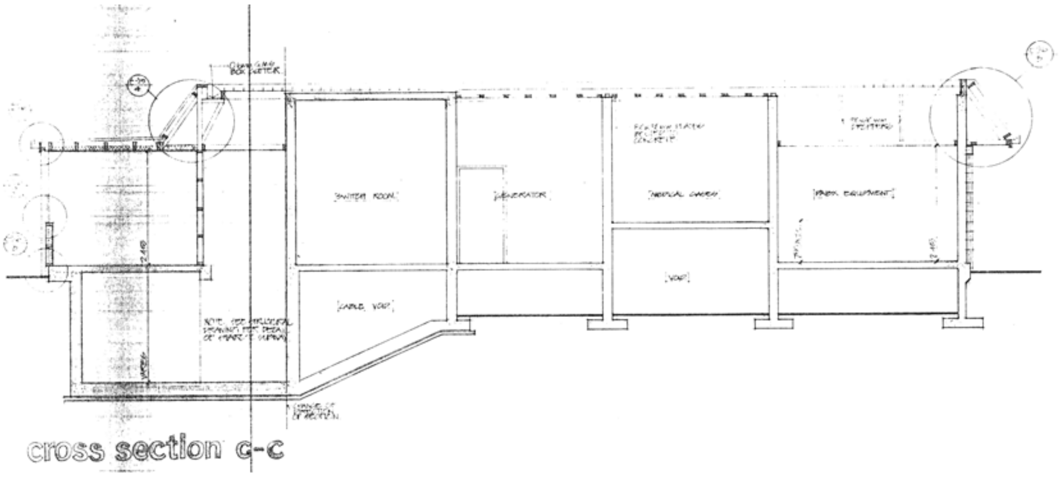


Figure 2-9: Engineering Services – East-West Building Section

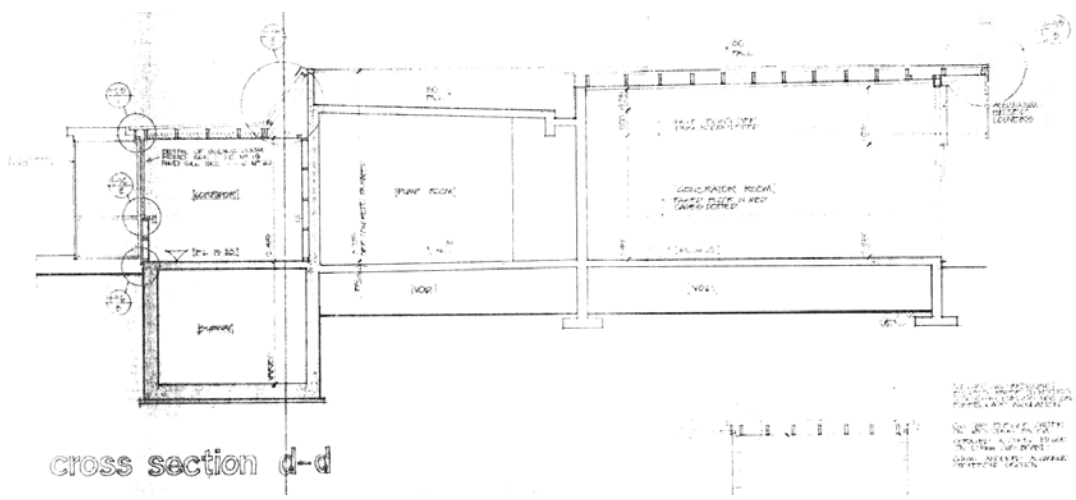


Figure 2-10: Engineering Services – North-South Building Section

2.2 LATERAL LOAD RESISTING SYSTEMS

The primary lateral load resisting system for the Engineering Services Building consists of reinforced concrete structural walls and reinforced concrete roof and ground floor slabs. The roof and floor slabs act as rigid diaphragms to distribute lateral loads to the concrete walls below. It was assumed that the timber roof did not provide out-of-plane resistance to the structural walls and did not transfer loads to or from the concrete roof diaphragm to the concrete structural walls. The concrete structural wall bracing lines at the ground floor level align with the sub-floor service tunnel or partial basement wall lines, which are all founded on continuous reinforced concrete strip footings.

It was assumed that the block partition wall at the loading dock provides in-plane resistance for itself and not out-of-plane resistance for the adjacent perpendicular concrete walls. The out-of-plane loads from this wall are supported by the floor slab and the capping beam spanning to the adjacent medical gas room concrete structural walls. The concrete structural wall between the medical gas room and the generator room is braced at roof level by a 100 SHS beam that spans to the concrete diaphragm above the transformer room and across the loading dock to the exterior PABX room concrete structural wall. See figure 2.2 for the building plan with room names.

2.3 PRE-EARTHQUAKE BUILDING CAPACITY – DIRECT CODE COMPARISON

The building capacity under earthquake actions discussed in this section is compared to the capacity of a similar building that would be designed to today's standards. A new building would be designed to the *Structural Design Actions Standard, Part 5: Earthquake Actions – New Zealand*, NZS 1170.5:2004 [10] and incorporating the amendments made to this standard as a result of the Lyttelton Earthquake as outlined in the Amendment 10 of the Building Code [9]. The implications of the recent amendments are discussed more fully in the Burwood Hospital Campus Base Report. The amendments essentially result in an increase to the basic seismic design loads of 36 %.

A limited number of the original structural and architectural drawings for the building are available, but the structural calculations and specifications are not, so the exact design and loading assumptions originally made are unknown. For the purposes of this report, seismic loading assumptions have been made based on a detailed review of the drawings available and physical observations of the building.

When the building was originally designed in the late 1970s, the loading standard at the time was the *New Zealand Loading Code* NZS 4203:1976 [11].

The current seismic loading code, NZS 1170.5, requires a new building to be designed to a Design Basis Earthquake (DBE), which is based upon the buildings physical location, local soil conditions, building type, fundamental period and importance level.

The Engineering Services Building is not regarded as an essential hospital facility by the CDHB and is therefore classified as an Importance Level 3 building in accordance with NZS 1170:2004 [10] The associated return period of the DBE event is 1000 years, with a risk factor for design of R = 1.3 (no post-disaster or special function). The subsoil for the site is taken as Soil Type D, which is consistent with the findings of a post-earthquake geotechnical investigation [5].

Based upon the period of construction, and the detailing of the lateral load resisting elements, the concrete portion of the building has been assessed as having nominal ductility, and as such the reinforced concrete walls have been assigned a ductility factor of **μ**=1.25.

It is likely that the Engineering Services Building was designed originally to NZS 4203:1976 with an Importance Factor of 1 (i.e. equivalent to an Importance Level 2 Structure as defined in 1170:2004). As the Engineering Services Building provides services to Importance Level 3 buildings on the Burwood Hospital Campus it is assessed as an Importance Level 3 building as defined in NZS 1170:2004. A comparison between the DBE of NZS 4203:1976 (Importance Factor 1) and NZS 1170:2004 (IL3) for the site and type of construction are plotted below. Based upon a fundamental building period of approximately 0.40 seconds, the Engineering Services building capacity is equivalent to approximately 75%DBE in both directions based on a comparison of the likely design loads.

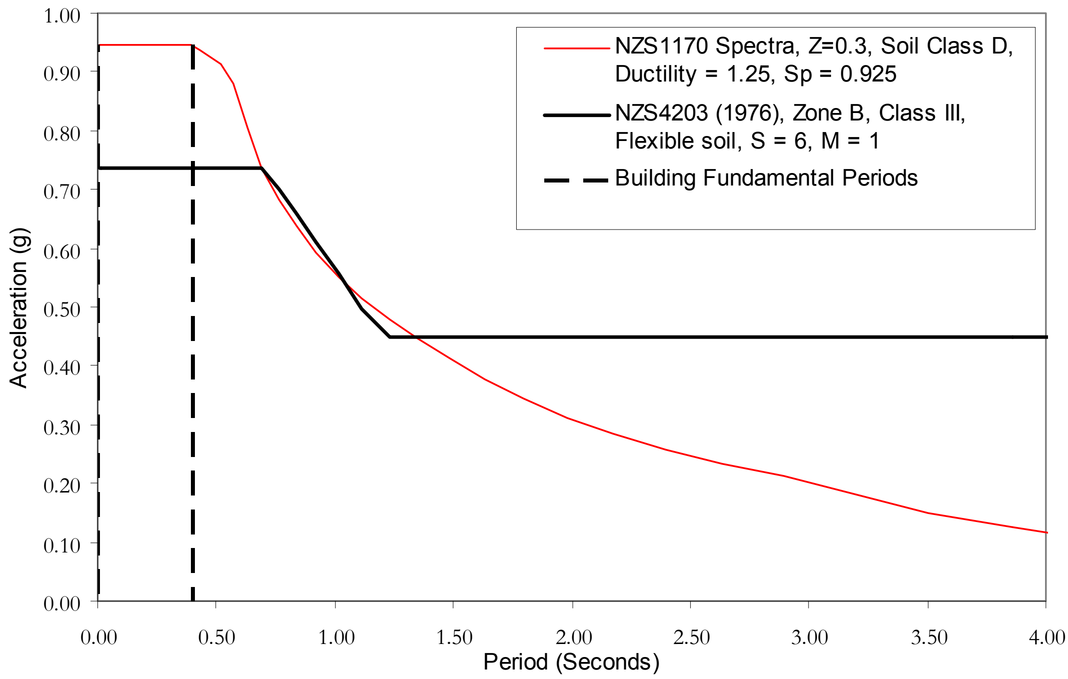


Figure 2-11: Comparison of Design Codes – NZS1170 (IL3) to NZS 4203 (IL2)

2.4 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 estimated 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.

Following the Lyttelton earthquake, a geotechnical report was conducted by Tonkin & Taylor, titled "Burwood Hospital Post Earthquake Geotechnical Assessment", dated June 2011 [5]. This report has been used to aid in the evaluation of the site conditions and the likely effect of the ground on the buildings past and future performance. The soil parameters described in the geotechnical report completed by Tonkin and Taylor have been used for the evaluation of the buildings existing foundation system.

The probable capacities have been calculated using the New Zealand Society for Earthquake Engineering guidelines presented in the *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes – NZSEE 2006* [16] and the requirements of NZS 1170:2004. The guidelines allow some relaxation of the requirements for existing buildings when compared to what would be required for a new building.

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. These are described in more detail in the Burwood Hospital Base report [1] and include strength governed elements such as short columns and deflection governed elements such as floor and stair elements with inadequate support seating.

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 (DBE), and have been expressed as a %DBE. This includes checks for both the strength and deflection requirements.

Due to the limited structural drawings available, several assumptions had to be made in regards to the existing properties of the building elements. Based upon the drawings available, inference has been made as far as the minimum steel reinforcement in the block walls. It was assumed that the block walls are reinforced with 2-HD12 at 800 centres each face. A minimum concrete compressive strength of 37.5MPa has been assumed as the probable strength for all concrete elements. This is based on the assumption of an original concrete strength of 25MPa at the time of construction.

The foundations have been assessed at an ultimate bearing capacity of 150kPa, for factored loads, as per recommendations provided by Tonkin and Taylor.

A summary of the %DBE for each primary element has been noted in Table 2-1 and Table 2-2.

Building Element	%DBE (IL3)	Comments
Concrete roof diaphragm – N-S …………………… E-W	100% 100%	
Block walls – E-W	100%	
Concrete structural walls – N-S ………………………….…...E-W	77% 100%	Out-of-plane capacity of medical gas room wall

Table 2-1: Superstructure – Seismic Assessment %DBE (IL3)

Building Element	%DBE (IL3)	Comments
Ground floor concrete slab – N-S …………………………… E-W	100% 100%	
Sub-floor walls – N-S ………………... E-W	100% 100%	
Foundations – N-S E-W	100% 100%	

Table 2-2: Sub-floor – Seismic Assessment %DBE (IL3)

The SHS tie on the south face of the building has the capacity to transfer the loads into the concrete roof diaphragm and concrete structural walls. The connection between the SHS and the wall is unknown. Further investigation is required to confirm the capacity of the connection.

A review of the drawings available and site observations revealed no Critical Structural Weaknesses (CSW's) that could lead to premature collapse of the building.

3. POST EARTHQUAKE BUILDING CONDITION

This section covers the structural damage sustained by the Engineering Services building at Burwood Hospital as a result of the Darfield Earthquake that struck at 4:36am on 4th September 2010, the Lyttelton Earthquake that struck at 12.51 pm on the 22nd February 2011, the earthquake at 2.20 pm on the 13th June 2011 and the December earthquake that struck at 3.18pm on 23rd December 2011. The Lyttelton Earthquake subjected the building to strong ground motions which likely exceeded the full design earthquake load for buildings of this nature and appears to have caused the bulk of the earthquake damage observed after the initial Darfield event.

3.1 THE LYTTELTON EARTHQUAKE

The fundamental period of the building is estimated to be approximately 0.4 seconds. 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 strong motion data presented in the Base Report [10], it appears the Lyttelton Earthquake produced ground shaking intensities of approximately 45#90% of the current Ultimate Limit State design spectra for an Importance Level 3 building.

It should be noted that the Lyttelton Earthquake was very short in terms of strong shaking produced with the strong motion only lasting for duration of 5#7 seconds.

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.

3.2 PRELIMINARY INVESTIGATIONS

Preliminary investigations were carried out to identify 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
- review of available structural engineering construction documentation

In conjunction with a review of the available drawings for the building the following areas were identified for potential damage:

- movement or damage to structure associated with ground movement and/or settlement

- cracking and joint failure of concrete sub-floor walls, service tunnels and foundations
- cracking in concrete shear walls or floor diaphragms
- signs of distress in external block veneer
- distress and cracking of plaster ceiling linings

A review of the above information on the building type and preliminary observations highlighted this building as requiring a more detailed inspection. The aim of the detailed inspections was to determine the full extent of the damage caused to the building, particularly those elements identified for potential damage above. These areas were targeted to identify if damage had occurred, and to what extent the damage had reduce the capacity of the buildings lateral load resisting system to resist future seismic events.

3.3 DETAILED STRUCTUAL OBSERVATIONS

Further detailed inspections and structural explorations have been carried out following the initial assessments to ascertain the full extent of structural damage. The majority of the detailed structural observations were completed on the 21st and 22nd of June 2012, with additional observations completed on the 9th July 2012 to inspect the timber roof framing to wall connections and on the 29th August to inspect the floor levels and slopes.

A full photographic record of the observations is available electronically on request. The detailed structural observation identified the following additional damage to those items noted in the initial rapid assessments:

- Cracking to the concrete service tunnel wall near service penetrations
- Cracking to wall and floor slab
- Cracking at corridor block wall joint between Engineering Building and Spinal Injury Unit

It should be noted that the structural observations to date have been limited in places due to wall and floor finishes concealing the surface of concrete elements. Further investigations could reveal additional damage. Additional investigations recommended are outlined in Section 3.7.

3.4 LEVELS SURVEY

A detailed survey of the ground floor levels in the Engineering Services Building was conducted by Fox & Associates and issued on 3rd August, 2012 [6]. The survey indicates a differential settlement of approximately 35mm, with the most significant differential settlements occurring at the southeast end of the building. The results of the survey indicate that the northern portion of the building has settled less relative to the south portion, the corridor to the west and the Spinal Injuries Building to the east. Cracking was observed at the junction of the building corridor at the northwest corner of the corridor junction to the Spinal Injuries Unit which is consistent with the settle indicated by the levels survey. The worst case permanent slope, based upon this survey, is a drop of approximately 16mm over a 3.5 meter length resulting in a slope in the elevated ground floor slab of approximately 0.45% or 1:220. The slope is outside the acceptable tolerance of *NZS 3109:1997* [15]. This could be remediated through localised lifting of the structure using mechanical or grout injection techniques. A discussion on how to reinstate the southeast end of the building has been included in Section 4.2.

For the extent of the differential settlement noted see Figure 3-1 and the levels survey included in Appendix C.

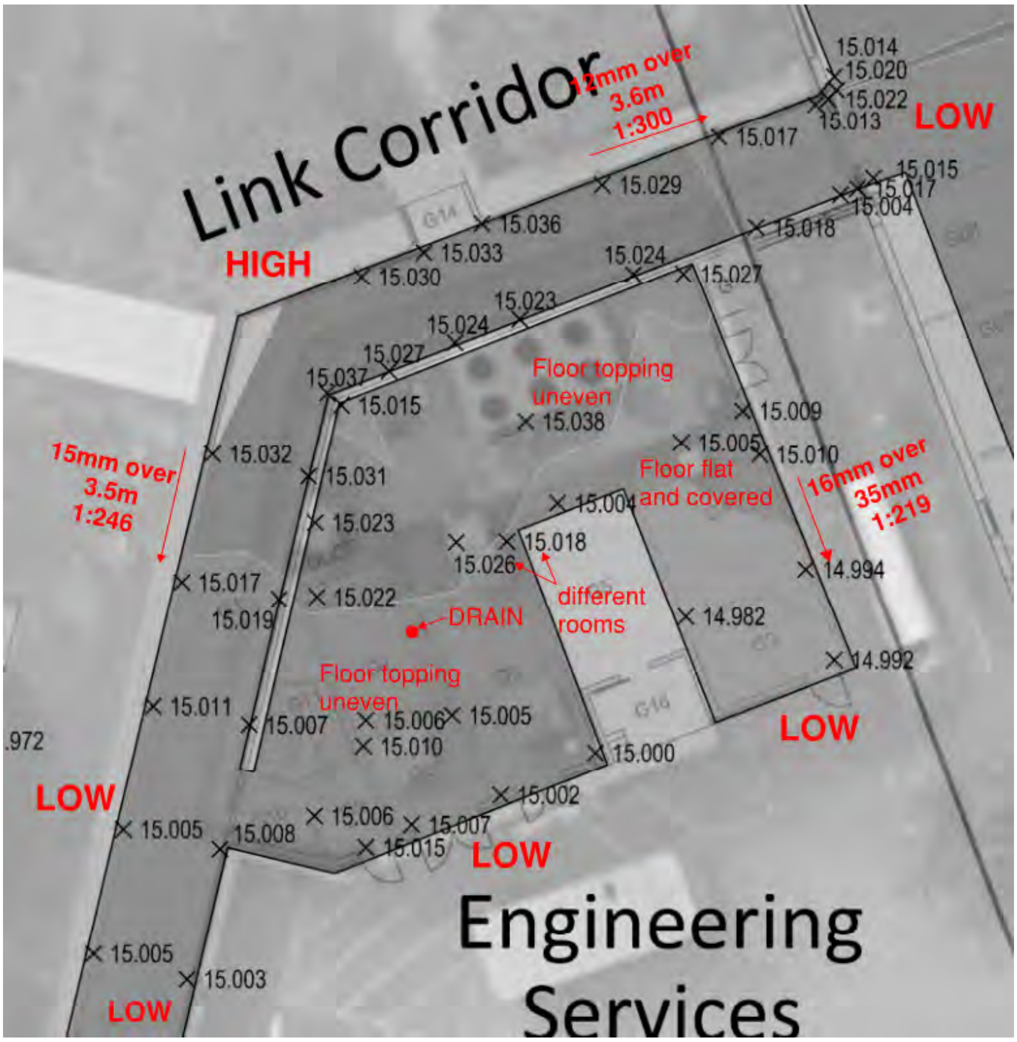


Figure 3-1- Ground Floor Level Survey Showing Falls

3.5 GEOTECHNICAL REVIEW

A review of the ground damage and conditions was carried out by Tonkin & Taylor for the Burwood Hospital Campus. A subsequent report titled *Burwood Hospital Post Earthquake Geotechnical Assessment* was issued in June 2011 [5]. The geotechnical review concluded that the settlement and damage to building foundations and slabs on the Burwood Hospital Campus was likely due to the liquefaction of underlying soil layers. It is believed that excessive pore water pressures have been dissipated and that further settlement is not expected, unless another significant event was to occur.

Based on this report and from our detailed damage observations both internally and externally it does not appear that the overall stability of the Engineering Services Building has been affected by earthquake induced settlement.

Based on the geotechnical report provided by Tonkin & Taylor [5] the potential for future total and differential settlements at the building site varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

3.6 SUMMARY OF BUILDING DAMAGE

The following is a summary of the observations made for the building, and our conclusions as to their condition and seismic load resisting capacity. These observations do not specifically distinguish between damage caused by the Darfield Earthquake, the Lyttelton Earthquake, the June Earthquake, the December Earthquake or any significant aftershocks thereafter. Despite not being able to specifically distinguish when individual damage observed occurred, it is believed that the majority of the damaged can be linked to the February 22nd event.

The Engineering Services Building appears to have performed as would be expected for a building of this type and age. The observed structural damage was cracking in the concrete walls and floors. The structural damage sustained by the building as a whole would be categorized as minor. A summary of the typical damage observed is as follows:

- **Cracking to Concrete Sub-floor, Service Tunnel and Partial Basement Walls** – Typical cracking noted in reinforced concrete service tunnel walls, up to 0.4 to 0.5mm in width at openings.

- **Cracking to Concrete Walls and Floor Slabs** – Cracking has been noted in the reinforced concrete wall and floor slabs. *Roof and floor finishes have not been removed to determine the full extent of cracking. In general, the top side ground floor and the underside of the concrete roof slab are exposed.*

- **Differential Settlement** – Differential settlement has occurred with the southern portion of building being lower than the north.

Table 4.1 provides a photographic summary of the typical damage observed.

3.7 ADDITIONAL INVESTIGATIONS RECOMMENDED

Further investigations are required in order to understand the full extent of damage to the Engineering Service Building. An exhaustive survey of the full extent of cracking to the building has yet to be completed due to the presence of floor finishes, wall coverings and dropped ceiling in some areas. Additional investigations that should be completed are as follows.

3.7.1 Additional Investigations Required for Assessment

- Remove wall linings at 100 SHS connection to exterior concrete PABX room wall above the loading dock and connection to concrete roof slab above generator room.

- Investigate the connection to roof framing above corridor to concrete wall along Engineering Service Building. Remove wall linings as required.

3.7.2 Additional Investigations Required During Repair

- Remove floor lining in PABX room to investigate concrete slab damage due to differential ground settlement.

3.8 POST-EARTHQUAKE BUILDING CAPACITY

Based upon our investigations to date, we do not consider the Engineering Services Building to have any notable reduction to the overall gravity load resistance of the structure or the lateral load resisting system.

The damage observed will require repair to restore the strength, stiffness, durability and performance of the individual structural components. This repair work is outlined in Section 4. Following the recommended repair of the structural damage, the lateral load resisting performance of the structure will be restored to approximately pre#earthquakes levels (see Section 2.4).

In its pre#earthquake and post#earthquake state the main lateral load resisting elements of the Engineering Services Building has been assessed to have a capacity greater than 33% DBE, and as such the building is not considered to be "Earthquake Prone."

4.1 TYPICAL DAMAGE & REPAIRS REQUIRED

This section covers the damaged noted during our detailed assessment of the building. Note that our observations have been restricted to structural aspects of the building 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.

Table 4#1 provides a photographic summary of the observed typical damage and typical repairs required for the Engineering Services Building. Figure 4#1 provides a floor plan with the location of the observed damage.

In general, the aim of the repair work indicated in this section is to restore the structure to its pre#earthquake state, as far as practicable, while maintaining the utility of the building. The repairs presented attempt to address any loss of strength, stiffness and durability of the structural elements due to the damage noted.

It should be noted that more damage may be identified during the repair works and (if required) additional repair details will be specified accordingly.

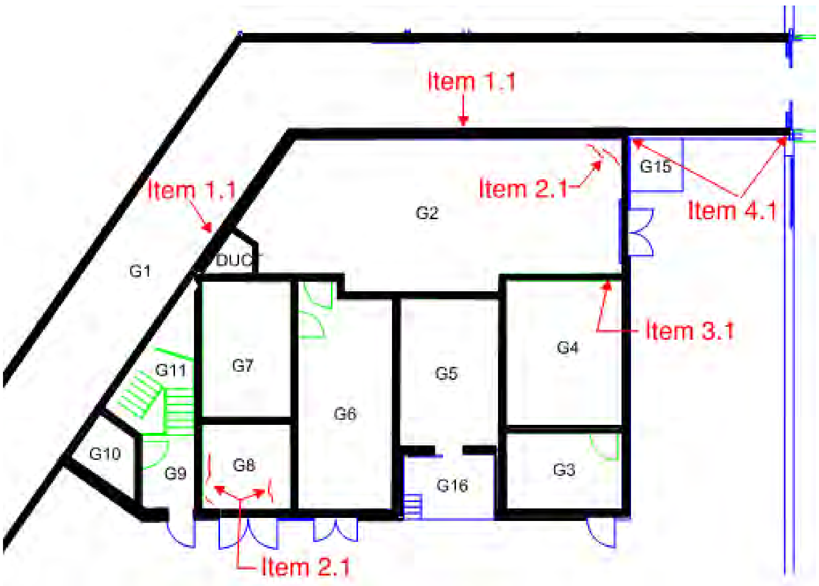


Figure 4-1: Ground Floor Plan – Required Repairs

.

Table 4-1: Photographs of Observed Typical Damage and Repairs Required

Damaged Item & Location	Damage	Recommendations	Example Photograph
1. Foundations and Sub#Floor Walls			
1.1 Sub#floor, service tunnel and partial basement walls	Cracking in concrete walls (typically less than 0.5mm)	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [3]. For cracks greater than 1mm, HCG to confirm the integrity of the existing reinforcement of the wall. If reinforcement is damaged, an engineered repair will be required.	
2. Concrete Roof and Floor Slabs			
2.1 Concrete floor slabs – ground floors	Cracking in floor slab in rooms G2 and G8	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [3]. For cracks greater than 1mm, HCG to confirm the integrity of the existing reinforcement of the wall. If reinforcement is damaged, an engineered repair will be required.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
3. Concrete Walls			
3.1 Wall between	Cracking in concrete wall	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [3]. For cracks greater than 1mm, HCG to confirm the integrity of the existing reinforcement of the wall. If reinforcement is damaged, an engineered repair will be required.	
4. Miscellaneous items			
4.1 Corridor wall to Engineering Services Building wall connection	Cracking at block wall joint	Further investigation needed.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
5. Differential Settlement			
5.1 Differential ground settlement	Differential ground settlement of approximately 35mm resulting in a worst case slope in the ground floor slab of approximately 0.45% (1:220)	The differential settlements noted at the south end of the building shall to be addressed. For further discussion on the remediation work required see Section 4#2. (Note: All re# levelling is to occur prior to any other structural or cosmetic repairs).	

4.2 DISCUSSION ON DIFFERENTIAL SETTLEMENT REMEDIATION

The level survey, completed by Fox & Associates, has indicated differential ground settlement of up to approximately 35mm across the length of the ground floor slab on the eastern side of the building. The worst differential settlement is concentrated at the southeast end of the building (see Appendix C for complete level survey) and has resulted in permanent slopes in the elevated ground floor slab of up to 0.45% (1:220). The slopes are beyond the typical acceptable level tolerances of *NZS 3109:1997* [15].

This can either be addressed by demolishing and reconstruction of this portion or through re#levelling.

To re#level the building, the southern portion of the building would be lifted up to the level of the northern rooms and corridor. The re#levelling solution for the Engineering Services building will need to be developed in conjunction with the re#levelling solution for the adjacent Spinal Injuries Unit and corridor as these building are connected.

The two primary re#levelling options available include the use of mechanical jacking or the use of either underpinning grout or engineered resin. There are pro's and con's of each solution which extend beyond structural performance which will need to be considered by CDHB. These include continuity of operation, degree of re#levelling accuracy and the willingness of the re#levelling sub#contractor to provide a producer statement, amongst other items.

From a structural standpoint, either option is acceptable provided the use of underpinning grout or engineered resin does not create any "hard points" under the building. If "hard points" are created during the re#levelling process the potential for future differential settlements can be increased. If this were to occur it would reduce the capacity of the building going forward.

Based upon the information provided by Tonkin & Taylor the soil profile under the Spinal Injuries Unit (medium dense sand overlying dense sand) lends itself to localized lifting through underpinning grout or engineered resin techniques and should not create any undesirable "hard points" as described above.

The building also lends itself nicely to the use of mechanical jacking due an elevated ground floor slab and the relatively good shape of the exterior and interior concrete sub#floor walls in this area. The exterior sub#floor walls are typically roughly 1 meter in depth, heavy reinforced and well detailed, and should easily span between jacking locations placed under the sub#floor walls.

The suitability of re#levelling the building through the use of either mechanical jacking or underpinning grout (or engineered resin) will need to be verified by qualified sub#contractors in conjunction with the geotechnical consultant.

It should be noted that both options discussed above are not expected to increase the seismic performance of the building or reduce the potential for future differential settlements. Instead the options presented are designed to re#level the building without making the future performance of the building any worse than it was prior to the earthquakes. To improve the future performance of the building, and reduce the potential for future differential settlements, would likely require the entire footprint of the building to be either piled or the ground under all the sub#floor wall footings, service tunnels and the partial basement improved. *Further geotechnical investigations would be required into the type and depth of ground improvement required.*

Based up the geotechnical report provided by Tonkin & Taylor [5] the potential for future total and differential settlements at the building site varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

During the re#levelling process there is also the risk that addition damage could occur to the building linings, exterior block veneer, etc. Appropriate contingencies should be provided.

5. REFERENCES

1. *Burwood Hospital – Detailed Seismic Assessment Report – Base Report*, Holmes Consulting Group, November 2011.

2. *Burwood Hospital – Detailed Seismic Assessment Report – Earthquake Repair Specification*, Holmes Consulting Group, July 2011.

3. *Burwood Hospital Services Building, Subway and Connecting Corridor,* Original structural drawings. Frederick, Sheppard and Partners, 1977.

4. *Additions Burwood Hospital Services Building Subway and Connecting Corridor* , Original architectural drawings. Cutter, Pickmere, Douglas Architects, 1978.

5 *Burwood Hospital Post Earthquake Geotechnical Assessment*, Tonkin and Taylor Ltd., June 2011.

6 *Burwood Elevation Survey – Revision E*, Fox & Associates, August 2012

7 *Burwood Hospital Campus – Seismic Risk Assessment Report*, Holmes Consulting Group, April 2002

8 *Burwood Hospital Campus – 2007 Seismic Risk Assessment Update,* Holmes Consulting Group, June 2007

9 *Compliance Document for New Zealand Building Code - Clause B1 – Structure, Amendment 10 (Canterbury)*, Department of Building and Housing, Wellington, 19 May 2011.

10 *Structural Design Actions Part 5: Earthquake Actions – New Zealand, NZS 1170.5:2004*, Standards New Zealand, 2004.

11 *Code of Practice for General Structural Design and Design Loadings for Buildings*, NZS4203:1976, Standards New Zealand, 1976

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

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

14 *Concrete Structures Standard, NZS 3101:2006*, Standards New Zealand, 2006

15 *Concrete Construction Standard, NZS 3109:1997*, Standards New Zealand, 1997

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

17 *Seismic Rehabilitation of Existing Buildings,* ASCE 41#06, American Society of Civil Engineers, 2007

18 *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury – Part 2 Evaluation Procedure*, Engineering Advisory Group, July 2011

19 *Revised Guidance on Repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence*, Department of Building and Housing, November 2011

20 *Practice Note – Design of Conventional Structural Systems Following the Canterbury Earthquakes*, SESOC, December 2011

APPENDIX A

Record of Observation

APPENDIX A – RECORD OF OBSERVATIONS & REPAIRS

Inspection date: 21, 22 June 2012 and 29 August 2012

KEY	
N	No repair required
Y	Repair required
F	Further investigation required
C	Repair complete

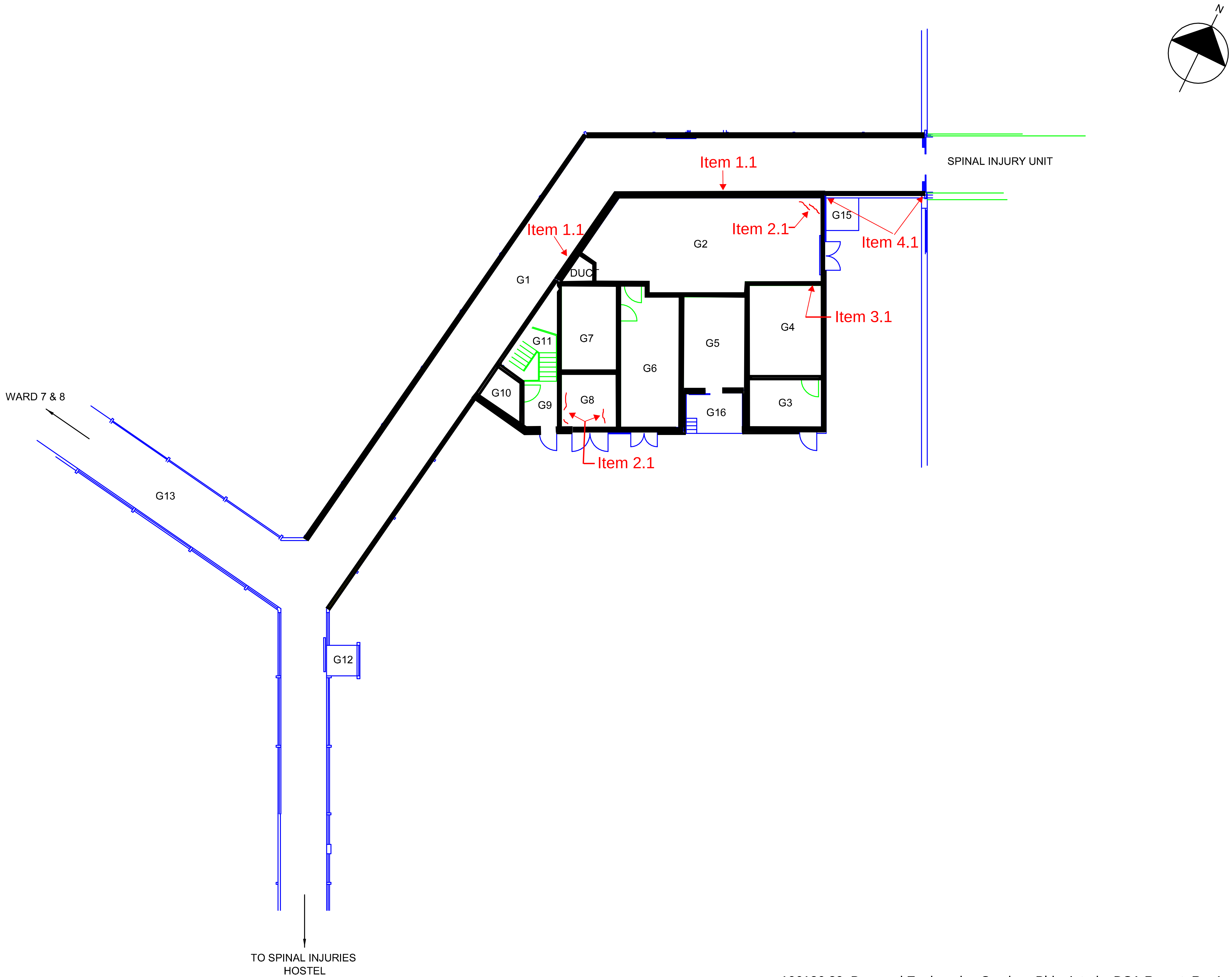
Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G1	Wall	Vertical cracking in concrete walls at opening	**Y**	Epoxy inject all cracks in concrete walls between 0.2mm and 1.0mm per the HCG Repair Specification. For cracks greater than 1mm are observed, advise HCG for additional inspection	Engineering Services (1)
G	G2	Floor	Diagonal cracking in concrete slab	**Y**	Epoxy inject all cracks in concrete walls between 0.2mm and 1.0mm per the HCG Repair Specification. For cracks greater than 1mm are observed, advise HCG for additional inspection	Engineering Services (2)
G	G8	Floor	Diagonal cracking in concrete slab	**Y**	Epoxy inject all cracks in concrete walls between 0.2mm and 1.0mm per the HCG Repair Specification. For cracks greater than 1mm are observed, advise HCG for additional inspection	Engineering Services (3)

Refer to Table 4.1 and HCG Specification for repair details

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G4	Wall	0.2mm diagonal crack in concrete wall	**Y**	Epoxy inject all cracks in concrete walls between 0.2mm and 1.0mm per the HCG Repair Specification. For cracks greater than 1mm are observed, advise HCG for additional inspection	Engineering Services (4)
G	Exterior	Wall	Cracking at block wall joint	**F**	Replace damaged lining with Gib Plasterboard	Engineering Services (5)

APPENDIX B

Reference Plan



106186.89_Burwood Engineering Services Bldg_Interim DSA Report_Rev1_14 Sept2012

AMENDMENT	DRAWN	CHECKED	DATE
MASTER DRAWING CREATED FROM 04000003 AND CDHB BORDER ADDED	BJT		15-06-09

BURWOOD HOSPITAL
ENGINEERING SERVICES, GROUND FLOOR AND LINK CORRIDOR
MASTER FLOOR PLAN

Canterbury
District Health Board
Te Poari Hauora o Waitaha

DESIGNED:
DRAWN: BJT
CHECKED:
DATE: 15-06-09

DWG No. 04022060
SCALE 1:100 AT A1
REVISED 15-06-09

XREFED DRAWING/S:

Maintenance and Engineering Department - Christchurch Hospital

APPENDIX C

Level/Elevation Survey

NOTES:

The source of the data and an estimate of its accuracy are stated below. Fox & Associates Ltd does not guarantee any data sourced from a third party. If the use of third party data is critical to the project then it should be independently verified.

ACCURACIES:

Levels observed using Trimble DiNi Level with an expected accuracy of 1ppm (1mm per km) and an estimated accuracy of +/-2mm relative to benchmark nails. Elderly Day Care and Old Surgical Block levels observed using hydrostatic level with an estimated accuracy of +/-4mm.

Horizontal position of levels is approximate only and relates to internal structures shown on building floor plans. Plans rotated and scaled into main drawing as cross-referenced underlays.

SURFACE DATA:

Contour surfaces are derivative products based on limited floor positions observed, and provided only to enable further investigation of potential building issues and should not be used in isolation to draw conclusions on the state of the buildings surveyed.

SURVEY DATUM:

Site benchmark: NAIL 8 RL = 14.532m.

Levels in terms of EHC9 RL = 14.19m Corner Burwood Rd and Helmsdale St.

Christchurch City Datum Post 13 June 2011 levels

FIELDWORK NOTES:

LOCATION	DATE	REF
Overall Site Control	20-Oct-11	2462.DAT
Surgical Services North	20-Oct-11	2462.DAT
Surgical Services South	21-Oct-11	2462.DAT
Spinal Unit	21-Oct-11	2462.DAT
Administration Building	28-Oct-11	246201.DAT
Nurses Hostel and Champion Centre	2-Nov-11	246202.DAT
Orthopaedic Rehabilitation Unit	10-Nov-11	246203.DAT
Elderly Day Care	18-Nov-11	N/A
Parafed Gym and Food Services	22-Nov-11	246204.DAT
Brain Injury Rehabilitation Service	22-Nov-11	246204.DAT
Physical Medicine	23-Nov-11	246205.DAT
Boilerhouse and Site Maintenance	23-Nov-11	246205.DAT
Melrose Chairs, Wards 9-10 Corridor	30-Nov-11	246206.DAT
Old Surgical Block and Wards 7 & 8	30-Nov-11	246206.DAT
Beacon House, BSU Hostel & Chapel	2-Apr-12	246212.DAT
Milner Lodge & Tapper Units	3-Apr-12	246213.DAT, 246214.DAT
Pain Mgmt, Medical Records, Admin B	5-Jun-12	246215.DAT
Surgical Services North infill	6-Jun-12	246216.DAT
Eng Services, BSU infill, Garage, Parafed-Food Link	4-Jul-12	246217.DAT
300 Burwood Rd, Food-Admin Link	13-Jul-12	246218.DAT

Audit observations (Revision E January 2012) are not included in this revision.

REVISION NOTES:

No.	APPROVED	DATE	NOTES
A	MJM	31-Oct-11	Administration and Allan Bean Centre elevations added
B	MJM	18-Nov-11	Nurses Hostel & Champion Centre, SOU elevations added
C	MJM	24-Nov-11	Phys Med, Parafed, Food Services, BIRS, Boilerhouse, EDC added
D	MJM	2-Dec-11	Wards 7 & 8, Surgical Block, Melrose Chairs & Corridor added
E	MJM	1-Feb-12	Audit observations post Dec 23 2011 (not included in this Revision)
F	MJM	18-Apr-12	Beacon House, BSU Hostel, Chapel, Tapper and Milner added
G	MJM	18-Jun-12	Med Records, Pain Mgmt, Admin B, and Surgical infill added
H	MJM	3-Aug-12	Eng Services, Garage, 300 Burwood Rd, corridors and infill

POINT REDUCTIONS:

Details of level reductions accompany surface elevation tables for each individual building.

AERIAL PHOTOGRAPHY

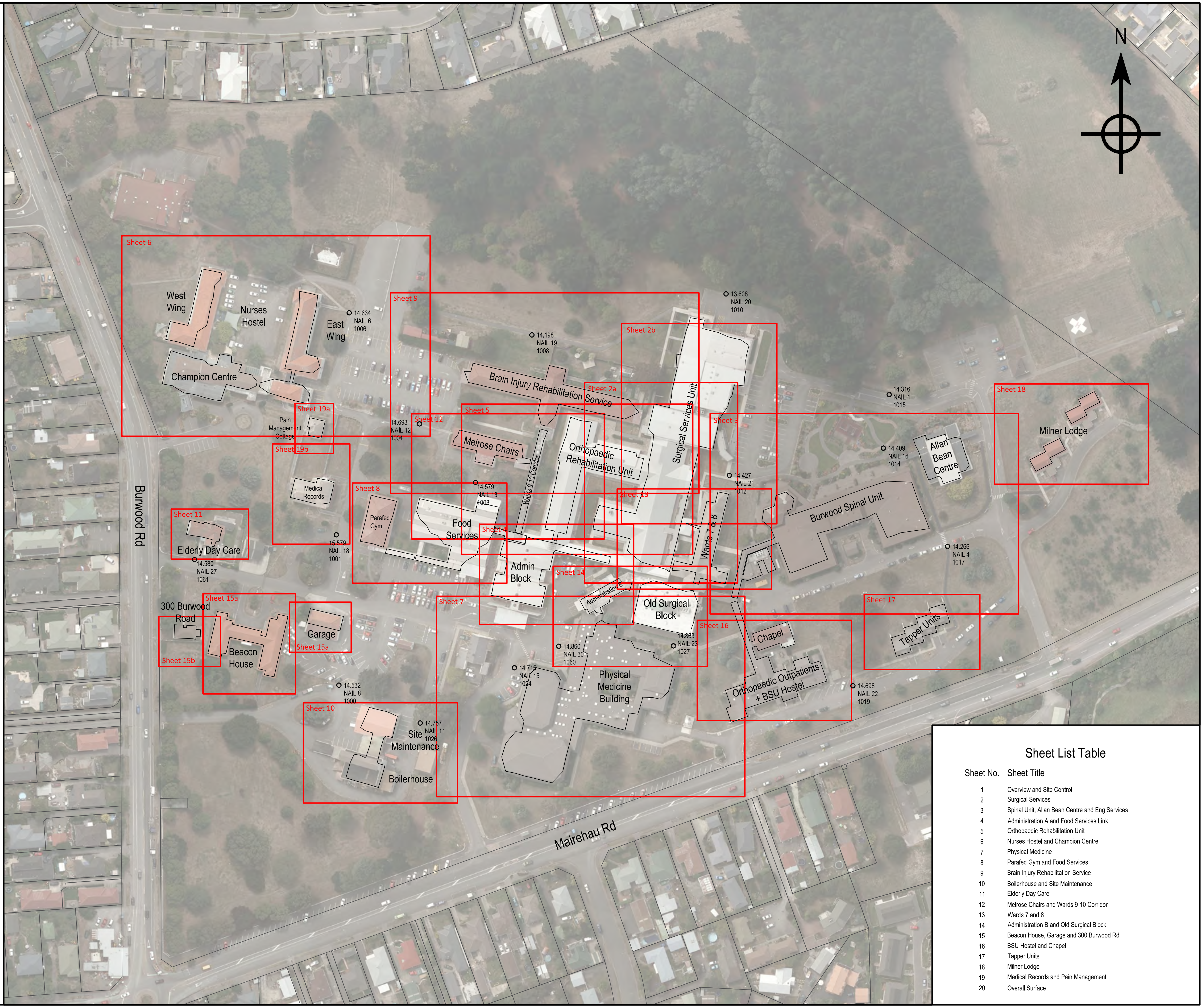
The aerial imagery is 10cm orthophotography provided by LINZ, flown on 24/02/2011. Crown Copyright reserved.

CERTIFICATION:

I, Michael Martin, hereby certify that this survey has been carried out by me, or under my direction, to the accuracies as set out above.

Michael Martin

Registered Professional Surveyor



Sheet List Table

Sheet No.	Sheet Title
1	Overview and Site Control
2	Surgical Services
3	Spinal Unit, Allan Bean Centre and Eng Services
4	Administration A and Food Services Link
5	Orthopaedic Rehabilitation Unit
6	Nurses Hostel and Champion Centre
7	Physical Medicine
8	Parafed Gym and Food Services
9	Brain Injury Rehabilitation Service
10	Boilerhouse and Site Maintenance
11	Elderly Day Care
12	Melrose Chairs and Wards 9-10 Corridor
13	Wards 7 and 8
14	Administration B and Old Surgical Block
15	Beacon House, Garage and 300 Burwood Rd
16	BSU Hostel and Chapel
17	Tapper Units
18	Milner Lodge
19	Medical Records and Pain Management
20	Overall Surface

No.	Revision	Approved	Date
G	----	----	----
F	----	----	----
E	----	----	----
D	----	----	----
C	----	----	----
B	----	----	----
A	See REVISION NOTES on Sheet 1 of this set	---	---

Scale	1:1000
Reduced	A3 1:2000
Designed	----
Drawn	DHP
Checked	MJM
Date	03/08/2012

106186.89_Burwood Engineering Services Bldg_Interim DSA Report_Rev1_14 Sept2012

Burwood Elevations Survey

Overview and Site Control

SURVEYORS · DESIGNERS · CONSULTANTS · LAND DEVELOPMENT

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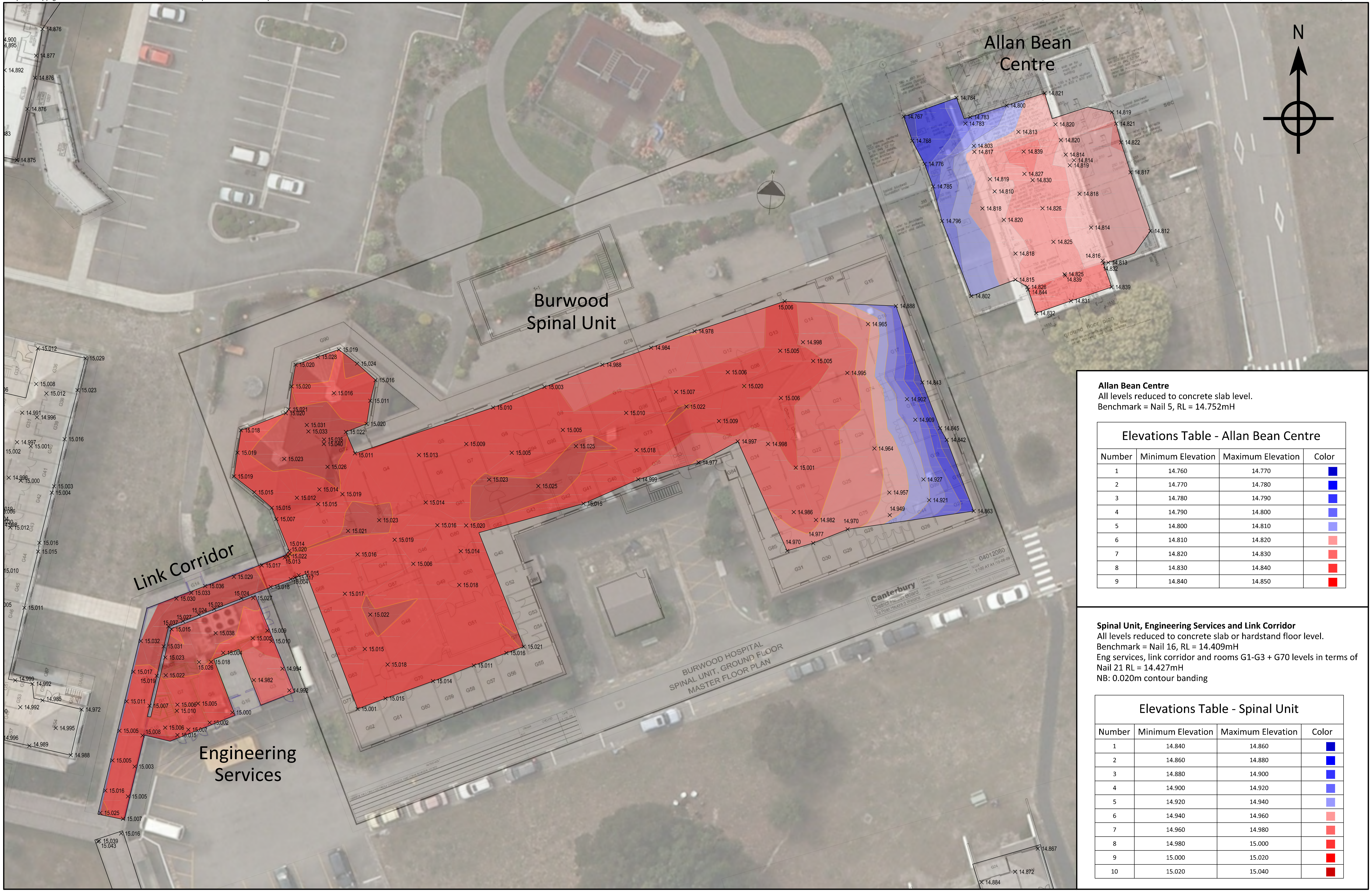
Job No. 2462C

Sheet No. 1 of 20

Rev. H

A1 Original Size

032005



Allan Bean Centre
All levels reduced to concrete slab level.
Benchmark = Nail 5, RL = 14.752mH

Elevations Table - Allan Bean Centre

Number	Minimum Elevation	Maximum Elevation	Color
1	14.760	14.770	
2	14.770	14.780	
3	14.780	14.790	
4	14.790	14.800	
5	14.800	14.810	
6	14.810	14.820	
7	14.820	14.830	
8	14.830	14.840	
9	14.840	14.850	

Spinal Unit, Engineering Services and Link Corridor
All levels reduced to concrete slab or hardstand floor level.
Benchmark = Nail 16, RL = 14.409mH
Eng services, link corridor and rooms G1-G3 + G70 levels in terms of Nail 21 RL = 14.427mH
NB: 0.020m contour banding

Elevations Table - Spinal Unit

Number	Minimum Elevation	Maximum Elevation	Color
1	14.840	14.860	
2	14.860	14.880	
3	14.880	14.900	
4	14.900	14.920	
5	14.920	14.940	
6	14.940	14.960	
7	14.960	14.980	
8	14.980	15.000	
9	15.000	15.020	
10	15.020	15.040	

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G	----	----	----
F	----	----	----
E	----	----	----
D	----	----	----
C	----	----	----
B	----	----	----
A	See REVISION NOTES on Sheet 1 of this set	---	---

Scale		1:200
Reduced	A3	1:400
Designed		----
Drawn		DHP
Checked		MJM
Date		03/08/2012

106186.89_Burwood Engineering Services Bldg_Interim DSA Report_Rev1_14 Sept2012

Burwood Elevations Survey

Spinal Unit, Allan Bean Centre and Eng Services

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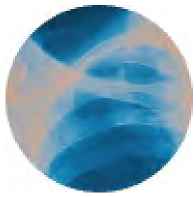
DETAILED SEISMIC ASSESSMENT REPORT

STRUCTURAL AND CIVIL ENGINEERS



BURWOOD HOSPITAL CAMPUS

REPORT 8 – NURSES HOSTEL WEST

PREPARED FOR

CANTERBURY DISTRICT HEALTH BOARD

106186.51

REPORT REV 5 – 31 OCT 2013

BURWOOD HOSPITAL CAMPUS – DETAILED SEISMIC ASSESSMENT REPORT

REPORT 8 – NURSES HOSTEL WEST (CHAMPION CENTRE, PUBLIC HEALTH NURSES AND VISION HEARING TESTERS OFFICES)

Prepared For:
CANTERBURY DISTRICT HEALTH BOARD

Date: 31 Oct 2013
Project No: 106186.51
Revision No: 5

Prepared By:

Joe Jones
PROJECT ENGINEER

Reviewed By:

Eric McDonnell
SENIOR PROJECT ENGINEER

Updated By:

Peter Grange
STRUCTURAL ENGINEER

Reviewed By:

Jenny Fisher
PROJECT DIRECTOR

Holmes Consulting Group LP
Christchurch Office

REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
12/12/11	1	Interim results of quantitative assessment (Phase 3) for discussion (some on site investigations still to be completed)
05/07/12	2	Updated format, analysis to include potential debonding at base of concrete walls, and repair and strengthening
09/07/12	3	New and updated repair and strengthening figures
30/09/13	4	Updated to include additional investigations completed
31/10/13	5	Updated for removal of chimney and brick wall removal and securing

CONTENTS

APPENDICES

TABLES

FIGURES

Holmes Consulting Group has been engaged by Canterbury District Health Board (CDHB) to complete a full structural review of the Burwood Campus following the Lyttelton Earthquake. A series of reports have been compiled as part of this process. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The specific building reports, like this one for the Surgical Block, should be read in conjunction with the base report, and refer to the repair specification.

This report identifies the structural damage sustained by the Nurses Hostel West building as a result of the series of Earthquakes, including the Darfield Earthquake that struck at 4:36am on the 4th September 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February 2011; the June Earthquake that struck at 2.20pm on the 13th June 2011 and the December Earthquake that struck at 3.18pm on the 23rd December 2011. The report summarises the effects of the damage on the lateral load capacity of the building, and provides structural repairs for the damage identified. The general form of the building, along with its capacity relative to current code levels, has been included for the buildings pre-earthquake undamaged state and post-earthquake state. Recommendations to increase the strength of the building to greater than 67% current code capacity have also been summarised.

The Nurses Hostel West building consists of two wings; a two storey West Wing and a single storey South Wing which are connected by a narrow hallway. The majority of the construction was completed in 1955 and consists of reinforced insitu concrete walls and timber framed roofs. The first floor of the West Wing has a suspended insitu reinforced concrete floor slab. The ground floor of both wings of the building have elevated timber floors spanning to continuous exterior concrete strip footings at the perimeter and are supported in the centre of the building by isolated concrete piers. In 2001, a one storey addition was added to the South Wing and wraps around the original construction. The addition is constructed of reinforced concrete block walls and flat timber roof framing.

The Champion Centre occupies the entire single storey South Wing of the building and a section of the ground floor of the two storey West Wing. The Public Health Nurses and Vision Hearing Testers occupy the remainder of the space in the West Wing.

The information available for the review included: the original 1955 architectural and structural drawings [3], the architectural drawings for the 2001 Champion Centre additions [4], a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [5], along with a level survey of the building completed by Fox & Associates [6].

For the purposes of this assessment the Nurses Hostel West building has been considered to be an Importance Level 2 building (IL2). This assumed Importance Level will need to be verified by CDHB with the Ministry of Education based upon its current use. The capacities if the building were considered an IL3 building are shown in brackets.

Based upon a review of the drawings available, and the site investigations completed, the primary lateral force resisting elements of the West and South Wings of the Nurses Hostel West building were assessed in their pre-earthquake undamaged state. The assessed capacity of the West and South Wings, relative to the demand imposed by the current loading code Design Basis Earthquake (DBE), is approximately 60% DBE (IL3 - 45%) and 45% DBE (IL3 - 35%) respectively. The limiting factor at the West Wing is the capacity of the first floor ceiling diaphragm while at the South Wing the limiting factor is the connection of the roof diaphragm of the 2001 addition to the concrete and concrete block walls.

In addition to the primary lateral load resisting elements noted above, the internal brick partition walls of the West Wing were initially assessed at between 15% and 20% DBE under face loading but have since been either removed or secured to 85% DBE (IL3 - 67%). The Porte Cochere has been assessed at 60% DBE.

The majority of the Nurses Hostel West building, particularly the West Wing, appears to have performed relatively well. The bulk of the structural damage appears to have occurred at the interface of the concrete block walls on the 2001 addition with the insitu concrete walls of original 1955 construction. The gaps that have opened up at these interface is believed to be at least partially settlement related. Earthquake induced differential settlements have been noted at both the West and South Wings, with permanent slopes in the ground floor framing of up to 1:130 or 0.77% being measured. The remaining damage is typified by cracking of the concrete walls, block walls, timber framed partition walls, ceiling linings and the slab of the service tunnel.

Addition localized damage has been noted to the brick partition walls of the West Wing toilet and washrooms along with pounding damage at the interface of the Porte Cochere with the 2001 addition of the South Wing. Damage has also been observed to the existing clay tile roof and to the exterior finishes of the Porte Cochere.

It is believed that the majority of the damage observed, including the onset of damage, occurred as a result of the 22nd February event. Further observations of the earthquake damage observed have been included in the body of this report.

While damage to the lateral load resisting system has occurred, the actual percentage reduction in the capacity of the building is hard to quantify. The differential settlement observed in the building will also have resulted in some reduction in the overall lateral load resisting capacity of the building, but again this is hard to quantify. The reduction in capacity will be the greatest at the South Wing where the perimeter footings of the 2001 addition appear to have settled away from the central portion of the wing.

Along with re-levelling of the building, the damage observed to the concrete walls, concrete block walls and first floor ceiling diaphragm, in particular, will require repair to restore the strength, stiffness, durability and performance of the individual structural components. The repair work required is outlined in Section 4. Upon completion of the recommended repairs, the lateral load resisting capacity of the building will be restored to approximately pre-earthquake levels.

Additional risks identified include the potential for the damaged clay roof tile assembly to 'shed' tiles during a significant seismic event, or the unbraced water tanks in the roof space to topple over.

The minimum repairs required to reinstate the building to its pre-earthquake undamaged condition, have been included in Section 4. In addition to the repairs, recommended strengthening concepts to increase the seismic performance of the building and bring its assessed capacity above 67% DBE have been included in Section 5.

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 Canterbury District Health Board to complete a full structural review of the Burwood Hospital Campus following the Lyttelton Earthquake. A series of reports have been completed as part of this. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The individual building reports, like this one, should be read in conjunction with the base report and refer to the repair specification.

The Burwood base report covers the purpose and scope of the structural review. The current statutory requirements relevant to earthquake damaged buildings are outlined and the level of shaking experienced at the site estimated. Earthquake induced ground settlement damage across the campus is also discussed. The repair specification has been prepared to include repair details for typical damage observed in buildings on the Burwood Hospital Campus and is referred to as required in the specific building reports.

1.1 SCOPE OF WORK

This report is on the Nurses Hostel West building, which houses the Champions Centre & Public Health Nurses Offices at the Burwood Hospital, Mairehau Rd, Christchurch. The report identifies the general form of the structure, along with the gravity and lateral load resisting systems. Each component of the structural system was reviewed based upon the information available and any potential Critical Structural Weaknesses (CSW's) were noted.

The report also identifies the structural damage observed to date as a result of the series of Earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February 2011; the June Earthquake that struck at 2.20pm on the 13th of June 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake, in particular, subjected the building to strong ground motions which significantly exceeded the current code loading demand for buildings of this nature.

The capacity of the Nurses Hostel West building has been assessed relative to current code loading in the buildings pre-earthquake undamaged state and in its post-earthquake damaged state. The post-earthquake assessment summarizes the effects of the damage identified on both the gravity and lateral load resisting elements. Repair options to restore the buildings capacity to pre-earthquake levels for strength, durability and stiffness have been included. The repair options aim to maintain the buildings utility. Where required, strengthening options have also been provided.

1.2 LIMITATIONS

Findings presented as a part of this project are for the sole use of the Canterbury District Health Board, 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 professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practising in this field at this time. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

Conclusions relate to the structural performance of the building under earthquake loads. We have not assessed the live load capacity of the floors, nor have we assessed the performance of non-structural components or building contents under earthquake loads.

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. Because all of the structure has not been available for detailed inspection or evaluation, this report is limited to those elements available and engineering judgement as to the likely condition of unseen elements. 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. PRE-EARTHQUAKE BUILDING CONDITION

This section discusses the form and capacity of the building prior to the Darfield Earthquake.

The information available for the review included: the original architectural and structural drawings [4], a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [5], along with a level survey of the building completed by Fox & Associates [6].

2.1 BUILDING FORM

The Nurses Hostel West building was constructed in two distinct phases. The original 1955 construction consists of a two-storey West Wing and a single storey South Wing. An addition to the single storey South Wing was designed in 2001 and constructed in the period thereafter.

Figure 2-1: The Champion Centre Entry and Porte Cochère

The Champion Centre occupies the entire single storey South Wing of the building and a section of the ground floor of the two storey West Wing. The Public Health Nurses and Vision Hearing Testers occupy the remainder of the space in the West Wing.

The general shape of the building mirrors that of the Nurses Hostel East building, and abuts it at the eastern end of the single storey South Wing.

2.1.1 Original 1955 Construction

The two storey West Wing of the building has a pitched roof clad with clay tiles which are supported by timber battens and roof purlins. Timber struts support the purlins at mid-span and at the ridge line and transfer the load to the top of 6" (152mm) insitu reinforced concrete perimeter and corridor walls below. The roof and ceiling framing are connected to the walls via a timber top plate which is anchored to the walls below. 6" (152mm) insitu reinforced concrete walls, perpendicular to the longitudinal perimeter and corridor walls, are located at approximately 5.3m centres. Non-load bearing timber partition walls are typically located between the concrete walls to divide the space, and are clad with fibrous plaster board. At the south end of the wing there is also a concrete water tank platform which spans between the top of the concrete corridor walls.

The first floor of the West Wing is constructed of a suspended 4.5" (114mm) insitu reinforced concrete floor slab which is supported by 7" (178mm) reinforced concrete walls below. The layout of the concrete ground floor walls is almost identical to the 1st floor walls above. Non-bearing timber framed walls, clad with fibrous plaster board, also occur at similar spacings to the floor above.

The concrete floor walls are supported by continuous reinforced concrete strip footings which are founded below ground level. The central corridor walls form a sub-floor service tunnel which is approximately 1.5m deep and retains approximately 1.0m of soil. The ground floor is typically timber framed floor and is supported by the reinforced concrete walls and isolated interior concrete piers. In isolated locations there is a suspended insitu concrete ground floor slab, which occurs at wet areas and over the partial basement at the south end of the West Wing (See Figure 2-3).

The first floor timber framed ceilings are typically clad with fibrous plasterboard, except along the corridors, and other isolated locations, which are clad with heavy plaster acoustic tiles. Each tile appears to be hard fixed to timber battens above.

At the south end of the West Wing, 4 ½" (144mm) thick unreinforced brick partition walls are located at the ground and first floor levels of the toilet and washroom areas. The original unreinforced brick chimney at the north end of the building has now been removed.

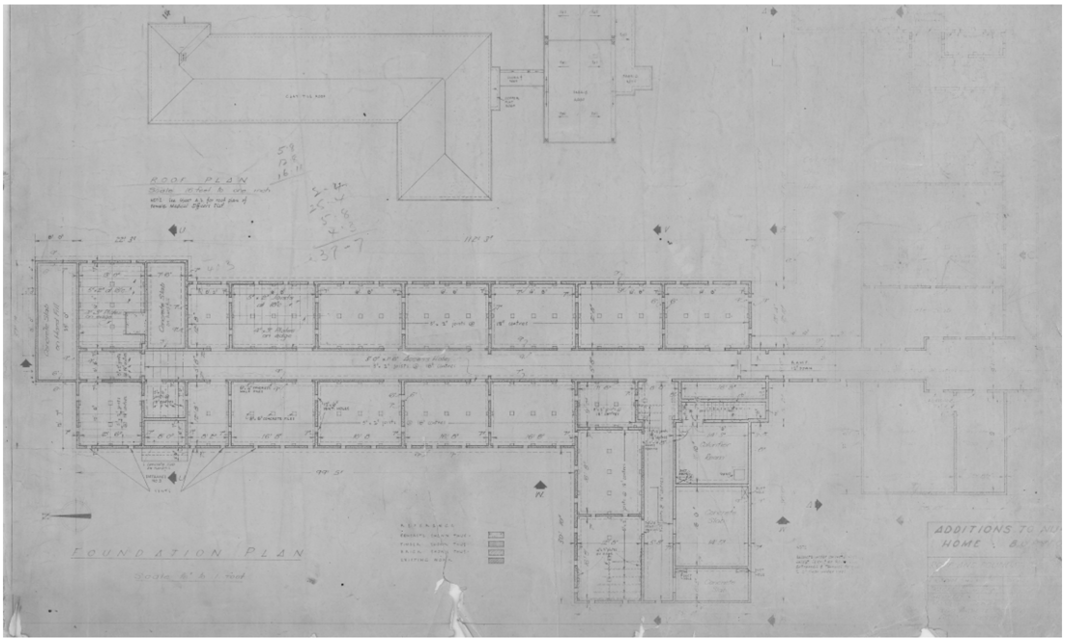


Figure 2-2: 1955 Construction – Foundation Plan

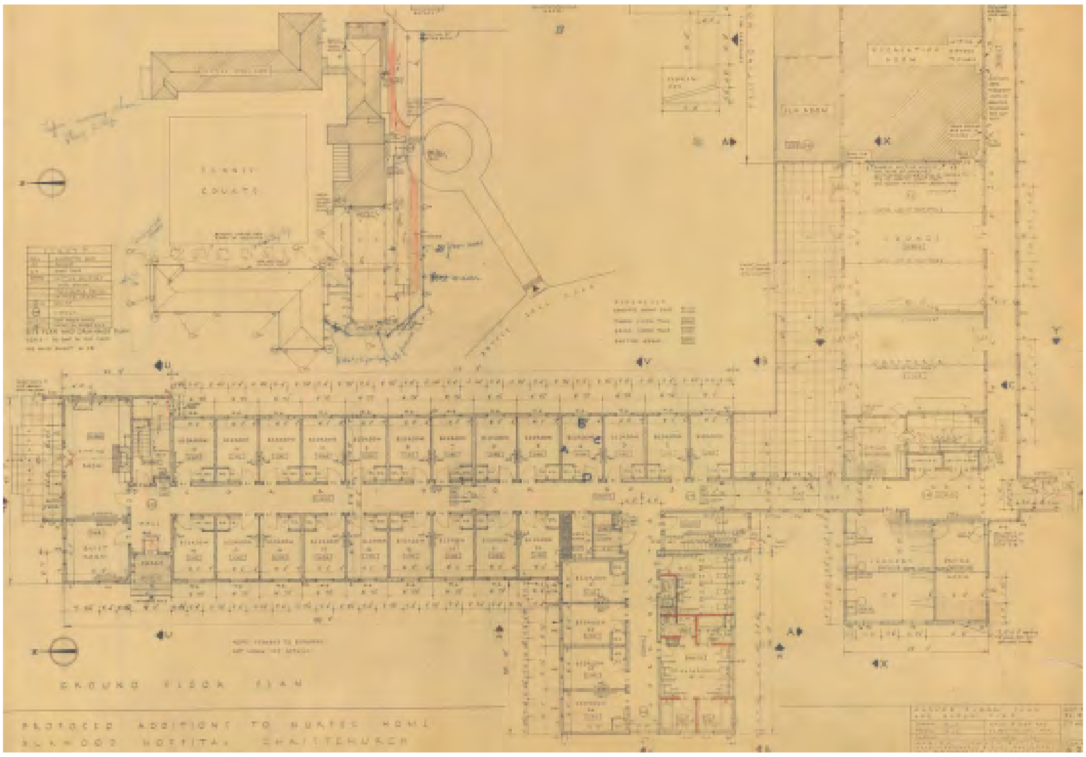


Figure 2-3: 1955 Construction – Ground Floor Plan

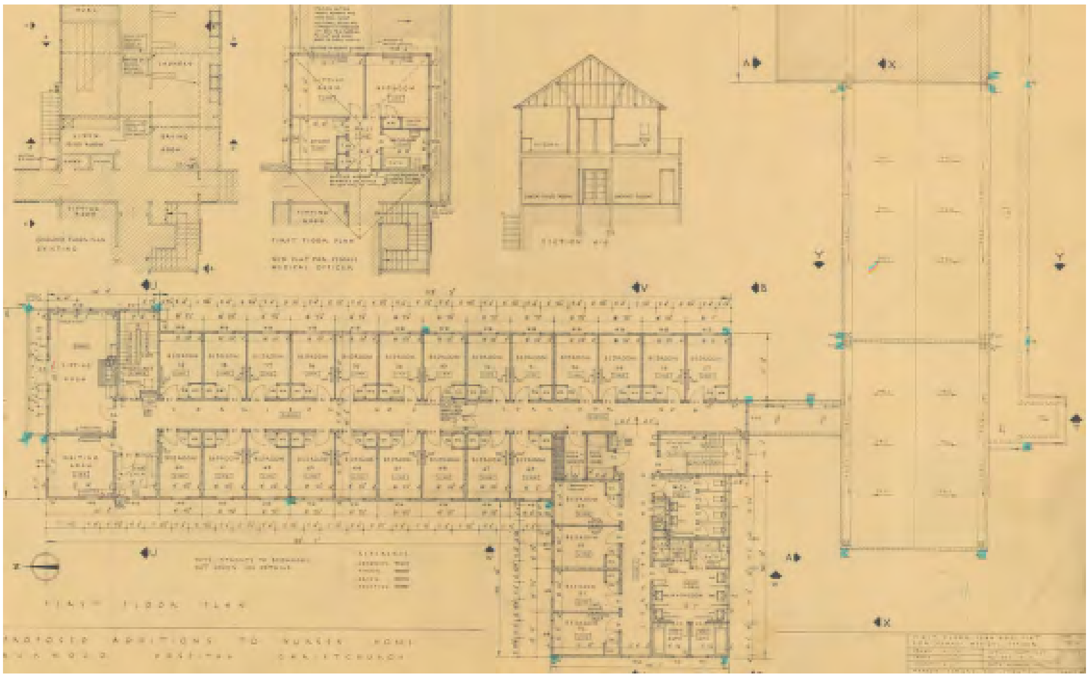


Figure 2-4: 1955 Construction – First Floor Plan

The central portion of the single storey South Wing (shown un-shaded in Figure 2-9) was constructed at the same time as the two storey West Wing. This section of the building was originally constructed with a flat roof of timber tongue and groove sheathing covered by a waterproof membrane. The sheathing is supported by timber roof purlins which span to timber roof trusses spaced at approximately 3.0m centres. The trusses are anchored to the side of 6"

(152mm) insitu reinforced concrete walls. There is also a lower timber framed mono sloped roof which spans the original corridor on the south end of the building. The majority of the original roofing has been covered by lightweight steel 'Decramastic' roof tiles which are supported by timber hipped framing constructed on top of the original roof trusses. The raised timber framed ground floor framing and foundations are similar to the West Wing.

The South Wing corridor ceilings, along with Activity Rooms 1 & 2 (see Figure 2-10) clad with heavy plaster acoustic tiles which are directly fixed to timber ceiling framing above.

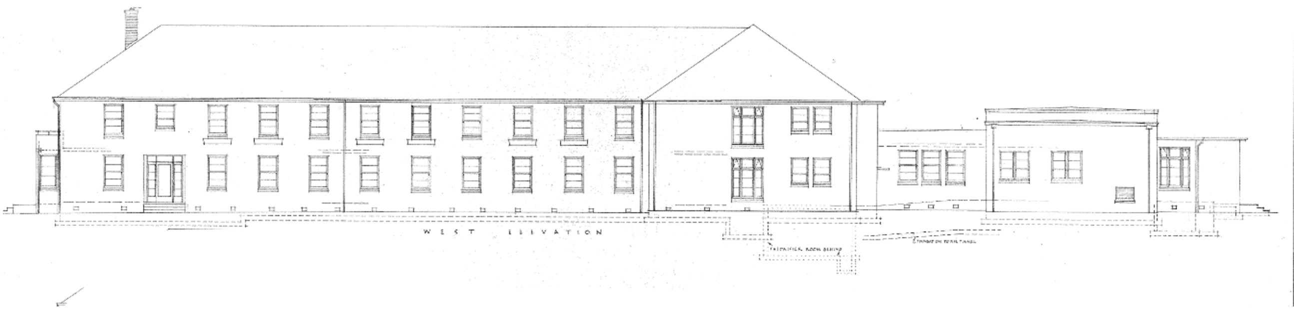


Figure 2-5: 1955 Construction – West Elevation

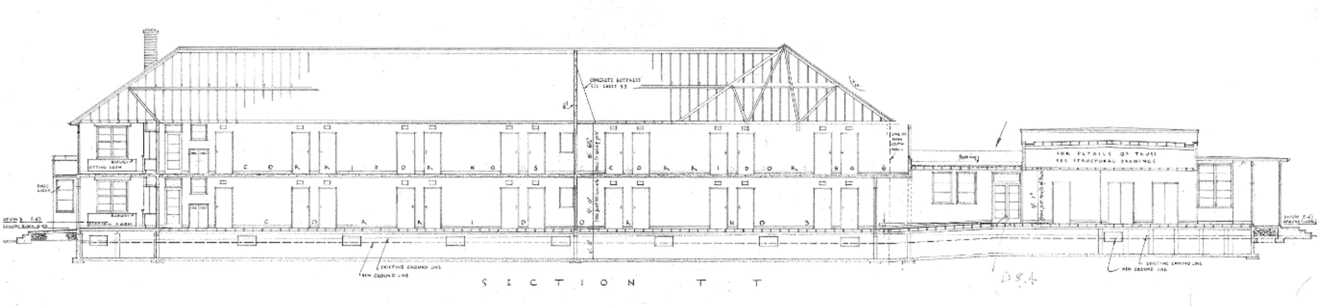


Figure 2-6: 1955 Construction – Typical Longitudinal Building Section

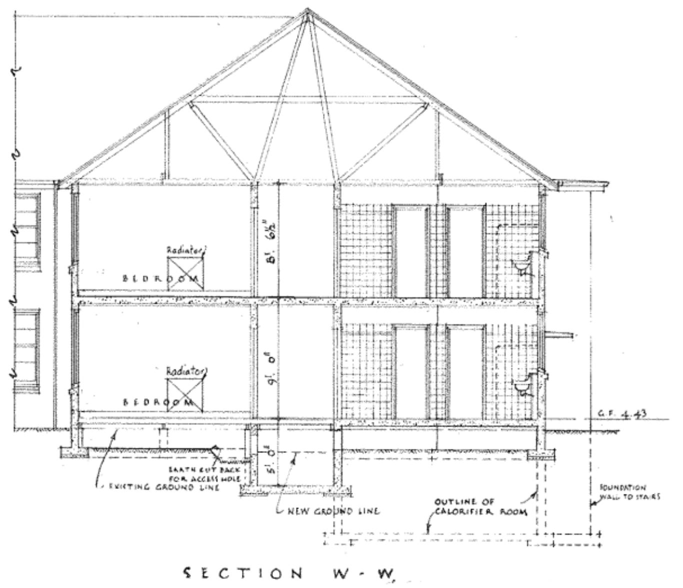


Figure 2-7: 1955 Construction – Typical Transverse Building Section

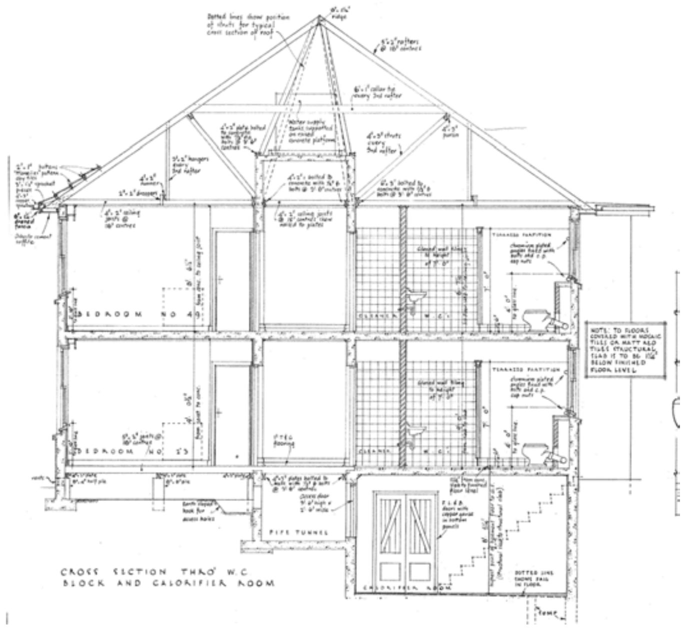


Figure 2-8: 1955 Construction –Building Section at Water Tank / Partial Basement

In the second storey attic space of the West Wing are several water tanks. Based upon observations completed to date there appears to be little or no fixings / lateral bracing of the water tanks.

Figure 2-9: Existing Water Tanks

2.1.2 2001 Champion Centre Addition

In 2001, a single storey addition was added to the South Wing of the building. The addition wraps around the original South Wing and is typically constructed of a gently sloping light weight timber roof and exterior reinforced concrete block walls (see Figure 2-9). The roof is clad with plywood sheathing and covered by a butynol membrane. The plywood sheathing is fixed to timber roofing purlins which are typically supported by the original 1955 6" (152mm) concrete walls, exterior 200 series reinforced concrete block walls, or internal timber framed bearing walls. At the concrete and block walls the roof framing is connected to walls through a timber ledger bolted to the sides of the walls with a combination of M12 bolt at 1000mm centres and M10 Dynabolts at 1200mm centres.

The ground floor addition is a raised timber floor, supported internally by isolated concrete piers and at the perimeter by the block walls and by the original insitu concrete walls. The block walls do not appear to have been constructed to a specific engineering design, but appear to comply with NZS4229:1999 [7]. The block walls are supported by continuous concrete strip footings. The connection between the block walls and the original insitu concrete walls is believed to be nominal at best.

The entry to the Champion Centre is covered by a Porte Cochère which spans over the entry driveway. The roof is constructed of plywood sheathing which is covered by a butynol membrane. The sheathing is supported by timber purlins which are in turn supported by steel girders and timber trusses which form the perimeter of the Porte Cochère. The timber trusses are supported by steel out-riggers which connect back to circular reinforced concrete cantilevered columns. The concrete columns are supported by shallow isolated reinforced concrete spread footings.

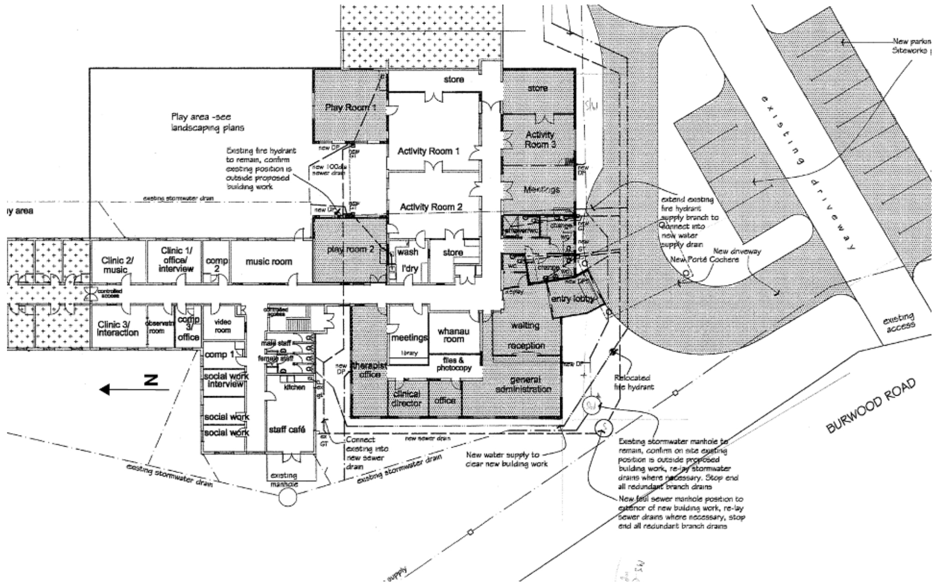


Figure 2-10: 2001 Additions – Ground Floor Plan

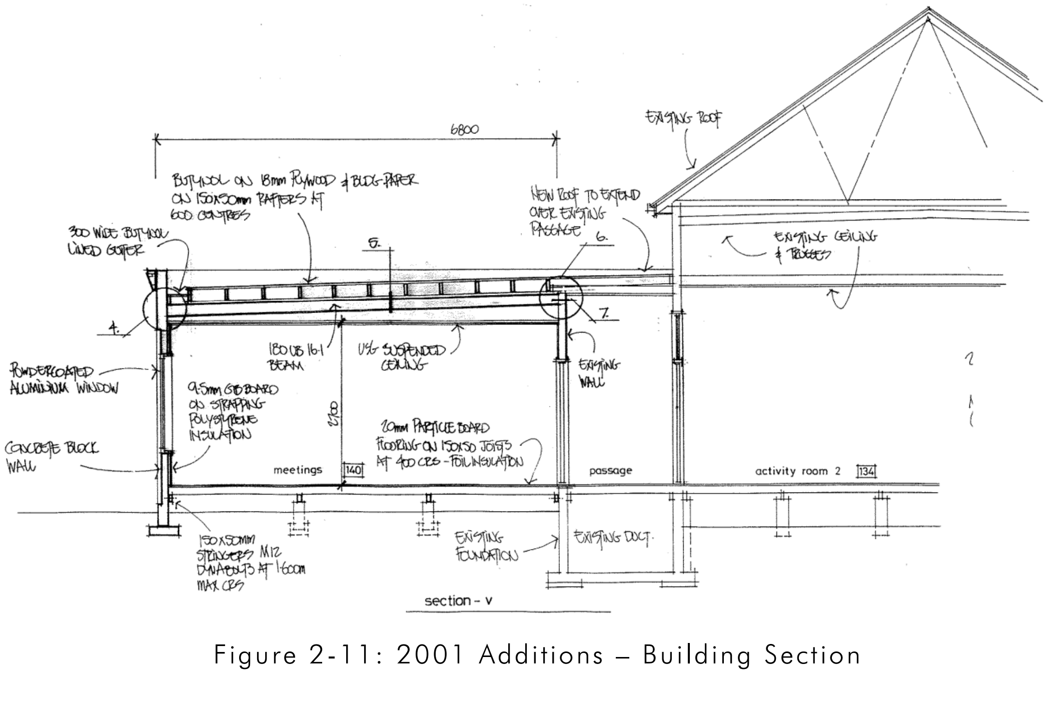


Figure 2-11: 2001 Additions – Building Section

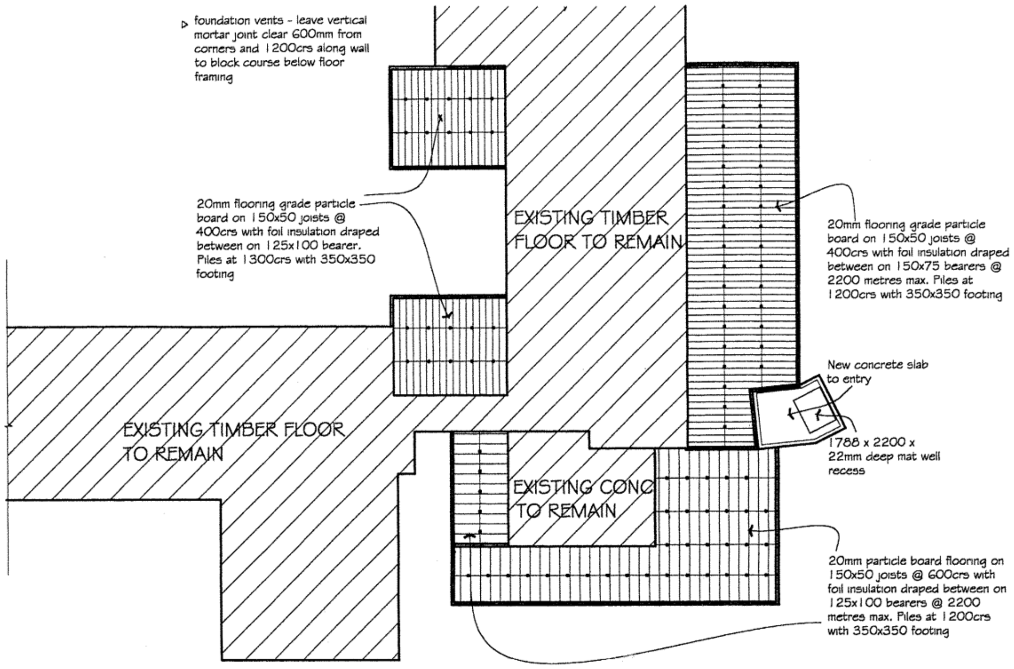


Figure 2-12: 2001 Additions – Ground Floor Framing Plan

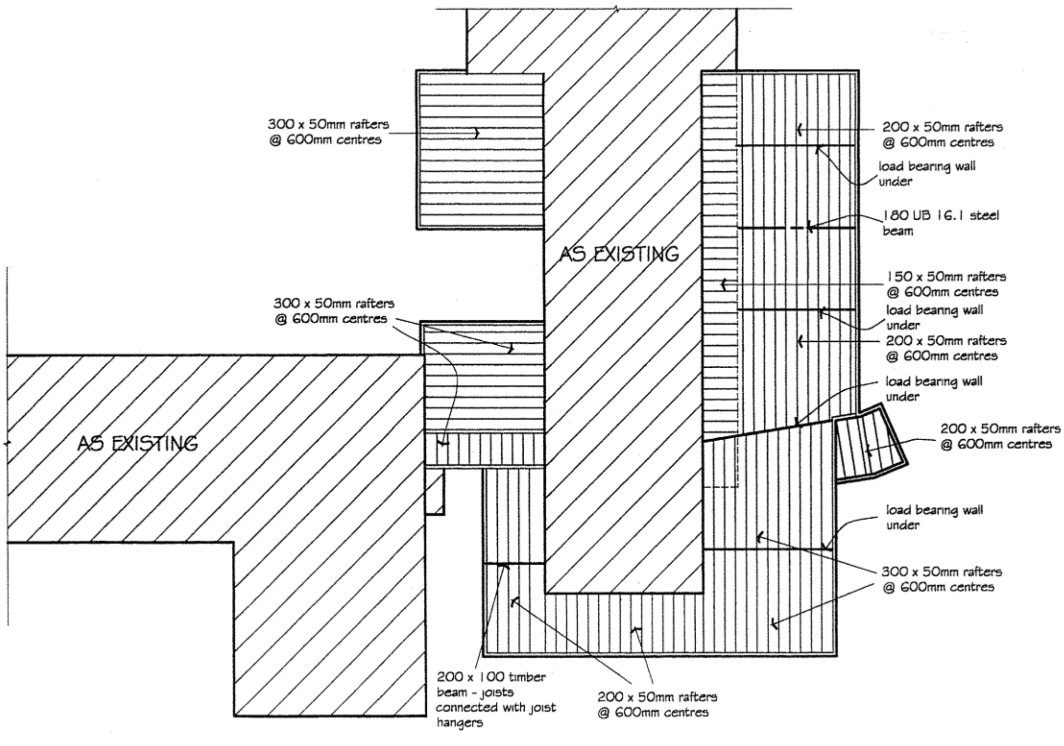


Figure 2-13: 2001 Additions – Roof Framing Plan

2.2 LATERAL LOAD RESISTING SYSTEMS

The primary lateral force resisting system of the two storey West Wing consists of insitu reinforced concrete shear walls. The walls are stacking full height (no vertical discontinuities) and are well dispersed. Based upon the original detailing of the walls, which includes smooth round reinforcing bars and short lap splices located at the base of the walls, there is a concern that the bars could become debonded from the surrounding concrete during a significant seismic event, thus losing their effectiveness.

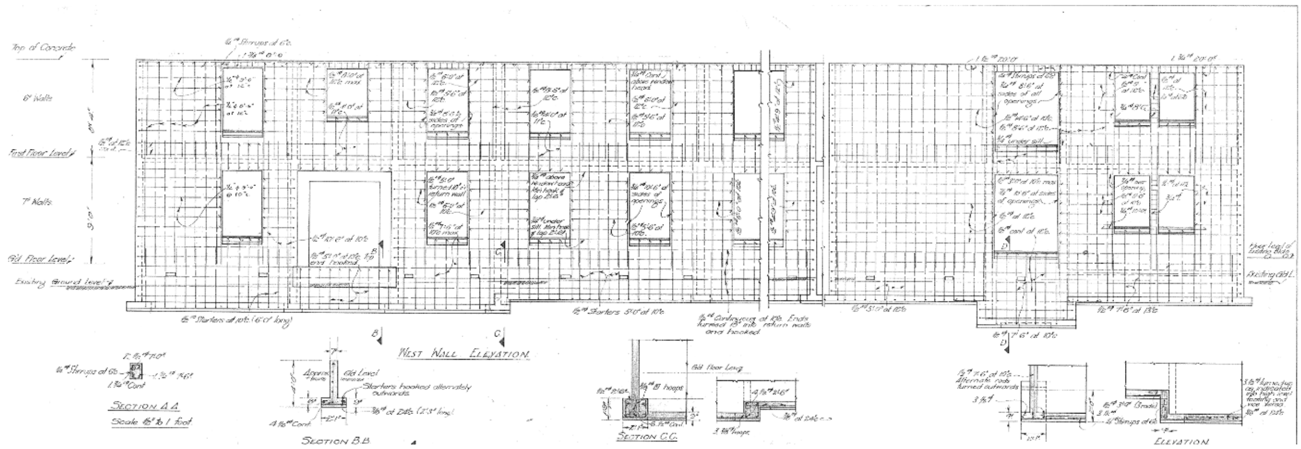


Figure 2-14: Typical Wall Elevation

In the roof plane of the West Wing there is no diaphragm or significant bracing to transfer the seismic mass of the roof tiles to the concrete walls below. Thus, in the transverse (east-west) direction of the building, lateral loads are transferred to the top of the walls directly through the roof framing. In the longitudinal (north-south) direction, lateral loads will be distributed to the perimeter walls below through weak -axis bending of the roof framing and by the roof hips at either end of the wing. At the ceiling level the fibrous plasterboard linings help distribute lateral loads in the longitudinal (north-south) direction but have insufficient strength to act as a diaphragm in the transverse (east-west) direction. In the transverse direction the top of the perimeter and corridor walls transfer seismic loads to the perpendicular walls through out-of-plane bending.

At the first floor level, the reinforced concrete slab acts as a diaphragm to distribute seismic loads to the concrete walls below which are in turn transferred to the continuous concrete footings. At the ground floor level, the tongue and groove timber sheeting over the raised floor framing acts to transfers inertial loads to the interior and exterior concrete shear walls.

The primary lateral force resisting system of the single story South Wing consists of a combination of the original 1955 insitu reinforced concrete shear walls, and the reinforced concrete block walls added in 2001. Lateral loads are distributed to the shear walls through the flexible timber roof diaphragms. At the ground floor level, the tongue and groove timber sheeting over the raised floor framing acts to transfers internal lateral loads to the shear walls which in turn transfer the loads to the continuous reinforced continuous strip footings below.

The primary lateral force resisting system of the Porte Cochère consists of a flexible roof diaphragm which transfers forces to the steel outriggers and in turn to the cantilevered reinforced concrete columns. The fixity at the base of the columns is created by shallow 1.5m square reinforced concrete pad foundations.

2.3 PRE-EARTHQUAKE BUILDING CAPACITY

2.3.1 Code Comparison

The building capacity under earthquake actions discussed in this section is compared to the capacity that a similar building would be designed to today. A new building would be designed to the *Structural Design Actions Standard, Part 5: Earthquake Actions – New Zealand*, NZS 1170.5:2004 [8] and incorporating the amendments made to this standard as a result of the Lyttelton Earthquake as outlined in the Amendment 10 of the Building Code [9]. The implications of the recent amendments are discussed more fully in the Burwood Hospital Campus Base Report, however, for a building of this type the amendments essentially result in an increase to the design loads of 36 % when compared to pre-earthquake design levels.

The original architectural and some structural drawings are available, but the structural calculations and specifications were not, so the exact design and loading assumptions originally made are unknown. For the purposes of this report seismic loading assumptions have been made based on a detailed review of the drawings available and physical observations of the building.

The original building was designed in 1955 to a predecessor of the current New Zealand Building Code; which was likely the New Zealand Standard Model Building By-Law NZSS95:1939 [10]. When these By-Laws were written, neither the seismology of the different areas within New Zealand, or the impact this could have on buildings was as well understood as it is today. Along with an increase in the seismic demands required by the change in the loading code over this period, the seismic detailing requirements have also progressed significantly resulting in more ductile and better performing buildings.

When the additions to the single storey South Wing of the building were designed in 2001 the current loading standard was the Code of Practice for General Structural Design and Design Loadings for Buildings, NZS 4203:1992 [11].

The current seismic loading code, NZS 1170.5, requires a new building to be designed for an earthquake, known as the Design Basis Earthquake (DBE), which is based upon the buildings physical location, local soil conditions, building type, fundamental period and importance level.

The Nurses Hostel West is not regarded as an essential hospital facility by the CDHB and is therefore classified as an Importance Level 2 building in accordance with NZS 1170:2004 [8] The associated return period of the DBE is 500 years, with a risk factor for design of R = 1.0 (no post-disaster or special function). The sub soil for the site is taken as Soil Type D, which is consistent with the findings of a post-earthquake geotechnical investigation [5].

Based upon the period of construction and the detailing of the time, the lateral load resisting system of the West and South Wings of the Nurses Hostel West building can be concluded to have nominal ductility. The insitu reinforced concrete and concrete block walls have been assessed with an assumed ductility of μ=1.25.

A comparison of the different design load levels for the building is plotted in Figure 2-15 and shows that based upon a fundamental building periods below 0.40 seconds.

As shown in Figure 2-15, the design loads have increased over 700% since 1955 and approximately 40% since 2001.

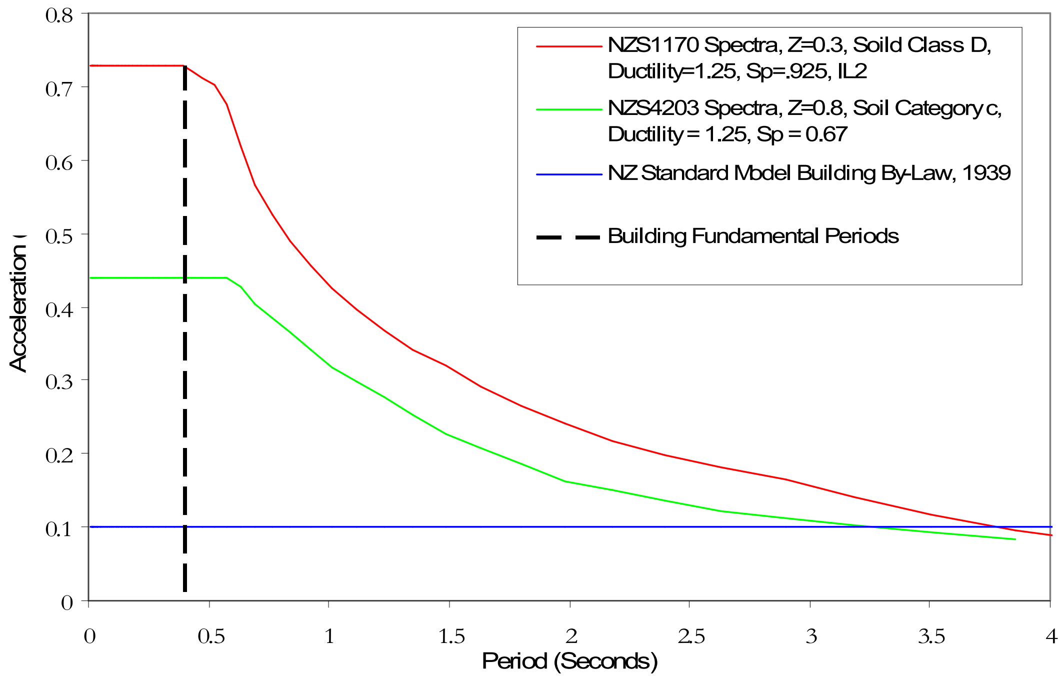


Figure 2-15: Comparison of Design Codes

2.3.2 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 estimated 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.

Following the Lyttelton earthquake, a geotechnical report was conducted by Tonkin & Taylor, titled "Burwood Hospital Post Earthquake Geotechnical Assessment", dated June 2011 [5]. This report has been used to aid in the evaluation of the site conditions and the likely effect of the ground on the buildings past and future performance. The soil parameters described in the geotechnical report complete by Tonkin and Taylor have been used for the evaluation of the buildings existing foundation system.

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 [12] and the requirements of NZS 1170:2004. 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. These are described in more detail in the Burwood Hospital Base report [1] and include strength governed elements such as short columns and deflection governed elements such as floor and stair elements with inadequate support seating.

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 (DBE), and have been expressed as a %DBE. This includes checks for both the strength and deflection requirements.

The structural analysis program, ETABS, by Computer Structures, Inc. was used in the aid of the equivalent static analysis of the West Wing.

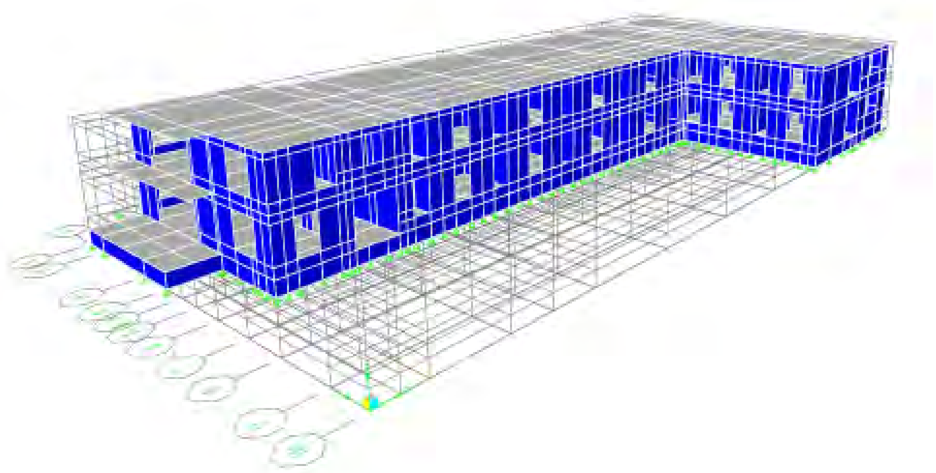


Figure 2-16: 3D Image of ETABS Model

For the purpose of this evaluation several assumptions also had to be made in regards to the existing material properties of the building. This included the assumed strength of the reinforced concrete walls (25 MPa) and the assumed grade of the smooth round reinforcing bars (33 ksi or 227 MPa).

Additional assumptions had to be made in regards to the timber framed elements of the building, specifically the existing bracing capacities of the roof, floor and ceiling diaphragms. The expected strength values for these elements were taken from NZSEE 2006, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes* [12] and ASCE 41-06, *Seismic Rehabilitation of Existing Buildings* [13]. These values could be further refined through destructive investigations of the existing materials. The assumed diaphragm and shear wall expected strength values are as follows:

- West Wing – 1st Floor Ceiling Diaphragm: Timber ceiling joists with fibrous plasterboard ceiling linings: Expected strength = 1.5kN/m with ductility, μ = 3.3.

- West Wing – 1st Floor Timber Bracing Wall: Timber framed stud walls with fibrous plasterboard cladding on each face. Expected strength = 3.0kN/m with ductility, μ = 3.3.

- South Wing - 1955 Roof Diaphragm: Straight tongue and groove board sheathing over timber roof framing. Expected strength = 2.8kN/m with ductility, μ = 3.5.

- South Wing 2001 Roof Diaphragm: 16mm plywood sheathing over timber roof framing. Expected strength = 6.0kN/m with ductility, μ = 3.5.

The foundations have been assessed at an ultimate bearing capacity of 150kPa, for factored loads, as per recommendations provided by Tonkin and Taylor.

A summary of the %DBE for each primary element has been noted in Table 2-1 & Table 2-2 below:

Building Element	%DBE (IL2)	%DBE (IL3)	Comments
Typical 1st Floor Ceiling Diaphragm – N-S E-W	60% 60%	45% 45%	Limited by bracing capacity of linings Limited by out-of-plane capacity of concrete walls
Isolated Ceiling Diaphragms – N-S North Sitting Room South Toilet/Washroom	45% 50%	35% 40%	Limited by span of diaphragm Limited by span of diaphragm
1st Floor Conc. Diaphragm – N-S E-W	100% 100%	100% 100%	
Ground Floor Diaphragm – N-S E-W	100% 100%	100% 100%	
Concrete Shear walls – N-S E-W	75% 70%	60% 55%	Limited by moment capacity at base of walls
Foundations – N-S E-W	100% 100%	100% 100%	
Brick Partition Walls – 1st Floor Gnd Floor	85% 85%	67% 67%	Remaining walls have been secured and connected into diaphragms.

Table 2-1: West Wing – Seismic Assessment % DBE

Building Element	%DBE (IL2)	%DBE (IL3)	Comments
Roof Diaphragm – N-S E-W	45% 45%	35% 35%	Limited by connection of roof framing to top of walls for in-plane shear and out-of-plane tension.
Ground Floor Diaphragm – N-S E-W	100% 100%	100% 100%	
Concrete and Block Walls – N-S E-W	100% 100%	100% 100%	
Foundations – N-S E-W	100% 100%	100% 75%	
Porte Cochere	60% 60%	45% 45%	Limited by bearing capacity of pad footings under cantilevered columns

Table 2-2: South Wing – Seismic Assessment % DBE

At the West Wing the capacity of the concrete walls is governed by the global moment capacity of the building, which has been reduced based upon the short lap lengths of the smooth reinforcing bars at the base of the walls. The analysis does reveal that yielding of the concrete spandrels over the door and window openings is likely to occur in a Design Basis Earthquake. Debonding of the reinforcing bars and/or failure of the lap splices at the base of the concrete walls is not expected to occur until approximately 80-90% DBE (IL3 - 60-70% DBE).

At the roof level, the ceiling diaphragm in the north-south direction has typically been assessed at 60% DBE (IL3 - 45% DBE) based upon the strength of the existing ceiling linings. The exception is at the north sitting room and the south toilet / washrooms. In the sitting room there is no transverse bracing walls to reduce the diaphragm span other than the timber partition wall. The span between the perimeter walls is also too far for the concrete wall to span out-of-plane, which has led to the assessed capacity of 45% DBE (IL3 - 35%).

In the south bathroom area, most of the brick walls have been removed while a plywood ceiling diaphragm and strapping across ceiling framing has been installed to take the out-of-plane loads of the remaining brick wall and to tie in the external concrete wall. This has brought the capacity of this part of this this section up to 85% DBE (IL3 - 67%).

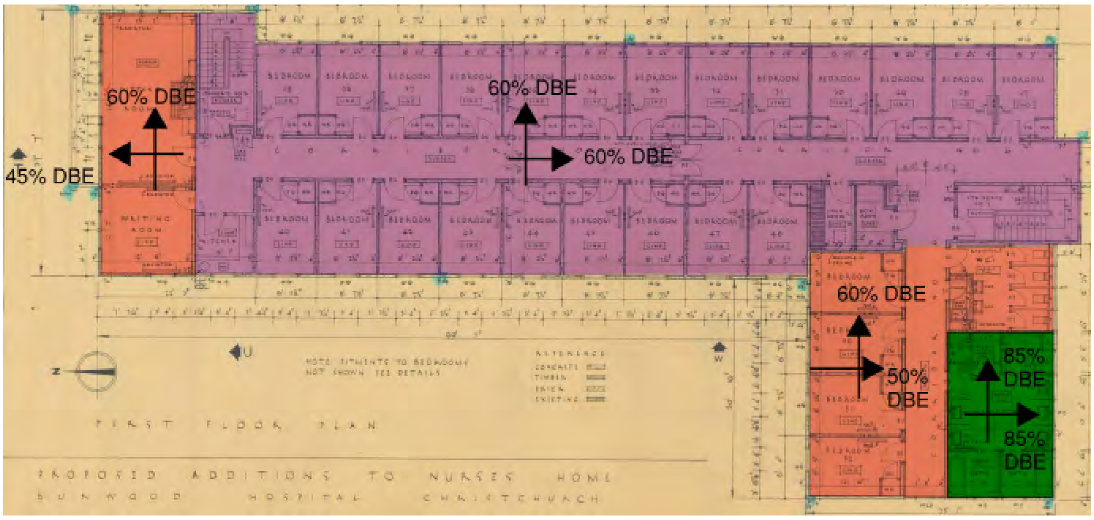


Figure 2-17: First Floor Ceiling Diaphragm - %DBE (IL2)

A review of the drawings available and site observations revealed no obvious critical structural weaknesses (CSW's) that could lead to premature collapse of the building.

3. POST EARTHQUAKE BUILDING CONDITION

This section covers the structural damage sustained by Nurses Hostel West building at Burwood Hospital Campus as a result of the series of earthquakes that includes the Darfield Earthquake that struck at 4:36am on 4th September 2010 and the Lyttelton Earthquake that struck at 12:51pm on the 22nd February 2011, the June Earthquake that struck at 2.20pm on the 13th June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd December 2011. The Lyttelton Earthquake subjected the building to strong ground motions which likely exceeded the full design earthquake load for buildings of this nature and appears to have caused the bulk of the earthquake damage observed after the initial Darfield event.

3.1 THE LYTTELTON EARTHQUAKE

The fundamental period of the buildings is estimated to be between 0.2 and 0.4 seconds. 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 strong motion data presented in the Base Report, it appears the Lyttelton Earthquake produced ground shaking intensities of approximately 60-120% of the current Ultimate Limit State design spectra for an Importance Level 2 building of nominal ductility.

It should be noted that the Lyttelton Earthquake was very short in terms of strong shaking produced with the strong motion only lasting for duration of 5-7 seconds. Rupture of an alpine fault is expected to contain 50 to 60 seconds of strong motion.

3.2 PRELIMINARY INVESTIGATIONS

Preliminary investigations were carried out to identify 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
- review of available structural engineering construction documentation
- damage observed during 'Rapid' Structural Assessments following the Lyttelton Earthquake & June 13th aftershocks
- review of previous Holmes Consulting Group assessments on the building [14],[15]

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

- cracking of the load-bearing concrete walls
- cracking of the load-bearing concrete block walls
- damage at the interface of 2001 and 1955 constructions
- connections of suspended timber flooring to foundation supports
- damage to roof framing at connections to insitu concrete and concrete block walls
- cracking in the suspended insitu concrete floors
- cracking to the concrete service tunnel
- displacement of ground around perimeter of building
- cracking in continuous concrete footings due to liquefaction induced differential settlement

A Rapid Level 2 Assessment was carried out on the 24th February 2011[16]. An additional Level 2 Assessment was conducted on the 14th June 2011 [17] following the June 13th earthquakes. Our structural observations involved a complete walk around the exterior and throughout the interior of the building. The following primary areas of damage were identified from the damage assessments:

- Cracking in external block walls
- Cracking in concrete walls particularly at openings

A review of the above information on the building highlighted this building as requiring a detailed inspection. The aim of the detailed inspection was to determine the cause and consequence of the damage as well as to determine the adequacy of the current lateral load resisting system.

3.3 DETAILED OBSERVATIONS

Further detailed inspections and structural explorations (including removal of finishes) have been carried out following the initial assessments to ascertain the full extent of structural damage. The detailed structural observations were completed between 27 September and 2 December, 2011. A full record of these observations can be found in Appendix A, with reference plans describing the location labelling used found in Appendix B. A full photographic record of the observations is available electronically on request. The detailed structural observation picked up the following damage in addition to the items noted in the initial rapid assessments:

- Separation and cracking at the interfaces of the 1955 and 2001 construction.
- Cracking throughout the service tunnel
- Cracking in the first floor suspended slab
- Cracking at top of brick partition walls
- Pounding damage between the Porte Cochère and Champion Centre fascias.

3.4 GEOTECHNICAL REVIEW

A review of the ground damage and conditions was carried out by Tonkin & Taylor for the Burwood Hospital Campus. A subsequent report titled *Burwood Hospital Post Earthquake Geotechnical Assessment* was issued in June 2011 [5]. The geotechnical review concluded that the settlement and damage to building foundations and slabs on the Burwood Hospital Campus was likely due to the liquefaction of underlying soil layers. It is believed that excessive pore water pressures have been dissipated and that further settlement is not expected, unless another significant event were to occur.

Based on this report and from our detailed damage observations both internally and externally it does not appear that the overall stability of the Nurses Hostel West building has been affected by earthquake induced settlement.

Based on the geotechnical report provided by Tonkin & Taylor [5] the potential for future total and differential settlements at the building site varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

3.5 LEVEL SURVEY

A detailed survey of the ground floor levels in the Nurses Hostel West building was conducted by Fox & Associates and issued on 18th November, 2011. An additional survey following the earthquakes on 23rd December 2011 and 2nd January 2012 was completed on 1st February 2012.

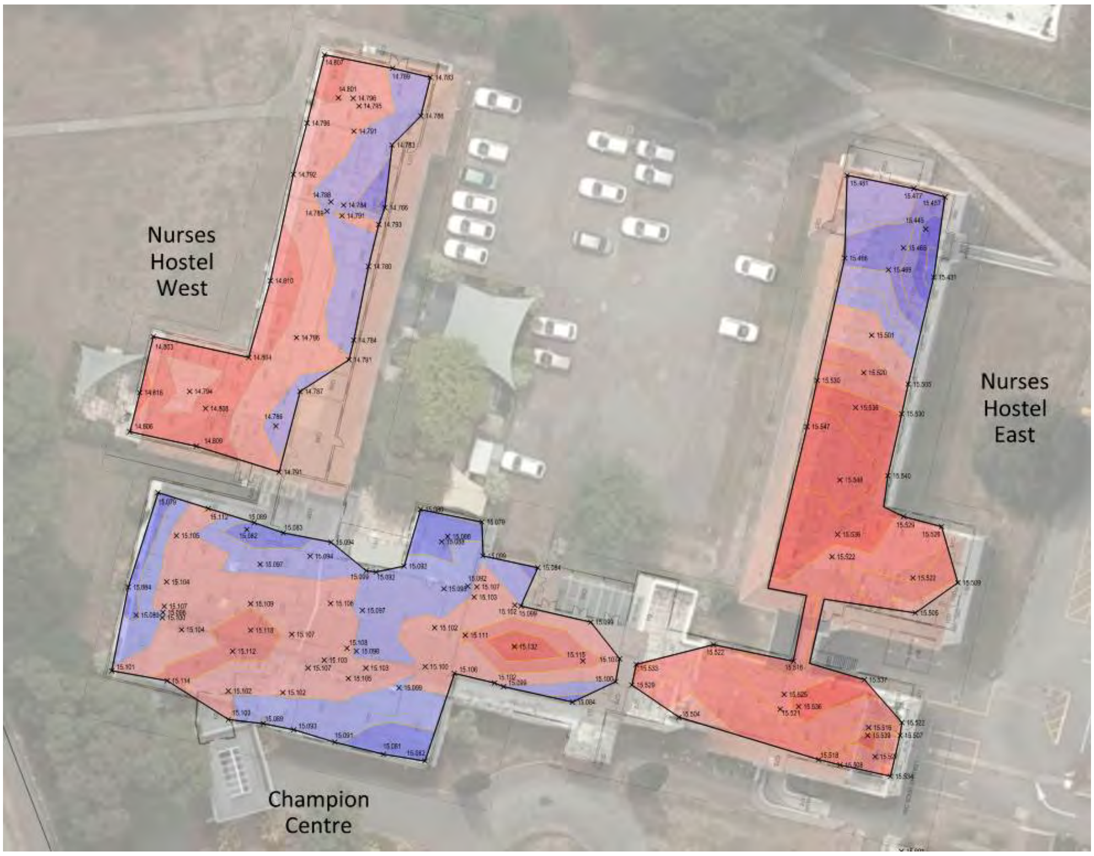


Figure 3-1: Level Survey- Ground Floor

The survey has indicated differential settlement at both the West and South Wings, resulting in permanent slopes in the ground floor framing. At the West Wing, a total differential settlement of 50mm was noted, with a typical drop of approximately 30mm was noted across the width of the building, resulting in a slope of approximately 1:340 (or 0.3%). Additional localized slopes in the floor framing of approximately 1:170 (0.59%) have been measured.

At the South Wing a total differential settlement of 53mm has been noted. In general, it appears as though the footings of the 2001 addition have settled more than the central 1955 portion of the South Wing, resulting in worst case slopes in the floor framing of approximately 1:130 (or 0.77%).

On 22nd January 2013 an additional survey was completed for the suspended first floor slab. The results of this survey indicate a total differential in measure elevation of 83mm across the slab. Worst case measured slopes of approximately 1:75 occur at the north end of the wing.

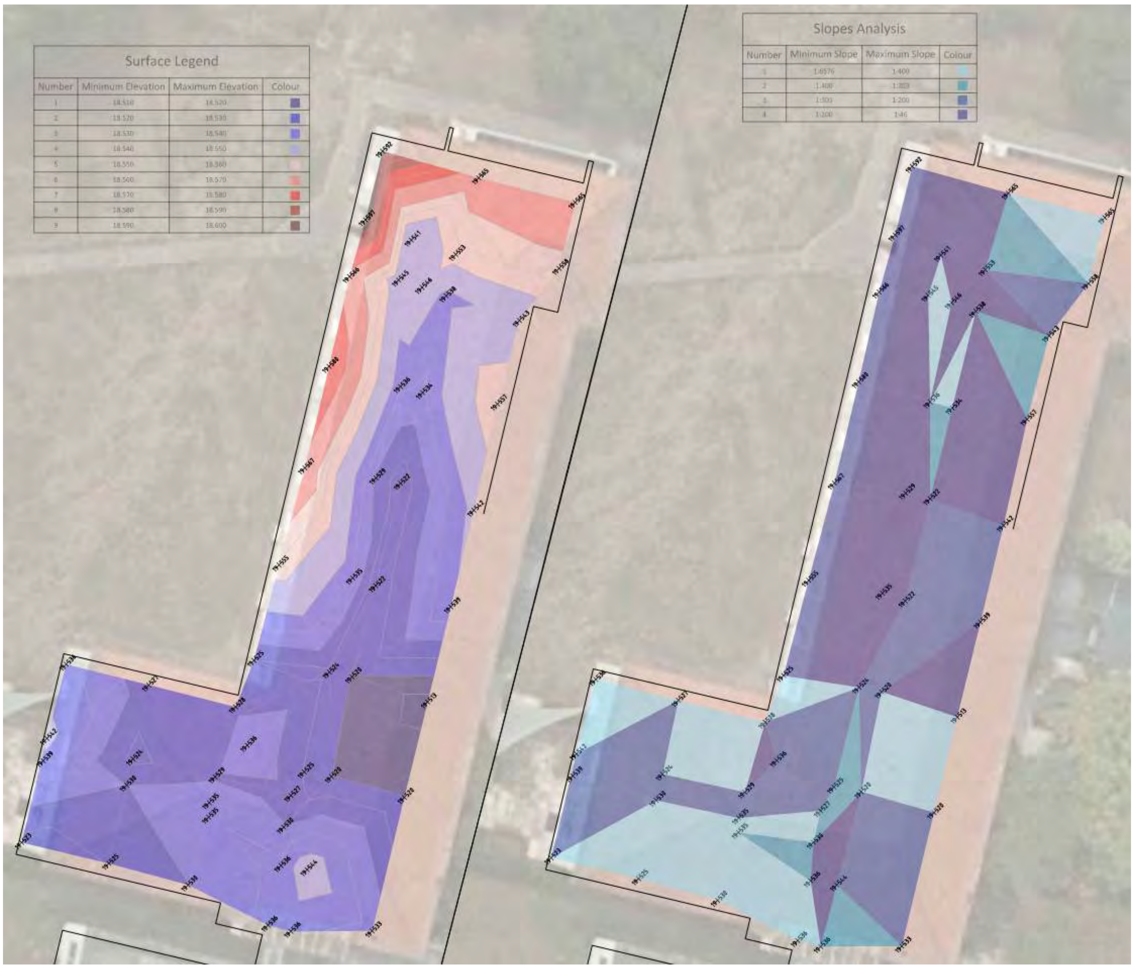


Figure 3-2: Level Survey- First Floor

The worst case slopes noted in the ground floor and first floor framing are outside the typical acceptable range and require re-levelling. For further discussion on re-levelling see Section 4.2.

The full level survey completed for the ground floor of the Nurses Hostel, and for the first floor of the West Wing, has been included in Appendix C.

3.6 SUMMARY OF BUILDING DAMAGE

The following is a summary of the observations made for the building, and our conclusions as to their condition and seismic load resisting capacity. These observations do not specifically distinguish between damage caused by the Darfield Earthquake, the Lyttelton Earthquake or any significant aftershocks, such as those that occurred on 13th June 2011, 23rd December 2011 or 2nd January 2012. Despite not being able to specifically distinguish when individual damage observed occurred, it is believed that the majority of the damaged (or the onset of damage) can be linked to the February 22nd event.

Our observations suggest that the building would have undergone a limited number of full cycles of primarily elastic deformation. The short duration of the strong ground motion recorded and the damaged observed would support this hypothesis.

The majority of the Nurses Hostel West building, particularly the West Wing, appears to have performed relatively well. The bulk of the structural damage appears to be settlement related or occur at the interface of the 2001 addition with the original 1955 construction.

The remainder of the damage to the structure can be considered minor or moderate damage and is typified by cracking to the reinforced concrete walls, particularly off the corners of doors and windows openings, and minor cracking to the reinforced concrete floor slabs. This includes cracking to the basement service tunnel walls. Damage has also been noted in the exterior brick veneer, interior plaster linings and other non-structural items. A summary of the typical damage observed is as follows:

- **Differential Ground Settlement -** Permanent slopes in the ground floor framing of the West and South Wings as a result of earthquake induced differential ground settlement.

- **Cracking to Reinforced Concrete Walls** – Typical cracking of up to 0.5mm has occurred in the concrete walls of the ground and first floor levels, particularly off the corners of windows and door openings. Typically cracking of up to 0.7mm has also occurred in the sub-floor and service tunnel walls at areas of reduced section (for vents and services).

- **Separation of Block Walls at Interface with Original 1955 construction** – At the South Wing, separation and cracking has occurred at the interface of the concrete block walls of the 2001 addition with the insitu concrete walls of original construction. It is believed the original connection between the two elements was insufficient and likely exacerbated by the differential ground settlement noted.

- **Cracking of Service Tunnel Slab** – In the service tunnel an upward bow and flexural cracking of the slab has been noted. This is likely due to the service tunnels settling more than the slab.

- **First Floor Ceilings** – At the first floor level of the West Wing, the ceiling linings are serving as a diaphragm in the longitudinal direction of the building. Typical cracking in the fibrous board ceiling has occurred in places particularly off of wall corners.

- **Cracking to Finishes -** Cracking to wall cladding and non-structural elements such as window reveals, door jambs and ground floor ceiling finishes has been noted throughout.

Isolated damage of note to the building is as follows:

- **Service Tunnel Walls** - In the service tunnel, below the South Wing there is a flexural 'pinwheel' crack in the side wall of the tunnel at the interface of an in-framing continuous concrete footing and concrete wall above. Vertical cracking of 1mm has also been noted in the service tunnel walls directly below the interface of the 1955 and 2001 constructions.

- **First Floor Slab -** Small lateral cracks, which extend the entire breadth of the first floor slab, where noted in two locations. In one location the crack has propagated down through the external ground floor wall. Both cracks appear to have occurred at existing construction joints.

- **Brick Partition Walls –** At the West Wing vertical cracks have been noted at the top of the brick partition walls of the toilet and washrooms.

- **Water Damage** – At the South Wing, water damage to interior ceiling tiles was noted in several locations at the interface between the 2001 and original 1955 construction. This may indicate separation in the roof framing has occurred and will require further investigation.

- **Porte Cochere -** Pounding damage between the Porte Cochère and South Wing fascia of the 2001 addition.

Damage to the clay tile roofs was not noted during the investigations, but they were not easily visible from ground level. Our investigations in the roof space were also limited. Based upon our experience with other clay tile roofs on the campus, and the lack of bracing in the roof plan, we would recommend that a thorough investigation of the roof tiles be completed by a qualified roofing contractor.

In Section 4, Table 4-1 provides a photographic summary of the typical damage observed. A full record of our detailed observations and repairs required can be found in Appendix A.

3.7 ADDITIONAL INVESTIGATIONS REQUIRED

Several assumptions were made in the completion of the pre-earthquake (undamaged state) and post-earthquake (damaged state) structural assessments. Additional investigations are required in a number of locations in order to verify these assumptions. *In response to SR1 (9 July 2012), the following investigations have been carried out.*

3.7.1 Investigations Required For Further Assessment

- At the West Wing, a detailed assessment of the condition of the existing clay tiles, including the fixings to the timber roof framing, is recommended to be completed by a qualified roofing contractor.

  An assessment of the clay roof tile assembly was completed by Wayman Roofing Services LTD. The report, dated 15th August 2012, indicates some movement in the tiles but to a much lesser extent than the Nurses Hostel East building. The damage noted to the tiles appears to be concentrated at the ridges and valleys (particularly at the north gable end). This report suggests that this damage will be difficult to repair due to the fragility of the tiles and the unavailability of the hips and ridge tiles used. It is recommended therefore, that the roof tiles are replaced with lightweight steel roofing and plywood diaphragm.

  Of perhaps bigger concern is borer damage noted in the timber battens supporting the tiles. This appears to be widespread but does not appear to extend to the roof purlins. It is difficult to quantify the extent of any reduction in capacity but it is recommended that the battens are removed and replaced should the roof tiles be replaced with lightweight steel roofing.

- At the West Wing, the brick partition walls of the toilet / washroom areas have been assessed at a low percentage in their pre-earthquake undamaged state. Earthquake induced cracking has also been noted at the top of the walls. As such we are recommending their removal and replacement with a light weight alternative. Prior to removal provide detailed mapping of the cracks on the wall surfaces for insurance purposes.

All the exposed walls have cracking in them.

- At the West Wing, expose the surface of worst cracks noted in a ground floor concrete spandrel spanning over an internal door opening to determine the crack width in the base concrete material.

  The worst case crack noted in a spandrel beam occurs at what appears to be an existing vertical construction joint above the door entrance to Playroom 2. The crack varies in width between 0.2 and 1.2mm. More typical diagonal cracking off the corner of door openings appears to be limited to approximately 0.5 to 0.8mm in width. The repair for the cracks in these walls is included in Section 4.

- At the West Wing, the adequacy of water tank fixings to the concrete platform below needs to be checked.

  No fixings have been provided at the base of the water tanks to the concrete plantforms, nor has any lateral bracing been provided at the top of the tanks. We recommend that lateral restraint for the tanks be provided.

- At the West Wing, complete an additional level survey of the first floor slab.

  The requested survey was completed by Fox & Associates. A summary of the findings is included in Section 3.5 and a copy of the survey has been attached in Appendix C.

- At the West Wing, investigate existing brick chimney for potential damage.

  The chimney has now been removed.

- At the South Wing, further investigations of the roof framing of the 2001 addition to the concrete and concrete block walls is recommended. This includes not only the fixings of the timber ledger to the walls but also a review of the existing waterproof membrane.

  The investigations completed by Naylor Love noted that the timber ledgers are connected to the concrete and concrete block walls with M12 Trubolts as originally assumed. No damage to the sample fixings observed was noted. The existing waterproof membrane was inspected by Wayman Roofing Services LTD in their report dated 15th August 2012. This suggests that the overall condition of the substrate and the butynol is good with a few minor issues where the butynol is stressed and may eventually puncture. This requires further investigation before a repair can be specified. The report is given in Appendix D.

- Investigate cracks in Porte Cochere columns to determine if cracks extend into base concrete material.

  The additional investigations completed by Naylor Love revealed that the cracks observed on the surface of the columns are a result of the plastic concrete forms and do not extend into concrete columns themselves. No repair work is therefore required.

3.7.2 Investigations to be Completed During Building Repairs

- At the West Wing, during the first floor ceiling repairs, the adequacy of the roof and ceiling framing fixings to the top of the perimeter and interior concrete walls needs to be confirmed.

3.8 POST-EARTHQUAKE BUILDING CAPACITY

Based upon our investigations to date, we do not consider the West and South Wings of the Nurses Hostel West building to have any notable reduction to the overall gravity load resistance of the structure. While damage to the lateral load resisting system has occurred, the actual percentage reduction in the capacity of the building is hard to quantify.

The damage observed to the concrete walls, concrete block walls and first floor ceiling diaphragm in particular will require repair to restore the strength stiffness, durability and performance of the individual structural components. The repair work required is outlined in Section 4. Following the recommended repair of the structural damage, the lateral load resisting performance of the structure will be restored to approximately pre-Darfield earthquake levels, which are summarized in Tables 2-1 and 2-2. (Approximately 60% DBE (IL3 - 45%) for the West Wing, 45% DBE (IL3 - 35%) for the South Wing and 60% DBE for the Porte Cochère).

The differential settlement observed in the building will also have resulted in some reduction in the overall lateral load resisting capacity of the building. The reduction in capacity will be the greatest at the South Wing where the perimeter footings of the 2001 addition appear to have settled away from the central portion of the building of original construction. While the effects of the differential settlement noted for the rest of the building are less severe, the settlements noted will have result in some reduction to the capacity of the building. In addition the settlements noted will limit the ability of the building to absorb future differential settlements before severe distress to the structure occurs.

In its pre-earthquake and post-earthquake state the primary lateral load resisting elements of the building have been assessed to have a capacity greater than 33% DBE, and as such the building is not considered to be "Earthquake Prone." While the interior brick partition walls are not part of the main lateral load resisting system they were assessed at between 15% and 20% DBE for face loading and thus are considered "earthquake prone" elements. The cracks noted in the walls as a result of the earthquakes have further reduced their capacity. As per recommendation, the brick walls have all been either removed or secured using LVL studs.

It should be noted that when compared to the loading code prior to the earthquakes, the brick partition walls would have been assessed below 33% DBE, and thus would have been considered "earthquake prone" elements prior to the earthquakes. Amendment 10 [9], which was put into place following the Lyttleton Earthquake, essentially resulted in an increase to the design loads of 36 % when compared to pre-earthquake design levels.

In addition to the minimum repairs, recommended strengthening concepts to increase the seismic performance of the building, and bring the assessed capacity above 67% DBE have been included in Section 5.

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4. OBSERVED DAMAGE AND REQUIRED REPAIRS

4.1 TYPICAL DAMAGE & REPAIRS REQUIRED

This section covers the damage noted during our detailed assessment of the building. Note that our observations have been restricted to structural aspects of the building 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.

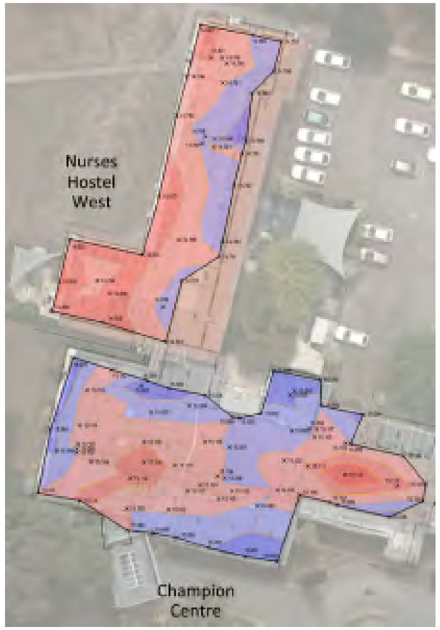
Table 4-1 provides a photographic summary of the observed typical damage and typical repairs required for the Nurses Hostel West building. Table 4-1 should be read in conjunction with Appendix A – Record of Observation and Appendix B – Reference Plans which provide the complete extent of the observed damage. The Repair Specification [2] referred to in Table 4-1 has been issued separately.

In general, the aim of the repair work indicated in this section is to restore the structure to its pre-earthquake state, as far as practicable, while maintaining the utility of the building. The repairs presented attempt to address the loss of strength, stiffness and durability of the structural elements due to the damage noted. Recommendations for strengthening works to achieve 67%DBE are included in Section 5.

It should be noted that more damage may be identified during the repair works and (if required) additional repair details will be specified accordingly.

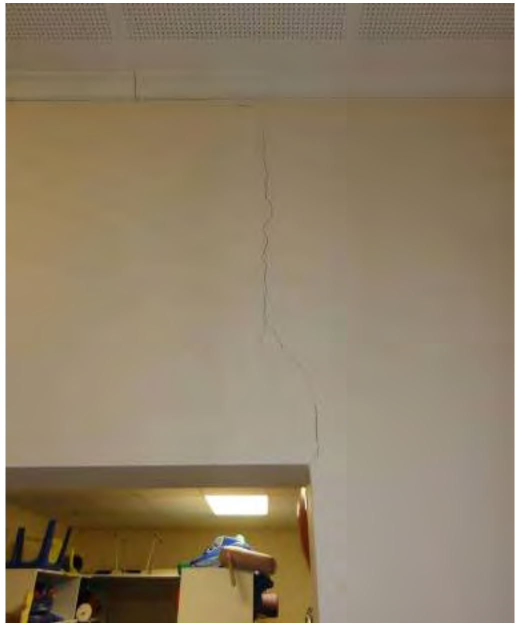
Further recommendations for improvement to the buildings seismic performance have been included in Section 5.

Table 4-1: Photographic Summary of Primary Damage Observed and Repairs Required

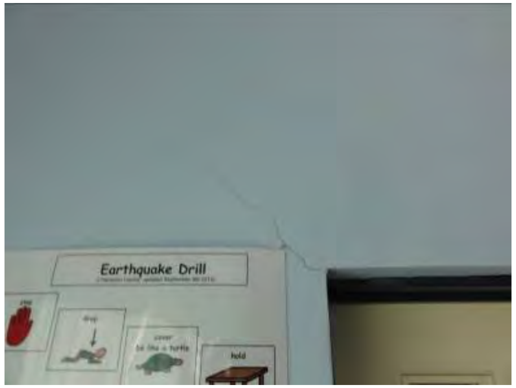
Damaged Item	Photo Ref: Location	Recommended Repair	Example
1. Foundations and Service Tunnel			
1.1. Differential ground settlement resulting in permanent slopes in the ground floor framing of the West and South Wings of up to 1:170 (0.59%) and 1:130 (0.77%) respectively.	Refer: Appendix C - Survey of Levels	The slopes noted in the ground floor of the West and South Wings of the building are outside the typical acceptable levels for timber framed construction and will require repair. For additional discussion on re-levelling see Section 4.2. Note: All proposed re-levelling of existing foundation elements is to occur prior to any additional repairs.	

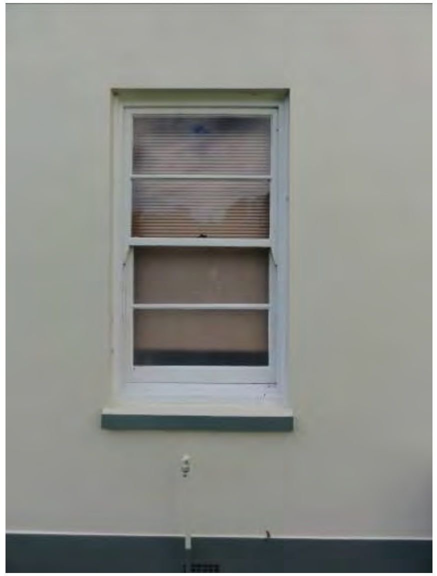
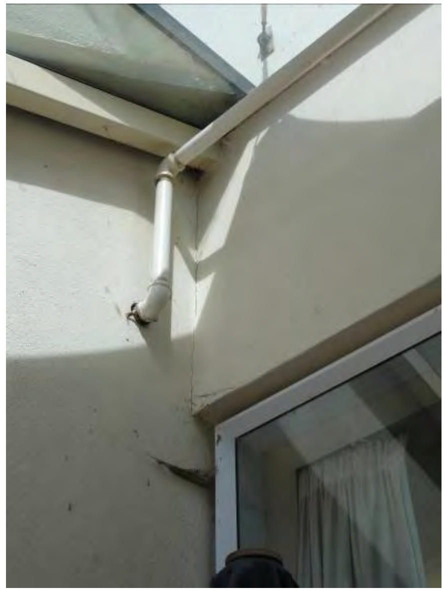
Damaged Item	Photo Ref: Location	Recommended Repair	Example
1.2. Vertical crack to tunnel wall. Pile of sand at base of crack. Crack appears to correspond with cracks noted at Ground Floor level	062: Service Tunnel	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [2]. For cracks greater than 0.5-0.6mm, HCG to confirm integrity of existing reinforcing steel. See Section 4.3 for additional discussion on repairs to concrete elements.	
1.3. Longitudinal crack running down the approximate centre of the service tunnel slab. Appears as typical throughout the service tunnel.	066: Service Tunnel (typical throughout service tunnel)	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [2]. For cracks greater than 0.5-0.6mm, HCG to confirm integrity of existing reinforcing steel. See Section 4.3 for additional discussion on repairs to concrete elements. *Inspection / repair of waterproof membrane to be completed by other.*	

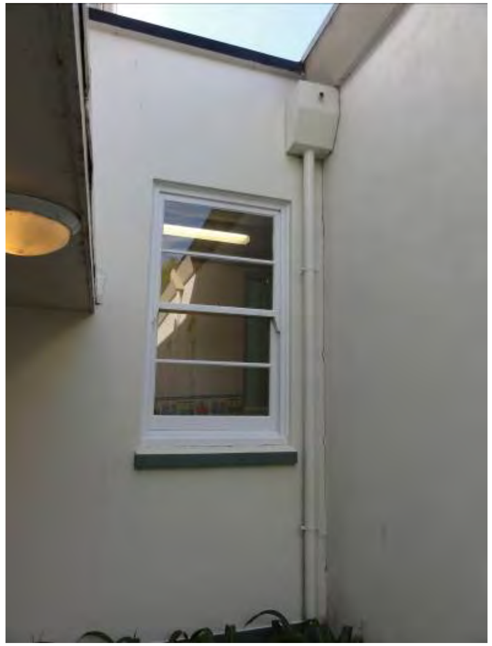
Damaged Item	Photo Ref: Location	Recommended Repair	Example
1.4. ~1mm vertical and diagonal cracks in tunnel wall propagating from vent openings of 1955 construction. Appear as typical throughout the service tunnel.	069: Service Tunnel Walls (typical throughout service tunnel)	Refer to item 1.2	
1.5. Diagonal cracks <1.5mm propagating from service openings in service tunnel walls (typical to service tunnel)	071: Service Tunnel (typical throughout service tunnel)	Refer to item 1.2	
1.6. 'Pinwheel' of 5 cracks 0.5-1mm propagating from single point in tunnel wall. No obvious horizontal displacement was noted. Photo 074 shows a perpendicular wall that abuts the service tunnel wall at the approximate location of the cracks.	072: Service Tunnel	Refer to item 1.2	

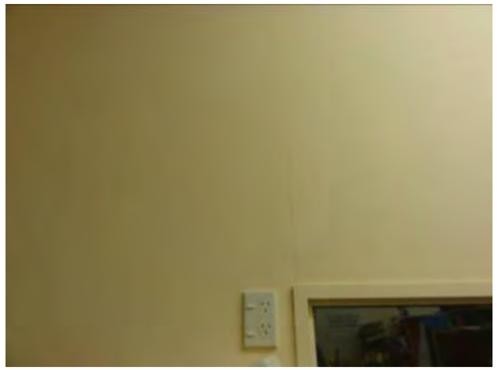
Damaged Item	Photo Ref: Location	Recommended Repair	Example
2. Ground Floor Walls			
2.1. 0.2 – 1.5mm cracks in finishes over insitu concrete and concrete block walls *(removal of finishes revealed cracks in the base concrete between 0.5 to 0.8mm)*	003: GND Level Store	At all visible cracks, remove finishes to expose crack to base insitu or block wall. Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [2]. For cracks greater than 0.5-0.6mm, HCG to confirm integrity of existing reinforcing steel. See Section 4.3 for additional discussion on repairs to concrete elements. For aesthetic repair of finishes, see repair specifications by others.	 

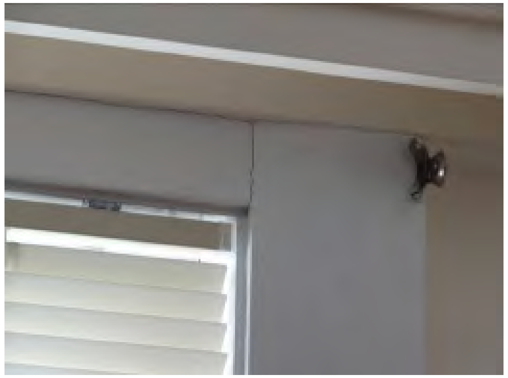
Damaged Item	Photo Ref: Location	Recommended Repair	Example
2.2. 5mm vertical crack in finishes over insitu concrete and concrete block walls *(removal of finishes revealed cracks in the base concrete between 0.2 and 1.2mm)*	025: GND Level Corridor	Refer to Item 2.1	 

Damaged Item	Photo Ref: Location	Recommended Repair	Example
2.3. Vertical cracks each side of concrete infill panel above doorway.	139: GND Level Store Room	Refer to Item 2.1	
2.4. 0.7mm crack at door head in cladding.	027: GND Level, Playroom 2	Refer to Item 2.1	

Damaged Item	Photo Ref: Location	Recommended Repair	Example
2.5. 0.5mm tapered diagonal crack in external insitu concrete wall. Crack propagating from bottom right and top left corners of window penetration.	103: GND Level External wall, appears to coincide with 1st Level slab crack refer item 4.1	See Item 2.1	
2.6. 1mm crack at interface between 1955 insitu concrete wall and 2001 concrete block walls	020: Play area	Based upon the extent of the damage observed the existing connection between the concrete block walls of the 2001 addition and the concrete walls of the original 1955 construction are insufficient. New connections at these locations will be required. See Section 4.4 for additional discussion.' Note: Re-levelling of the perimeter footings maybe required to close the "gaps" formed between the concrete block and concrete walls.	

Damaged Item	Photo Ref: Location	Recommended Repair	Example
2.7. 10mm full height vertical crack between 1955 insitu concrete wall and 2001 concrete block walls	082: GND Level external wall at interface between 1955 and 2001 structures	See Item 2.6.	

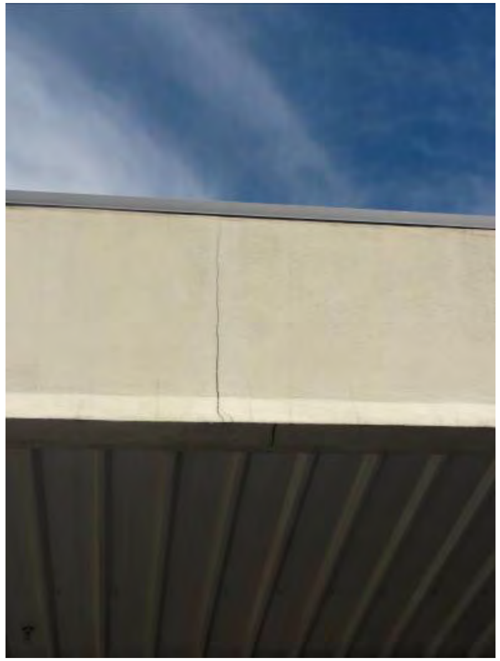
Damaged Item	Photo Ref: Location	Recommended Repair	Example
2.8. Cracking to unreinforced brick partition walls		Damage to the brick partition walls has been noted. This includes cracking at the top of the wall and separation of the top of the wall from the supporting ceiling framing. It is recommended that the walls be demolished and replaced with light weight timber framed walls. See Section 4.6 for additional discussion. *Note: In every location where the finishes were removed cracking was observed in the brick partition walls beneath (Updated Naylor Love report, 11/09/12).*	 
2.9. 0.7mm vertical crack in wallboard of lightweight partition wall	009: GND Level, Store Room	Aesthetic repair to wallboard. *Repair specification by others.*	

Damaged Item	Photo Ref: Location	Recommended Repair	Example
2.10. Damage to miscellaneous finishes		Aesthetic repair. *Repair Specification by others.*	
3. Ground Level Floors			
3.1. Exposed reinforcing. And untreated penetrations through slab.	065: GND floor slab in subfloor void under laundry	Not earthquake related but should be patched with high strength, non-shrink grout to provide cover to exposed reinforcement in accordance with HCG specification.	

Damaged Item	Photo Ref: Location	Recommended Repair	Example
4. First Level Floors			
4.1. Horizontal crack to concrete slab across width of building at what appears to be an existing construction joint in the slab.	045: 1st Level slab.	At all visible cracks, remove finishes to expose crack to base concrete slab. Epoxy inject all cracks in concrete slabs >0.2mm as per the HCG Repair Specification [2]. For cracks greater than 0.5-0.6mm, HCG to confirm integrity of existing reinforcing steel. See Section 4.3 for additional discussion on repairs to concrete elements.	
5. Roof Framing and Ceilings			
5.1. Moisture affected tiles at South Wing	011: Activity Room 3 ceiling tiles	Review integrity of flashings and waterproof membranes adjacent to moisture affected tiles. In addition check that timber ledger and/or fixings have not been damaged. *To be completed and reviewed by others.* Investigations showed that the water tank above was not restrained for lateral movement. It is recommended that lateral restraints are provided.	

Damaged Item	Photo Ref: Location	Recommended Repair	Example
5.2. 0.5-2mm cracking to heavy plaster ceiling tiles through hallway	055: 1st Level ceiling tiles. (typical to 1st Level ceilings)	Replace damaged tiles as required. If heavy plaster replacement tiles are to be used, ensure they are hard fixed to the ceiling framing above on all four sides.	
5.3. Cracking of fibrous plaster ceiling linings particularly at interface of top of wall.		At the first floor level replace any damaged fibrous plaster ceiling boards with new gypsum plasterboard linings. Any ceiling lining to remain at the first floor level are to be re-fixed to the ceiling framing above. See Section 4.5 for additional information.	

Damaged Item	Photo Ref: Location	Recommended Repair	Example
5.4. Minor damage noted in roof waterproof membrane at South Wing		*Repair Specification by others. See report completed by Wayman Roofing Services LTD, dated 15th August 2013*	
5.5. Minor damage observed to clay roof tile assembly		*Repair Specification by others. See report completed by Wayman Roofing Services LTD, dated 15th August 2013*	

Damaged Item	Photo Ref: Location	Recommended Repair	Example
5.6. Borer damage observed to timber battens		*Repair specification by others.*	
6. Porte Cochère			
6.1. Vertical cracks in the finishes of the Fascia/Parapet of the steel framed roof.	092: Porte Cochère fascia/parapet	This crack is most likely due to creep of the timber fascia truss and not associated with earthquake damage. Repair is recommended to protect the integrity of structural elements below finishes	

Damaged Item	Photo Ref: Location	Recommended Repair	Example
6.2. 1.0mm horizontal crack to columns.	090: Porte Cochère concrete column	Further investigation is required to determine if the crack extends into the base concrete. *(Note: cracking does not extend into the base concrete beneath, Naylor Love report, 30/08/2012).*	
6.3. Pounding between fascia of Porte Cochère and the roof of the Champion Centre.	153: Porte Cochère	The current 20mm gap does not provide adequate space between the two buildings to prevent pounding under an SLS or ULS event. The degree of expected pounding is unlikely to cause significant structural damage, however it will cause damage to the finishes of varying degrees. Should this be deemed unacceptable, the non-structural corner of the fascia may be trimmed back to provide 100mm clearance between the Porte Cochère and the Champion Centre roof.	

4.2 DISCUSSION ON DIFFERENTIAL SETTLEMENT REMEDIATION

The level survey, completed by Fox & Associates, has indicated earthquake induced differential ground settlements of approximately 50mm and 53mm has occurred at the West and South Wings respectively. The worst case resulting slope in the ground floor framing of the West Wing is approximately 1:170, while the worst case slope noted at the South Wing is approximately 1:130. The survey completed also noted permanent slopes in the suspended first floor slab of the West Wing up to 1:75, with a total differential in the slab elevation of 83mm. The worst case slopes noted in the ground floor and first floor framing are outside the typical acceptable range and require re-levelling.

While the low points at the ground floor and first floor of the West Wing both occur along the eastern side of the building, there are several other locations which do not correlate as nicely which will make the re-levelling process more difficult.

At the West Wing the re-levelling process will need to first address the remediation of the slopes at the first floor level. This can achieved either through the use of mechanical jacking (beneath the continuous concrete footings/sub-floor walls) or through the use of underpinning grout techniques.

Once the first floor has been re-levelled adjustments can be made to the elevated timber framing at the ground floor level. At the West Wing the timber floor framing is supported by exterior and interior concrete walls and isolated interior concrete piers. Slopes in the ground floor can be remediated by detaching sections of the floor framing from the foundation elements below, adjusting them back to level and re-attaching them to the existing foundations.

At the South Wing it is likely the continuous footings under the perimeter concrete block walls of the 2001 addition will be required to be lifted back to level. This is to remediate sloped noted in the ground floor framing and help close the gaps that have formed at the interface of the block walls and the concrete walls of original 1955 construction. As with the West Wing this can likely be achieved through the use of either mechanical jacking or underpinning grout techniques.

There are risks and opportunities for each solution which extend beyond structural performance which will need to be considered by CDHB. These include continuity of operation, degree of re-levelling accuracy and the willingness of the re-levelling sub-contractor to provide a producer statement, amongst other items.

From a structural standpoint, either option is acceptable provided the use of underpinning grout does not create any “hard points” under the building. If “hard points” are created during the re-levelling process the potential for future differential settlements can be increased. If this were to occur it would reduce the capacity of the building going forward.

Based upon the information provided by Tonkin & Taylor the soil profile at the Burwood Hospital site (medium dense sand overlying dense sand) should lend itself to localized lifting through underpinning grout techniques and should not create any undesirable “hard points” as described above. The building also lends itself nicely to the use of mechanical jacking due an elevated ground floor slab and the relatively good shape of the perimeter concrete sub-floor walls and footings in this area. However, the suitability of re-levelling the building through the use of either mechanical jacking or underpinning grout will need to be verified by qualified sub-contractors in conjunction with the geotechnical consultant.

It should be noted that both options discussed above are not expected to increase the seismic performance of the building or reduce the potential for future differential settlements. Instead the options presented are designed to re-level the building without making the future performance of the building any worse than it was prior to the earthquakes. To improve the future performance of the building, and reduce the potential for future differential settlements, would likely require the entire footprint of the building to be either piled or the ground under all the sub-floor wall footings, service tunnels and the partial basement improved. Further geotechnical investigations would be required into the type and depth of ground improvement required.

Based up the geotechnical report provided by Tonkin & Taylor [5] the potential for future total and differential settlements at the building site varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

During the re-levelling process there is also the risk that addition damage could occur to the building linings, exterior block veneer, etc. Appropriate contingencies should be provided.

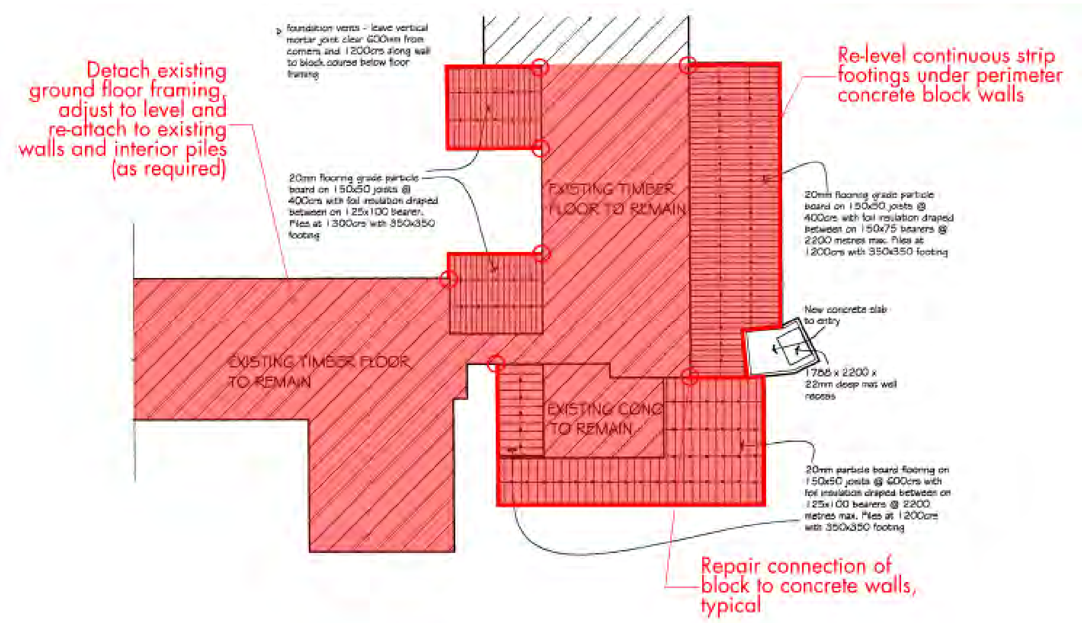


Figure 4-1: South Wing – Foundation / Ground Floor Plan – Repairs Required

4.3 REPAIR OF REINFORCED CONCRETE & BLOCK ELEMENTS

Cracking has been observed to sections of the reinforced concrete and concrete block walls as discussed in Section 3. This includes horizontal and vertical cracking along with diagonal cracking off the corners of window and doors openings. Cracking has also been observed in the first floor slab. The majority of the cracks observed to the concrete elements appear to be approximately 0.5mm in width or less, although some larger cracks have been noted in the concrete sub-floor walls (~1.0mm) and the concrete spandrels at the ground floor level (~0.7mm).

Based on the results of the testing of the reinforcing steel at Riverside Hospital and 235 Antigua Street, a vertical or diagonal crack exceeding 0.5-0.6mm in width would indicate that a significant level of strain hardening is likely to have occurred. The width of a horizontal crack is not an indication of the extent of strain hardening or debonding as the gravity loads close the cracks. The results of the testing completed to date in other buildings indicates that debonding or strain hardening is likely to have occurred where diagonal cracks extend to near the base of the wall or where there are horizontal cracks.

Little or no cracking has been observed at the base of the concrete walls of the West Wing which is the location which would be of primary concern for debonding. However, there has been isolated cracking noted in the concrete spandrels at the ground floor level which have been identified as having the potential to yield under the ULS Design Basis Earthquake. Cracking observed in these locations could indicate that the reinforcing steel has been strain hardened or the smooth round bars have debonded from the surrounding concrete.

In general though, based upon the size and extent of the cracking noted, we do not believe debonding or strain hardening of the reinforcing steel has occurred on a large scale. It appears as though the strength and stiffness of the majority of the cracked concrete elements can be restored to approximately pre-earthquake levels through epoxy injection of the cracks. Testing of the reinforcing steel at the worst case cracks in the concrete spandrels is recommended as outline in Section 3.7. If debonding or strain hardening at any concrete elements has occurred additional repair of these elements will be required.

4.4 REPAIR OF CRACKS AT INTERFACE OF 2001 ADDITION AND ORIGINAL 1955 CONSTRUCTION

Separation has occurred at the interface of the concrete block walls of the 2001 addition and the concrete walls of the original 1955 construction and requires repair. As noted in Section 4.2, remediation of the differential ground settlement at the South Wing is required to bring the 2001 addition back up to level and close the gap between the walls. Once this has been completed the connections between the two elements will need to be repaired to current code requirements. This will likely consist of new reinforcing steel drilled and epoxied across the joint to tie the two elements together.

4.5 BRICK PARTITION WALLS

Cracking has been observed to the internal brick partition walls in the toilet and washrooms of the West Wing at the ground floor and first floor levels. As outlined in Table 2-1, these walls had an assessed pre-earthquake capacity between 15% and 20% DBE, making them "Earthquake Prone." The recommended repair was to demolish and replace these walls with new light-weight partition walls. These brick walls have now been either removed or secured with timber framing such that the capacity is 85% DBE (IL3 - 67%).

4.6 REPAIR OF FIRST FLOOR CEILING DIAPHRAGM

At the first floor level of the West Wing, the fibrous board ceiling linings act as a diaphragm in the longitudinal direction to help distribute lateral loads the concrete shear walls below. Based upon the movement observed, including cracking at the interface of the ceiling linings with the top of the concrete walls, it is believe the ceiling linings and associated fixings have been damaged throughout and require repair. In order to reinstate the pre-earthquake strength and stiffness to the ceiling diaphragms, the repair recommendation is to remove all cracked or damaged sections of the ceiling linings and replace them with new gypsum board sheathing. The new gypsum board sheathing is to be fixed in accordance with GIB ceiling diaphragm specifications (or equivalent). All existing ceiling linings to remain are to be re-fixed to the existing ceiling framing in a similar manner. A new finish is then to be applied to all interior walls.

Note: The fixings of the ceilings to the perimeter and interior concrete walls below will need to be checked for damage and the ability to transfer the bracing demands. Refer to Section 3.7.2.

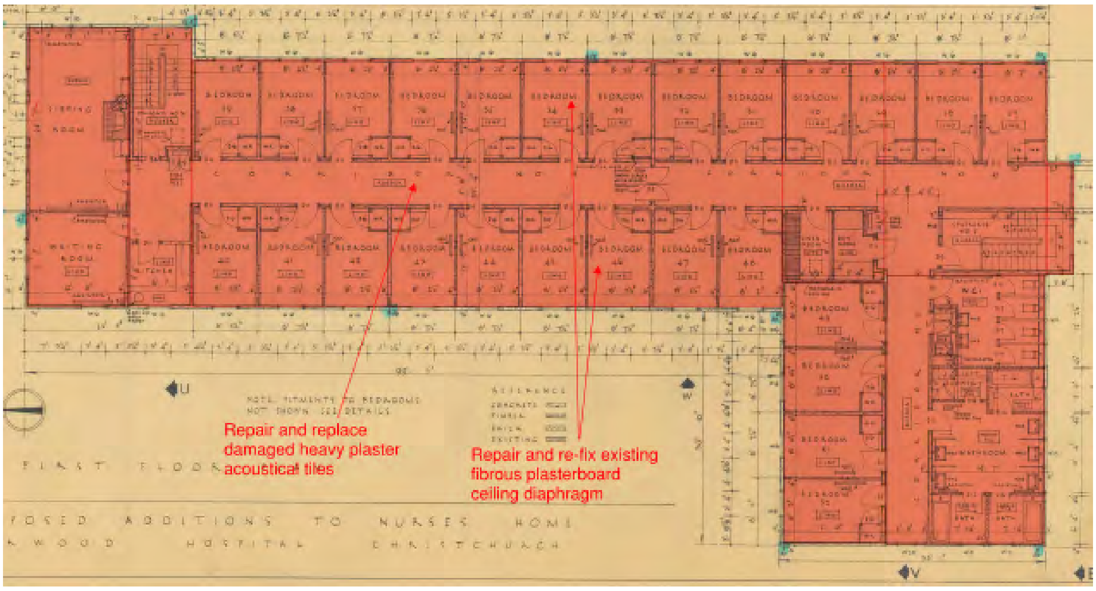


Figure 4-2: West Wing - First Floor Plan
– Repairs Required

4.7 REPAIR / INVESTIGATION OF CLAY ROOF TILES

Damage to the clay tile roofs was not noted during our investigations, but they were not easily visible from ground level. Our investigations in the roof space were also limited. Based upon our experience with other clay tile roofs on the campus, and the lack of bracing in the roof plan, we would recommend that a thorough investigation of the roof tiles be completed by a qualified roofing contractor. If the connection of the existing roof tiles to the timber roof framing has been compromised they have the potential of being dislodged and "shed" during a significant seismic event.

If the investigation calls for replacement of the existing roof assembly, we would recommend the roof be replaced with a light-weight alternative, such as a standing seem metal roof over a layer of plywood sheathing, in lieu of in kind material. Not only would this approach decrease the seismic mass of the building but it could also be installed to a sufficient strength level to act as a diaphragm in both the longitudinal and transverse direction, and support the top of the concrete walls under face loading. If this option was implemented, the repair noted in Section 4.6 would become aesthetic in nature only.

An assessment of the clay roof tile assembly was completed by Wayman Roofing Services LTD. The report, dated 15th August 2012, indicates some movement in the tiles but to a much lesser extent than the Nurses Hostel East building. The damage noted to the tiles appears to be concentrated at the ridges and valleys (particularly at the north gable end). This report suggests that this damage will be difficult to repair due to the fragility of the tiles and the unavailability of the hips and ridge tiles used. It is recommended therefore, that the roof tiles are replaced with lightweight steel roofing and plywood diaphragm.

5. STRENGTHENING RECOMMENDED

The main lateral load resisting system of the West and South Wings of the Nurses Hostel West building is provided by reinforced concrete and concrete block walls, along with roof, ceiling and floor diaphragms of various materials. As noted in Section 2, Pre-Earthquake Building Condition, and Section 3, Post-Earthquake Building Condition, the lateral load resisting capacity of each section of the building has been assessed as a percentage of the loads imposed by the Design Basis Earthquake (DBE). Based upon the analysis completed the lateral load resisting capacity of the West Wing, South Wing and Porte Cochere have been assessed at approximately 60% DBE (IL3 - 45%), 45% DBE (IL3 - 35%) and 60% DBE respectively. At the West Wing, while the typical diaphragm capacity has been assessed at approximately 60% DBE (IL3 - 45%) there are two sections of the ceiling diaphragm that have been assessed at approximately 45% DBE (IL3 - 35%) (north sitting room) and 50% DBE (IL3 - 40%) (south toilet and wash rooms). While not part of the primary lateral load resisting system the brick partition walls were assessed below 33% DBE and have since been removed or secured to 85% DBE (IL3 - 67%).

Provided the repairs specified in Section 4 are implemented, including the replacement of the heavy brick partition walls with a light weight alternative, the seismic capacity of the building will be restored to pre-earthquake levels and potentially even slightly increased due to a reduction in seismic mass.

Additional recommended strengthening to achieve a capacity of 67% DBE, and improve the overall seismic performance of the building, have been included in sub-sections below.

5.1 STRENGTHENING WORKS TO ACHIEVE 67% DBE

West Wing: Provide a new plywood diaphragm or bracing in the existing roof and/or ceiling plane. The bracing shall be designed to restrain the head of the concrete walls against out of plane inertial forces and distribute the loads to the in-plane walls. Available options include:

- Removing the existing clay roof tiles and providing new plywood sheathing over existing roof framing. Due to borer damage found throughout the roof battens, these should be replaced during this process.

- Provide new bracing at ceiling level. This would involve deconstruction the existing ceiling linings and replacing them with new plywood linings. Nominal diagonal bracing in the roof plane and diagonal struts down the internal concrete bracing walls should also be provided.

On installation of the new bracing, the connections between the existing timber framing and top of the interior and exterior concrete walls would require strengthened to accommodate the selected diaphragm/bracing system.

Additional timber collector elements will also be required in the ceiling plane at the north sitting room and the south toilet / washrooms in order to drag load back into the building.

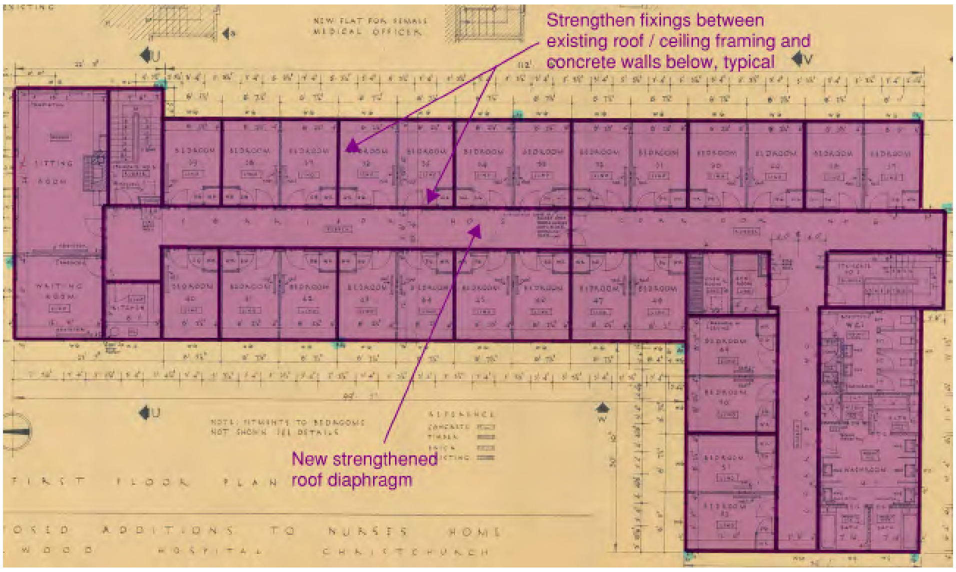


Figure 5-1: West Wing – First Floor Plan - Strengthening Recommended

South Wing: Strengthen the connections between the existing timber roof framing and top of the interior and exterior concrete and concrete block walls below. The existing diaphragm fixings between the 1955 tongue and groove roof sheathing, and the 2001 plywood sheathing, to the roof framing below will need to be checked during the strengthening to confirm there adequacy.

At the 2001 South Wing additions, additional sub-floor bracing of the ground floor framing is recommended at a maximum of 5m spacings. This can be achieved by strengthening the existing floor framing connection to the original 1955 concrete sub-floor walls.

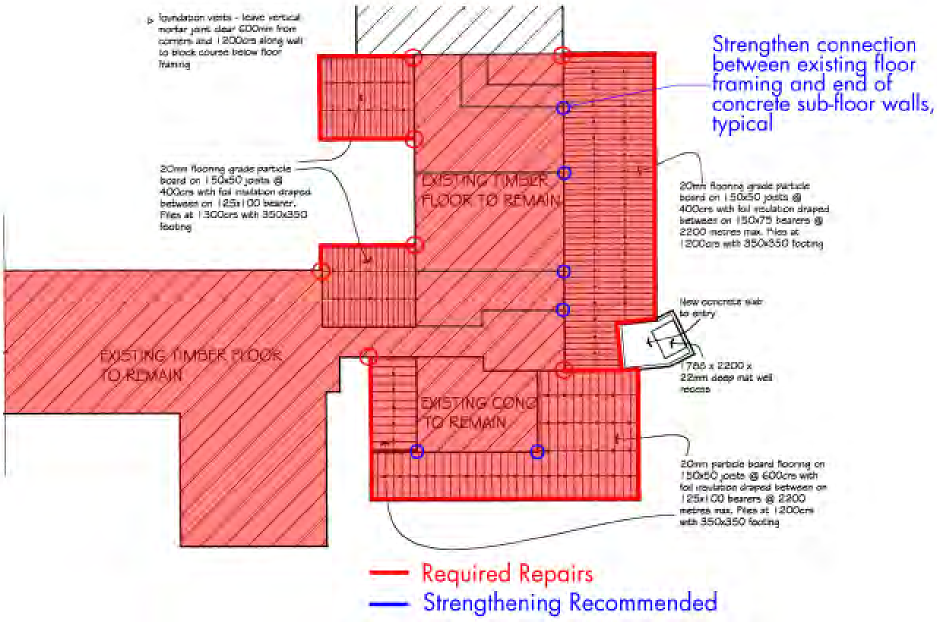


Figure 5-2: South Wing – Foundation / Ground Floor Plan - Strengthening Recommended

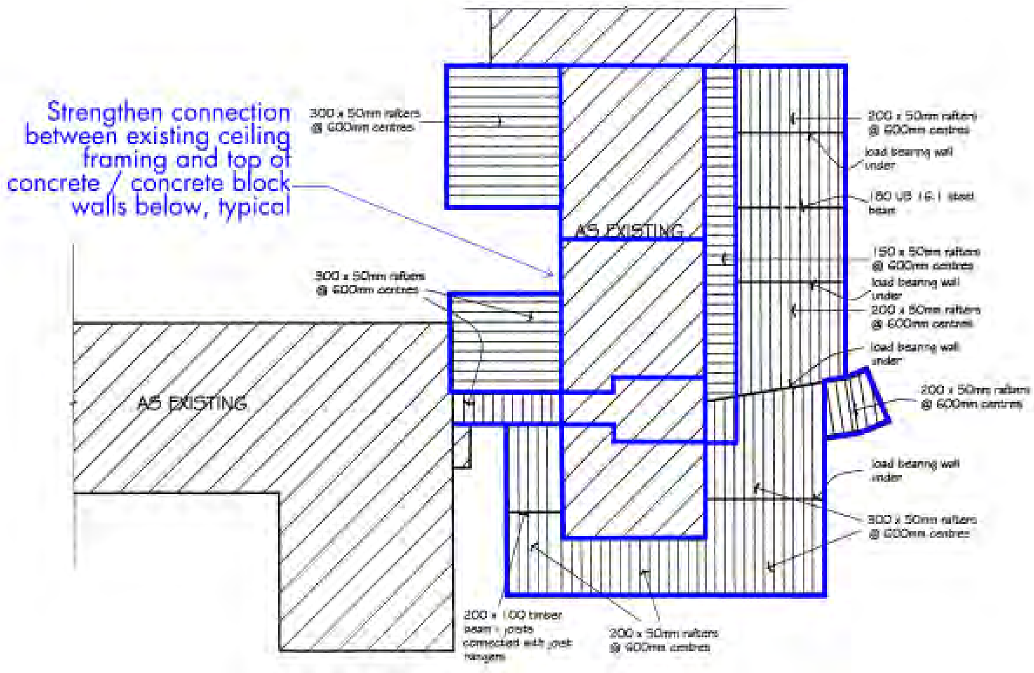


Figure 5-3: South Wing – Roof Plan - Strengthening Recommended

Porte Cochère: Strengthening of the Porte Cochere structure will be required to achieve a target of 67% DBE. Additional moment capacity can be provided at the base of the columns by enlarged the existing pad footings under the columns. Additional clearance is also recommended at the interface between the Porte Cochere and the facia of the 2001 South Wing addition. A 100mm seismic gap is the minimum recommended and could be achieved by trimming back the corner of the Porte Cochere. In its current state pounding damage is expected in Serviceability Limit State (SLS1) seismic events.

6. REFERENCES

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2 *CHDB – Burwood Campus – Detailed Seismic Assessment Report – Repair Specification*, Holmes Consulting Group, November 2011

3 *Proposed Additions to Nurses Home, Burwood Hospital Christchurch.* Original architectural drawings, Manson, Seward and Stanton Architects and Civil Engineers, 1954

4 *Champion Centre Relocation – Extensions and Alterations.* Original Architectural Drawings, Tony Adams Architectural Design, 2000

4 *Burwood Hospital Christchurch – Survey of Existing Buildings – Nurses Hostel West,* Cutter Pickmere Douglas - Architects, 1976

4 *Burwood Hospital- Master Drawing Survey- Nurses Hostel West,* Maintenance and Engineering Department, Christchurch Hospital, 2009

5 *Burwood Hospital Post Earthquake Geotechnical Assessment*, Tonkin and Taylor Ltd. June 2011

6 *CDHB – Burwood Field Survey*, Fox & Associates, July 2011 (revised November 2011)

7 *NZS4229:1999 – New Zealand Standard Concrete Masonry Buildings Not Requiring Specific Engineering Design.* Standards New Zealand, 1999

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

9 Department of Building and Housing, *Compliance Document for New Zealand Building Code - Clause B1 – Structure, Amendment 10 (Canterbury)*, Department of Building and Housing, Wellington, 19 May 2011

10 *NZS 95 – 1939, New Zealand Standard Code of Building By-Laws*

11 *Code of Practice for General Structural Design and Design Loadings for Buildings*, NZS4203:1992, Standards New Zealand, 1992

11 *New Zealand Standard Model Building Bylaw – Part IV Basic Design Loads To Be Used In Design And Methods Of Application*, N.Z.S.S 95, New Zealand Standards Institute, 1955

11 *New Zealand Standard Model Building Bylaw – Part V Reinforced and Plain Concrete Construction,* N.Z.S.S. 95, New Zealand Standards Institute, 1955

11 *State Schools Property Management Handbook*. New Zealand Ministry of Education. Updated December 2011

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

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

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13 *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury – Part 2 Evaluation Procedure*, Engineering Advisory Group, July 2011

13 *Revised Guidance on Repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence*, Department of Building and Housing, November 2011

14 *Burwood Hospital Campus – Seismic Risk Assessment Report*. Holmes Consulting Group, 2002

15 *Burwood Hospital Campus – 2007 Seismic Risk Assessment Update,* Holmes Consulting Group, June 2007

16 *CDHB – Burwood Hospital Campus – Rapid Visual Inspection: 106186.03SR1,* Holmes Consulting Group, 24 February 2011

17 *CDHB – Burwood Hospital – Rapid Seismic Assessment Post June 13th Earthquakes: 106186.03SR4* Holmes Consulting Group, 14 June 2011

APPENDIX A

Record of Observations

APPENDIX A – RECORD OF OBSERVATIONS & REPAIRS

Inspection date: 27 September - 2 December 2011

KEY	
N	No repair required
Y	Repair required
F	Further investigation required
C	Repair complete

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
SUB	59	Timber Floor Framing	No damage. Indicative configuration and condition of timber floor framing	**N**	-	125-131
SUB	Laundry (G51) Subfloor	GND Floor Concrete Slab	Spalling of concrete and exposed reinforcing. Untreated penetrations through slab. Appear to be a result of previous services or intentional drilling	**Y**	Not earthquake related but should be patched with epoxy grout to provide cover and durability to exposed reinforcement	065
SUB	Service Tunnel	Tunnel Wall	5mm vertical crack to RC retaining wall. Pile of sand at base of crack. Crack appears to correspond with Cracks noted at GND Floor level	**F**	Cracks > 1mm require further investigation to confirm the integrity of the steel reinforcement. Refer to HCG specification	062
SUB	Service Tunnel	Tunnel Wall	3-5mm vertical crack in RC retaining wall. Appears to correspond with cracks in the GND level walls	**F**	Cracks > 1mm require further investigation to confirm the integrity of the steel reinforcement. Refer to HCG specification	064

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
SUB	Service Tunnel	Tunnel Wall	'Wheel' of 5 cracks 0.5-1mm propagating from single point in RC wall. No obvious horizontal displacement was noted. Photo 074 shows a perpendicular wall that abuts the service tunnel wall at the approximate location of the cracks. The level of the footing of the perpendicular wall was not investigated.	**Y**	For all cracks greater than 0.2mm and less than 1mm, epoxy inject cracks in accordance with HCG specification.	072-074
SUB	Service Tunnel	Tunnel Wall	0.5-1mm tapered diagonal cracks typical to service openings in the RC wall underneath the West Wing	**Y**	For all cracks greater than 0.2mm and less than 1mm, epoxy inject cracks in accordance with HCG specification.	080-081
SUB	Service Tunnel	Tunnel Wall	Diagonal cracks <1.5mm propagating from service openings in service tunnel RC walls (typical to service tunnel)	**Y**	For all cracks greater than 0.2mm and less than 1mm, epoxy inject cracks in accordance with HCG specification. Cracks > 1mm require further investigation to confirm the integrity of the steel reinforcement. Refer to HCG specification	070, 071
SUB	Service Tunnel	Tunnel Wall	~1mm vertical and diagonal cracks in RC wall propagating from vent openings in the exterior wall of the 1955 building. Appear as typical throughout the service tunnel.	**F**	For all cracks greater than 0.2mm and less than 1mm, epoxy inject cracks in accordance with HCG specification. Cracks > 1mm require further investigation to confirm the integrity of the steel reinforcement. Refer to HCG specification	069
SUB	Service Tunnel	Concrete Slab	2-5mm crack in concrete floor slab of service tunnel. Existing architectural documentation reflects that the slab is a 'U' shaped link between the retaining walls.	**F**	Cracks > 1mm require further investigation to confirm the integrity of the steel reinforcement. Refer to HCG specification	0063

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
SUB	Service Tunnel	Concrete Slab	1-5mm crack running down the approximate centre of the service tunnel slab. Appears as typical throughout the service tunnel. Dirt and fines in the crack in many locations indicates that the crack may have existed in some form before the earthquakes but is likely exaggerated since the quakes.	**F**	For all cracks greater than 0.2mm and less than 1mm, epoxy inject cracks in accordance with HCG specification. Cracks > 1mm require further investigation to confirm the integrity of the steel reinforcement. Refer to HCG specification	066-068
GND	55	Heavy Tile Ceiling	Cracking between Ceiling and cornice	**Y**	Aesthetic repair to ceiling specified by others. Ceiling may require replacement as per Section 4.	017-018
GND	56	Lightweight Tile Ceiling	Moisture affected tiles. Note: some moisture noted before quakes, however considerably worse since earthquakes	**F**	Specialist contractor to investigate waterproofing. Ceiling may require replacement as per Section 4.	019
GND	59	Lightweight Tile Ceiling	Moisture affected tiles. Note: some moisture noted before quakes, however considerably worse since earthquakes	**F**	Specialist contractor to investigate waterproofing.	011-012
GND	13	Lightweight Tile Ceiling	Moisture affected ceiling tile	**F**	Specialist contractor to investigate waterproofing.	038
GND	6	Heavy Tile Ceiling	1mm crack to ceiling. Cracking between cornice and walls typical throughout hallway	**Y**	Aesthetic repair to ceiling specified by others. Ceiling may require replacement as per Section 4.	010
GND	Porte Cochere	Concrete Columns	1.0mm horizontal crack to columns. Unable to distinguish if this crack is part of the structural concrete or is only in the render	**F**	Further investigation required to determine if the crack extends into the base concrete material	090, 093

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	Play Area	Ext. Wall and Glazed Roof	1-2mm crack at interface between external block walls and the glazed roof	**F**	As the glazed roof is attached to both the 1955 insitu concrete walls and the 2001 concrete block additions, repair methodology to be determined pending investigations outlined in main report	021-022
GND	Porte Cochere	Fascia	Damage to fascia of Porte Cochere and Champion Centre appears indicative of pounding damage caused by excessive drifts of Porte Cochere. 20mm gap between fascias was measured on site.	**Y**	Gap between Porte Cochere and Champion Centre can be extended by trimming the Porte Cochere framing at this location. 100mm gap should be sufficient to prevent future pounding in a design event.	152-154
GND	Porte Cochere	Fascia and Drainage	Fascia of Porte Cochere appears to have deflected significantly, causing ponding of water which should readily drain. This is likely due to creep of the timber fascia truss and not due to earthquake damage	**N**	Not Earthquake related	147-151
GND	58	Joinery	5mm crack between cupboard and wall	**Y**	Aesthetic repair only. Repair specification by others.	007
GND	Porte Cochere	Parapet/Fascia	Vertical cracks in the Fascia/Parapet of the timber framed roof. Likely as a result of creep of timber fascia truss	**N**	Not Earthquake related	088, 089, 091, 092, 097
GND	31	Concrete Slab	Horizontal crack to slab across length of room	**Y**	For all cracks greater than 0.2mm and less than 1mm, epoxy inject cracks in accordance with HCG specification.	045
GND	35	Concrete Slab	0.2mm crack to concrete 1st floor slab soffit	**Y**	For all cracks greater than 0.2mm and less than 1mm, epoxy inject cracks in accordance with HCG specification.	057-058

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	39	Concrete Slab	0.2mm crack to concrete 1st floor slab soffit	**Y**	For all cracks greater than 0.2mm and less than 1mm, epoxy inject cracks in accordance with HCG specification.	060-61
GND	45	Concrete Slab	Horizontal crack to slab across length of room	**Y**	For all cracks greater than 0.2mm and less than 1mm, epoxy inject cracks in accordance with HCG specification.	047
GND	46	Concrete Slab	Horizontal crack to slab across length of room	**Y**	For all cracks greater than 0.2mm and less than 1mm, epoxy inject cracks in accordance with HCG specification.	046
GND	3	Concrete Wall	Window sill has cracked. Cornice above window has cracked from wall	**Y**	Remove finishes to expose crack to base concrete wall. For cracks greater than 0.2mm and less than 1.0mm epoxy inject in accordance with HCG specification. Aesthetic repair to wallboard, specification by others. For cracks identified as greater than 1.0mm in base concrete wall, advise engineer for inspection to confirm the integrity of steel reinforcement.	040-041
GND	3	Concrete Wall	5mm vertical crack at door head	**Y**	Remove finishes to expose crack to base concrete wall. For cracks greater than 0.2mm and less than 1.0mm epoxy inject in accordance with HCG specification. Aesthetic repair to wallboard, specification by others. For cracks identified as greater than 1.0mm in base concrete wall, advise engineer for inspection to confirm the integrity of steel reinforcement.	025

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	5	Concrete Wall	0.1mm diagonal crack propagating from door head	**Y**	Remove finishes to expose crack to base concrete wall. For cracks greater than 0.2mm and less than 1.0mm epoxy inject in accordance with HCG specification. Aesthetic repair to wallboard, specification by others. For cracks identified as greater than 1.0mm in base concrete wall, advise engineer for inspection to confirm the integrity of steel reinforcement.	059
GND	10	Concrete Wall	Cracking in door head behind plastic laminate wall dressing	**Y**	Remove finishes to expose crack to base concrete wall. For cracks greater than 0.2mm and less than 1.0mm epoxy inject in accordance with HCG specification. Aesthetic repair to wallboard, specification by others. For cracks identified as greater than 1.0mm in base concrete wall, advise engineer for inspection to confirm the integrity of steel reinforcement.	028-029
GND	10	Concrete Wall	2-3mm movement of wall cladding	**Y**	Remove finishes to expose crack to base concrete wall. For cracks greater than 0.2mm and less than 1.0mm epoxy inject in accordance with HCG specification. Aesthetic repair to wallboard, specification by others. For cracks identified as greater than 1.0mm in base concrete wall, advise engineer for inspection to confirm the integrity of steel reinforcement.	030

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	11	Concrete Block Wall	1mm crack above window head	**Y**	Remove finishes to expose crack to base block wall. For cracks greater than 0.2mm and less than 1.0mm epoxy inject in accordance with HCG specification. Aesthetic repair to wallboard, specification by others. For cracks identified as greater than 1.0mm in base block wall, advise engineer for inspection to confirm the integrity of steel reinforcement.	031
GND	11	Concrete Wall	Cracking to door head, frame and vertical crack to wall propagating from door head.	**Y**	Remove finishes to expose crack to base concrete wall. For cracks greater than 0.2mm and less than 1.0mm epoxy inject in accordance with HCG specification. Aesthetic repair to wallboard, specification by others. For cracks identified as greater than 1.0mm in base concrete wall, advise engineer for inspection to confirm the integrity of steel reinforcement.	032-033
GND	12	Concrete wall	Door jamb has moved approx. 5mm, Corner of wall cladding has 'popped'	**Y**	Remove joinery and finishes to expose crack to base concrete wall. For cracks greater than 0.2mm and less than 1.0mm epoxy inject in accordance with HCG specification. Aesthetic repair to joinery and wallboard, specification by others. For cracks identified as greater than 1.0mm in base concrete wall, advise engineer for inspection to confirm the integrity of steel reinforcement.	035-037

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	13	Concrete Wall	Wall end has moved/cracked adjacent to ceiling	**Y**	Remove joinery and finishes to expose crack to base concrete wall. For cracks greater than 0.2mm and less than 1.0mm epoxy inject in accordance with HCG specification. Aesthetic repair to joinery and wallboard, specification by others. For cracks identified as greater than 1.0mm in base concrete wall, advise engineer for inspection to confirm the integrity of steel reinforcement.	039
GND	23	Concrete Wall	0.5 mm diagonal crack to wall cladding, bubbled contact on corresponding side of wall. Crack propagates from door head	**Y**	Remove joinery and finishes to expose crack to base concrete wall. For cracks greater than 0.2mm and less than 1.0mm epoxy inject in accordance with HCG specification. Aesthetic repair to joinery and wallboard, specification by others. For cracks identified as greater than 1.0mm in base concrete wall, advise engineer for inspection to confirm the integrity of steel reinforcement.	043-044
GND	49	Concrete Wall	Crack between window reveal, exterior wall and internal cladding	**Y**	Remove joinery and finishes to expose crack to base concrete wall. For cracks greater than 0.2mm and less than 1.0mm epoxy inject in accordance with HCG specification. Aesthetic repair to joinery and wallboard, specification by others. For cracks identified as greater than 1.0mm in base concrete wall, advise engineer for inspection to confirm the integrity of steel reinforcement.	042

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	50	Concrete Wall	0.7mm crack at door head in cladding. Could not determine if crack continued to structural wall	**Y**	Remove joinery and finishes to expose crack to base concrete wall. For cracks greater than 0.2mm and less than 1.0mm epoxy inject in accordance with HCG specification. Aesthetic repair to joinery and wallboard, specification by others. For cracks identified as greater than 1.0mm in base concrete wall, advise engineer for inspection to confirm the integrity of steel reinforcement.	027
GND	51	Concrete Wall	0.1mm vertical crack at door head	**Y**	Remove joinery and finishes to expose crack to base concrete wall. For cracks greater than 0.2mm and less than 1.0mm epoxy inject in accordance with HCG specification. Aesthetic repair to joinery and wallboard, specification by others. For cracks identified as greater than 1.0mm in base concrete wall, advise engineer for inspection to confirm the integrity of steel reinforcement.	026
GND	55	Partition Wall	.2mm vertical crack. Appears to occur at photo 009 location	**Y**	Aesthetic repair to wallboard finishes, repair specification by others.	016

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	57	Concrete Wall	2mm horizontal + vertical crack to wall cladding between reinforced concrete wall and timber furring.	**F**	Remove joinery and finishes to expose crack to base concrete wall. For cracks greater than 0.2mm and less than 1.0mm epoxy inject in accordance with HCG specification. Aesthetic repair to joinery and wallboard, specification by others. For cracks identified as greater than 1.0mm in base concrete wall, advise engineer for inspection to confirm the integrity of steel reinforcement.	008
GND	57	Partition Wall	0.7mm vertical crack to wall.	**Y**	Aesthetic repair to wallboard finishes, repair specification by others.	009
GND	58	Concrete Wall	0.2 – 1.5mm cracks to plaster cladding and ceiling. We could not confirm if the cracks extended through to the structure where cladding was present. Cracks typically propagating from corners of door	**F**	Repair methodology to be determined pending investigation	001-006
GND	58	Concrete Wall	Cracking of reinforced concrete wall. Worst cracking appears to be at or propagating from the interface of an infill panel constructed from reinforced concrete and fixed between the existing reinforced concrete walls. We could not determine the fixing or reinforcing configuration of the infill panel from visual observation.	**F**	Repair methodology to be determined pending investigation	116-124 & 132-139
GND	62	Partition Wall	0.2mm vertical crack	**Y**	Aesthetic repair to wallboard finishes, repair specification by others.	013-014

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	65	Partition Wall	Joinery separated from wall approx 15mm	Y	Aesthetic repair to wallboard finishes, repair specification by others.	015
GND	External	Block Wall	0.2mm crack, 200mm from corner edge of 190mm reinforced block wall	Y	Epoxy inject crack in accordance with HCG specification.	094
GND	External	Block Wall	0.2mm vertical crack to window opening of 2001 addition. Typical to external windows on building perimeter	Y	Epoxy inject crack in accordance with HCG specification.	083, 084, 085
GND	External	Concrete Wall	1mm tapered diagonal crack propagating from window sill	Y	Epoxy inject crack in accordance with HCG specification.	098
GND	External	Concrete Wall	0.5mm horizontal crack propagating from door jamb	Y	Epoxy inject crack in accordance with HCG specification.	099
GND	External	Concrete Wall	0.5mm horizontal crack propagating from window head	Y	Epoxy inject crack in accordance with HCG specification.	102
GND	External	Concrete Wall	0.5mm tapered diagonal crack propagating from Bottom right and top left corners	Y	Epoxy inject crack in accordance with HCG specification.	103
GND	External	Concrete Wall	0.5mm tapered diagonal crack propagating from Bottom right and top left corners	Y	Epoxy inject crack in accordance with HCG specification.	104
GND	External	Concrete Wall	0.2mm horizontal and 0.5mm vertical crack propagating from window corner	Y	Epoxy inject crack in accordance with HCG specification.	105
GND	External	Block Wall	0.5 mm vertical crack to window sill	Y	Epoxy inject crack in accordance with HCG specification.	106
GND	External	Concrete Wall	Cracking to sill and jamb reveals, 0.5mm diagonal tapered crack propagating from window head	Y	Epoxy inject crack in accordance with HCG specification.	100, 101

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	External abutment of 2001 and 1955 constructions	Intersection of Concrete and Block Walls	10mm full height vertical crack between 1955 building and 2001 addition.	**F**	Repair methodology to be determined pending investigation	082
GND	External abutment of 2001 and 1955 constructions	Upper Level Cladding	Patched cracks in the upper section of wall between the Champion Centre and the adjacent Public Health Nurses Offices (different construction periods). These cracks are in the proximity of moisture affected tiles in the Champion Centre.	**F**	Repair methodology to be determined pending investigation	095, 096
GND	Play Area	Intersection of Concrete and Block Walls	1mm crack between 1955 and 2001 structures	**F**	Repair methodology to be determined pending investigation	020
GND	Play Area	Upper Level Cladding	Cracking to cladding directly below eaves. Cracks appear at centres consistent with cladding ends. Did not access roof to inspect closely.	**N**	Not earthquake related	023-024
GND	Entrance	Block Walls and Fascia	Crack at perpendicular joint between main building and the entrance. Cracking of fascia cladding at parapet.	**Y**	Epoxy inject crack in accordance with HCG specification. Aesthetic repair to finishes, refer to specification by others. For cracks that extend through finishes, refer to specification.	086, 087
1st	45	Heavy Tile Ceiling	0.5-2mm cracking to ceiling tiles through hallway	**Y**	Aesthetic repair to ceiling, repair specification by others. Ceiling may require replacement as per Section 4.	055

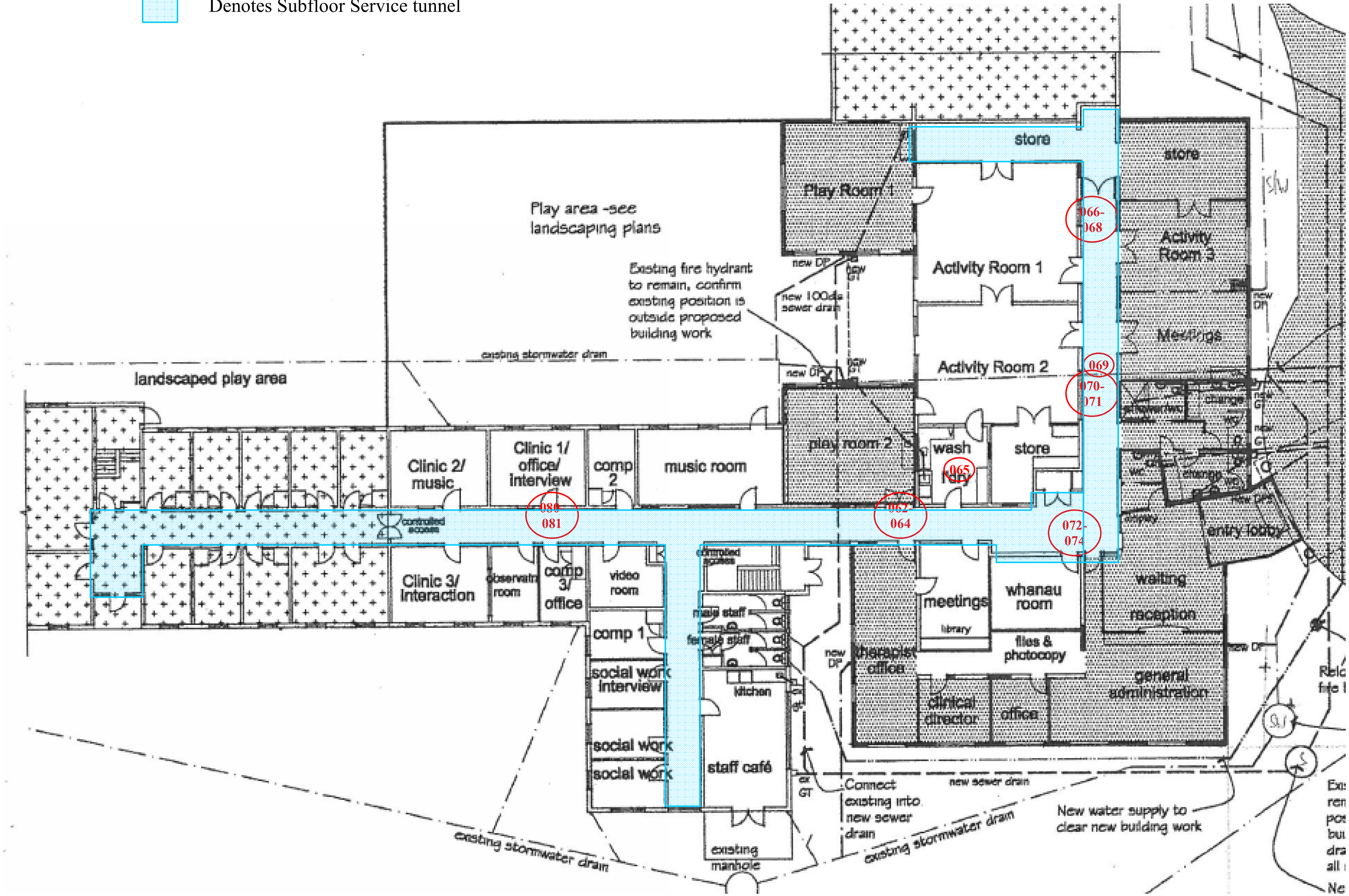
Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
1st	47	Cornices	1-2mm crack between cornice and wall (typical to 1st floor)	**Y**	Aesthetic repair to cornice, repair specification by others. Ceiling may require replacement as per Section 4.	051
1st	19	Heavy Tile Ceiling	0.2mm crack to ceiling	**Y**	Aesthetic repair to ceiling, repair specification by others. Ceiling may require replacement as per Section 4.	056
1st	17	Cornice	1mm crack between cornice and wall	**Y**	Aesthetic repair to cornice, repair specification by others. Ceiling may require replacement as per Section 4.	049
1st	19	Cornice	1mm crack between cornice and wall	**Y**	Aesthetic repair to cornice, repair specification by others. Ceiling may require replacement as per Section 4.	048
1st	24	Suspended Concrete Slab	1.2mm crack in floor across length of room. Crack is reflected in soffit of slab when viewed from underneath. Crack appears to have existed prior to earthquakes as evidenced by glue within crack, however appears to have widened further as a result of the earthquakes	**Y**	Remove finishes to expose full length of crack in concrete slab. For cracks greater than 0.2mm and less than 1.0mm epoxy inject in accordance with HCG specification. Aesthetic repair to any damaged finishes, specification by others. For cracks identified as greater than 1.0mm in the concrete slab, advise engineer for inspection to confirm the integrity of steel reinforcement.	110-115

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
1st	19	Vents in Concrete Wall	0.2mm diagonal crack propagating from vents (typical to 1st floor hallway)	**Y**	Remove finishes to expose crack to base concrete wall. For cracks greater than 0.2mm and less than 1.0mm epoxy inject in accordance with HCG specification. Aesthetic repair to wallboard, specification by others. For cracks identified as greater than 1.0mm in base concrete wall, advise engineer for inspection to confirm the integrity of steel reinforcement.	053
1st	2	Wall	Hairline diagonal crack to door head	**Y**	Remove finishes to expose crack to base concrete wall. For cracks greater than 0.2mm and less than 1.0mm epoxy inject in accordance with HCG specification. Aesthetic repair to wallboard, specification by others. For cracks identified as greater than 1.0mm in base concrete wall, advise engineer for inspection to confirm the integrity of steel reinforcement.	050
1st	24	Wall to Roof Collectors	No Damage. Photo show configuration and condition of roof framing to reinforced concrete walls at this location.	**N**	-	107-109
1st	22	Windows	0.5mm crack to reveals/sills (typical to 1st floor windows)	**Y**	Aesthetic repair to window joinery. Repair specification by others	054
1st	45	Windows	0.5mm cracks to window sills/jambs (typical to 1st floor)	**Y**	Aesthetic repair to window joinery. Repair specification by others	052

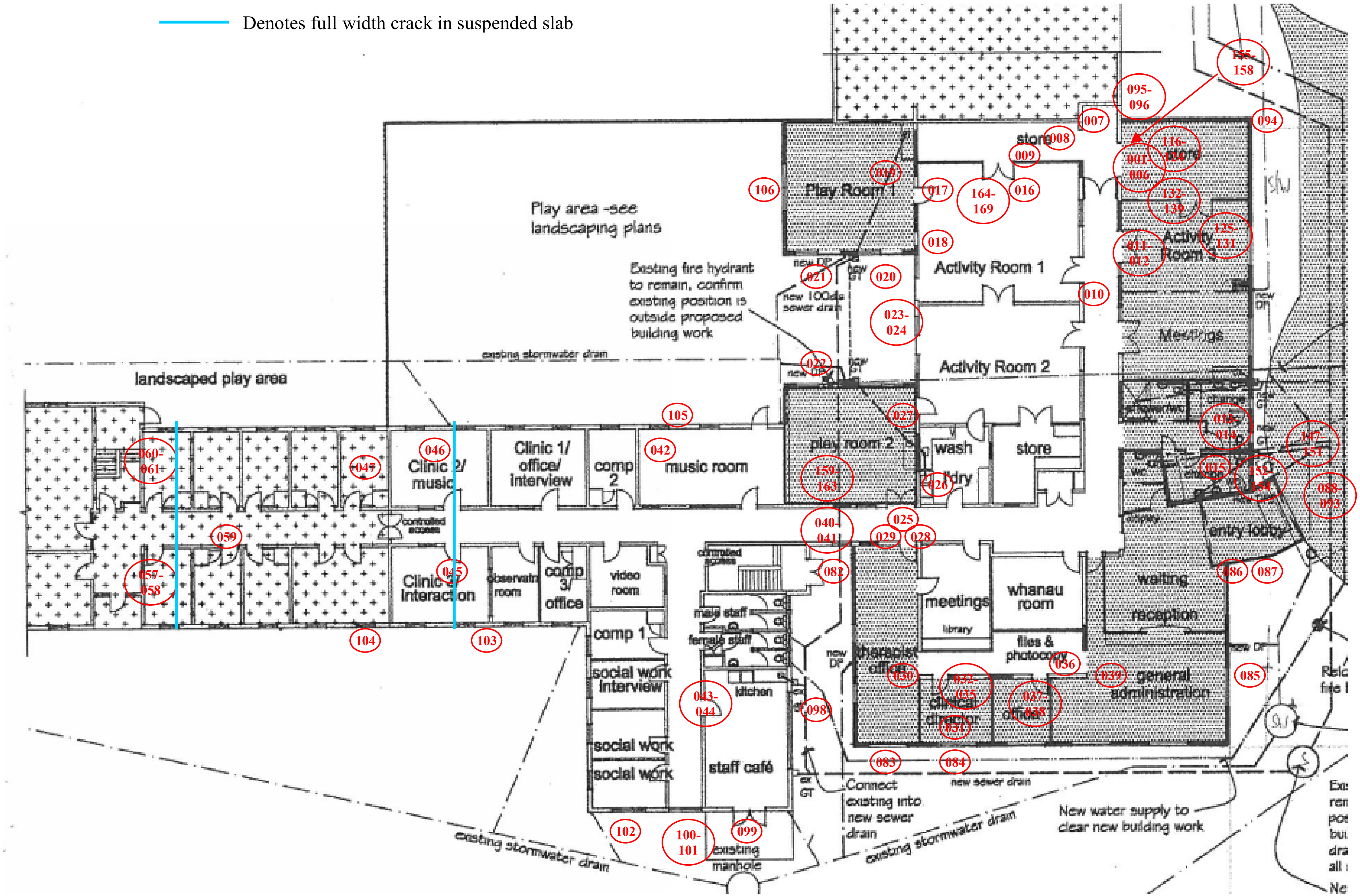
APPENDIX B

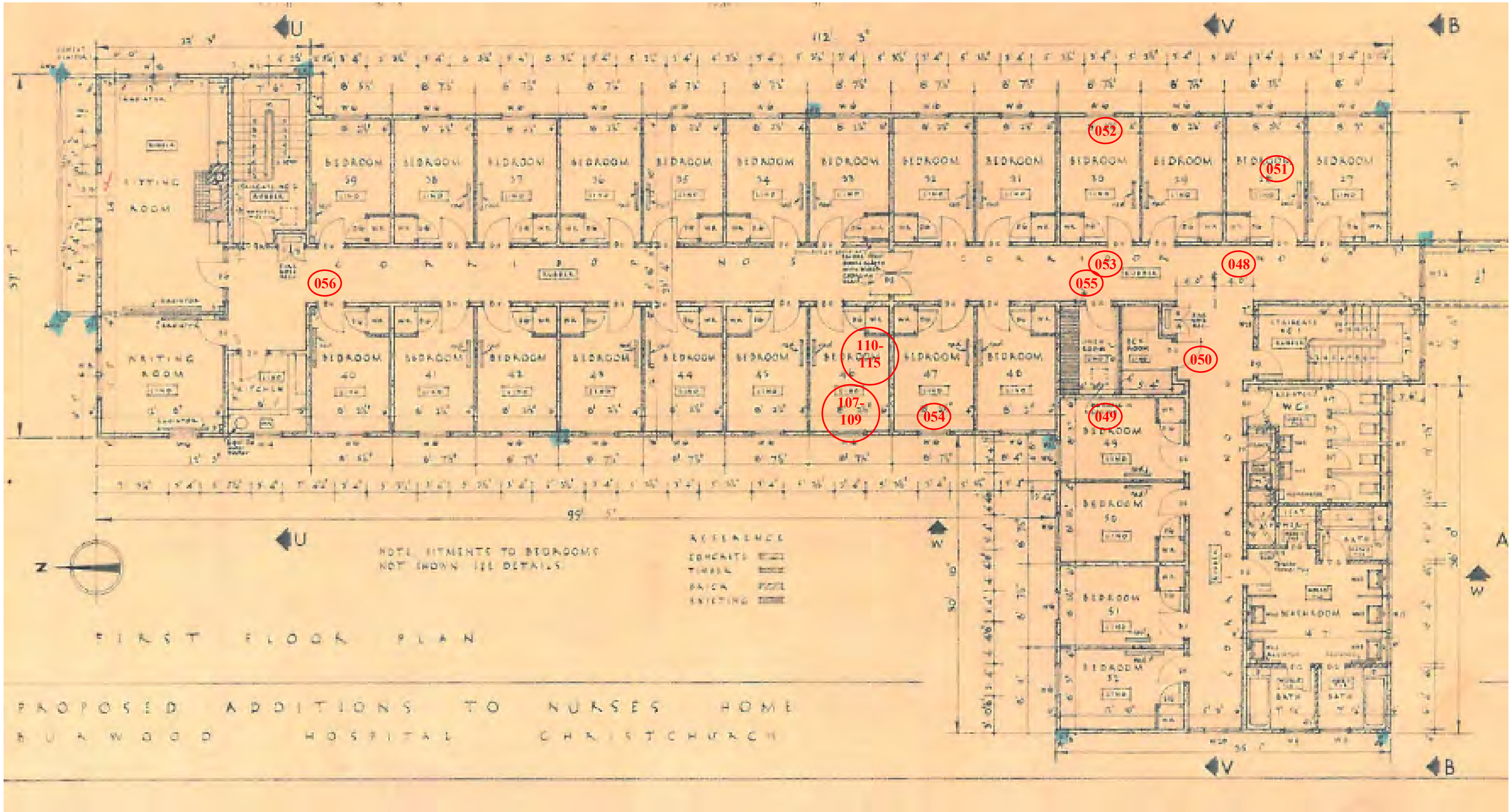
Reference / Key Plans

Denotes Subfloor Service tunnel



Denotes full width crack in suspended slab




052
051
053
048
055
056
050
110-
115
107-
109
054
049
BEDROOM
SITTING ROOM
WRITING ROOM
KITCHEN
WASHROOM
W.C.
NOTE. FITMENTS TO BEDROOMS NOT SHOWN SEE DETAILS
REFERENCE
CONCRETE
TIMBER
BRICK
EXISTING
FIRST FLOOR PLAN
PROPOSED ADDITIONS TO NURSES HOME
BURWOOD HOSPITAL CHRISTCHURCH

APPENDIX C

Survey of Levels

Levels reduced to concrete or hardstand floor.
Benchmarks = Nails 6, 26, 25 and 24

Colour ramping applies only to specific area as labelled.
Champion 1 and 2 surfaces displayed with 0.010m banding.
Social Work building displayed with 0.020m banding.

Elevations Table - Champion 1

Number	Minimum Elevation	Maximum Elevation	Color
1	14.760	14.770	
2	14.770	14.780	
3	14.780	14.790	
4	14.790	14.800	
5	14.800	14.810	
6	14.810	14.820	

Elevations Table - Champion 2

Number	Minimum Elevation	Maximum Elevation	Color
1	15.070	15.080	
2	15.080	15.090	
3	15.090	15.100	
4	15.100	15.110	
5	15.110	15.120	
6	15.120	15.130	
7	15.130	15.140	

Elevations Table - Social Work

Number	Minimum Elevation	Maximum Elevation	Color
1	15.430	15.450	
2	15.450	15.470	
3	15.470	15.490	
4	15.490	15.510	
5	15.510	15.530	
6	15.530	15.550	

A1 Original Size

No.	Revision	Approved	Date
G			
F			
E			
D			
C	----	----	----
B	Nurses Hostel & Champion Centre, SOU elevations added	MJM	18/11/2011
A	Administration and Allan Bean Centre elevations added	MJM	31/10/2011

Scale		1:200
Reduced	A3	1:400
Designed		----
Drawn		DHP
Checked		MJM
Date		18/11/2011

032005

Burwood Elevations Survey

Nurses Hostel and Champion Centre

SURVEYORS
URBAN DESIGNERS
Fox & Associates
CONSULTANTS
LAND DEVELOPMENT
www.foxsurvey.co.nz
0800 FOX SURVEY
P.O.Box 13-390
CHRISTCHURCH

Job No. 2462C
Sheet No. 6 of 6
Rev. B

Appendix D

Roof Condition Report

MEMBER OF ROOFING ASSOCIATION N.Z.

WRS P.O. BOX 9354 CHRISTCHURCH
WAYMAN ROOFING SERVICES LTD.
TEL: 338 0877 FAX: 338 7489 MOBILE (027) 451 1170
Email sales@waymanroofing.co.nz

15 August 2012

Naylor Love Construction
P.O. Box 31006
Christchurch

Attention: Brendon Keenan

Re: Burwood Hospital – Zone 1 B - Nurses Hostel West Wing Roof Report
Note: Our inspection was limited by the access that was possible from an E.W.P.

Brendon,

Although the request was to report on the west wing, at the end of the report we have included a few comments and pictures of the east wing for your information.

West Wing

Firstly, visually from ground level, there seems to be less damage sustained to the west wing (which appears to be the newer building of the two wings) compared with damage to the east wing.

Every tile on any row that we inspected was tied (with wire). We concluded that the whole roof has been done this way which is very unusual (but not necessarily incorrect). It is probably why there are only a few of areas of excessive "shuffling" or "dancing" of the tiles.

Photo's 1,2&3 are on the west side of the roof and show typical movement of the tiles but this could not be classed as excessive.


photo 1


photo 2

photo 3

Photo's 4&5 are in the same area and show damage to the drainage channels, some of which are obviously more recent than others.

photo 4

photo 5

Photo's 6&7 are also in the same area and this is caused by frost damage.

photo 6

photo 7

These photos are typical of most of the roof and although a lot of the damage cannot be attributed to the earthquake, the mere fragile state of the roof will make it very difficult to access the roof to effect repairs.

Photo 8 is an example of the timber forming a 125mm bell cast at the bottom of the roof that would need to be removed it the roof was replaced with long run iron. What is not very clear in the photo is substantial amount of borer in the 50x25 tile batten and one would have to assume that this would have to be widespread. Photo 9 is wider shot of the same area.

photo 8

photo 9

The south gable end shows very little signs of any sort of damage.

Photo's 10&11 are of the north gable end and show where the ridge tiles and pointing have broken away.

photo 10

photo 11

The east side of the roof was harder to access but the condition of the roof is similar. Photo's 12&13 are up above the children's play area and can't be accessed easily but there signs of reasonable movement on this face.


photo 12


photo 13

East Wing

As mentioned previously, the east roof seems to have suffered more damage but this construction is different in that the tile battens are fixed over a tar paper underlay on top of a hit and miss 25mm sarking, (photo 14).


photo 14

This will help disguise any roof leaks that may be occurring, and because of the structure, it makes it very difficult to wire tie the tiles, only about every second tile on every row was tied, which is normal. Unfortunately, this allows more shuffling or dancing of the tiles to occur. This can be seen in photo's 15 & 16.


photo 15


photo 16

Our conclusion is that the tiles to the west wing have sustained minor earthquake damage which is going to very difficult to fix due to the nature of the fragility of the tiles, plus the fact that accessories such as hip and ridge tiles will be nearly impossible to salvage or purchase replacements. The east wing has sustained moderate damage which is also going to be very difficult to fix for the same reasons.

Regards,

John Wayman

DETAILED SEISMIC ASSESSMENT REPORT

STRUCTURAL AND CIVIL ENGINEERS

BURWOOD HOSPITAL CAMPUS

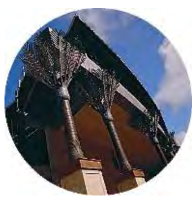
REPORT 9 – ORTHOPAEDIC REHABILITATION UNIT

PREPARED FOR

CANTERBURY DISTRICT HEALTH BOARD

106186.65

REVISION 3 – 02 APRIL 2015



BURWOOD HOSPITAL CAMPUS – INTERIM DETAILED SEISMIC ASSESSMENT REPORT

REPORT 9 – ORTHOPAEDIC REHABILITATION UNIT BUILDING

Prepared For:
CANTERBURY DISTRICT HEALTH BOARD

Date:	02 April 2015
Project No:	106186.65
Revision No:	3

Prepared By:

Joe Jones
PROJECT ENGINEER

Reviewed By:

Eric McDonnell
SENIOR PROJECT ENGINEER

Updated By:

Peter Grange
STRUCTURAL ENGINEER

Reviewed By:

Jenny Fisher
PROJECT DIRECTOR

Holmes Consulting Group LP
Christchurch Office

REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
06/12/11	1	Interim results of quantitative assessment (Phase 3) for discussion (some on site investigations still to be completed)
10/10/12	2	Revisions include general updates to format and terminology and repair works completed to date
31/03/15	3	Updated to include completion of the works to date, Project Director signature and minor updates to limiting capacities. Changes in this revision are indicated by a vertical line to the left hand side of text.

CONTENTS

APPENDICES

TABLES Page

FIGURES Page

Holmes Consulting Group has been engaged by Canterbury District Health Board (CDHB) to complete a full structural review of the Burwood Campus following the Lyttelton Earthquake. A series of reports have been compiled as part of this process. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The specific building reports, like this one for the Orthopaedic Rehabilitation Unit, should be read in conjunction with the base report, and refer to the repair specification.

This report identifies the structural damage sustained by the Orthopaedic Rehabilitation Unit as a result of the series of earthquakes, including the Darfield Earthquake that struck at 4:36am on the 4th September 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February 2011; the June Earthquake that struck at 2.20pm on the 13th of June 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The report summarises the effects of the damage on the lateral load capacity of the building, and provides structural repairs for the damage identified. The general form of the building, along with its capacity relative to current code levels, has been included for the buildings pre-earthquake undamaged state and post-earthquake state. Recommendations for improving the seismic performance of the building have also been identified.

The Orthopaedic Rehabilitation Unit was designed in 2001 and constructed in the period thereafter. The building is 'L' shaped in plan and abuts the Administration Building on the South end of the building and Surgical Orthopaedic Unit (SOU) and Theatres on the East end. The building is a single storey structure consisting of reinforced precast concrete wall panels and a light weight roof constructed of steel roof purlins and beams. In the short or transverse direction of the building, lateral loads are resisted by a series of steel beams embedded in precast concrete blade columns forming portal frames on either side of the building. In the longitudinal direction lateral loads are resisted by precast concrete wall panels. The ground floor is constructed of a reinforced concrete slab on ground, and the foundation system consists of isolated concrete spread footings under the precast wall panels and interior steel columns.

The information available for the review included the original structural drawings [3], a post-earthquake geotechnical report for the campus conducted by Tonkin & Taylor[4], and a level survey of the building conducted by Fox & Associates[5].

In general, the structural damage observed to date is believed to have been primarily attributed to liquefaction and ground shaking induced lateral stretching, noted by a serious of cracks that have opened up in the ground floor concrete slab on grade, typically at existing shrinkage control joints. The cracking noted in the slab are most significant in the longitudinal direction of the building with cracks at the control joints typically between 3-5mm with a maximum of 12mm. The cumulative amount of cracking noted over the 63 metre length of the building is approximately 45mm. The cracking in the slab has resulted in minor cracking in exterior concrete footings and damage to interior partition walls and finishes.

It is believed that the majority of the damage observed, including the onset of damage, occurred as a result of the 22nd February event. Further observations of the earthquake damage observed have been included in the body of this report.

Based upon a review of the drawings available and site investigations completed the Orthopaedic Rehabilitation Unit has been assessed in its pre-earthquake undamaged state. For the purposes of this assessment the Orthopaedic Rehabilitation Unit has been considered to be an Importance Level 3 building (IL3, R=1.3).

Based on this review the assessed capacity of the primary lateral load resisting elements of the building, relative to the demand imposed by the current loading code Design Basis Earthquake (DBE), have been assessed at approximately 35 % DBE. The limiting factors on the assessed capacity of the building is the capacity of the spread footings below the blade columns to resist over-turning and sliding.

Post-earthquake the blade columns have been strengthened to 67% DBE (IL3) for sliding and overturning by the installation of slab ties and foundation extensions respectively. This work was completed in January 2015.

The damage noted can be considered relatively minor from a structural standpoint, and we do not believe the building's overall capacity has been significantly reduced from its pre-earthquake state. With that said some reduction in capacity has likely occurred due deformation of the building as a result of the earthquake induced differential ground settlement and lateral stretching noted. Structural repairs are required to address the damage noted but once the repairs have been completed the building's lateral load capacity should be restored to approximately pre-earthquake levels. Further observations of the earthquake damage observed have been included in the body of this report.

The minimum repairs required have been included in Section 4. These include the repair of the existing slab on grade, repair of cracks observed in the foundations and the damage noted to the interior partition walls. As it is impractical to push the building back together and restore the capacity of the structure lost due to the deformation noted, we are recommending the installation of new tie elements across the slab to prevent future spreading from occurring. This addition of the elements also increased the sliding capacity of the blade columns to 85% DBE (IL3). The recommended repairs to the slab on grade and the partition walls was completed in January 2015.

Recommendations for localized strengthening to improve the seismic performance of the building and increase the assessed capacity above 67% DBE have been included in Section 5. As noted above, the strengthening of the building to 67% DBE (IL3) was completed in January 2015.

We note that differential movement between buildings at the locations of seismic gaps may have occurred; services crossing at the junctions between buildings should be checked by relevant specialist contractors to confirm integrity.

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. Because all of the structure has not been available for detailed inspection or evaluation, this report is limited to those elements available and engineering judgement as to the likely condition of unseen elements. 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.

1. INTRODUCTION

Holmes Consulting Group has been engaged by Canterbury District Health Board to complete a full structural review of the Food Services Block at Burwood Hospital following the Lyttelton Earthquake. A series of reports have been completed as part of this. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The individual building reports, like this one, should be read in conjunction with the base report and refer to the repair specification.

The Burwood Hospital base report covers the purpose and scope of the structural review. The current statutory requirements relevant to earthquake damaged buildings are outlined and the level of shaking experienced at the site estimated. Earthquake induced ground settlement damage across the campus is also discussed. The repair specification has been prepared to include repair details for typical damage observed in buildings on the Burwood Hospital and is referred to as required in the specific building reports.

1.1 SCOPE OF WORK

This report is on the Orthopaedic Rehabilitation Unit located on the Burwood Hospital Campus at 255 Mairehau Road, Burwood, Christchurch. The report identifies the general form of the structure, along with the gravity and lateral load resisting systems. Each component of the structural system was reviewed based upon the information available and any potential Critical Structural Weaknesses (CSW's) were noted.

The report also identifies the structural damage observed to date as a result of the series of Earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake, in particular, subjected the building to strong ground motions which significantly exceeded the current code loading demand for buildings of this nature.

The capacity of the Orthopaedic Rehabilitation Unit has been assessed relative to current code loading in the buildings pre-earthquake undamaged state and in its post-earthquake damaged state. The post-earthquake assessment summarises the effects of the damage identified on both the gravity and lateral load resisting elements. Repair options to restore the buildings capacity to pre-earthquake levels for strength, durability and stiffness have been included. The repair options aim to maintain the buildings utility. Where required, strengthening options have also been provided.

1.2 LIMITATIONS

Findings presented as a part of this project are for the sole use of the Canterbury District Health Board, 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. Because all of the structure has not been available for detailed inspection or evaluation, this report is limited to those elements available and engineering judgement as to the likely condition of unseen elements. 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. PRE-EARTHQUAKE BUILDING CONDITION

This section discusses the form and capacity of the building prior to the Darfield Earthquake.

2.1 BUILDING FORM

The Orthopaedic Rehabilitation Building was designed in 2001 and constructed in the period thereafter. The structural design for the building was provided by Powell Fenwick Consultants Ltd.

The building is 'L' shaped in plan. The main Ward Area is approximately 62m by 13m in plan and the Eastern Wing is approximately 29m by 14m. The building is of single storey configuration with plant areas within the roof space of the Eastern Wing. The southern end of the building abuts the corridor of the Administration Building and eastern end abuts the Surgical Orthopaedic and Surgical Services Building.

Figure 2-1: Orthopaedic Rehabilitation Building, Burwood Hospital Campus

The information available for the review included the original structural drawings [3], a post-earthquake geotechnical report for the campus conducted by Tonkin & Taylor[4], and a level survey of the building conducted by Fox & Associates[5].

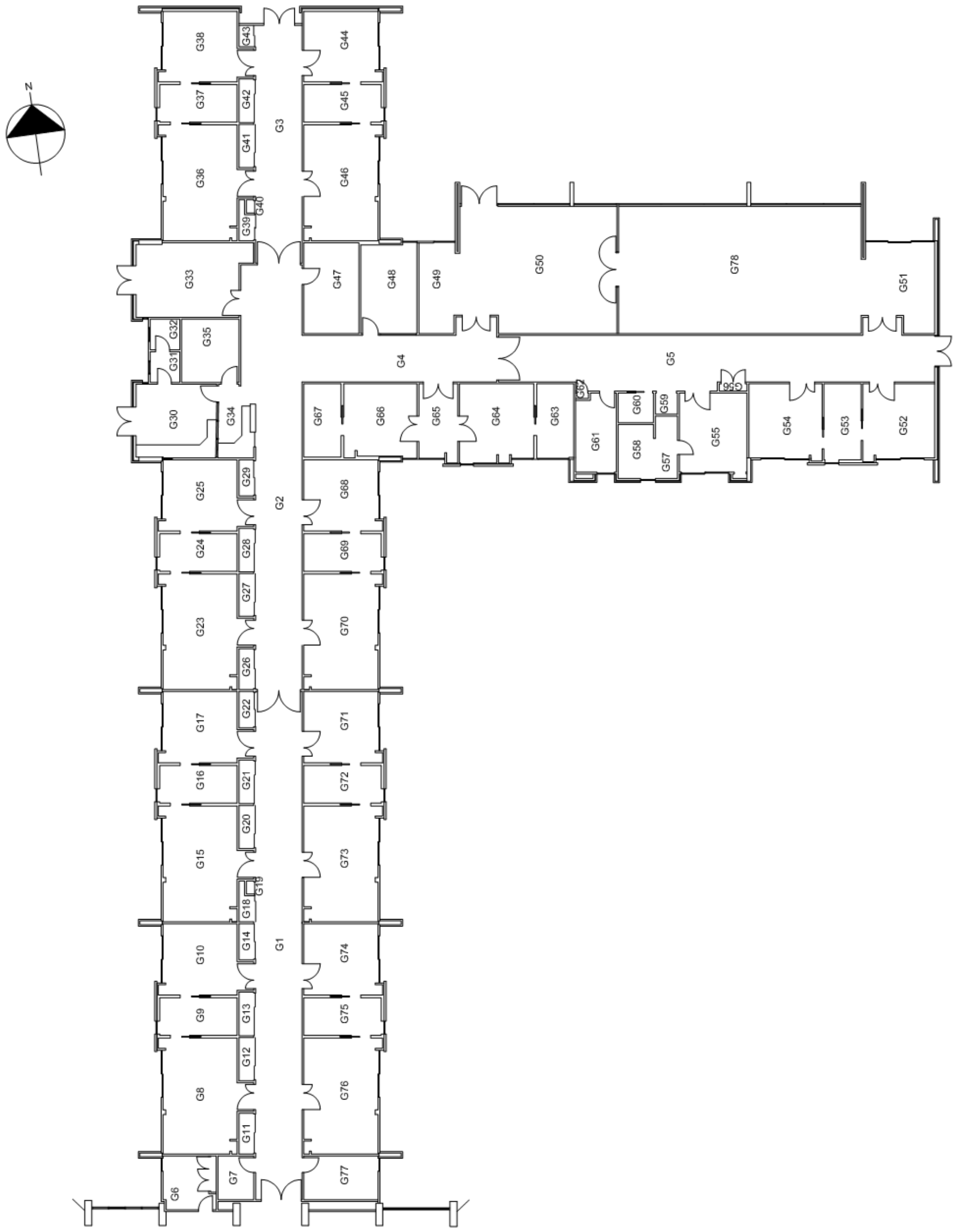


Figure 2-2 Architectural Floor and Room Numbering Plan

The Orthopaedic Rehabilitation Unit has a light weight corrugated metal roof over plywood sheathing which is supported by cold-formed steel 'DHS' roof purlins, spaced at approximately 1200mm centres. The steel roof purlins are in turn supported by steel portal beams which run in the transverse direction of the building. The ends of the beams are typically embedded in the top of the exterior precast concrete blade columns on either side of the building.

Along the exterior of the building, timber beams span between the exterior precast wall panels and support the edge of the roof framing. There are also isolated interior steel columns as required to support the plant, located in the ceiling space of the Eastern Wing.

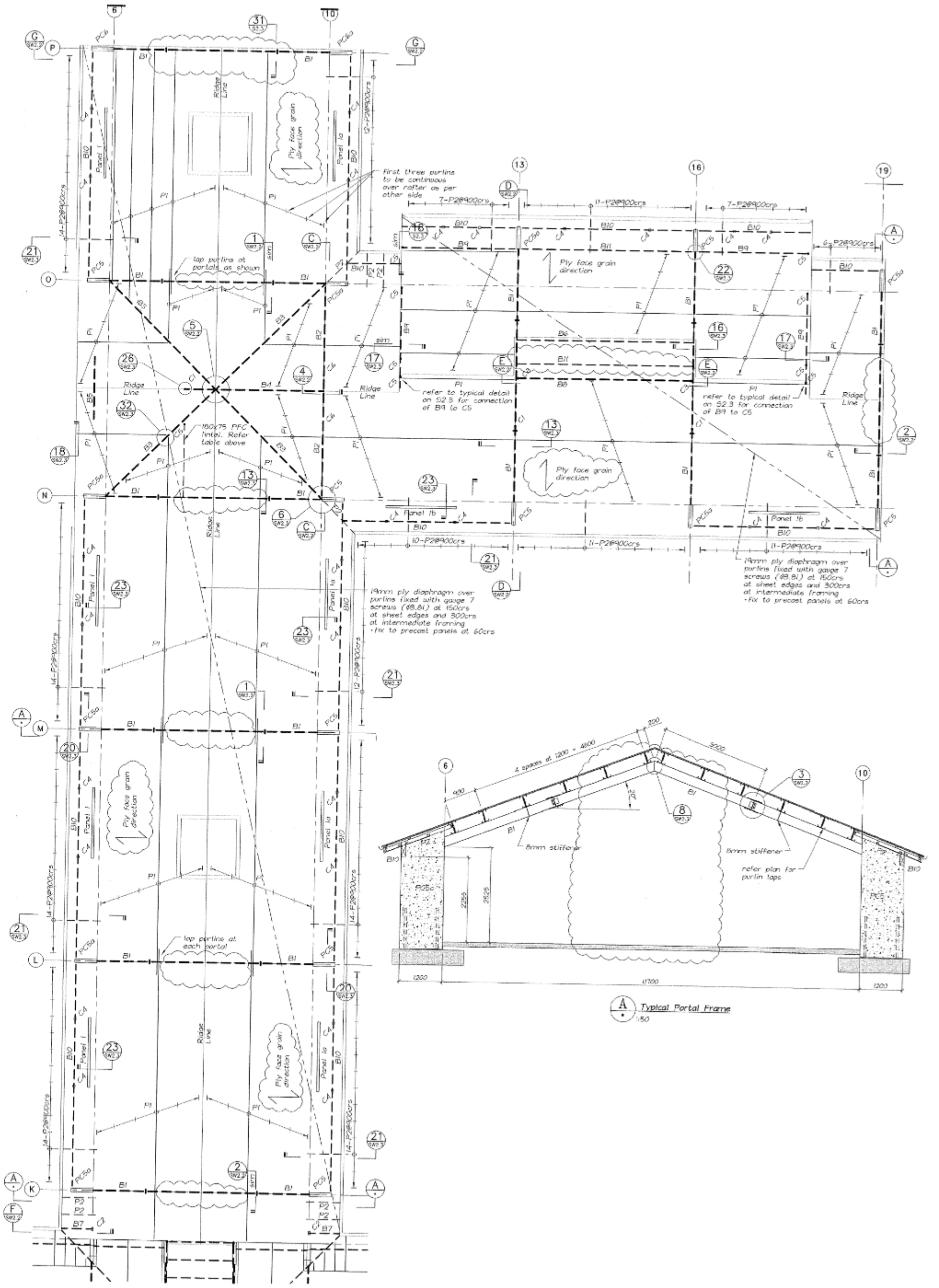


Figure 2-3 Structural Roof Framing Plan

The interior walls are typically non-load bearing, lightweight timber partition walls with lightweight gypsum board cladding. The interior partition walls are supported by an insitu mesh reinforced concrete slab on grade, over approximately 150mm of improved soil. The slab has a 450mm deep thickened edge beam along the perimeter.

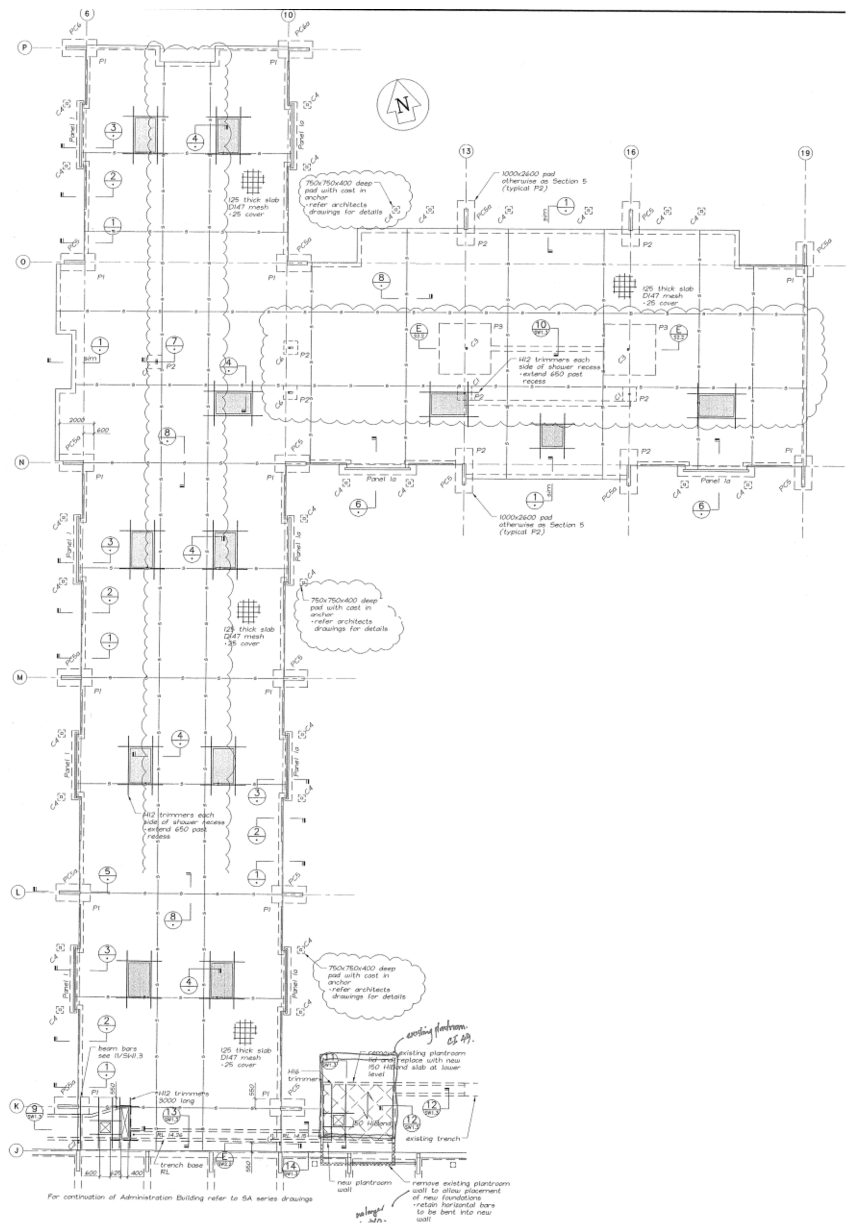


Figure 2-4 Structural Ground Floor / Foundation Plan

The precast concrete wall panels, precast concrete blade columns and interior steel columns are supported on isolated reinforced concrete spread footings.

Where the building abuts adjacent structures, the floor slab is dowelled in to the abutting building slabs while the superstructure is seismically separated with a 50mm gap.

2.2 LATERAL LOAD RESISTING SYSTEM

The lateral force resisting system for the building typically consists of a flexible roof diaphragm, formed by the plywood sheathing over cold-formed steel roof purlins. The diaphragm transfers seismic loads to the lateral load resisting elements below which consist of the steel beam / concrete blade column portal frames running in the transverse direction of the building and exterior precast wall panels running in the longitudinal direction of the building.

2.3 PRE-EARTHQUAKE BUILDING CAPACITY – DIRECT CODE COMPARISON

The building capacity under earthquake actions discussed in this section is compared to the capacity that a similar building would be designed to today. A new building would be designed to the *Structural Design Actions Standard, Part 5: Earthquake Actions – New Zealand*, NZS 1170.5:2004[9] and incorporating the amendments made to this standard as a result of the Lyttelton Earthquake as outlined in the Amendment 10 of the Building Code [8]. The implications of the recent amendments are discussed more in-depth in the Burwood Hospital Campus Base Report however, for a building of this type the amendments essentially result in an increase to the design loads of 36 % when compared to pre-earthquake design levels.

When the building was designed in 2001 the current loading standard at the time was the Code of Practice for General Structural Design and Design Loadings for Buildings, NZS 4203:1992 [10].

The original structural drawings are available, but the structural calculations and specifications were not, so the exact design and loading assumptions originally made are unknown. For the purposes of this report seismic loading assumptions have been made based on a detailed review of the drawings available and physical observations of the building.

A new building is required to be designed for an earthquake known as the Design Basis Earthquake (DBE), which is based upon the buildings physical location, local soil conditions, building type, fundamental period and importance level. The Orthopaedic Rehabilitation Unit is classified as an Importance Level 3 building in accordance with NZS 1170:2004 [9]. The associated return period of the DBE is 1000 years, with a risk factor for design of R = 1.3. The sub soil for the site is taken as Soil Type D, which is consistent with the findings of a post-earthquake geotechnical investigation [4].

Based upon the period of construction, and the detailing of the lateral load resisting elements, the system as a whole in each direction has been concluded to have limited ductility. For the purpose of this analysis, the precast blade columns and exterior precast wall panels have been assumed to rock at an associated ductility factor of approximately μ=2.0.

A comparison of the Design Basis Earthquake of NZS 4203:1992 [10] and NZS 1170:2004[9] for the site is plotted below. Based upon a fundamental building period below 0.50 seconds, the seismic demands required by the loading code have increased by approximately 40% since 2001. As a result a building designed to 100% of the DBE at the time of construction would currently have a capacity to resist approximately 70% of the demands imposed by the current code level DBE.

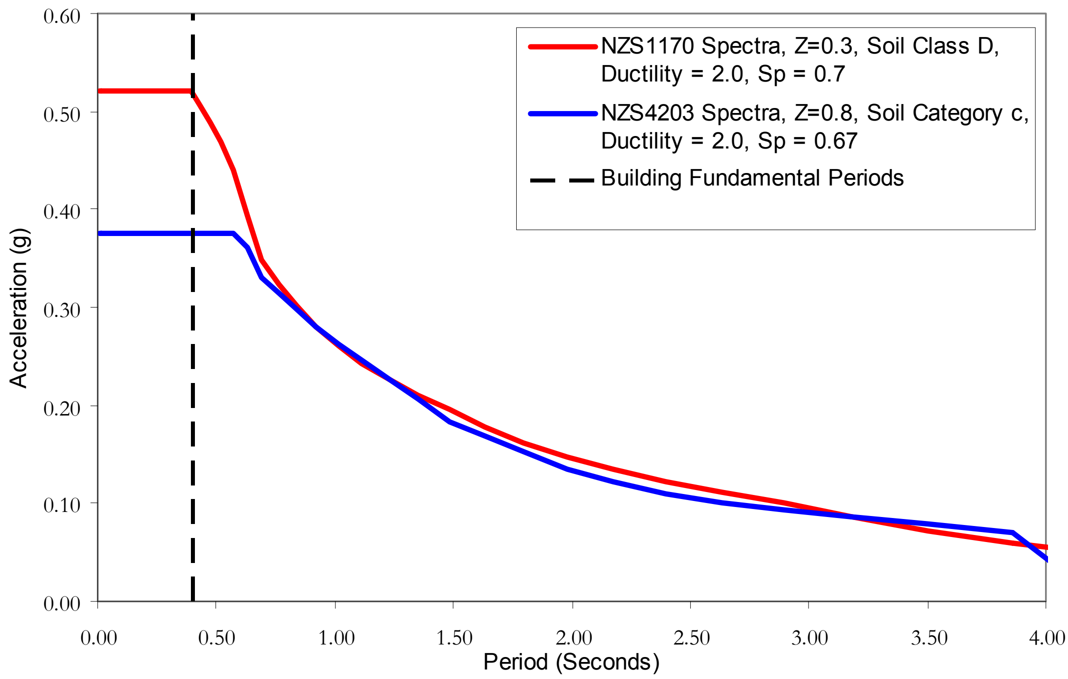


Figure 2-5: Comparison of Design Codes

2.4 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 estimated capacity when compared to current loading standards. The equivalent static analysis was carried out based upon the original structural drawings, and incorporation of on site measurements and as built observations.

Following the Lyttelton earthquake, a geotechnical report was conducted by Tonkin & Taylor, titled "Burwood Hospital Post Earthquake Geotechnical Assessment", dated June 2011 [4]. This report has been used to aid in the evaluation of the site conditions and the likely effect of the ground on the buildings past and future performance. The soil parameters described in the geotechnical report completed by Tonkin and Taylor has been used for the evaluation of the buildings existing foundation system.

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 DBE 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. These are described in more detail in the Burwood Hospital Base report [1] and include strength governed elements such as short columns and deflection governed elements such as floor and stair elements with inadequate support seating.

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 (DBE), and have been expressed as a %DBE. This includes checks for both the strength and deflection requirements.

Because neither the original structural calculations, specifications nor the general notes were available some assumptions had to be made in regards to the existing material properties of building elements in order to complete the seismic assessment. For example a compressive strength of 30 MPa has been assumed for the precast concrete wall panels. The foundations have been assessed at an ultimate bearing capacity of 150kPa (for factored loads) and a coefficient of friction against sliding of 0.58, as per recommendations provided by Tonkin and Taylor.

Based upon our analysis, the limiting factors on the assessed capacity of the building, in its pre-earthquake undamaged state, is the assessed over-turning and sliding capacity of the spread footings below the blade columns. These items have been assessed at 35% DBE under IL3 loading.

The loads applied to the blade columns and the footings are a combination of seismically induced lateral loads and an outward thrust imposed at the top of the blade columns by the steel portal frame under gravity loads. Note that post-earthquake the slab repairs completed (included in Appendix D) have incorporated new tie elements across the building connecting the spread footings under the precast blade columns, taking out the lateral thrust under gravity loads and increasing the assessed capacity of the building against sliding to 85%DBE.

Post-earthquake strengthening of the foundation to the blade columns has been completed, increasing the capacity of the blade columns for overturning to 67% DBE (IL3). The increased capacities have been included in Table 2-1.

The steel portal frame beams have been assessed at approximately 55% DBE (IL3) and are limited by the buckling capacity of the unrestrained bottom flange of the PFC beams.

A summary of the %DBE for each primary element has been noted in Table 2-1.

Building Element	**%DBE (IL2)**	**%DBE (IL3)**	**Comments**
Roof Diaphragm and Collectors	100%	100%	N/A
Steel Portal Frame Beams	70%	55%	Limited by y-axis buckling of unrestrained bottom flange of PFC section
Transverse Precast Blade Column	87%	67%	Limited by capacity of spread footing beneath precast blade column. Post-earthquake, foundations to the blade columns were extended increasing the overturning capacity of the blade columns from 35% DBE (IL3). This work was completed in January 2015.
Longitudinal Precast Wall Panels	100%	100%	N/A
Foundations - Sliding	100%	85%	Lack of positive tie at the base of the portal frame column footings makes them susceptible to sliding failure. Post-earthquake, foundation repairs incorporated a new continuous tie in the SOG between the precast blade columns increasing the assessed capacity from 40% DBE (IL2). This work was completed in January 2012.
Foundations - Bearing	100%	100%	Based upon ultimate bearing capacity of 150kPa

Table 2-1: Seismic Assessment %DBE

A review of the drawings available and site observations revealed no obvious critical structural weaknesses (CSW's) that could lead to premature collapse of the building. However, the drawings have indicated that there are no tie elements connecting the isolated spread footings, particularly in the transverse direction of the building. Strengthening to remove this Critical Structural Weakness was completed in January 2012 as noted above.

3. POST-EARTHQUAKE BUILDING CONDITION

This section covers the structural damage sustained by the Orthopaedic Rehabilitation Building at Burwood Hospital Campus as a result of the series of earthquakes that includes the Darfield Earthquake that struck at 4:36am on 4th September, 2010 and the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011, the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake subjected the building to strong ground motions which likely exceeded the full design earthquake load for buildings of this nature and appears to have caused the bulk of the earthquake damage observed after the initial Darfield event.

3.1 THE LYTTELTON EARTHQUAKE

The Fundamental Period of the building is estimated to be approximately 0.4 seconds. 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 strong motion data presented in the Base Report, it appears the Lyttelton Earthquake produced ground shaking intensities of approximately 60-120% of the current Ultimate Limit State design spectra for an Importance Level 2 building.

It should be noted that the Lyttelton Earthquake was very short in terms of strong shaking produced with the strong motion only lasting for duration of 5-7 seconds. Rupture of an alpine fault is expected to contain 50 to 60 seconds of strong motion.

3.2 PRELIMINARY INVESTIGATIONS

Preliminary investigations were carried out to identify 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
- review of available structural engineering construction documentation
- damage observed during 'Rapid' Structural Assessments following the Lyttelton Earthquake & June 13th aftershocks

In conjunction with a review of the structural drawings for the building the following areas were identified for potential damage:

- movement or damage to structure associated with ground movement and/or settlement
- cracking and joint failure of ground slabs and foundations

- cracking at base of precast wall panels and stressing of drossbach connections to spread footings below

- connection of steel portals to top of precast blade columns

- roof framing connections (collectors) at end of precast wall panels running in the longitudinal direction of the building

- connections of roof framing to exterior wall panels

- roof framing at seismic joint interfaces to adjoining buildings

A Rapid Level 2 assessment was carried out on the 24th February 2011[6]. An additional Level 2 assessment was conducted on the 14th June 2011 [7] following the June 13th earthquakes. Our structural observations involved a complete walk around the exterior and throughout the interior of the building. The following primary areas of damage were identified from the damage assessments:

- spreading of concrete floor slabs at locations of shrinkage control joints

- minor cracking in ceilings and interior partition walls

- roof framing at seismic joint interfaces to adjoining buildings

A review of the above information on the building type and preliminary observations highlighted this building as requiring a more detailed inspection. The aim of the detailed inspections was to determine the full extent of the damage caused to the building, particularly those elements identified for potential damage above. These areas were targeted to identify if damage had occurred, and to what extent the damage had reduce the capacity of the buildings lateral load resisting system to resist future seismic events.

3.3 DETAILED OBSERVATIONS

Further detailed inspections and structural explorations (including removal of finishes) have been carried out following the initial assessments to ascertain the full extent of structural damage. The detailed structural observations were completed between 3 November 2011 and 18 November 2011, and are summarised in Section 3. A full record of these observations is attached on Appendix A, with a reference plan indicating locations on Appendix B. A full photographic record of the observations is available electronically on request. The detailed structural observation picked up the following damage in addition to the items noted in the initial rapid assessments:

- further understanding of the extent of cracking to the concrete slab on grade (approximately 45mm in total over 63 meter length of building with a maximum crack width of 12mm noted)

- fan cracking pattern in exterior concrete footings at intersection of cracks noted in slab on grade

3.4 GEOTECHNICAL REVIEW

A review of the ground damage and conditions was carried out by Tonkin & Taylor for the Burwood Hospital Campus. A subsequent report titled *Burwood Hospital Post Earthquake Geotechnical Assessment*, was issued in June 2011 [4]. The geotechnical review concluded that the settlement and damage to building foundations and slabs on the Burwood Hospital Campus was likely due to the liquefaction of underlying soil layers. It is believed that excessive pore water pressures have been dissipated and that further settlement is not expected to occur, unless another significant event were to occur.

Only minor differential settlements have been observed to date. However, the report does note the potential for future total and differential settlements of between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

Despite only limited differential settlements, extensive spreading has been noted to the concrete slab on grade, which has been expressed by a serious of cracks that have opened up in the ground floor slab, typically at existing shrinkage control joints. Due to the flat topography of the site, it is believed that large scale lateral spreading is unlikely to occur at this site. Tonkin and Taylor have reported:

> *The lateral stretching observed is most likely attributed to residual deformation from ground movements during the earthquakes which have been estimated up to 200mm at the site. Part of this displacement is likely to be [due to] cyclic ground movements induced by liquefaction. Geotechnical testing indicates that approximately 50mm of liquefaction induced horizontal movement occurred during the earthquakes."*

Based upon the geotechnical advice provided by Tonkin & Taylor, in a future SLS event a cumulative movement of approximately 20mm has been predicted across the 63 meter length of the building with up to a 5mm opening occurring across any particular joint. In the case of a future ULS event up to 100mm cumulative movement can be expected across the length of the building with up to 25mm across any particular joint.

Note: The slab on grade repair recommendations, included in Appendix D, have been completed to tie the sections of slab together and address the lateral stretching concerns noted above. This includes the addition of new continuous tie elements, incorporated in the existing slab on grade.

3.5 LEVEL SURVEY

A detailed survey of the ground floor levels of the Orthopaedic Rehabilitation Unit was conducted by Fox & Associates and issued on 18th November, 2011 [4]. The survey indicates a maximum differential settlement of approximately 42mm over the footprint of the building. The worst case measured slope in the ground floor slab is approximately 1:400 (0.25%) across the breadth of the building. No other significant localised settlements were noted in the survey. For the extent of the differential settlement noted see the level survey included in Appendix C.

While the slopes noted in the ground floor slab may be within the acceptable range for this building, the Orthopaedic Rehabilitation Unit is interconnected with the Administration Building which requires re-levelling. When the Administration Building is re-levelled the Orthopaedic Building will likely be required to be lifted at the same time. For additional discussions on re-levelling see Section 4.

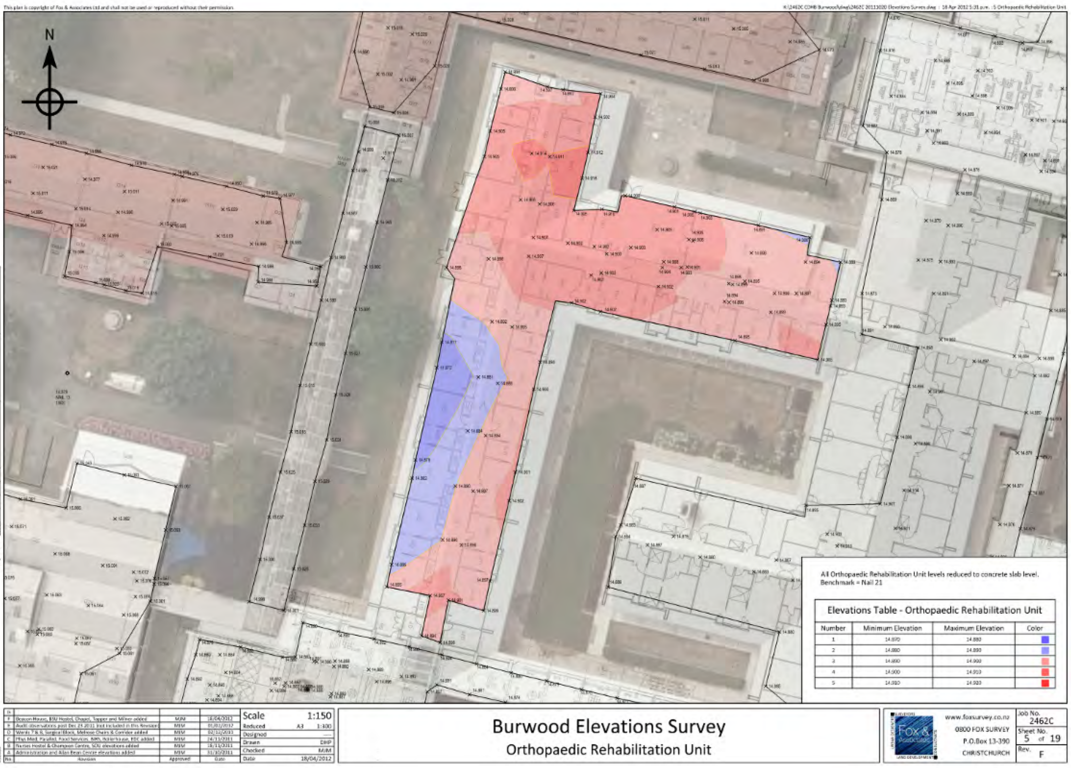


Figure 3-1: Level Survey

3.6 SUMMARY OF BUILDING DAMAGE

The following is a summary of the observations made for the building, and our conclusions as to their condition and seismic load resisting capacity. These observations do not specifically distinguish between damage caused by the Darfield Earthquake, the Lyttelton Earthquake or any significant aftershocks, such as those that occurred on the 13th June 2011, .the 23rd December 2011 or the 2nd January 2012. Despite not being able to specifically distinguish when individual building damage observed occurred, it is believed that the majority of the damaged, or at least the onset of damage, can be linked to the February 22nd event.

The majority of the Orthopaedic Rehabilitation Unit appears to have performed relatively well with the majority of the damage related to earthquake induced lateral stretching. Observations of some structural elements of the building were limited due to difficulties in accessing the elements hidden within building finishes. An attempt to observe a sample of the critical elements and connections has been made for the purposes of evaluation and reporting. Our observations suggest that the building would have undergone a limited number of full cycles of primarily elastic deformation. The short duration of the strong ground motion recorded and the damaged observed would support this hypothesis. A summary of the building damage observed can be typified as follows:

- **Spreading of concrete floor slabs** – Extensive spreading and cracking was noted in the concrete slab on grade, which were typically located at existing control joints. The cracking was most significant in the longitudinal direction with cracks in the control joints spanning the transverse direction of the building typically between 3-5mm, and a maximum of 12mm. The cumulative amount of cracking noted over the 63 metre length of the building was approximately 45mm. Similar cracking occurred in the transverse direction of the building to a lesser degree.

The shrinkage control joints are detailed so that the wire mesh reinforcing is stopped short on either side of the joint, and thus no reinforcing is present to tie the sections of the slab on grade together. Only minor differential vertical displacements (1-3mm) were noted in isolated locations across the slab joints. For the extent of the cracking to the slab on grade see the crack map included in Section 4.

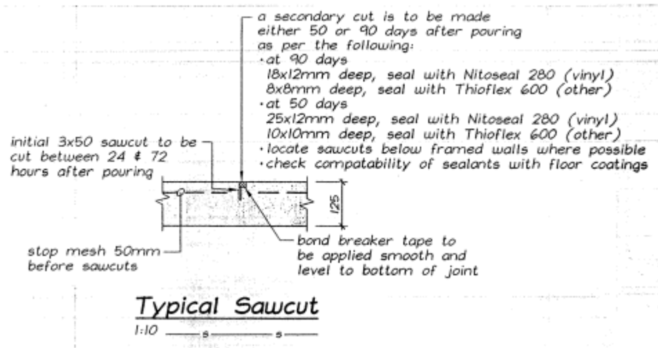


Figure 3-2: SOG Construction

- **Cracking to Exterior Concrete Strip Footings** – In general, the exterior footings are concealed below the surface and were not visible for observation. A small sample of the footings were exposed and in some locations "fan" shaped cracking pattern was noted at the intersection with the cracks in slab on grade. The exterior footings act to tie the slab together which explains the cracking pattern noted. No significant permanent rotations of isolated footings were noted; however a verticality survey of precast panels has not been carried out.

- **Precast Concrete Panels** - In general, no significant rotations were noted in the precast wall panels. At the exterior precast blade columns hairline cracking was noted. At this location the base of the wall was exposed and a hairline crack was observed in the bedding joint between the base of the panel and the spread footing below. It is not believed the movement observed has resulted in the yielding of the reinforcing in the Drossbach Ducts but this has not been tested. At the top of the blade columns hairline cracking was noted were the portal frame steel beams are embedded into the panel, in one of two locations which were exposed.

- **Roof Framing & Seismic Joints** – In general, no damage was noted in the roof framing except at the interface with the Administration Building. At this location rotations and deflections were noted in the roof framing and surrounding finishes.

- **Cracking to Interior Partition Walls** – Minor cracking to the interior partition walls was noted throughout. Larger cracks were noted adjacent to the cracks in the slab on grade. It is believe the damage is due to a combination of ground movement and shaking.

- **Cracking to Ceiling & other Finishes** - Minor cracking has been observed at wall and ceiling interfaces indicated racking of the ceiling, which is typical for the flat ceilings throughout. Larger cracks of up to 5mm have been noted adjacent to areas were larger cracks in the slab on grade have occurred.

Table 4-1, in Section 4 provides a photographic summary of typical damage observed. A full record of our detailed observations and repairs required can be found in Appendix A.

3.7 POST EARTHQUAKE BUILDING CAPACITY

Based upon our investigations to date, we do not consider the Orthopaedic Rehabilitation Unit to have any significant reduction to the overall gravity load resistance of the structure. While damage to the lateral load resisting system has occurred, the actual percentage reduction in the capacity of the building is hard to quantify.

The movement noted in the slab on grade is not believed to have significantly affected the existing capacity of the building as there was no reinforcing present across the control joints prior to the earthquake. We also believe the roof framing is flexible enough to have absorbed the 45mm of movement noted across the 63 meter length of the building, without imposing undue stress on the base of the precast wall panels. With that said, the wall panels will be slightly out of alignment resulting in some reduction in capacity.

While it is believed that the predicted movements noted for future Serviceability Limit State (SLS) and Ultimate Limit State (ULS) events can be absorbed without disproportionate damage or partial collapse of the building, we believe the accumulative stress to the precast elements under a ULS event will likely require the repair or replacement of these elements. The movement predicted for the SLS event is also likely to result in the damage of floor finishes and interior partition walls, requiring the future repair or replacement of these elements. For an Importance Level 3 Building, this is a once in twenty-five year event.

The damage observed will require repair to restore the strength, stiffness durability and performance of the individual structural components. The repair work recommended is outlined in Section 4. Following the recommended repair of the structural damage, the lateral load resisting performance of the structure will be restored to approximately pre-earthquakes levels (see Section 2).

Recommendations for strengthening to improve the seismic performance of the building and bring the assessed capacity of the building to above 67% DBE have been included in Section 5. In January 2015, the strengthening required to increase the capacity of the building to 67% BDE (IL3) was completed.

4. OBSERVED DAMAGE AND REQUIRED REPAIRS

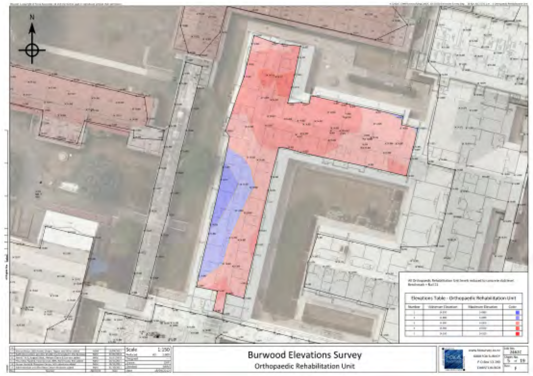
This section covers the damage noted during our detailed assessment of the building. Note that our observations have been restricted to structural aspects of the building only. Waterproofing elements, electrical and mechanical equipment, fire protection and safety systems, service connections, water supplies and sanitary fittings have not been inspected or reviewed, and secondary elements such as windows and fittings have not generally been reviewed.

Table 4-1 provides a photographic summary of the observed damage and typical repairs required for the Orthopaedic Rehabilitation Unit. Table 4-1 should be read in conjunction with Appendix A – Record of Observation. The Repair Specification [2] referred to in Table 4-1 has been issued separately.

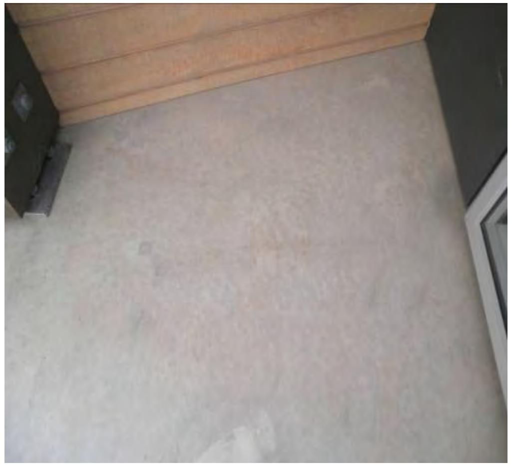
In general, the aim of the repair work indicated in this section is to restore the structure to its pre-earthquake state, as far as practicable, while maintaining the utility of the building. The repairs presented attempt to address the loss of strength, stiffness and durability of the structural elements due to the damage noted.

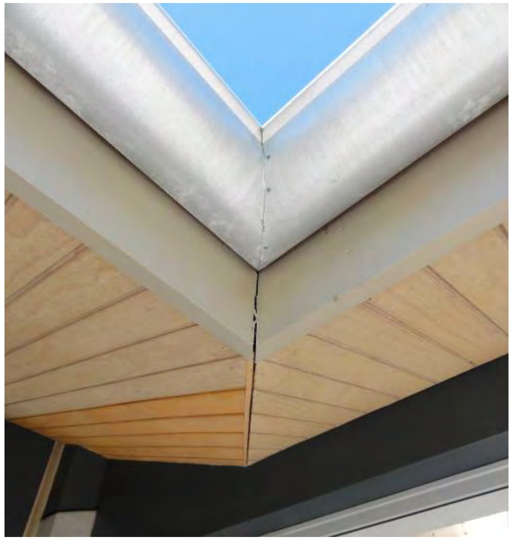
Further recommendations for localised strengthening to improve the seismic performance of the building and bring the assessed capacity of the building to above 67% DBE have been included in Section 5.

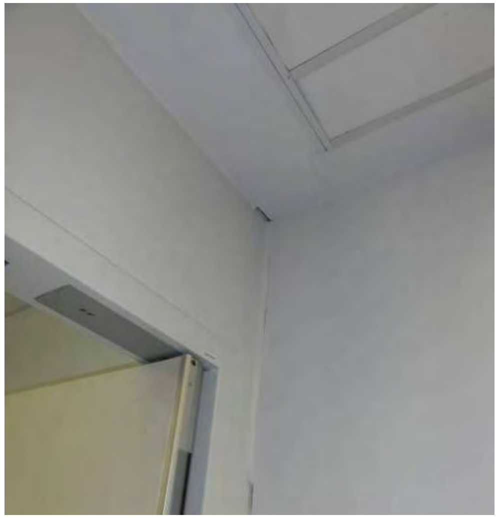
Table 4-1: Photographic Summary of Primary Damage Observed & Repairs Required

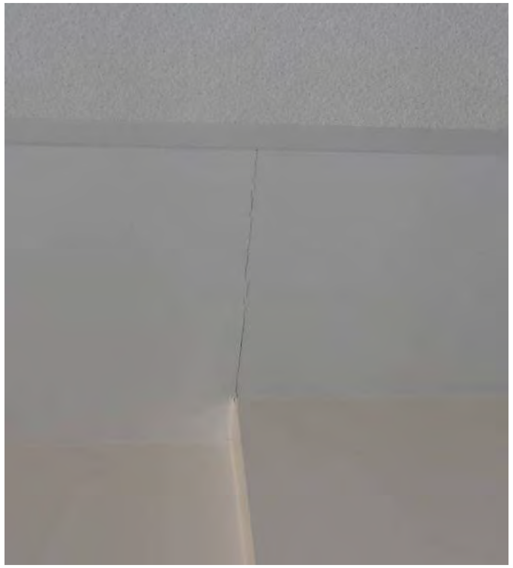
Item & Location	Observations	Recommended Repair	Photo Reference
1. Concrete Floors			
1.1. Slab on Grade	The survey indicates a maximum differential settlement of approximately 42mm over the footprint of the building. The worst case measured slope in the ground floor slab is approximately 1:400 (0.25%) across the breadth of the building.	If CDHB believe the slopes noted in the slab on grade to be unacceptable they can be re-levelled through the use of either underpinning grout of mechanical jacking techniques. For additional information see Section 4.3	
1.2. Room G48	Spreading of mesh reinforced concrete slab on grade has occurred, noted by cracks of up to 12mm located at the sawn shrinkage control joints. No shear reinforcing has been noted across the joints or documented in original structural engineering drawings.	Slabs are to be reinforced across all cracks and joints, and then grouted. Refer to joint and crack locations in Figure 4-1 and required repair sketches included in Appendix D. ***REPAIR COMPLETED***	

Item & Location	Observations	Recommended Repair	Photo Reference
1.3. Corridor G5	A number of cracks have been observed in the ground floor slab with minor vertical offsets across the joints noted	Slab to be reinforced across all cracks and joints, and then subsequently grouted. Refer to joint and crack locations in Section 4.1 and required repair sketches in Appendix D. Vertical offsets to be repaired with floor levelling compound. ***REPAIR COMPLETED***	

Item & Location	Observations	Recommended Repair	Photo Reference
2. Precast panel elements			
2.1. Exposed faces and brick cladding	No significant damage noted	No repair required.	
2.2. Footing Connection Grid O-16	0.2mm crack at the interface between grout bedding and the base of the precast	Inspect all connections between concrete blade columns and foundations. Where crack size exceeds 0.2mm, epoxy inject in accordance with HCG specification. ***REPAIR COMPLETED***	
2.3. Interface with Steel Portal Frame Beam Grid O-16	Series of hairline cracks at the location where the steel portal frame beam is cast in to the precast concrete blade column.	Inspect all steel beam / concrete blade column connections. Where cracks are greater than 0.3mm in width, epoxy inject in accordance with HCG specification. ***REPAIR COMPLETED***	

Item & Location	Observations	Recommended Repair	Photo Reference
3. Roof Framing			
3.1. Seismic Gap	10mm separation of eaves and fascia board adjacent to connection between Administration Building corridor and the Orthopaedic Rehabilitation Building. Apparent 'bow' or 'wave' in the lightweight metal roof sheeting at this location	Provide Aesthetic Repair. Note similar damage can be expected in a future SLS event.	
3.2. Seismic Gap	Apparent 'bow' or 'wave' in the lightweight metal roof sheeting at this location	Further exploration required.	

Item & Location	Observations	Recommended Repair	Photo Reference
4. Interior Partition Walls			
4.1. Typical to Corridor and Wards	0.1-2mm cracks typical throughout building, particularly at re-entrant corners of doors and bulkheads.	Aesthetic repair required.	
4.2.	Cracking at seismic joint.	Aesthetic repair required. Note similar damage can be expected following a future SLS event. ***REPAIR COMPLETED***	

Item & Location	Observations	Recommended Repair	Photo Reference
5. Ceilings			
5.1. Typical to Corridor and Wards	0.1-2mm cracking to ceilings and bulkheads	Aesthetic repair required.	
5.2. Corridor G5	5mm crack in ceiling board. Corresponds to lateral spreading of concrete floor slab below.	Aesthetic repair required. ***REPAIR COMPLETED***	

4.1 LATERAL STRECTHING IN SLAB ON GRADE

As discussed in Section 3, ground shaking and liquefaction induced lateral stretching has occurred and is noted by the cracks that have formed across the unreinforced control joints of the existing slab on grade. The locations and degree of cracking are shown below in Figure 4-1. The amount of stretching was measured at each notably cracked joint. The cumulative amount of spreading measured on site was 45mm in the North-South Direction and 35mm in the East-West direction. Spreading at each joint where significant damage was observed was typically between 5mm and 10mm. The largest crack noted on site was 12mm. No significant differential vertical displacements were noted across the joints. The building settlement appears to relatively uniform with only minor vertical stepping across the unreinforced joints.

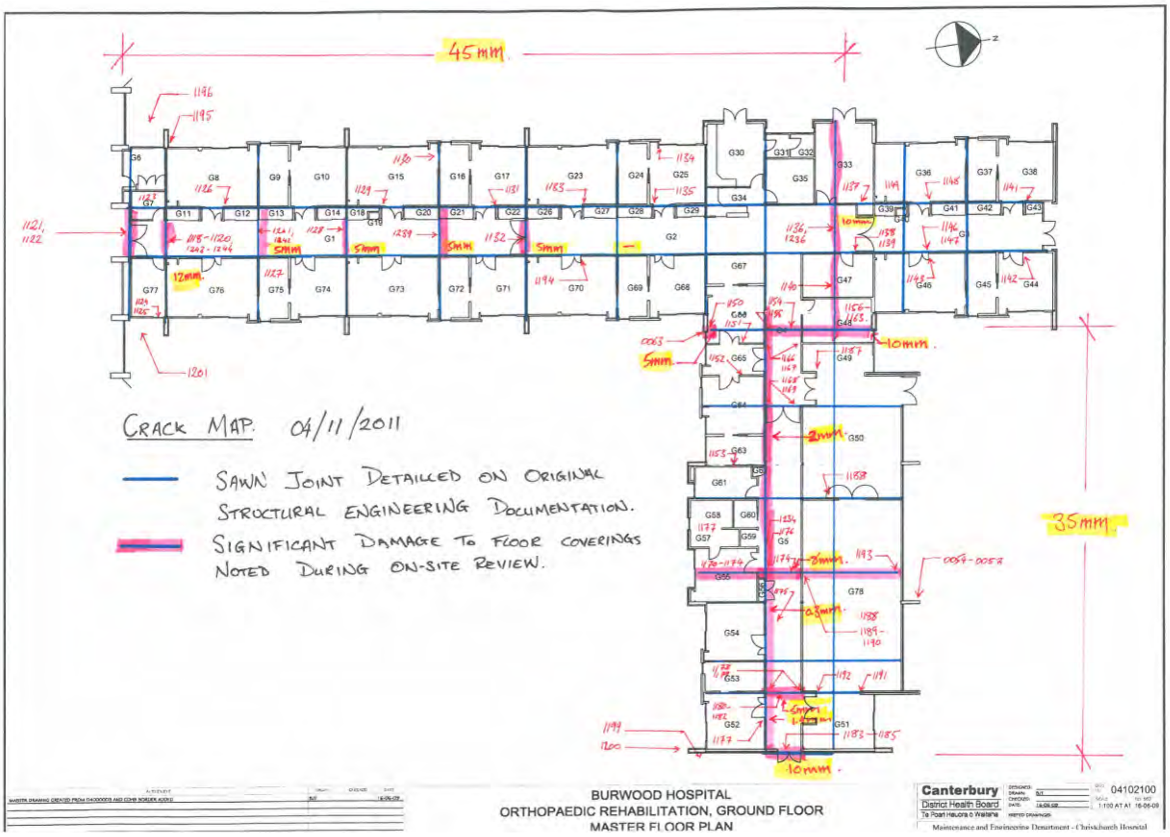


Figure 4-1 Crack map

This degree of lateral stretching is believed to have occurred as there is no reinforcement across the control joint which, if present, would act to restrain the sections of slab when subject to the ground movement. There is also no tie between the foundation elements in the transverse direction to prevent lateral spread of the foundations. NZS3101:2006 [11] requires spread footings to be tied together with an element capable of resisting a minimum of 10% of the factored axial load on the footing, under both compression and tension.

4.2 REQUIRED REPAIR OF SLAB ON GRADE CRACKS

Repair of the cracks across the shrinkage control joints is required to reinstate the structural performance and durability of the slab. To prevent similar damage from occurring in a future serviceability level event and to prevent structural issues arising due to lateral spread in an ultimate limit state event, the slab will be required to be physically 'stitched' back together placing new D12 reinforcing bars across the existing joints at 600mm centres. A chase will be required to be cut in the slab in order to place the D12 bars. The chase would then be packed with high-strength non-shrink grout.

The installation of the D12 reinforcing bars will require portions of the mesh to be cut which is acceptable. At joint intersections, the joints highlighted in red, in Figure 4-2, are to take precedence over the joints shown in yellow. The added bars are not to intersect. Between the added D12 reinforcing bars the existing cracks are to be repaired using Sika Grout 212 where practical. At smaller, or concealed cracks (under existing walls), the cracks are to be repaired with Sikadur 52 low viscosity crack injection epoxy.

As it is impractical to push the building back together and restore the capacity of the structure lost due to the deformation noted, we are recommending the installation of new tie elements across the slab to prevent future spreading from occurring. This will require the installation of new tie elements incorporated with the slab repairs. These will consist of continuous bars placed in chases in the slab and dowelled into the concrete blade column footings on either side of the building.

For the complete slab on grade repair plans and details see the drawings attached in Appendix D.

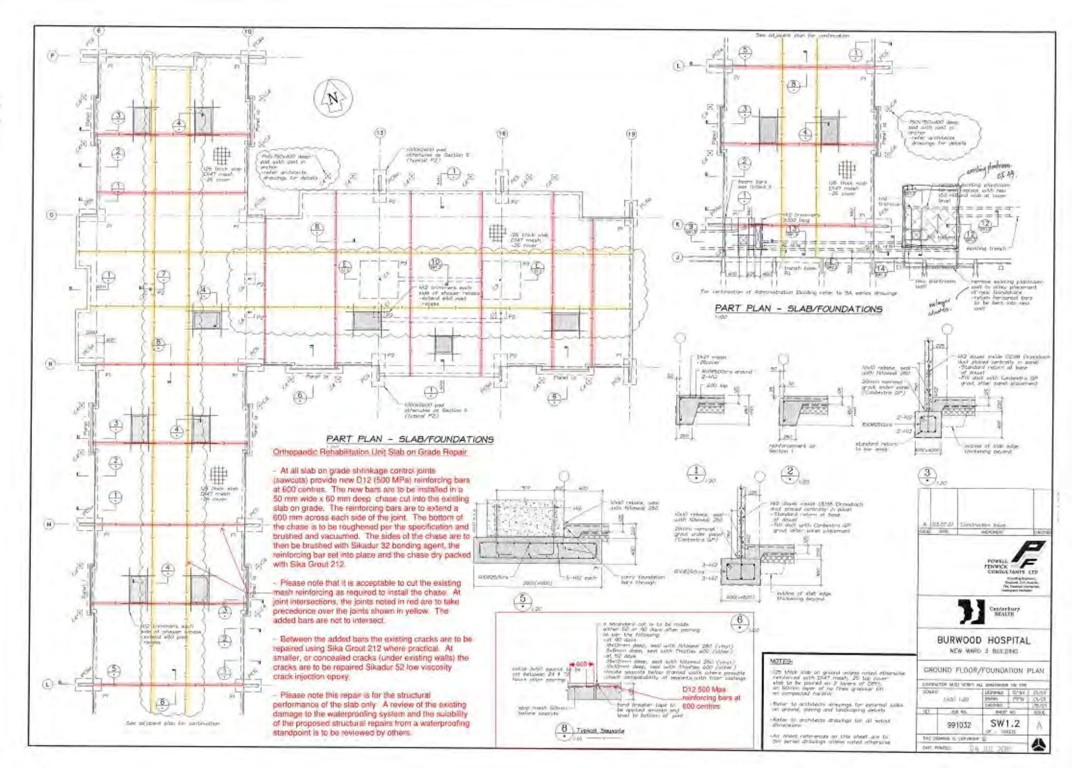


Figure 4-2: Slab on Grade Repair

Stitching the joints together, will reduce the capacity of the slab to compensate for natural volumetric changes in the concrete, however the majority of concrete curing shrinkage has already occurred and the slab is in a temperature controlled environment and not subject to major fluctuations in temperature induced expansion or contraction.

Please note this repair is for the structural performance of the slab only. A review of the existing damage to the waterproofing system and the suitability of the proposed structural repairs from a waterproofing standpoint are to be reviewed by others. Note that the existing waterproofing membrane, which consists of two layers of DPM over 50mm of granular fill, has been exposed in one location with no damage observed.

Note: The slab on grade repairs have been completed as described above.

4.3 DISCUSSION ON RE-LEVELLING

While the slopes noted in the ground floor slab may be within the acceptable range for this building, the Orthopaedic Rehabilitation Unit is interconnected with the Administration Building which requires re-levelling. When the Administration Building is re-levelled the Orthopaedic Building will likely be required to be lifted at the same time. This could require lifting the entire structure up to 57mm. For additional information on the re-levelling options available see the *Burwood Hospital Campus - Administration Building Detailed Seismic Assessment Report, Revision 2*, dated 6th August 2012[12].

5. STRENGTHENING RECOMMENDED

The main lateral load resisting system of the Orthopaedic Rehabilitation Unit is provided by precast concrete blade column and steel beam portal frames in the transverse direction of the building and by precast wall panels in the longitudinal direction.

As noted in Section 2, Pre-Earthquake Building Condition, and Section 3, Post-Earthquake Building Condition, the lateral load resisting capacity of each section of the building has been assessed as a percentage of the loads imposed by the Design Basis Earthquake (DBE). The capacity of the building is governed by the over-turning and sliding capacity of the spread footing beneath the precast concrete blade columns at either end of the portal frames (35% DBE).

Recommended strengthening to improve the seismic performance of the building and bring the assessed capacity of the building to above 67% DBE have been included in Section 5.1.

5.1 STRENGTHENING WORKS TO ACHIEVE 67% DBE (IL3)

Based on our detailed structural assessment, we have identified the base of the precast blade columns and the associated footings as requiring strengthening to bring the lateral force resisting system of the building above 67% DBE, assuming an Importance Level 3 (IL3) building.

The loading imposed on both of these elements is a combination of seismically induced lateral loads and a gravity load induced thrust imposed by the steel portal frame beams on the top of the precast blade columns. Strengthening of the blade columns is required for overturning and sliding.

5.1.1 Strengthening for Overturning

Two options to strengthen the blade columns for overturning have been identified:

- Connect the blade columns with a tie rod at ceiling level
- Extend the blade column foundation

5.1.2 Strengthening with Tie Rods

The strengthening proposed is to connect the end of the steel roof beams with a M24 threaded pre-tensioned tie-rod in order to remove the gravity load induced thrust. This tie rod may be fabricated as per details in Appendix D. The removal of the gravity load reduces the imposed moment at the base of the blade column and the sliding to the associated footings to a degree that both elements would be at approximately 100% DBE.

The tie-rods will need to be notched into existing partition walls which will provide the vertical support for the rod. We note that the installation of the rod will require the repair of the notched studs and may interfere with existing fire rating to the steel roof beams and walls which would need to be reinstated. A fire engineering consultant should be contacted for a review of the proposed work.

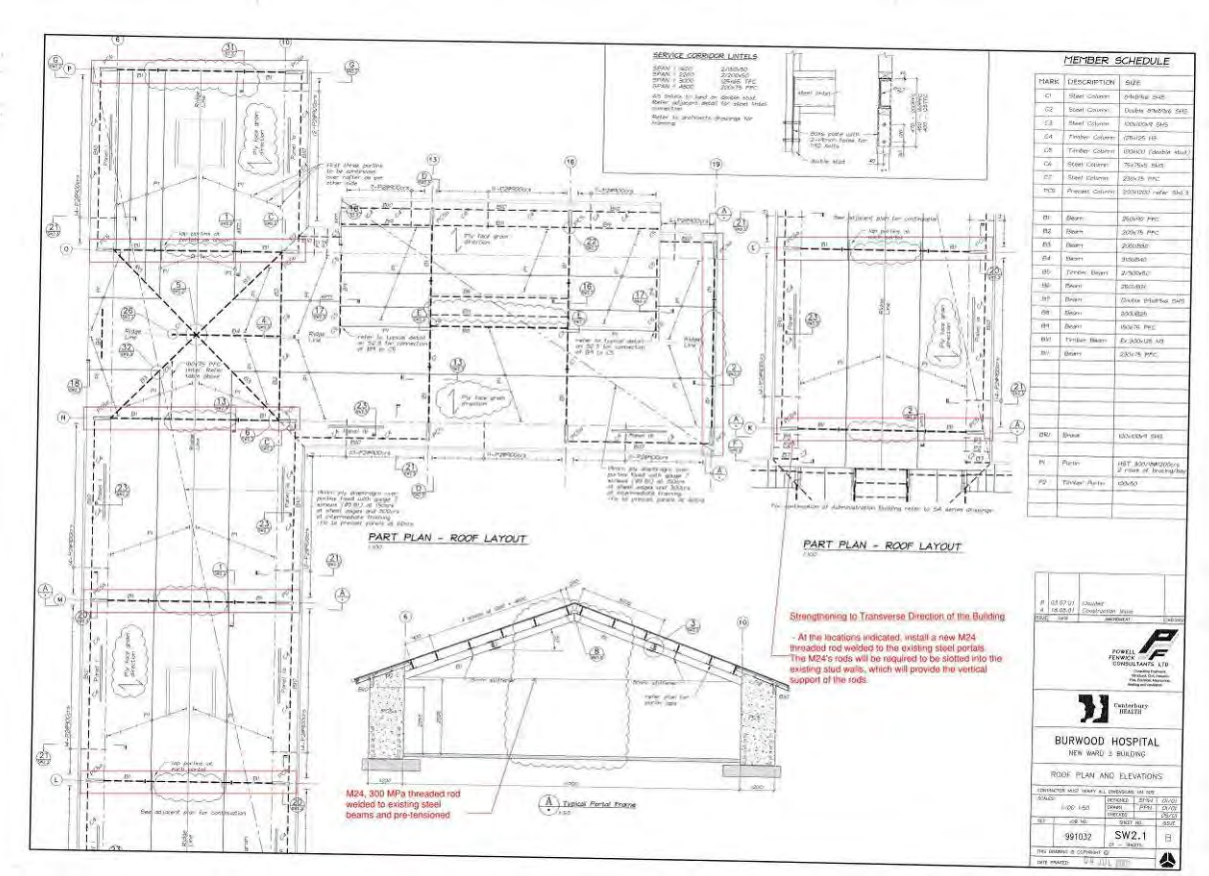


Figure 5-1: Recommended Strengthening for Overturning to Achieve 67%DBE

5.1.3 Strengthening by Extending the Blade Column Foundation

The overturning capacity of the blade columns can be increased to 67% DBE (IL3) by extending the foundations on the outside face of the building. This strengthening option is detailed in drawings titled CDHB Burwood Ortho Rehab Ward Strengthening – Consent Exception issue and dated 05 March 2014 and numbered 106186.65 S01-01 and S01-02.

In January 2015, the blade column foundation extensions were completed, increasing the overturning capacity of the building to 67% DBE (IL3).

To strengthen the blade columns for sliding, ties are required to be provided between the columns on either side of the building. These slab ties have been recommended for the repair of the slab cracks and lateral spread damage as outlined in Section 4.

In January 2012, the installation of floor slab ties was completed increasing the sliding capacity of the blade columns to 85% DBE (IL3).

6. REFERENCES

1 *Burwood Hospital Campus – Detailed Seismic Assessment Report – Base Report*, Holmes Consulting Group, November 2011.

2 *Burwood Hospital Campus – Detailed Seismic Assessment Report – Repair Specification*, Holmes Consulting Group, November 2011.

3 *Burwood Hospital – New Ward 3 Building – Structural Engineering Construction Documentation,* Powell Fenwick Consultants LTD, 2001

4 *Burwood Hospital Post Earthquake Geotechnical Assessment*, Tonkin and Taylor Ltd. June 2011.

5 *CDHB – Burwood Field Survey*, Fox & Associates, July 2011 (revised November 2011).

6 *CDHB – Burwood Hospital Campus – Rapid Visual Inspection: 106186.03SR1,* Holmes Consulting Group, February 2011

7 *CDHB – Burwood Hospital – Rapid Seismic Assessment Post June 13th Earthquakes*, Holmes Consulting Group, 14 June 2011.

8 Department of Building and Housing, *Compliance Document for New Zealand Building Code - Clause B1 – Structure, Amendment 10 (Canterbury)*, Department of Building and Housing, Wellington, 19 May 2011.

9 *Structural Design Actions Part 5: Earthquake Actions – New Zealand, NZS 1170.5:2004*, Standards New Zealand, 2004.

10 *Code of Practice for General Structural Design and Design Loadings for Buildings*, NZS4203:1992, Standards New Zealand, 1992

11 *NZS3101:2006 Concrete Structures Standard*, Standards New Zealand, 2006

12 *Burwood Hospital Campus - Administration Building Detailed Seismic Assessment Report, Revision 2*, dated 6th August 2012

13 *New Zealand Society for Earthquake Engineering Guidelines for the assessment of the structural performance of buildings in earthquakes* – NZSEE 2006

APPENDIX A

Record of Observations

APPENDIX A – RECORD OF OBSERVATIONS & REPAIRS

Inspection date: 3 - 18 November 2011

KEY	
N	No repair required
Y	Repair required
F	Further investigation required
C	Repair complete

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	G78, G51	Bulkhead	0.2mm cracks to bulkhead wall board. Cracking at interface between bulkhead and ceiling	Y	Aesthetic repair to bulkhead	1191, 1192
GND	G33	Ceiling	2mm crack in ceiling board	Y	Aesthetic repair to ceiling.	1137
GND	G4	Ceiling	2-3mm crack in corridor ceiling	Y	Aesthetic repair to ceiling.	1166, 1167
GND	G4	Ceiling	2-3mm crack in corridor ceiling	Y	Aesthetic repair to ceiling.	1168, 1169
GND	G5	Ceiling	2mm crack in ceiling	Y	Aesthetic repair to ceiling.	1176
GND	G5	Ceiling	2-5mm tapered horizontal crack in ceiling corresponding to spreading in concrete floor below	Y	Aesthetic repair to ceiling.	1178 - 1179
GND	G55	Ceiling	0.3mm tapered vertical crack to wall and ceiling. Propagates from door head	Y	Aesthetic repair to ceiling.	1170
GND	G65	Ceiling	Horizontal crack at interface of wall and ceiling	Y	Aesthetic repair to ceiling.	1152
GND	G66	Ceiling	Horizontal crack at interface of wall and ceiling	Y	Aesthetic repair to ceiling.	1151
GND	G36	Ceiling / Sprinkler	Separation and damage of ceiling around sprinkler. Several instance of this damage was noted throughout building	Y	Aesthetic repair to ceiling.	1149

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	External	Eaves	5mm crack/delamination of fascia boards and eaves linings adjacent to seismic gap between buildings	**F**		1196
GND	External	Fascia and Eaves	10mm separation of timber fascia and eaves lining adjacent to seismic gap between buildings. Eaves lining has dislodged in this location	**F**		1201
GND	G1	Floor Slab	Crack in slab at joint location covered with carpet. Apparent step in floor	**C**	Repair cracks in floor slab as per Section 4	1118
GND	G1	Floor Slab	Movement in slab at seismic joint concealed by vinyl floor covering. Vinyl has lifted at the seismic joint	**Y**	Repair cracks in floor slab as per Section 4	1121-1122
GND	G1	Floor Slab	Horizontal crack in slab at apparent shrinkage joint. Crack is concealed by floor covering. Vertical step apparent	**C**	Repair cracks in floor slab as per Section 4	1128
GND	G1	Floor Slab	Horizontal crack in slab at apparent shrinkage joint. Crack is concealed by floor covering. Vertical step apparent	**C**	Repair cracks in floor slab as per Section 4	1132
GND	G1	Floor Slab	Horizontal crack in slab at apparent shrinkage joint. Crack is concealed by floor covering. Vertical step apparent. Crack continues through rooms G33, G47 and G48 as shown on crack location plan	**C**	Repair cracks in floor slab as per Section 4	1136
GND	G4	Floor Slab	Horizontal crack in slab at apparent shrinkage joint. Crack is concealed by floor covering. Carpet has lifted at the edge of the corridor	**C**	Repair cracks in floor slab as per Section 4	1154, 1155

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	G47	Floor Slab	Horizontal crack in slab at apparent shrinkage joint. Crack is concealed by floor covering.	C	Repair cracks in floor slab as per Section 4	1140
GND	G48	Floor Slab	Approximately 10mm lateral spreading of sawn joints in concrete slab. Joints are detailed in structural documentation as unreinforced across saw cut. No reinforcement was noted in the N-S joint, however mesh was noted as continuous across E-W joint. No significant vertical displacement was noted across joints	C	Repair cracks in floor slab as per Section 4	1156 - 1160
GND	G48	Floor Slab	Spreading of crack through stiffened edge beam. Cracks propagate from spreading of sawn joint.	C	Repair cracks in floor slab as per Section 4	1161
GND	G5	Floor Slab	Damage to carpet floor covering. Concrete slab is concealed. Possible spreading of sawn joint or cracking in concrete slab.	Y	Repair cracks in floor slab as per Section 4	1175
GND	G5	Floor Slab	5mm lateral spreading of slab. No significant vertical displacement apparent at this location.	C	Repair cracks in floor slab as per Section 4	1180 - 1182
GND	G5	Floor Slab	Crack in floor at abutment of Ortho Rehab BLDG to Surgical Ortho BLDG. There appears to be both horizontal and vertical displacements at this building joint.	Y	Repair cracks in floor slab as per Section 4	1183
GND	G55 and G5	Floor Slab	Approximately 5mm lateral spreading of slab. No significant cracking away from sawn joint noted. Approximately 1-2mm vertical displacement noted across joint. No shear reinforcing across slab sawn joint was present or detailed in structural documentation.	Y	Repair cracks in floor slab as per Section 4	1171 - 1174

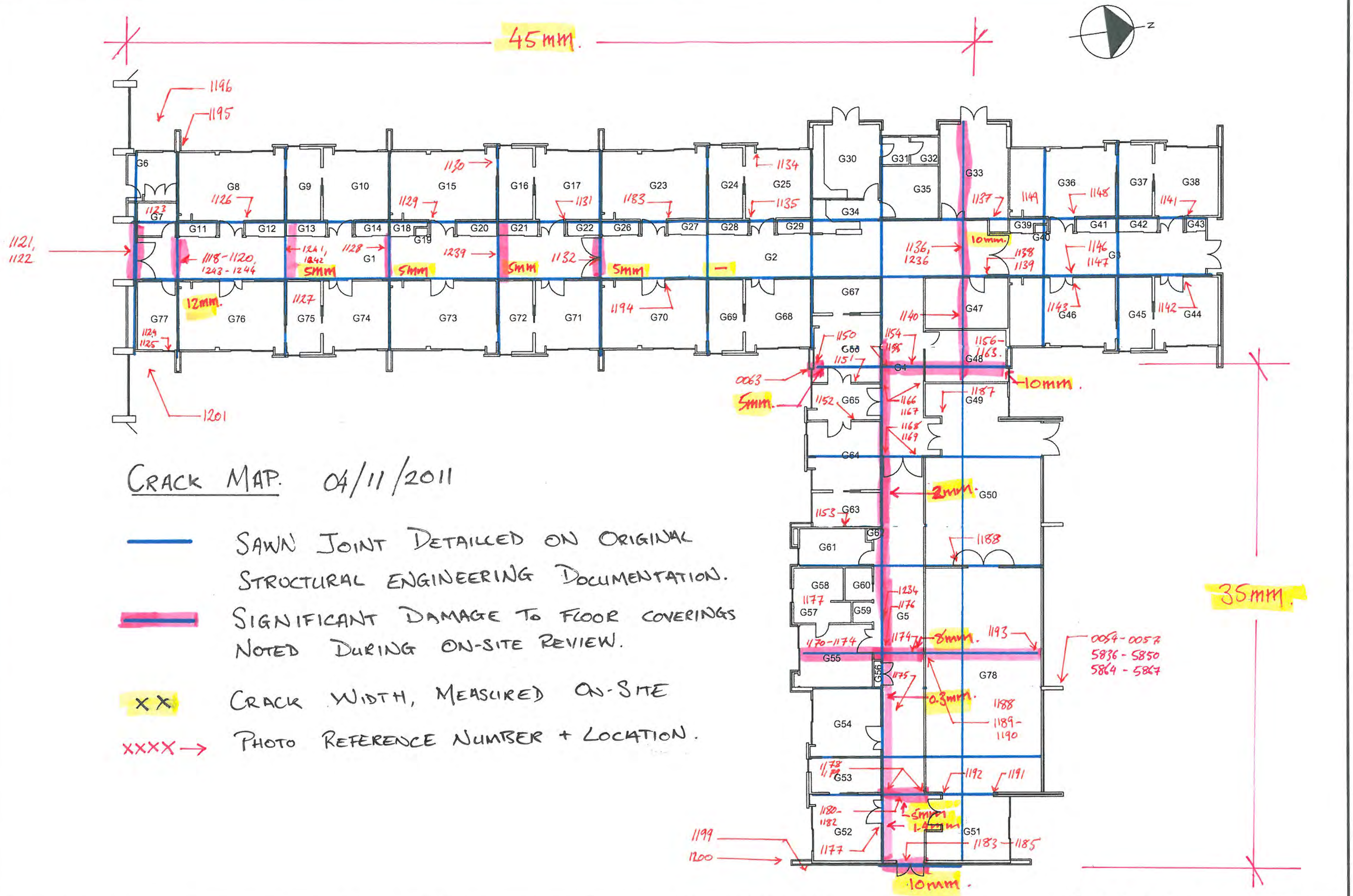
Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	G63	Floor Slab	Large waves in vinyl where it has delaminated from the concrete floor slab	**Y**	Repair cracks in floor slab as per Section 4	1153
GND	G66	Floor Slab	Damaged vinyl at slab edge adjacent to window	**Y**	Repair cracks in floor slab as per Section 4	1150
GND	External	Paving	10-20mm of spreading at of paving at joint between buildings.	**N**	Landscape Item	1200
GND	External	Precast Blade Column	Spalling of concrete from face of column	**N**	Spalling is due to removal of nail used to hold in eaves board. This is a result of the investigation, but is not earthquake related and doe not significantly affect the capacity of the column.	5848
GND	External	Precast Blade Footings	0.2mm horizontal crack at interface of grout between precast concrete blade column and footing	**N**	Damage may not be due to earthquake. Does not significantly affect the capacity of the member	5842, 5864
GND	External	Roof	Bend/Wave in lightweight metal sheeting and gutter adjacent to seismic gap between buildings	**F**		1195
GND	External	Roof	Noticeable bow/curvature in roof sheeting. May be typical deflection or could be indicative of bowed or buckled roof framing members. Further investigation required.	**F**		1197, 1198
GND	External	Roof	Negative curvature/wave of roof sheet adjacent to seismic gap. Further investigation of member and connections required.	**F**		1202
GND	G1	Wall	2mm vertical crack in wall at door head	**Y**	Aesthetic repair to wall board.	1119
GND	G1	Wall	Vertical cracks in wall at door head	**Y**	Aesthetic repair to wall board.	1120

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	G1	Wall	Cracking in ceiling edge board corresponding to cracks in rooms. This damage appears as typical throughout the corridors	Y	Aesthetic repair to wall board.	1146, 1147
GND	G17	Wall	0.3mm tapered vertical crack in bulkhead	Y	Aesthetic repair to wall board.	1131
GND	G23	Wall	0.3mm tapered vertical crack in bulkhead	Y	Aesthetic repair to wall board.	1133
GND	G25	Wall	0.3mm tapered crack to wallboard	Y	Aesthetic repair to wall board.	1134
GND	G25	Wall	0.3mm tapered vertical crack in bulkhead	Y	Aesthetic repair to wall board.	1135
GND	G36	Wall	0.3mm vertical crack to bulkhead	Y	Aesthetic repair to wall board.	1148
GND	G38	Wall	0.2mm vertical crack to wallboard	Y	Aesthetic repair to wall board.	1141
GND	G44	Wall	0.5mm vertical crack to wallboard	Y	Aesthetic repair to wall board.	1142
GND	G50	Wall	0.2mm vertical crack in wallboard adjacent to door head	Y	Aesthetic repair to wall board.	1187
GND	G70	Wall	0.2mm vertical crack to wallboard adjacent to door head	Y	Aesthetic repair to wall board.	1194
GND	G75	Wall	0.5mm tapered diagonal cracks propagating from ceiling access hole	Y	Aesthetic repair to wall board.	1127
GND	G77	Wall	3mm crack propagating from window frame and through wall	Y	Aesthetic repair to wall board.	1125
GND	G78	Wall	Vertical cracking to wall. Approximately 0.3mm	Y	Aesthetic repair to wall board.	1189, 1190
GND	G78	Wall	5mm vertical crack to wallboard	Y	Aesthetic repair to wall board.	1193
GND	G8	Wall	0.5mm vertical crack in bulk head in ceiling	Y	Aesthetic repair to wall board.	1126
GND	G15	Wall	0.3mm tapered vertical crack in bulkhead	Y	Aesthetic repair to wall board.	1129
GND	G15	Wall	0.5mm horizontal crack propagating from door head	Y	Aesthetic repair to wall board.	1130
GND	G46	Wall	0.2mm vertical crack to wall. Propagates through ceiling	Y	Aesthetic repair to wall board.	1143

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	G52	Wall	0.2mm vertical crack in wallboard adjacent to door head	Y	Aesthetic repair to wall board.	1177
GND	G77	Wall	Diagonal cracks in wallboard propagating from window corners	Y	Aesthetic repair to finishes.	1124
GND	G4	Wall and Ceiling	4mm vertical crack to wall and ceiling	Y	Aesthetic repair to finishes.	1164, 1165
GND	G48	Wall and Ceiling	Cracking at interface of wall and ceiling	Y	Aesthetic repair to finishes.	1162, 1163
GND	G49	Wall and Ceiling	0.2mm vertical crack to wall. Cracking at interface of wall and ceiling at bulkhead.	Y	Aesthetic repair to finishes.	1186
GND	G5	Wall and Ceiling	Series of horizontal and vertical cracks <5mm to wall and ceiling at building joint.	Y	Aesthetic repair to finishes.	1184, 1185
GND	G7	Wall and Ceiling	Crack propagating from door jamb, crack continues through ceiling at this location	Y	Aesthetic repair to finishes.	1123
GND	G78	Wall and Ceiling	Crack at interface between wall and ceiling	Y	Aesthetic repair to finishes.	1188
GND	G1	Wall and Door	2mm vertical crack in wallboard. Approximately 10mm of movement in stainless steel cladding apparent	Y	Aesthetic repair to finishes.	1138, 1139
GND	External	Wall and Eaves	10-20mm of spreading of wall and eaves at joint between buildings.	Y	Aesthetic repair only. Contractor is to remove finishes and confirm the integrity of hidden structure.	1199
GND	External	Floor Slab	5mm crack in floor slab adjacent to location of internal shrinkage control joint	C	Repair cracks in floor slab as per Section 4	63

APPENDIX B

Reference/Key Plans



AMENDMENT	DRAWN	CHECKED	DATE
MASTER DRAWING CREATED FROM 04000003 AND CDHB BORDER ADDED	BJT		16-06-09

BURWOOD HOSPITAL
ORTHOPAEDIC REHABILITATION, GROUND FLOOR
MASTER FLOOR PLAN

Canterbury District Health Board
Te Poari Hauora o Waitaha

DESIGNED:
DRAWN: BJT
CHECKED:
DATE: 16-06-09
XREFED DRAWINGS:

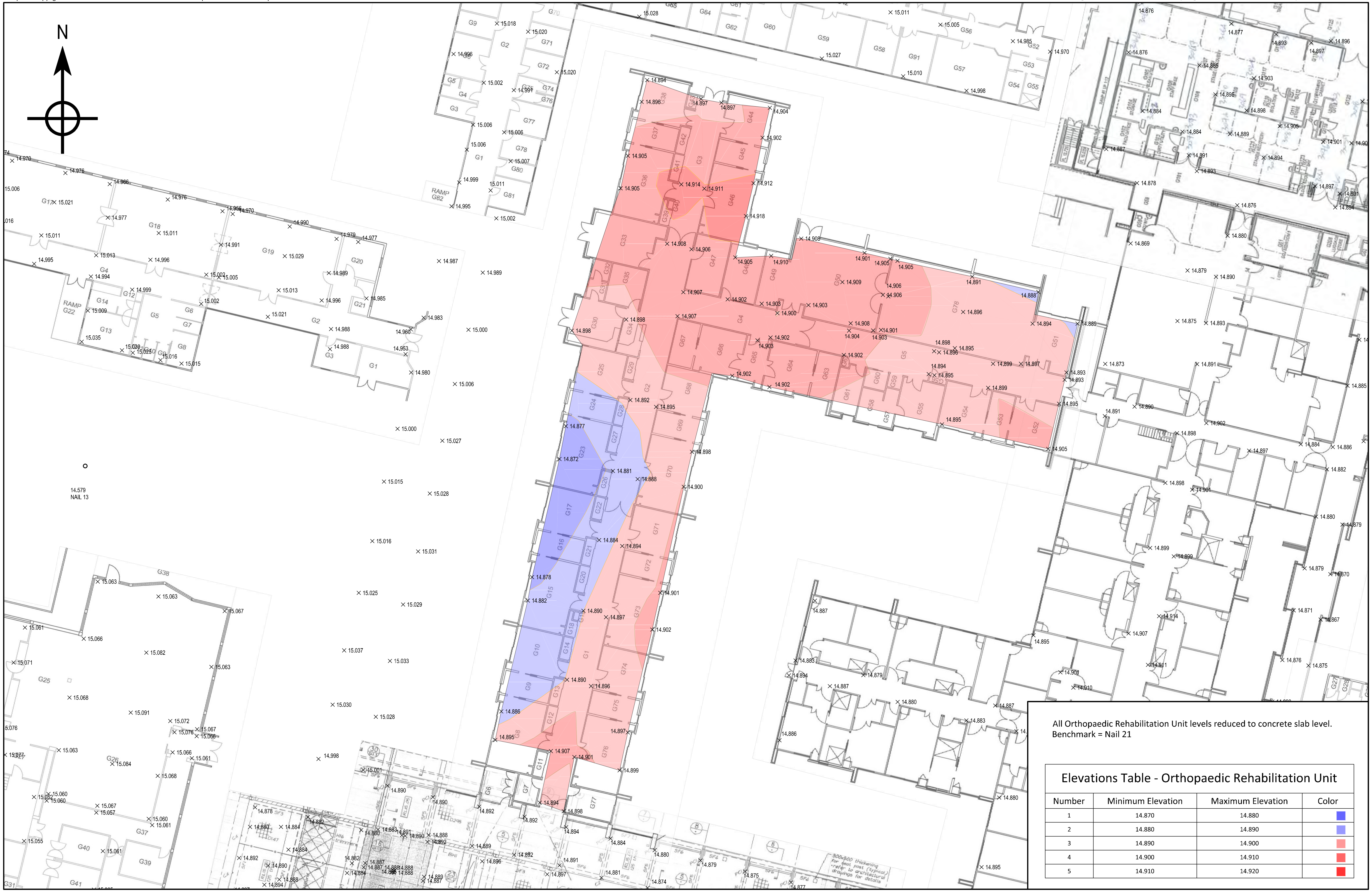
DWG No. 04102100
SCALE 1:100 AT A1
REV-STD 16-06-09

Maintenance and Engineering Department - Christchurch Hospital

APPENDIX C

Survey of Levels

K:\2462C CDHB Burwood\dwg\2462C 20111020 Elevations Survey.dwg : 02 Dec 2011 11:58 a.m. : 5 Orthopaedic Rehabilitation Unit



All Orthopaedic Rehabilitation Unit levels reduced to concrete slab level.
Benchmark = Nail 21

Elevations Table - Orthopaedic Rehabilitation Unit

Number	Minimum Elevation	Maximum Elevation	Color
1	14.870	14.880	
2	14.880	14.890	
3	14.890	14.900	
4	14.900	14.910	
5	14.910	14.920	

No.	Revision	Approved	Date
G			
F			
E			
D	Wards 7 & 8, Surgical Block, Melrose Chairs & Corridor added	MJM	02/12/2011
C	Phys Med, Parafed, Food Services, BIRS, Boilerhouse, EDC added	MJM	24/11/2011
B	Nurses Hostel & Champion Centre, SOU elevations added	MJM	18/11/2011
A	Administration and Allan Bean Centre elevations added	MJM	31/10/2011

Scale		1:150
Reduced	A3	1:300
Designed		----
Drawn		DHP
Checked		MJM
Date		02/12/2011

Burwood Elevations Survey
Orthopaedic Rehabilitation Unit

SURVEYORS URBAN DESIGNERS LAND DEVELOPMENT CONSULTANTS
Fox & Associates
www.foxsurvey.co.nz
0800 FOX SURVEY
P.O.Box 13-390
CHRISTCHURCH

Job No. 2462C
Sheet No. 5 of 14
Rev. D

A1 Original Size

032005

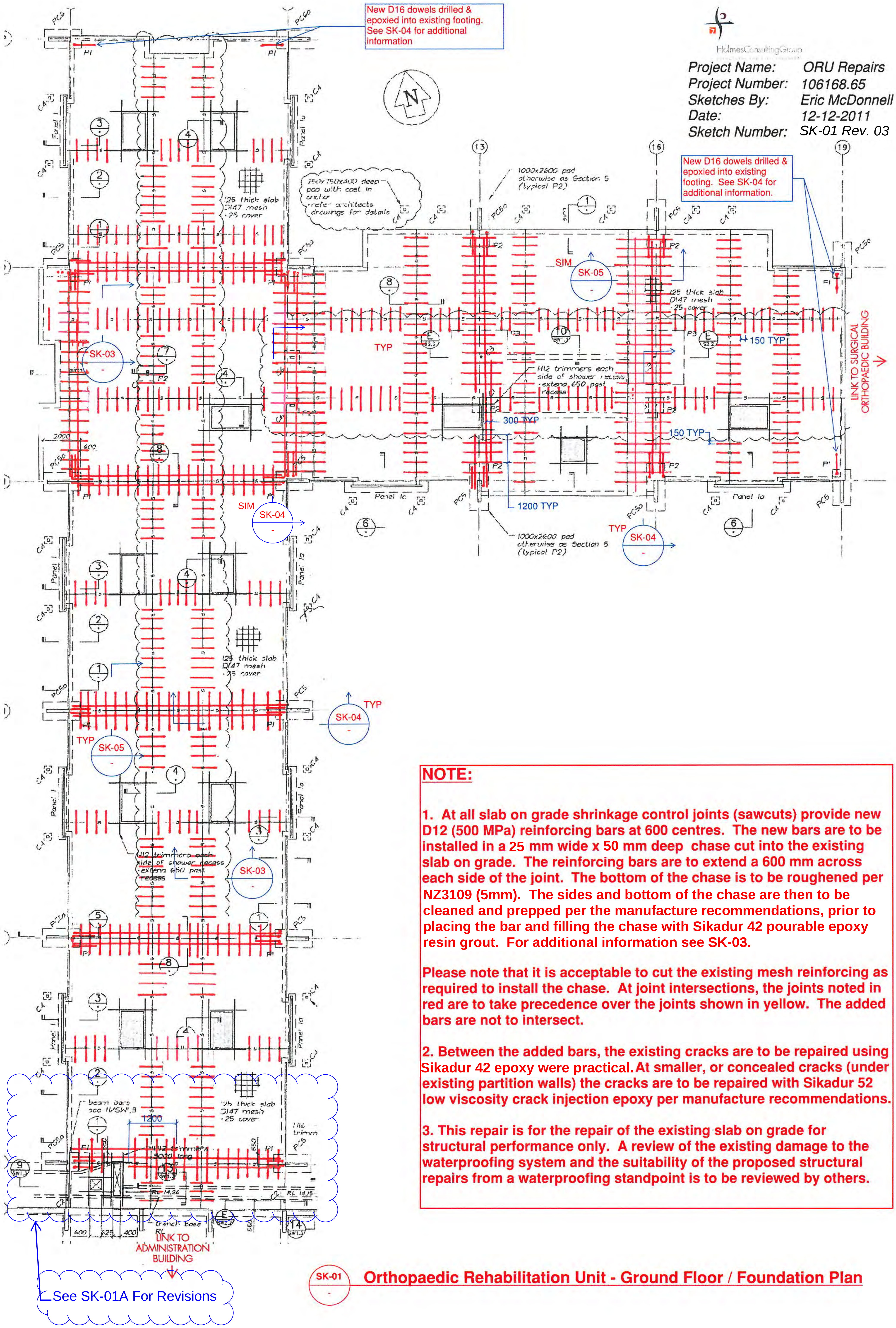
APPENDIX D

Recommended Repairs

Sketches

HolmesConsultingGroup

Project Name: ORU Repairs
Project Number: 106168.65
Sketches By: Eric McDonnell
Date: 12-12-2011
Sketch Number: SK-01 Rev. 03



NOTE:

1. At all slab on grade shrinkage control joints (sawcuts) provide new D12 (500 MPa) reinforcing bars at 600 centres. The new bars are to be installed in a 25 mm wide x 50 mm deep chase cut into the existing slab on grade. The reinforcing bars are to extend a 600 mm across each side of the joint. The bottom of the chase is to be roughened per NZ3109 (5mm). The sides and bottom of the chase are then to be cleaned and prepped per the manufacture recommendations, prior to placing the bar and filling the chase with Sikadur 42 pourable epoxy resin grout. For additional information see SK-03.

Please note that it is acceptable to cut the existing mesh reinforcing as required to install the chase. At joint intersections, the joints noted in red are to take precedence over the joints shown in yellow. The added bars are not to intersect.

2. Between the added bars, the existing cracks are to be repaired using Sikadur 42 epoxy were practical. At smaller, or concealed cracks (under existing partition walls) the cracks are to be repaired with Sikadur 52 low viscosity crack injection epoxy per manufacture recommendations.

3. This repair is for the repair of the existing slab on grade for structural performance only. A review of the existing damage to the waterproofing system and the suitability of the proposed structural repairs from a waterproofing standpoint is to be reviewed by others.

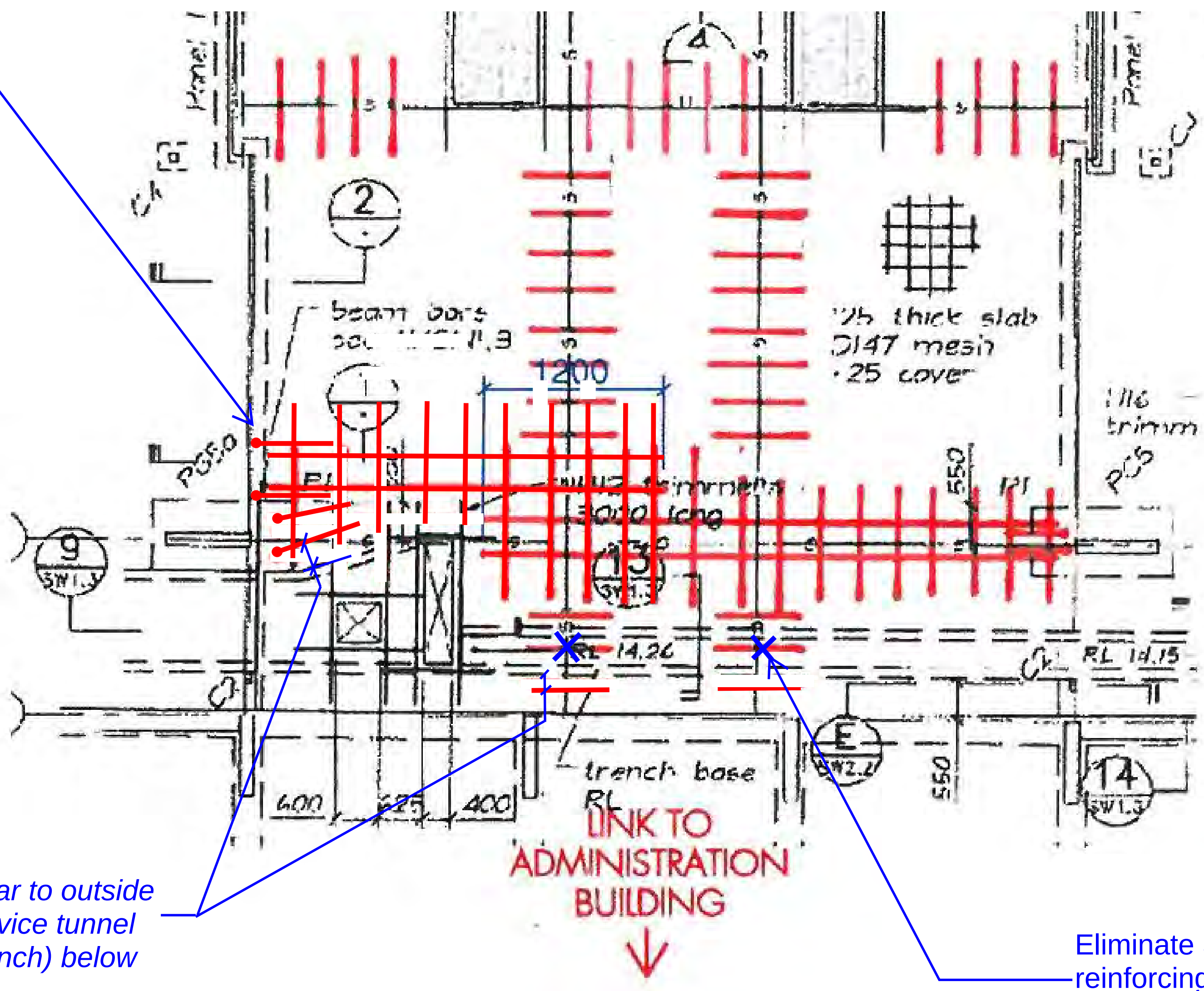
SK-01

Orthopaedic Rehabilitation Unit - Ground Floor / Foundation Plan

Revise bar layout to avoid conflict with service trench. Rotate dowels into existing blade column footings as required. At added dowels into exterior strip footing, install dowels as close to exterior wall as practical and to a depth of 175mm

HolmesConsultingGroup

Project Name:	*ORU Repairs*
Project Number:	*106168.65*
Sketches By:	*Eric McDonnell*
Date:	*12-12-2011*
Sketch Number:	*SK-01A Rev. 01*

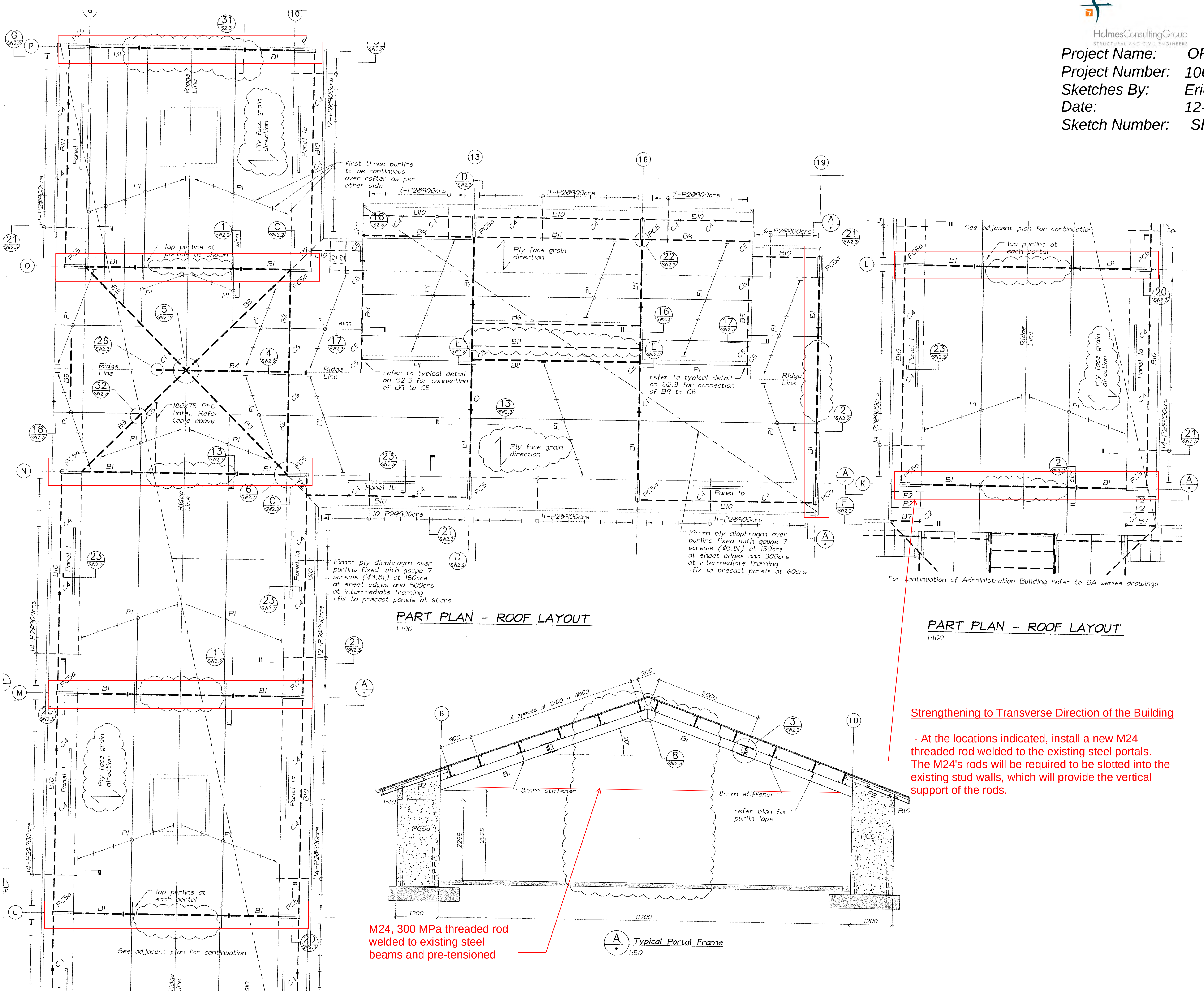


75 mm clear to outside face of service tunnel wall (or trench) below

Eliminate Stitch reinforcing over existing trench

HolmesConsultingGroup
STRUCTURAL AND CIVIL ENGINEERS

Project Name: ORU Repairs
Project Number: 106168.65
Sketches By: Eric McDonnell
Date: 12-12-2011
Sketch Number: SK-02 Rev. 0



Strengthening to Transverse Direction of the Building

- At the locations indicated, install a new M24 threaded rod welded to the existing steel portals. The M24's rods will be required to be slotted into the existing stud walls, which will provide the vertical support of the rods.

M24, 300 MPa threaded rod welded to existing steel beams and pre-tensioned

HolmesConsultingGroup

Project Name: ORU REPAIRS

Project No: 10618665

Calcs By: ERIC McDONNELL

Date: 19/12/2011

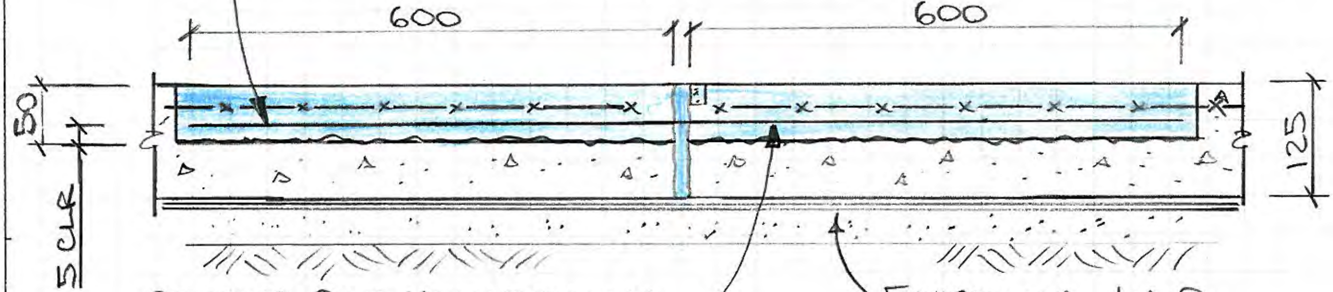
Page No:

Sketch No: SK-03

Revision: 1

CALCS/SKETCHES

AT ALL EXISTING SHRINKAGE CONTROL JOINTS (SAWCUTS) PROVIDE NEW D12 (500 MPa) REINFORCING BARS AT 600 CENTRES. NEW REINF. BARS ARE TO BE INSTALLED IN A 25mm WIDE x 50 mm DEEP CHASE CUT INTO THE EXISTING SLAB ON GRADE. THE BOTTOM OF THE CHASE IS TO BE ROUGHENED TO 5mm AMPLITUDE PER NZ3109. ALL THREE SIDES ARE THEN TO BE PREPPED & CLEANED, PER MNFTR. RECOMMEDATIONS, PRIOR TO PLACING BAR & FILLING CHASE W/ SIKADUR 42 POURABLE EPOXY RESIN GROUT.



CENTER BAR HORIZOTALLY IN CHASE. PROVIDE 5mm CLEAR, ALL SIDES.

EXISTING W.P. MEMBRANE

SK-03 - TYPICAL SLAB 'STITCH' REPAIR

NOT TO SCALE

NOTE: REVIEW OF WATERPROOFING TO BE COMPLETED BY OTHERS

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Project Name: ORU REPAIRS

Project No: 106185.65

Calcs By: ERIC McDONNELL

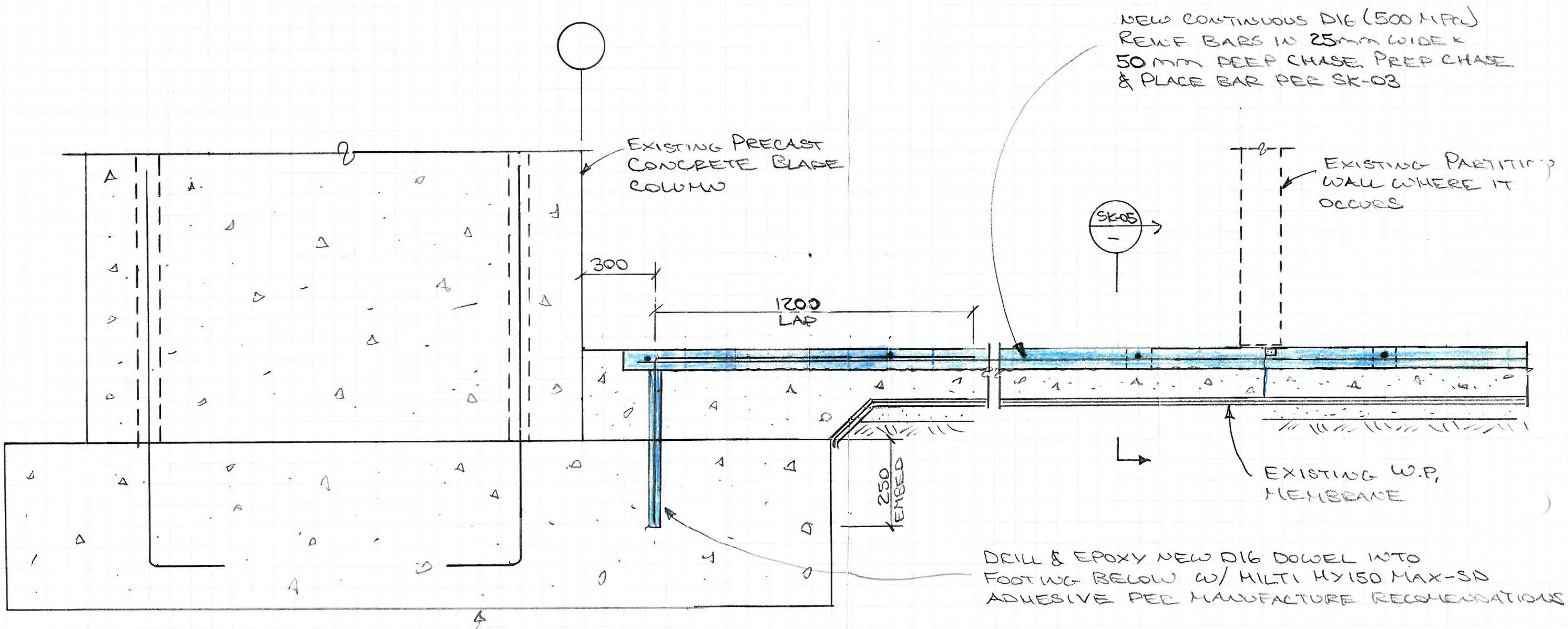
Date: 19/12/2011

Page No:

Sketch No: SK-04

Revision: 1

CALCS/SKETCHES



NOTE: REVIEW OF WATERPROOFING TO BE COMPLETED BY OTHERS

SK-04 SLAB ON GRADE TIE AT PRECAST BLADE COLUMNS

NOT TO SCALE

HolmesConsultingGroup

Project Name: OEU REPAIRS

Project No: 106186.65

Calcs By: ERIC McDONNELL

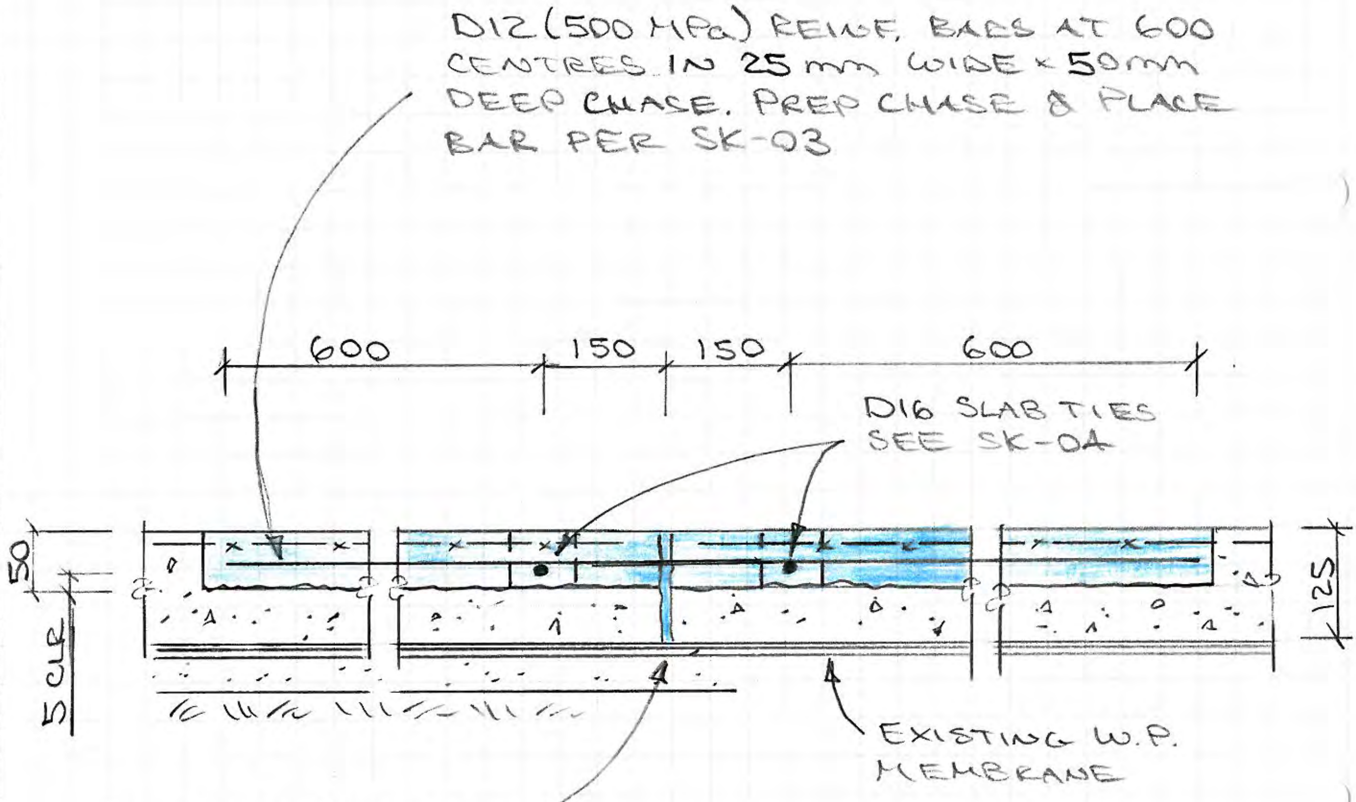
Date: 19/12/2011

Page No:

Sketch No: SK-05

Revision: 1

CALCS/SKETCHES



NOTE! REVIEW OF WATERPROOFING TO BE COMPLETED BY OTHERS.

SK-05 / — STITCH ACROSS SLAB TIE

NOT TO SCALE

CDHB BURWOOD ORTHO REHAB WARD STRENGTHENING

CHRISTCHURCH

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STRUCTURAL AND CIVIL ENGINEERS

Drawing Index

S01-01 floor plan
S01-02 construction details

Rev	Date	Appd	Reason

Consultants

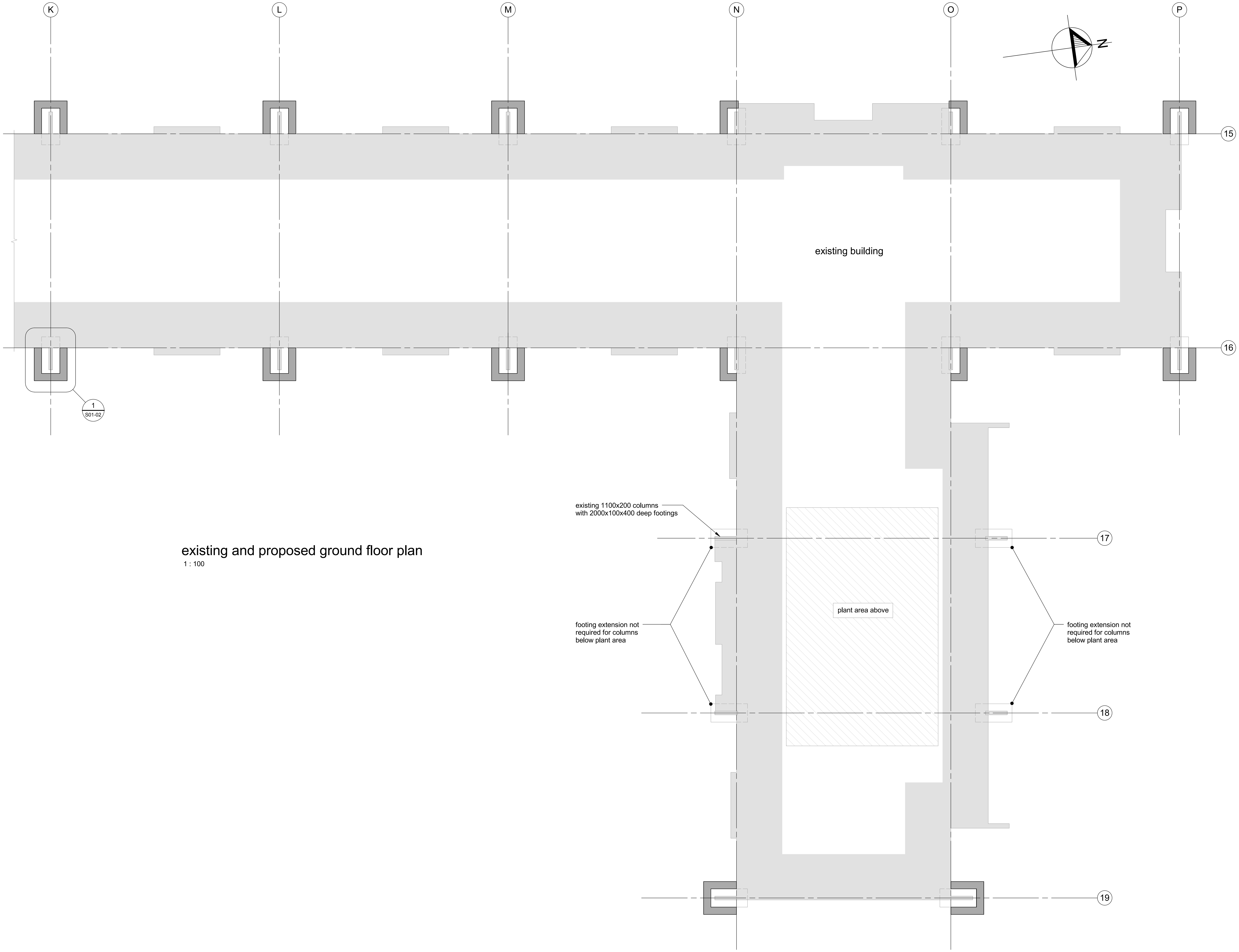
HolmesConsultingGroup
STRUCTURAL AND CIVIL ENGINEERS
Level 2
41 Ballarat Street
PO Box 1266
Queenstown 9348
New Zealand
T +64 3 441 3055
F +64 3 441 3011
www.holmesgroup.com

CDHB BURWOOD
ORTHO REHAB WARD
STRENGTHENING

Drawn Author Scale (at A1)
Filename 106186.65_R14

Sheet Title
drawing index

Job No	Sheet No	Rev
106186.65	S00-01	



existing and proposed ground floor plan

1 : 100

notes:

1. All drawings to be read in conjunction with the HCG Structural Specification.

legend:

column footing extension

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1	05/03/14	PJG	Consent Exemption

Consultants

HolmesConsultingGroup
STRUCTURAL AND CIVIL ENGINEERS
Unit 5
295 Blenheim Road
PO Box 6718
Christchurch 8442
New Zealand
T +64 3 366 3366
F +64 3 379 2169
www.holmesgroup.com

CDHB BURWOOD ORTHO REHAB WARD STRENGTHENING

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Sheet Title

floor plan

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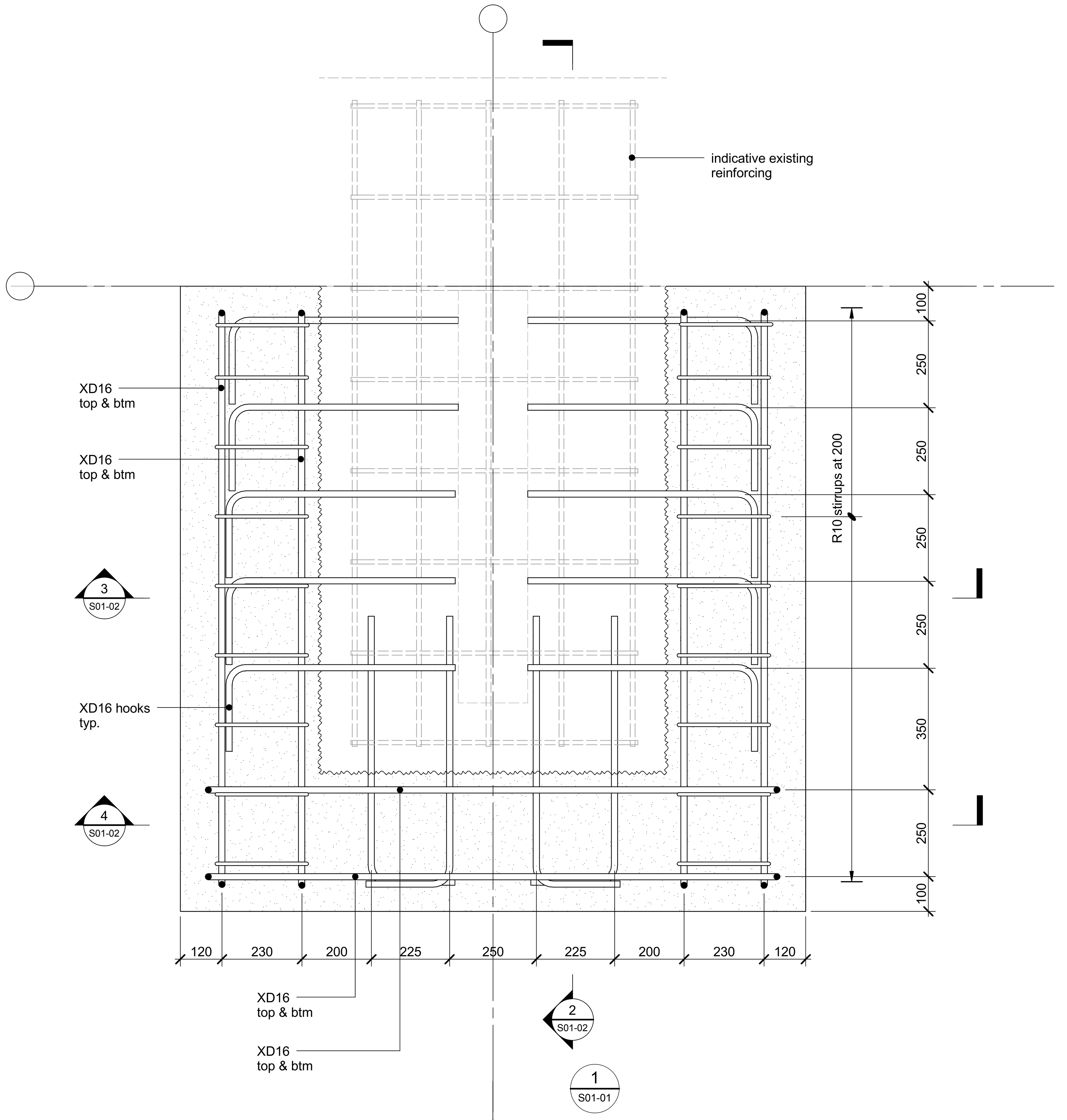
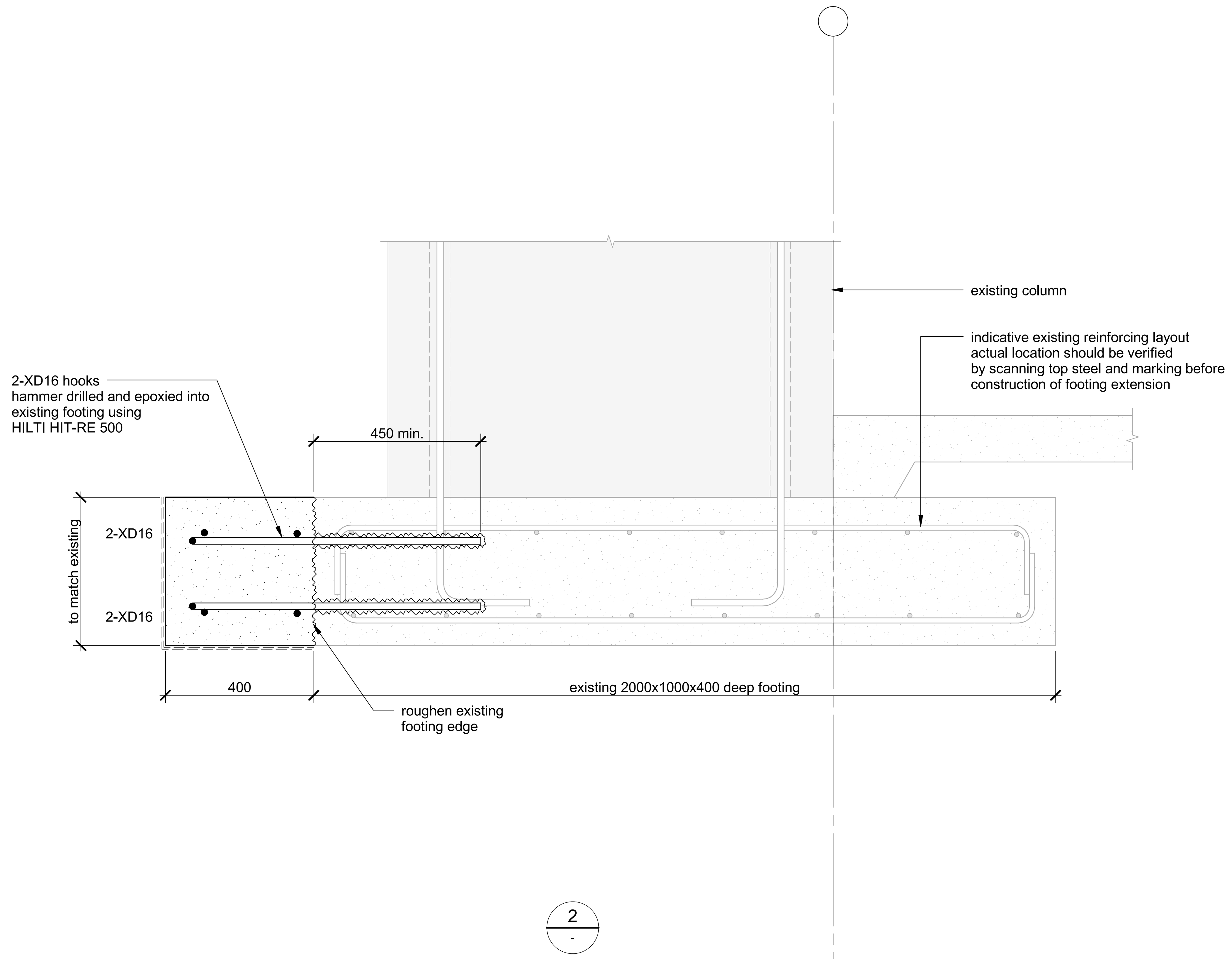
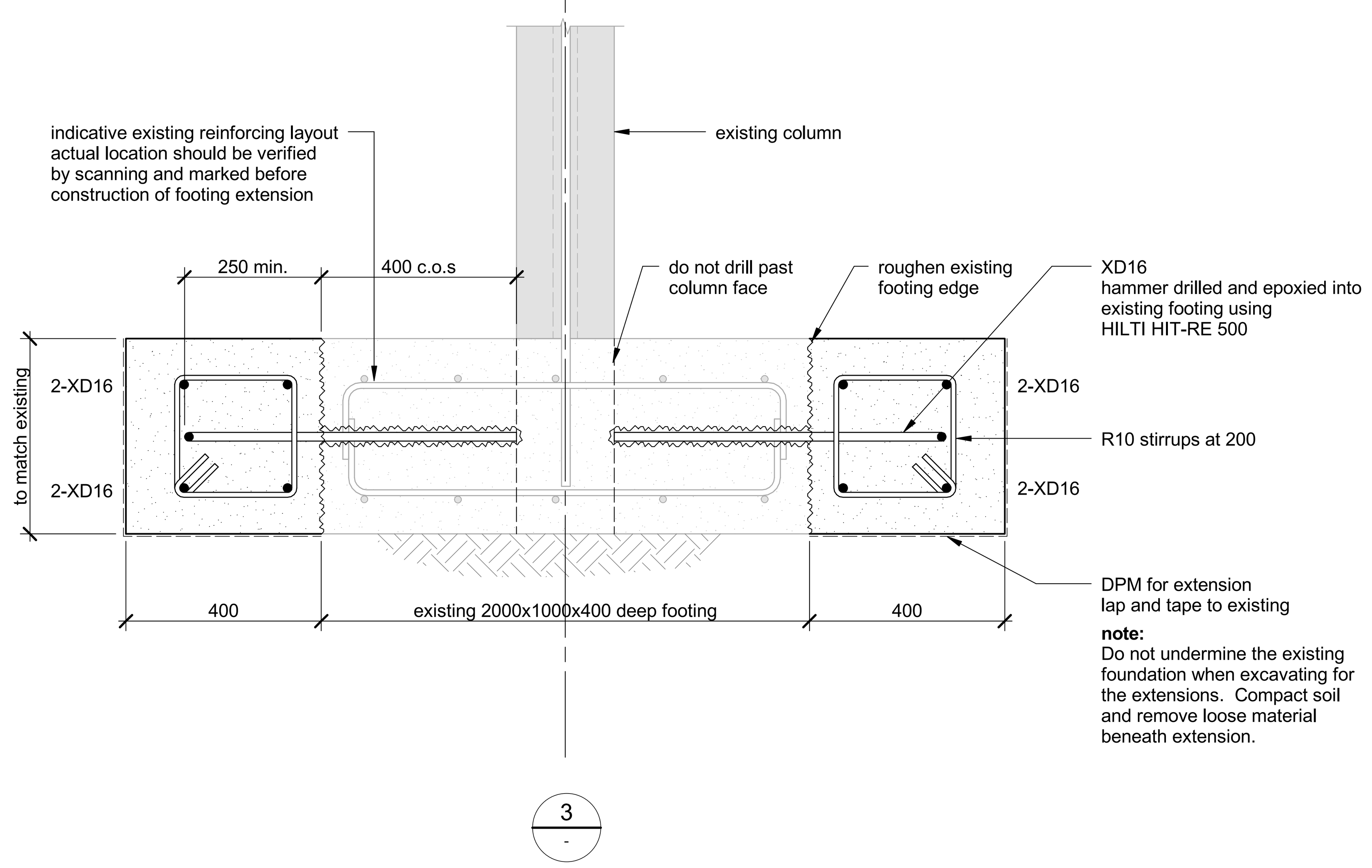
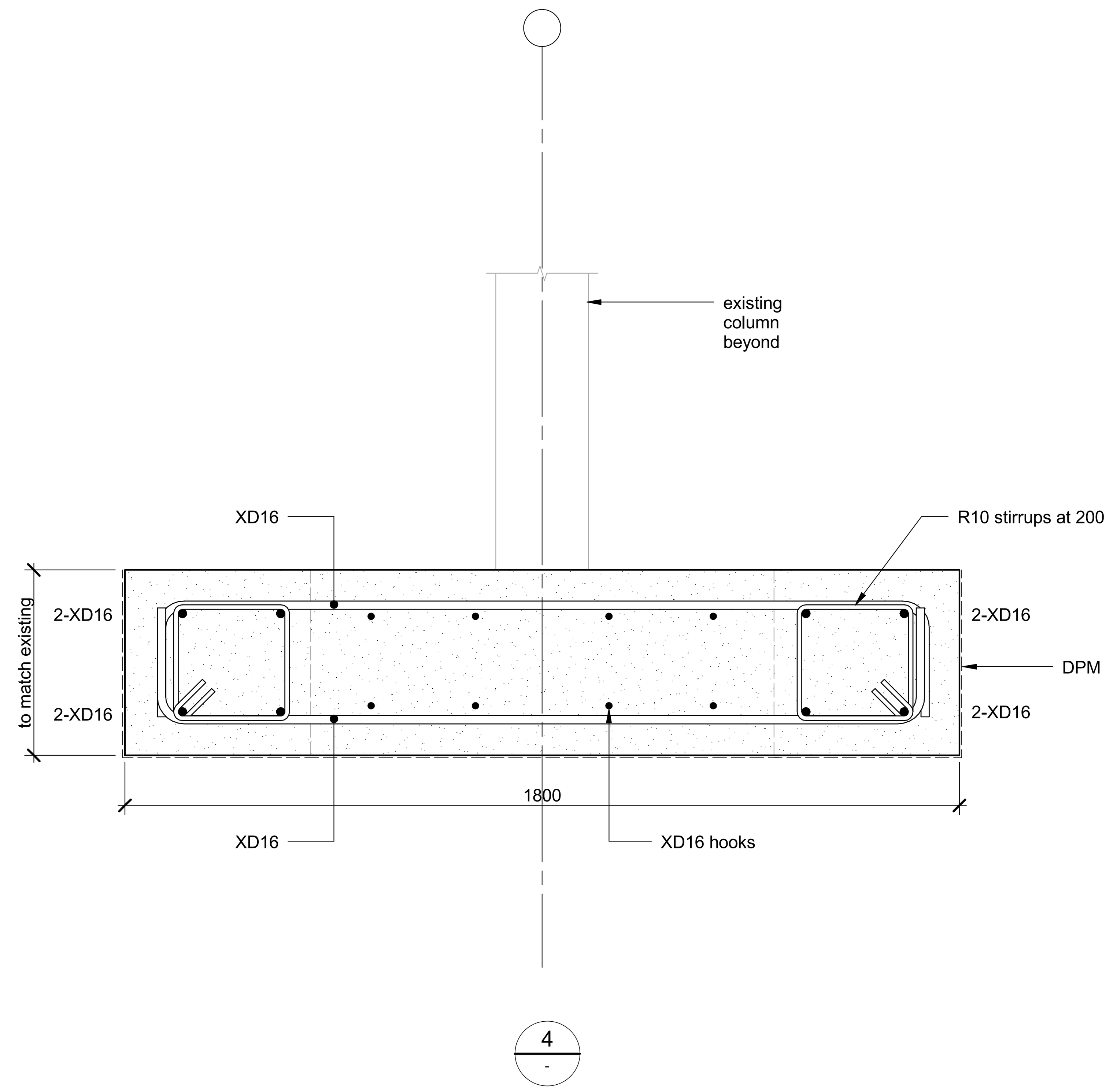
106186.65	S01-01	1









notes:

1. All drawings to be read in conjunction with the HCG Structural Specification.

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Consultants

HolmesConsultingGroup
STRUCTURAL AND CIVIL ENGINEERS
Unit 5
295 Blenheim Road
PO Box 6718
Christchurch 8442
New Zealand
T +64 3 366 3366
F +64 3 379 2169
www.holmesgroup.com

CDHB BURWOOD ORTHO REHAB WARD STRENGTHENING

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Filename 106186.65_R14

Sheet Title

construction details

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DETAILED SEISMIC ASSESSMENT REPORT

STRUCTURAL AND CIVIL ENGINEERS

BURWOOD HOSPITAL CAMPUS

REPORT 11 – PHYSICAL MEDICINE

PREPARED FOR

CANTERBURY DISTRICT HEALTH BOARD

106186.56

INTERIM REPORT REV 4 – 29 OCTOBER 2014

BURWOOD HOSPITAL CAMPUS – DETAILED SEISMIC ASSESSMENT REPORT

REPORT 11 – PHYSICAL MEDICINE

Prepared For:
CANTERBURY DISTRICT HEALTH BOARD

Date: 29 October 2014
Project No: 106186.56
Revision No: 4

Prepared By: Reviewed By:

Ian Matthew
SENIOR PROJECT ENGINEER

Lisa Oliver
PROJECT ENGINEER

Updated By: Reviewed By:

Matthew Franklin
STRUCTURAL ENGINEER

Jenny Fisher
PROJECT DIRECTOR

Holmes Consulting Group LP
Christchurch Office

REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
24/7/12	1	Interim for Review
26/7/12	2	Update to amend inconsistencies in executive summary
30/5/14	3	Update to include additional investigations and updated capacities
29/10/14	4	Updated for the correct capacity of the physiotherapy and occupational therapy wing. Updated to include IL3% of current DBE.

CONTENTS

APPENDICES

TABLES

FIGURES

Holmes Consulting Group has been engaged by Canterbury District Health Board (CDHB) to complete a full structural review of the Burwood Campus following the Lyttelton Earthquake. A series of reports have been compiled as part of this process. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The specific building reports, like this one for the Physical Medicine Block, should be read in conjunction with the base report, and refer to the repair specification.

This report identifies the structural damage sustained by the Physical Medicine Block as a result of the series of earthquakes, including the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The report summarises the effects of the damage on the lateral load capacity of the building, and provides structural repairs for the damage identified. The general form of the building, along with its capacity relative to current code levels, has been included for the buildings pre-earthquake undamaged state and post-earthquake state. Recommendations for improving the seismic performance of the building have also been included.

The Physical Medicine Block was designed in 1981 and constructed in the period there after. The building is a single storey structure with a partial basement and service tunnels below the ground floor slab throughout most of the building. The roof consists of a lightweight standing seem metal roof over timber rafters and timber trusses. There are reinforced concrete frames supporting the roof sections over the Gymnasium, Hydrotherapy Pool and Workshop. A number of the perimeter and interior frames are infilled with concrete block walls. Fully grouted structural reinforced concrete block walls are also located in the Workshop and Plantroom. The ground floor consists of an elevated precast floor system with an insitu concrete topping in the Occupational Therapy, Hydrotherapy Pool, and Physiotherapy areas, while the Gymnasium, Workshop and Plantroom floors are slab on grade construction.

The information available for the review included: the original 1981 architectural and structural drawings by Cutter Pickmere Douglas Architects [3] and Frederick Sheppard and Partners [4], respectively, a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [5], and a level survey of the building completed by Fox & Associates [6].

A considerable amount of minor to moderate damage has been noted across the footprint of building. This includes typical damage at the interface of the infill block walls and the concrete frames, cracking of the block walls, cracking and separation at the joints of the wall ceiling framing, cracking of the wall and ceiling linings and cracking of the cantilevered concrete columns in the Gymnasium.

A level survey completed for the building also indicates that earthquake induced differential ground settlement has occurred at the site resulting in typical permanent slopes in the ground floor around 1:350. Associated damage has been noted in both vertical and horizontal movement at joints in the floor slabs, and cracking to the infill timber and block walls, and block veneer. Options for re-levelling the building have been included in Section 4.1.

It is believed that the majority of the damage observed, including the onset of damage, occurred as a result of the 22nd February event. Further observations of the earthquake damage observed have been included in the body of this report.

Based upon a review of the drawings available, and the site investigations completed, the primary lateral force resisting elements of the building were assessed in their pre-earthquake undamaged state. The assessed capacity of the building is above 33% of the demand required by the current loading code Design Basis Earthquake (DBE), although the gym has been assessed at 35% (27% IL3), limited by column flexure.

For the purposes of this assessment, the Physical Medicine Block has been considered to be an Importance Level 2 building (IL2), IL3 values have been included in brackets.

The reduction in the lateral capacity of the building due to the earthquake damage observed is hard to quantify. The particular elements that have been most affected by earthquake damage include the infill block walls at the perimeter and interior concrete frames, ceiling framing to interior wall and cracking of the columns in the Gymnasium. The differential settlement observed will also have resulted in some reduction in the capacity of the concrete frames and interior block walls.

Permanent repair and strengthening measures required to reinstate the building to its pre-earthquake undamaged condition have been included in Section 4. In addition to the minimum repairs, recommended strengthening concepts to increase the seismic performance of the building and bring the assessed capacity above 67% DBE have been included in section 5.

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. Secondary elements, such as windows and fittings, have not generally been reviewed.

1. INTRODUCTION

Holmes Consulting Group has been engaged by Canterbury District Health Board to complete a full structural review of the Physical Medicine Block at Burwood Hospital following the Lyttelton Earthquake. A series of reports have been completed as part of this. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The individual building reports, like this one, should be read in conjunction with the base report and refer to the repair specification.

The Burwood Hospital base report covers the purpose and scope of the structural review. The current statutory requirements relevant to earthquake damaged buildings are outlined and the level of shaking experienced at the site estimated. Earthquake induced ground settlement damage across the campus is also discussed. The repair specification has been prepared to include repair details for typical damage observed in buildings on the Burwood Hospital and is referred to as required in the specific building reports.

1.1 SCOPE OF WORK

This report is on the Physical Medicine Block located on the Burwood Hospital Campus at 255 Mairehau Road, Burwood, Christchurch. The report identifies the general form of the structure, along with the gravity and lateral load resisting systems. Each component of the structural system was reviewed based upon the information available and any potential Critical Structural Weaknesses (CSW's) were noted.

The report also identifies the structural damage observed to date as a result of the series of earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake, in particular, subjected the building to strong ground motions which significantly exceeded the current code loading demand for buildings of this nature.

The capacity of the Physical Medicine Block has been assessed relative to current code loading in the buildings pre-earthquake undamaged state and in its post-earthquake damaged state. The post-earthquake assessment summarises the effects of the damage identified on both the gravity and lateral load resisting elements. Repair options to restore the buildings capacity to pre-earthquake levels for strength, durability and stiffness have been included. The repair options aim to maintain the buildings utility. Where required, strengthening options have also been provided.

1.2 LIMITATIONS

Findings presented as a part of this project are for the sole use of the Canterbury District Health Board, 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. Because all of the structure has not been available for detailed inspection or evaluation, this report is limited to those elements available and engineering judgement as to the likely condition of unseen elements. 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. PRE-EARTHQUAKE BUILDING CONDITION

This section discusses the form and capacity of the building prior to the Darfield Earthquake.

2.1 BUILDING FORM

The Physical Medicine Block is located at the Canterbury District Health Board's Burwood Hospital Campus, approximately 7 km north-east of Christchurch City. The building was originally designed in 1981 and constructed in the period there after. The eastern elevation of the Physical Medicine Building is shown in Figure 2-1.

Figure 2-1: East Elevation of Physical Medicine Block

The Physical Medicine Block comprises of five primary sections, the Occupational Therapy and Physiotherapy areas, the Gymnasium, the Hydrotherapy Pool, the Workshop, and the Plantroom. The five areas are indicated in the floor plan in Figure 2-2. The Occupational Therapy and Physiotherapy sections are primarily gypsum board and plywood braced single storey timber frame structures on concrete block wall strip foundations. The Gymnasium and Hydrotherapy Pool structures are supported on cantilevered concrete columns with concrete block and timber/plywood and concrete block infill walls. The Plant Room is housed in a concrete block walled structure.

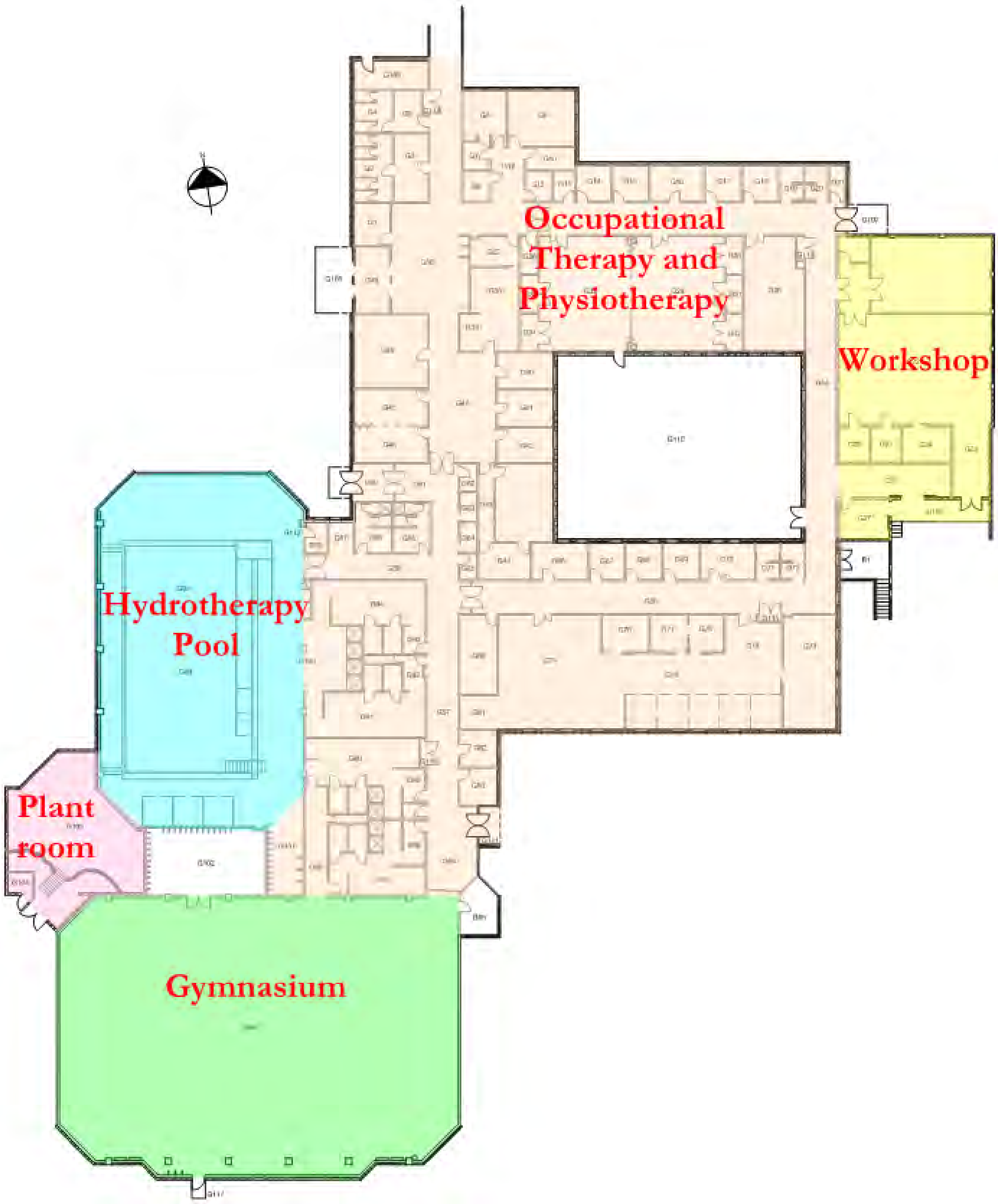


Figure 2-2: Floor Plan of Physical Medicine Block indicating different building areas

The information available for the review included: the original 1981 architectural and structural drawings by Cutter Pickmere Douglas Architects [3] and Frederick Sheppard and Partners [4] respectively, a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [5], and a level survey of the building completed by Fox & Associates [6].

Typical Roof Framing - The roof throughout is constructed of a lightweight standing seam metal roof over a 12mm plywood diaphragm. The plywood is typically supported by 150 x 50mm timber purlins at 600mm centres which are in turn supported by timber trusses. Suspended ceilings (timber framed) are present beneath the trusses throughout most of the building.

Occupational Therapy and Physiotherapy sections – The method of construction in these areas is typically single storey light timber framing. The office and treatment rooms are arranged in a U shape around a central courtyard (with the workshop enclosing the fourth side of the courtyard). Each wing is approximately 16m wide and 22 m long (see Figure 2-2). The building is clad in partially filled reinforced masonry concrete block veneer.

The floor is predominantly constructed of precast, prestressed concrete "Unispan" units with 75 mm of concrete topping and a cement plaster screed of varying thickness. The suspended floor is supported by continuous concrete sub-floor walls (these can be seen in Figure 2-3). The remainder of the floor is a reinforced concrete slab on grade.

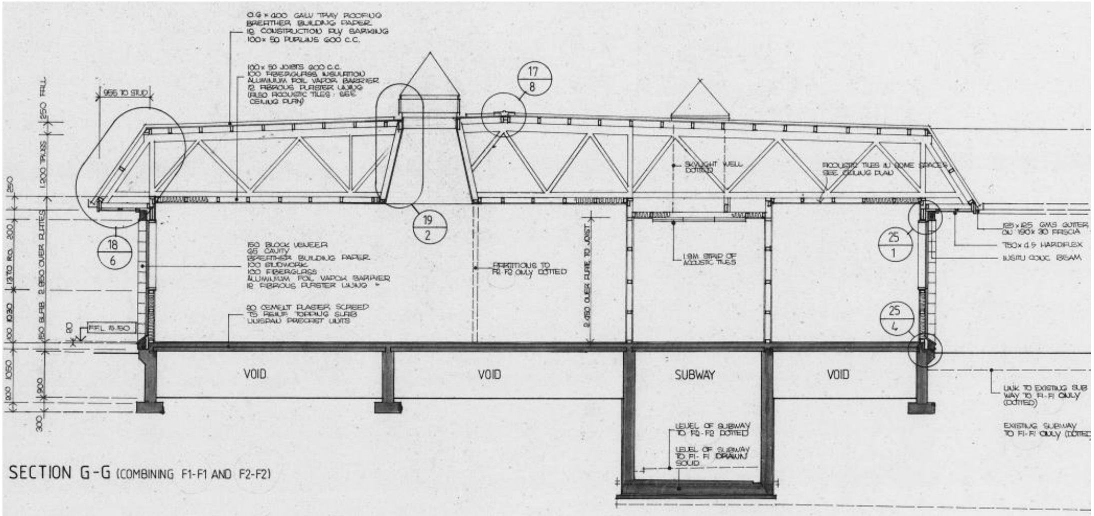


Figure 2-3: Occupational Therapy and Physiotherapy – Section G-G

Hydrotherapy Pool – The Hydrotherapy Pool roof is supported by long span timber trusses bearing on perimeter concrete cantilever columns. The Hydrology Pool room is 28m by 16m with a combination of plywood and concrete block infill walls. The floor surrounding the pool is constructed of precast, prestressed concrete "Unispan" units with 75 mm of concrete topping and a cement plaster screed of varying thickness. Sections across and along the building are shown in Figure 2-4 and Figure 2-5 respectively.

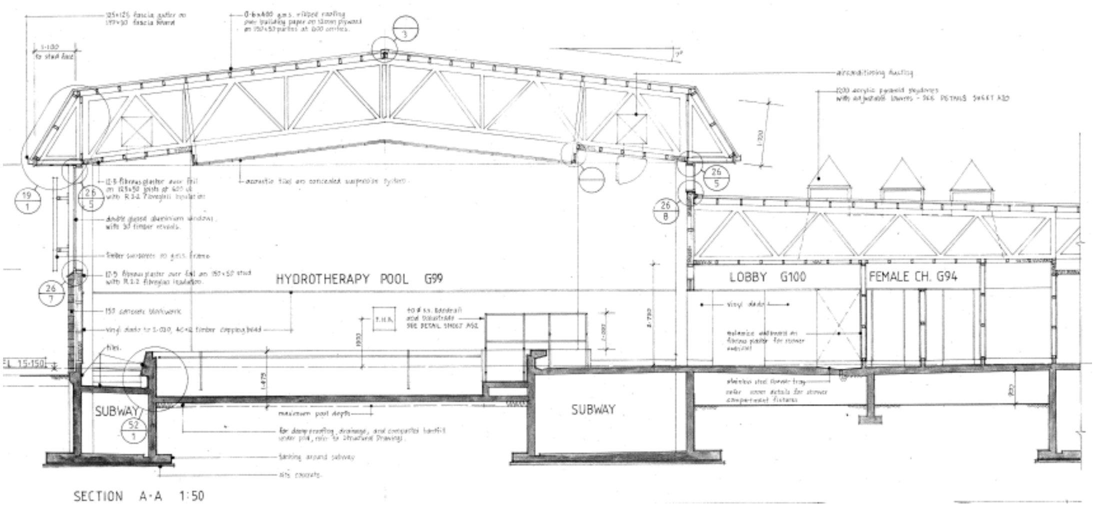


Figure 2-4: Hydrotherapy – Section A-A

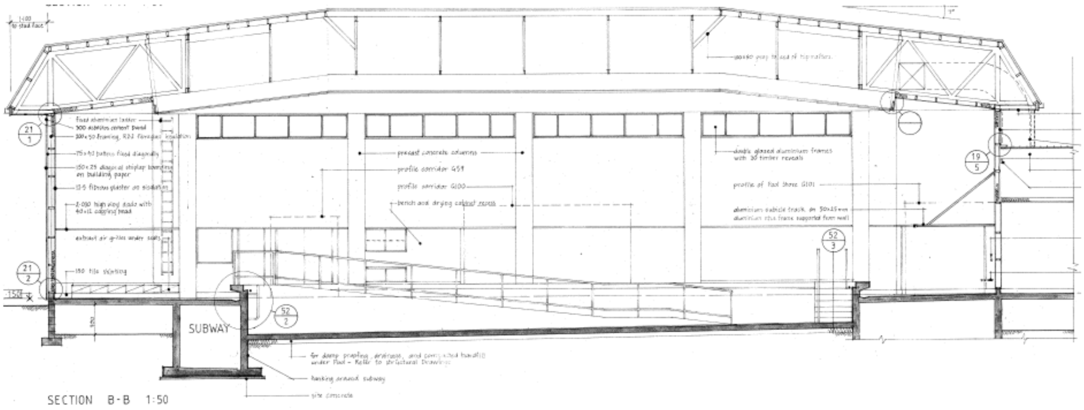


Figure 2-5: Hydrotherapy – Section B-B

Gymnasium - The roof over the Gymnasium is supported by long span timber trusses bearing on concrete cantilever columns. The room is approximately 32m by 22m (see Figure 2-6).

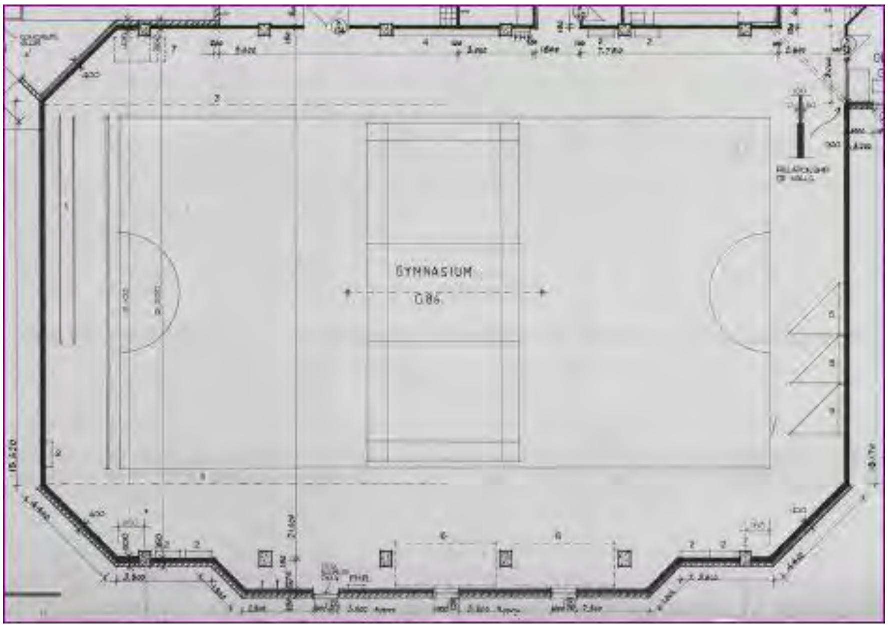


Figure 2-6: Gymnasium – Floor Plan

The walls are a combination of timber frame with plywood and partially filled concrete block infill walls. The floor of the Gymnasium is concrete slab on grade construction. Sections through the Gymnasium are shown in Figure 2-7 and Figure 2-8.

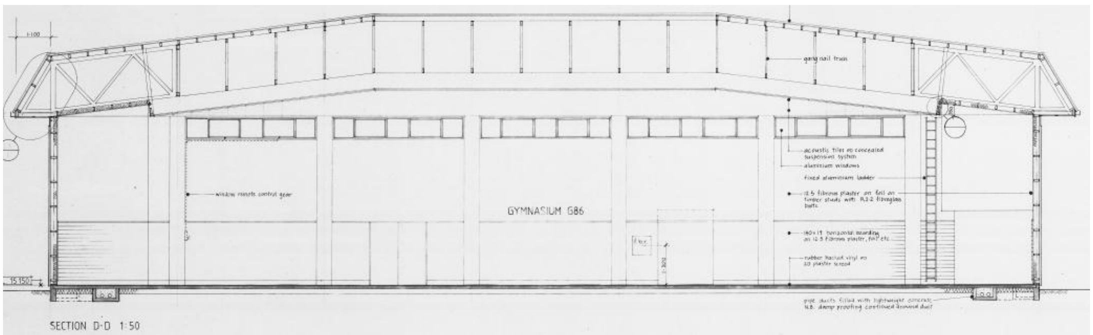


Figure 2-7: Gymnasium – Long Section D-D

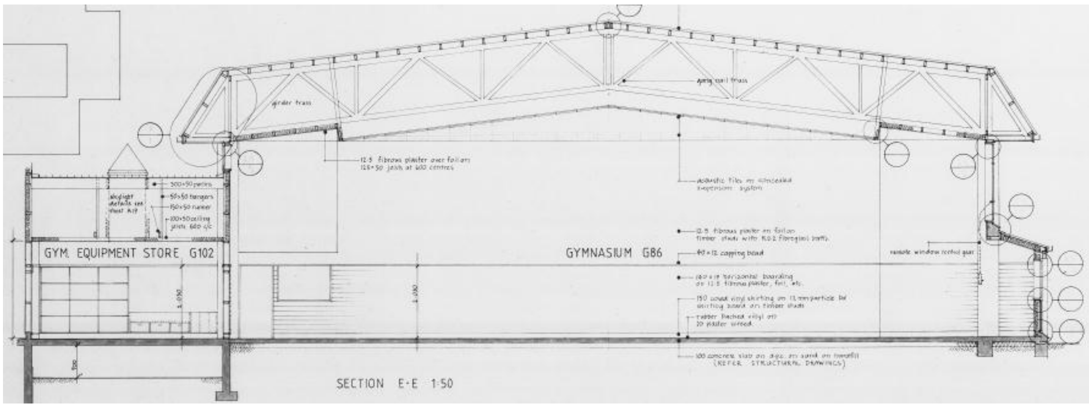


Figure 2-8: Gymnasium –Section E-E

Workshop and Plantroom– These two areas have fully grouted reinforced concrete block walls. Concrete columns are interspersed with the block walls in the Workshop. The Workshop is approximately 20m by 12m, while the Plantroom is a complex shape of approximately 20m x 15m (see Figure 2-4). The ground floor of the Plantroom is a steel metal deck, below this it has a full basement level.

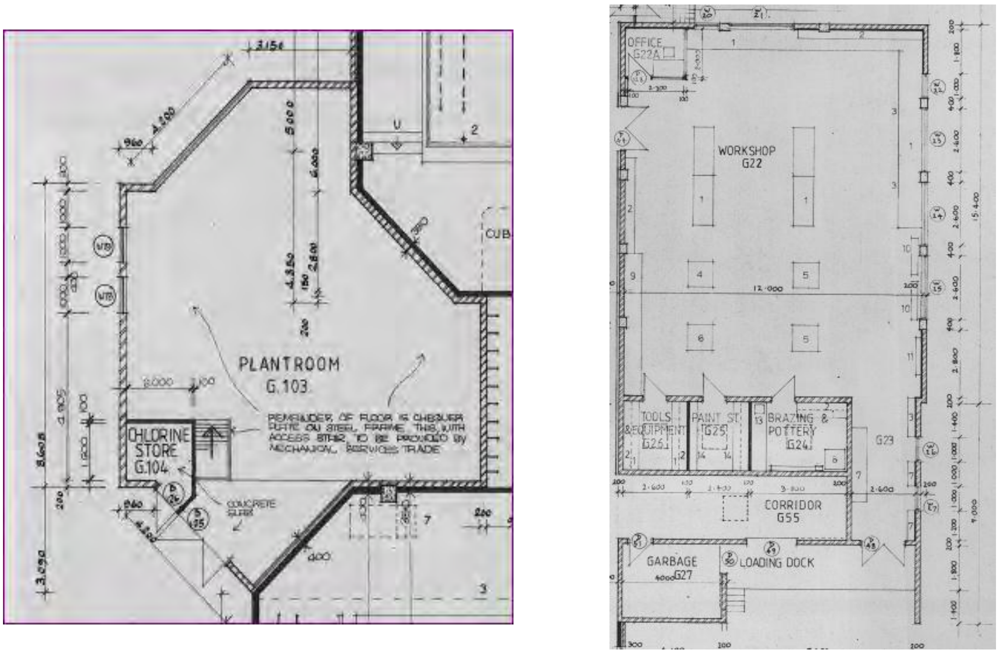


Figure 2-9: Plantroom and Workshop – Floor Plans

2.2 LATERAL LOAD RESISTING SYSTEMS

This section describes the primary lateral load resisting system of the building sections described in Section 2.1.

Occupational Therapy and Physiotherapy sections – Lateral loads are transferred from the plywood roof diaphragm to the bracing walls by diagonal cross bracing and extensions of the plywood shear wall through the roof space. Lateral loads from bracing walls are transferred to the continuous concrete sub-floor walls and footings below via the concrete floor diaphragm.

Gymnasium - The primary lateral load resisting system of the Gymnasium is by cantilever concrete columns across the building and concrete frame action along the building. The loads are transferred to the columns by the plywood roof diaphragm. Lateral loads from the concrete frames are transferred directly to the continuous concrete sub-floor walls and footings below.

Hydrotherapy pool - The primary lateral load resisting system of the Hydrotherapy Pool is by cantilever concrete columns in both directions. The loads are transferred to the columns by the plywood roof diaphragm. Lateral loads from the concrete columns are transferred directly to the continuous concrete sub-floor walls and footings below. The ground floor diaphragm of the Hydrotherapy Pool area provides restraint to the cantilevered concrete columns and distributes the lateral loads to the exterior concrete sub-floor walls.

Workshop and Plantroom – Fully grouted concrete masonry walls provide the lateral load resisting system. The load is transferred to the walls by a combination of direct fixing, the plywood roof diaphragm and diagonal cross bracing to the roof eves.

2.3 PRE-EARTHQUAKE BUILDING CAPACITY – DIRECT CODE COMPARISON

The building capacity under earthquake actions discussed in this section is compared to the capacity that a similar building would be designed to today. A new building would be designed to the *Structural Design Actions Standard, Part 5: Earthquake Actions – New Zealand*, NZS 1170.5:2004[7] and incorporating the amendments made to this standard as a result of the Lyttelton Earthquake as outlined in the Amendment 10 of the Building Code [8]. The implications of the recent amendments are discussed more in-depth in the Burwood Hospital Campus Base Report however, for a building of this type the amendments essentially result in an increase to the design loads of 36 % when compared to pre-earthquake design levels.

When the building was originally designed in 1981, the loading standard at the time was the, *Code of Practice for General Structural Design and Design Loadings for Buildings*, NZS4203:, Standards New Zealand, 1976 [9]. When this standard was written, neither the seismology of the different areas within New Zealand, nor the impact this could have on buildings was as well understood as it is today. Along with an increase in the seismic demands required by the change in the loading code over this period, the seismic detailing requirements have also progressed significantly resulting in more ductile and better performing buildings.

The current code requires a new building to be designed for an earthquake known as the Design Basis Earthquake (DBE), which is based upon the buildings physical location, local soil conditions, building type, fundamental period and importance level.

The original structural calculations for the Physical Medicine Block were not available; therefore the exact design and loading assumptions originally made are unknown. For the purposes of this report seismic loading assumptions have been made based on a detailed review of the drawings available and physical observations of the building.

The Physical Medicine Block is classified as an Importance Level 2 building in accordance with NZS 1170:2004 [9]. The associated return period of the DBE is 500 years, with a risk factor for design of R = 1.0. The sub soil for the site is taken as Soil Type D, which is consistent with the findings of a post-earthquake geotechnical investigation [5]. Based upon the period of construction, and the detailing of the reinforced concrete frames and block walls, they have been considered to be nominal ductility, and as such assigned a ductility factor of μ=1.25.

A comparison between the Design Basis Earthquake of NZS 4203:1976 and NZS 1170:2004 for the site is plotted below in Figure 2-9. Based upon a fundamental building period below 0.50 seconds, the seismic demands required by the loading code for an elastically responding structure have increased by approximately 20% since 1976. This means that if the building were design to 100% of the DBE at the time of construction, the building may currently have a capacity to resist approximately 80% of the demands imposed by the current code level DBE.

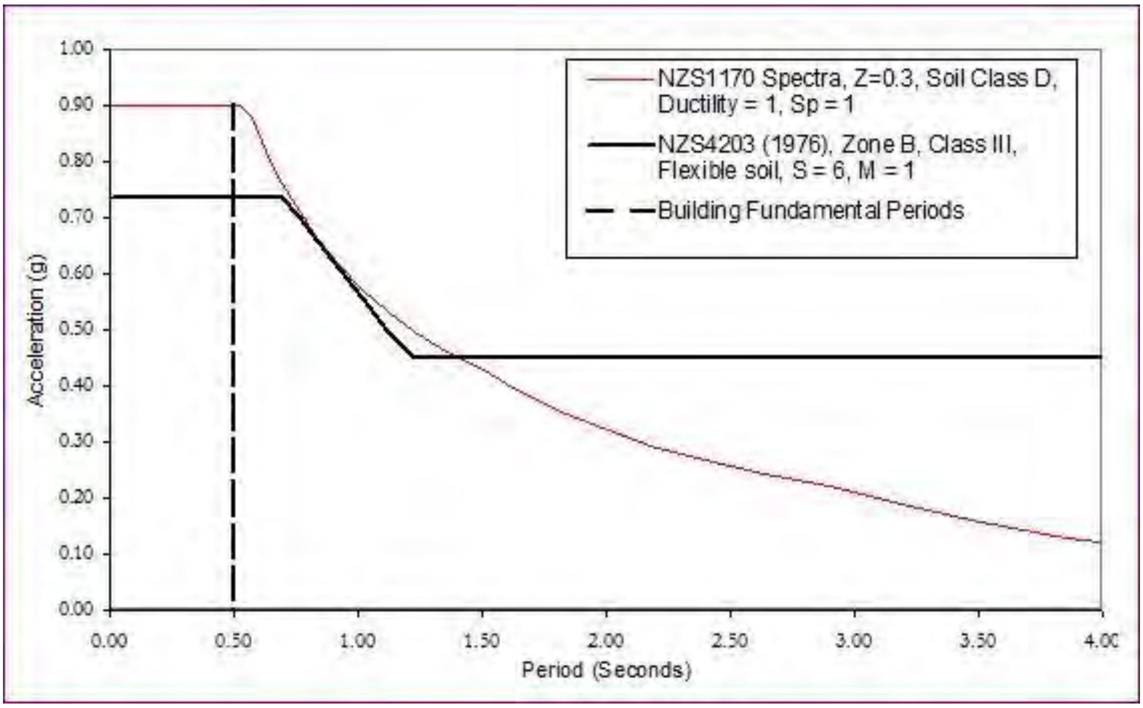


Figure 2-10: Comparison of Design Codes

2.4 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 estimated capacity when compared to current loading standards. The equivalent static analysis was carried out based upon the original structural drawings and incorporation of on-site measurements and as built observations.

The foundations have been assessed at an ultimate bearing capacity of 150kPa, for factored loads, as per recommendations provided by Tonkin and Taylor.

Following the Lyttelton earthquake, a geotechnical report was conducted by Tonkin & Taylor, titled "Burwood Hospital Post Earthquake Geotechnical Assessment", dated June 2011 [5]. This report has been used to aid in the evaluation of the site conditions and the likely effect of the ground on the buildings past and future performance. The soil parameters described in the geotechnical report completed by Tonkin and Taylor has been used for the evaluation of the buildings existing foundation system.

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 [10]. 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 DBE 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 (CSW). 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. These are described in more detail in the Burwood Hospital Base report [1] and include strength governed elements such as short columns and deflection governed elements such as floor and stair elements with inadequate support seating.

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 the strength and deflection requirements.

A summary of the %DBE calculated for each primary element has been noted in Table 2-1 and Table 2-2.

Building Element	%DBE (IL2)	%DBE (IL3)	Comments
Occupational Therapy and Physiotherapy	50%	40%	Limited by ply wall fixings
Gymnasium	35%	27%	Limited by soil bearing capacity
Hydrotherapy Pool	70%	55%	Limited by column flexure
Workshop	67%	52%	Limited by fixings in ply bracing above block walls
Plantroom	100%	100%	

Table 2-1: Seismic Assessment %DBE in North-South Direction

Building Element	%DBE (IL2)	%DBE (IL3)	Comments
Occupational Therapy and Physiotherapy	100%	100%	
Gymnasium	67%	52%	Limited by foundation beam flexure
Hydrotherapy Pool	70%	55%	Limited by column flexure
Workshop	80%	60%	Limited by fixings in ply bracing above block walls
Plantroom	100%	100%	

Table 2-2: Seismic Assessment % DBE in East-West Direction

Note that unless otherwise stated, all capacities are given as % of IL2.

Occupational Therapy and Physiotherapy sections – Fixings detailed in the original drawings are not consistent with current code. As such, assessment of the lateral load resisting capacity of these buildings has been based on specific analysis of plywood and diagonal brace fixing capacities and a reduced bracing capacity of 1.5kN/m/sheet of gypsum board. The pre-earthquake capacity of these walls, in both directions, has been assessed as being approximately 50% of current DBE loading (40% IL3).

Gymnasium – In the E-W direction flexural failure of the foundation beams has been assessed to occur at approximately 35% of current DBE loading (27% IL3). Apart from this, the building capacity typically exceeds 67% of current DBE loading.

These figures are lower than the 80% figure from direct code comparison. For both directions the assessment of design actions has been based on a ductility of 1.25, in line with SESOC guidelines. This is likely to be lower than what was assumed in the original design. Additionally, the original designer may have been able to use a higher bearing capacity for the stabiliser design, based on geotechnical investigations at the time. In the E-W direction the capacity of several bracing systems may have been combined to meet load capacity requirements. Several elements are present in the gymnasium that will provide some lateral load resisting capacity, but due to a lack of deformation compatibility these cannot work in unison and thus the building capacity is limited to that of the stronger individual element. The combination of the above explains the difference between the assessed capacities and that estimated by the direct code comparison.

Hydrotherapy pool – The moment capacity of the cantilever concrete columns in the Hydrotherapy Pool achieved approximately 70% of current DBE loading in both directions (55% IL3).

Workshop – The workshop was assessed to perform well against the current DBE loadings, limited by the plywood fixings. The fixings are not in accordance with current practise and are limiting the building to approximately 80% of current DBE loading in both directions (60% IL3).

Plantroom – The near continuous nature of the perimeter block wall meant that the plantroom lateral load capacity was estimated to exceed the current code DBE loading requirements. The block walls were also assessed to have an out of plane capacity in excess of current code DBE loading requirements.

If the capacity of a primary gravity or lateral load resisting element in a structure falls below 33% DBE it is considered to be "Earthquake Prone" in terms of section 122 of the Building Act. Current Christchurch City Council policy states that buildings identified as "Earthquake Prone" may be required to be strengthened to 67% of current code requirements when seeking consent for repairs.

All of the sections of the Physical Medicine Block have been assessed as having capacities above 33% DBE, though the gym has been assessed at below 67% DBE. Methodology to improve the seismic performance of the building, and bring the assessed capacity of the building above 67% DBE has been included in Section 5.

In some areas of the Occupational Therapy and Physiotherapy areas of the building, heavy ceiling tiles have been used. The ceiling grid is fixed to timber ceiling framing rather than being suspended from the roof (which has a plywood diaphragm) and therefore is likely to have adequate performance in a moderate earthquake and it is not expected that tiles would fall out of the grid in such an event.

3. POST-EARTHQUAKE BUILDING CONDITION

This section covers the structural damage sustained by the Physical Medicine Block at Burwood Hospital Campus as a result of the series of earthquakes that includes the Darfield Earthquake that struck at 4:36am on 4th September, 2010 and the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011, the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake subjected the building to strong ground motions which likely exceeded the full design earthquake load for buildings of this nature and appears to have caused the bulk of the earthquake damage observed after the initial Darfield event.

3.1 THE LYTTELTON EARTHQUAKE

The fundamental period of the building is estimated to be less than <0.5 seconds. 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 strong motion data presented in the Base Report, it appears the Lyttelton Earthquake produced ground shaking intensities of approximately 60-120% of the current Ultimate Limit State design spectra for a new Importance Level 2 building.

It should be noted that the Lyttelton Earthquake was very short in terms of strong shaking produced with the strong motion only lasting for duration of 5-7 seconds. Rupture of the Alpine Fault is expected to contain 50 to 60 seconds of strong motion.

3.2 PRELIMINARY INVESTIGATIONS

Preliminary investigations were carried out to identify 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
- review of available structural drawings
- damage observed during 'Rapid' Structural Assessments following the Lyttelton Earthquake, June 13th aftershocks and subsequent events

The following areas were identified for potential damage:

- movement or damage to structure associated with ground movement and/or settlement
- cracking and displacement of concrete sub-floor walls, service tunnels and foundations

- distress and cracking of concrete frames
- cracking to perimeter concrete block infill walls
- distress and cracking of internal concrete block walls
- distress to ceiling roof diaphragm
- distress at the roof truss to concrete frame connections

Rapid visual assessments were carried out on the 24th February 2011[11] following the 22nd February earthquake, and on the 14th [12] and 16th June 2011 [13] following the June 13th earthquakes. Two additional Rapid Visual Structural Assessments were conducted on the 4th January 2012 [14] and 31st January 2012 [15], following the 2nd January 2012 event. These structural observations involved a complete walk around the exterior and throughout the interior of the building. The following primary areas of damage were identified from the damage assessments:

- separation between the ceiling framing and block walls
- cracking and distress to timber partition walls
- cracking/separation between infill block walls and the surrounding internal concrete frames
- damage to internal wall linings
- stepped cracking in mortar joints of the perimeter block walls
- signs of ground movement around the loading dock
- significant damage to the main entry raised ceiling and reduced seating of heavy ceiling tiles
- cracking of mortar joints to the reinforced concrete block walls
- hairline cracking of perimeter concrete frames
- localised damage to the ceiling framing at the connection to the perimeter concrete frames

As a result of the damage observed after the 31st January 2012 [15] temporary securing of the ceiling tiles in the raised entry ceiling was instructed.

3.3 DETAILED STRUCTURAL OBSERVATIONS

Further detailed inspections have been carried out following the initial assessments to gain a better understanding of the damage sustained. The majority of the detailed structural observations were completed on the 14th March 2012.

A full record of these observations can be found in Appendix A, with reference plans describing the location labelling used found in Appendix B. A full photographic record of the observations is available electronically on request. The detailed structural observation identified the following additional damage to those items noted in the initial rapid assessments:

- Settlement of Gym slab on grade relative to the adjacent suspended floor slab to the main corridor. Vertical and horizontal movement recorded

- Horizontal hairline cracking to the Gym concrete columns

3.4 GEOTECHNICAL REVIEW

A review of the ground damage and conditions was carried out by Tonkin & Taylor for the Burwood Hospital Campus. A subsequent report titled *Burwood Hospital Post Earthquake Geotechnical Assessment* was issued in June 2011 [5]. The geotechnical review did not specifically cover the Pysical Medicine Block; however it did cover several adjacent buildings which are founded on similar material. The review concluded that the settlement of the foundations was likely due to liquefaction of the underlying soil layers. It is believed that excessive pore water pressures have been dissipated and that further settlement is not expected to occur, unless another significant event was to occur.

Based on the geotechnical report provided by Tonkin & Taylor [16] the potential for future total and differential settlements at the building site varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

3.5 LEVEL SURVEY & VERTICALITY STUDY

A detailed survey of the ground floor levels in the Physical Medicine Block was conducted by Fox & Associates and issued on 24th November 2011 [6]. The purpose of the level survey was to measure the slopes in the ground floor as a result of earthquake induced differential ground settlement. The results of the survey are included in Appendix C and are shown in Figure 3-1. Areas of dark red are the highest, with areas of dark blue the lowest.

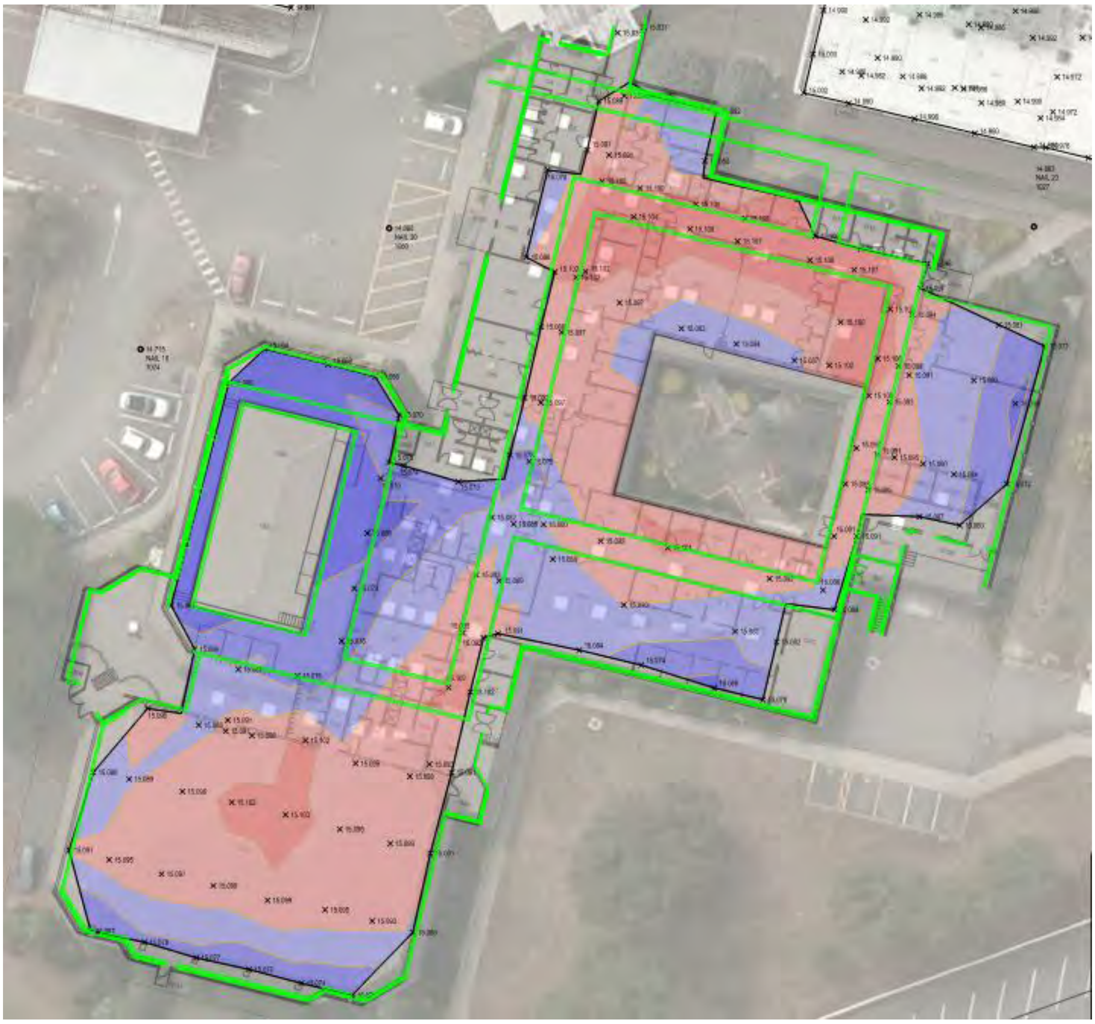


Figure 3-1: Level Survey measurements with sub-floor service tunnel locations super-imposed in green

The following is a summary of the differential settlements and resulting slopes in the ground floor for the main sections of the building:

Occupational Therapy and Physiotherapy - The level survey of the Occupational Therapy and Physiotherapy areas have indicated a high ridge along the line of the service tunnel beneath the northern wing in the middle of the building. Lower levels have been noted along the southern perimeter of the building (the survey was not able to capture the west and parts of the northern perimeter walls). In total an elevation change of 35mm has been noted over approximately a 13m length of the floor slab in the southern wing, resulting in a worst case slope of 0.29% or 1:370.

It is believed that as a result of being founded in deeper and stiffer material, the partial basement and service tunnels have settled less than the remaining foundation elements, particularly the continuous concrete walls and shallow footings under the perimeter concrete frames.

Gymnasium – The level survey result in the Gymnasium indicates a high point near the centre of the gymnasium, with the ground floor sloping downward toward the exterior of the building. It is believed that the perimeter footings have likely settled more than the interior piles due to a higher concentration of gravity loading on the exterior concrete frames and infill walls. The level survey has indicated differences in floor levels up to 41mm over a length of approximately 14m, resulting in a worst case slope in the floor of 0.29% or 1:340.

Hydrotherapy Pool –The level survey was not able to record levels within the pool. The floor around the pool exhibited little differential movement other than at the Southern end of the building. At this location differences in floor levels up to 23mm over a length of approximately 4.5m were recorded, resulting in a worst case slope in the ground floor of approximately 0.5% or 1:200; however, this could be due to built in slopes to stop water pooling. It is also possible that the floor in this room has settled more than the rest of the building due to a higher concentration of gravity loading from the pool.

Workshop - The level survey of the workshop indicated a cross fall from west to east. In total an elevation change of 31mm has been noted over approximately a 10m length of the floor slab in the southern wing, resulting in worst case resulting slope of 0.31% or 1:320.

Plantroom – No levels were recorded in the plantroom.

3.6 SUMMARY OF BUILDING DAMAGE

The following is a summary of the observations made for the building, and our conclusions as to their condition and seismic load resisting capacity. These observations do not specifically distinguish between damage caused by the Darfield Earthquake, the Lyttelton Earthquake or any significant aftershocks, such as those that occurred on the 13th June 2011, the 23rd December 2011 or the 2nd January 2012. However the majority of the observed damage was a result of the Lyttelton Earthquake. Some of the damage has become worse after the subsequent aftershocks, but no significant new damage has been noted.

A summary of the building damage observed can be typified as follows:

- **Differential Ground Settlement –** Differential ground settlements, resulting in slopes in the ground floor of approximately 1:320 in the Gymnasium, Occupational Therapy and Physiotherapy areas. Associated damage has been noted in both vertical and horizontal movement at joints in the floor slabs, and cracking to the infill timber and concrete block walls, and block veneer. In particular, significant step cracking has occurred in the concrete block wall below the sprinkler stop valve (this is located at the north-west corner of the building.

- **Damage to Perimeter Concrete Frames and Infill Walls –** Minor cracking is evident in several of the perimeter concrete frame columns. It is possible that these cracks were larger during shaking, and have since closed to some degree. More severe cracking has been observed at the interface of the unreinforced block infill walls with the reinforced concrete frame columns. The cracks at the infill walls to column interface are believed to be as a result of seismic induced face loading. Stepped cracking in the mortar joints of the infill walls has also been observed.

- **Damage to Interior Concrete Frames and Infill Walls –** At the four internal concrete frame lines, separation of the unreinforced infill block walls has been noted at the interface with the concrete frame columns. This is believed to be as a result of out-of-plane seismic loading on the walls.

- **Damage to Block Walls –** Cracking has been sustained to the majority of the reinforced concrete block walls of the Workshop and Plantroom. The damage appears to be a result of in-plane loading, and possibly differential settlement.

- **Distress to Wall and Ceiling Linings –** The internal wall linings on the timber framed walls within the Occupational therapy and Physiotherapy areas have suffered cracking and general distress In the Gymnasium and Hydrotherapy Pool areas some of the walls have separated from adjacent block walls and concrete frames.

- **Damage to floor and swimming pool tiles in Physiotherapy Pool area –** In several areas around the pool floor tiles appear to have delaminated from the floor substrate. This appears to be cosmetic damage only; however, when tiles are removed the concrete substrate will be reviewed to ensure there is no structural damage. Some tiles within the pool itself have also cracked.

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. Secondary elements, such as windows and fittings, have not generally been reviewed.

3.7 FURTHER INVESTIGATIONS REQUIRED

The Pre-earthquake (undamaged state) and Post-earthquake (damaged state) Structural Assessments have been made based on the original structural drawing and site observation of damages sustained. Based on non-invasive site observations to date the building appears to have been constructed generally in accordance with the 1891 structural drawings [4].

3.7.1 Investigations Required for Damage Assessment

- Removal of floor coverings. These may conceal earthquake induced cracking damage to the concrete floor. A small sample of floor coverings should be removed to determine if damage has occurred, especially in areas where differential settlements have been measured.

  Additional investigations revealed no significant earthquake induced cracks. Minor cracks noted are believed to have been pre-existing and generally over precast plank joints.

3.7.2 Investigation Required During Repairs

The following investigations are required during repairs:

- The fixings of the timber framing in the bracing walls will be required to be checked for damage and ability to transfer new bracing loads.

- Checking the steel strap bracing within gypsum board lined walls

- Checking the plywood wall fixings.

- Check the pool floor and walls during removal of cracked tiles.

  No significant cracking has been noted to pool walls and floor. Tiles have been replaced.

3.8 POST-EARTHQUAKE BUILDING CAPACITY

Based upon our investigations to date, we do not consider the Physical Medicine Block to have any significant reduction to the overall gravity load resistance of the structure

The areas where lateral load resisting capacity has been reduced as a result of the earthquake damage observed is outlined below:

- **Block Walls –** The stepped and diagonal cracking observed in the reinforced concrete block walls has resulted in some reduction to the capacity of the walls.

- **Wall and Ceiling Linings in the Occupational Therapy and Physiotherapy Areas –** Damage has been noted to the wall and ceiling linings. As these linings provide the lateral load resistance in this portion of the building the damage noted will have resulted in some loss of strength and stiffness.

- **Differential Ground Settlement -** The differential settlement observed in the building will have resulted in a reduction in the overall lateral load resisting capacity of the building, particularly to the concrete frames and internal block walls. There will also have been a reduction in the ability of the building to absorb future differential settlements.

The actual percentage reduction in the overall lateral load resisting capacity of the building is hard to quantify. As noted above, most of the damage observed will have had a primarily localized affect. The damage observed will require repair to restore the strength, stiffness and durability performance of the individual structural components. The repair work is outlined in Section 4. Following the recommended repair of the structural damage, the lateral load resisting performance of the structure will be restored to approximately pre-earthquakes levels.

In Section 2, the Pre-Earthquake capacity each section of the Physical Medicine Block has been assessed as being above 33% DBE. Permanent repair and strengthening measures will be required to reinstate the building to its pre-earthquake undamaged condition have been included in Section 4. In addition to the minimum repairs, recommended strengthening concepts to increase the seismic performance of the building and bring the assessed capacity above 67% DBE have been included in section 5.

4. OBSERVED DAMAGE AND REQUIRED REPAIRS

This section covers the damage noted during our detailed assessment of the building. Note that our observations have been restricted to structural aspects of the building 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.

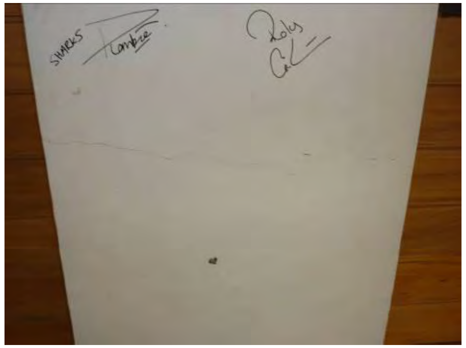
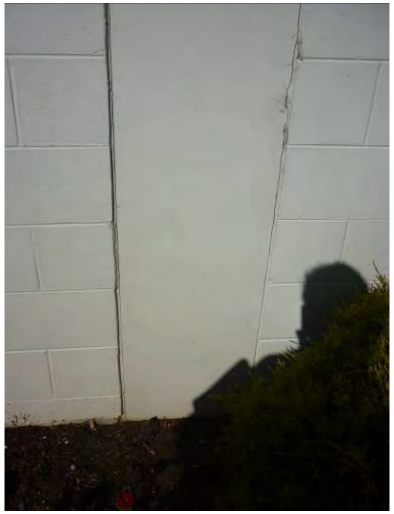
Table 4-1 provides a photographic summary of the observed damage and typical repairs required for the Physical Medicine Block. A full damage assessment is included Appendix A – Record of Observation and Appendix B – Reference Plans. The Repair Specification [2] referred to in Table 4-1 has been issued separately.

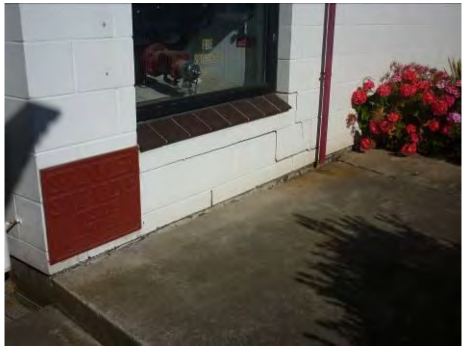
In general, the aim of the repair work indicated in this section is to restore the structure to its pre-earthquake state, as far as practicable, while maintaining the utility of the building. The repairs presented attempt to address the loss of strength, stiffness and durability of the structural elements due to the damage noted.

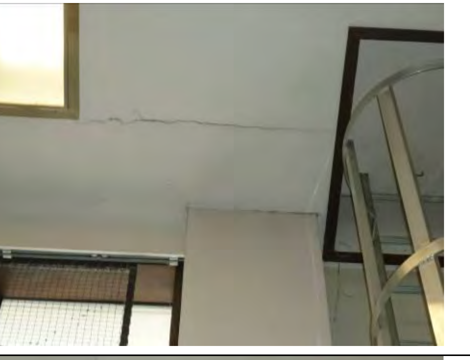
It should be noted that more damage may be identified during the repair works and (if required) additional repair details will be specified accordingly.

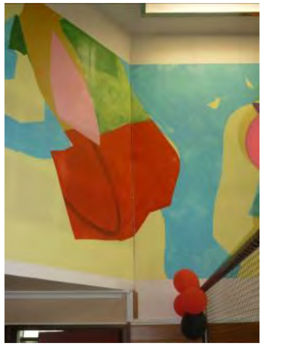
Further recommendations for improvement to the buildings seismic performance, and to bring the assessed capacity of the building above 67% DBE have been included in Section 5.

Table 4-1: Summary of Photographs of Observed Damage and Repairs Required

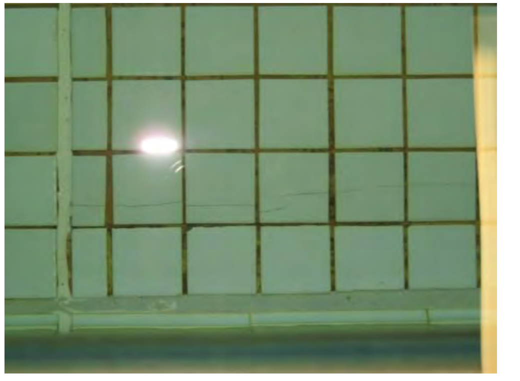
Damaged Item & Location	Damage	Recommendations	Example Photograph
1. Concrete Frames and Walls and Infill Block Walls			
1.1. Gymnasium Concrete Frames	Cracking in concrete columns	Epoxy inject cracks in column less than 1mm, in accordance with HCG specification For cracks greater than 1mm, contact HCG to confirm the integrity of the column reinforcement. If reinforcement is damaged, additional repair may be required	
1.2. Interface of Infill Concrete Block Walls and Concrete Frames	Separation of the infill block wall at the interface with the concrete frame beam and column members	Damaged mortar joints will need to be removed and the wall re-pointed. See Section 4.2 for additional information	

Damaged Item & Location	Damage	Recommendations	Example Photograph
1.3. Infill Concrete Block Walls	Stepped cracking in mortar joints of block walls	The damaged mortar between blocks will need to be removed and the wall re-pointed, cracks will need epoxy injection. See Section 4.2 for additional information	
1.4. Concrete block wall	Cracking of block wall around Fire Sprinkler Valve Room.	All damage concrete blocks will need to be removed and replaced with in kind material. Any cracks through grouted cells will be required to be epoxy injected. Section 4.2 for additional information For additional strengthening options see Section 5.	
1.5. Service Tunnel Walls	Cracking in service tunnel walls	Epoxy inject all cracks in the wall between 0.2mm & 1mm as per the HCG specification. If cracks of greater than 1mm are observed in the walls advice HCG for addition inspection.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
1.6. Pool walls	Cracking pool walls (observed from within service tunnels)	When pool is empty of water, remove the existing tiles at the crack location and apply a crack repair product for water retaining structures such as Xypex.	
2. Internal wall and ceiling linings			
2.1. Ceiling/roof diaphragms (All Areas)	Cracking in ceiling linings	The ceiling linings provide the lateral load transfer throughout the block. Any damaged gypsum board/plywood sheets are to be replaced. All roof/ceiling linings to remain are to be re-fixed to the framing in order to restore the pre-earthquake strength and stiffness. Refer to Section 4.4	
2.2. Structural internal wall linings (Workshop, Occupational Therapy and Physiotherapy)	Cracking in wall linings	The wall linings provide the bracing for this portion of the building. Any damaged gypsum board/plywood sheets are to be replaced. All wall linings to remain are to be re-fixed to wall framing in order to restore the pre-earthquake strength and stiffness. Refer to Section 4.3	

Damaged Item & Location	Damage	Recommendations	Example Photograph
2.3. Non-structural internal wall linings (Gymnasium and Hydrotherapy Pool)	Cracking in wall linings	These will require cosmetic repair only.	
3. Floor Slabs			
3.1. Differential settlement in slab on grade floors (Gymnasium & Workshop)	Differential ground settlement of approximately 45mm resulting in a worst case slope in the ground floor slab of approximately 0.31% (1:320)	For further discussion on the remediation work required see Section 4-1. (Note: All re-levelling is to occur prior to any other permanent structural or cosmetic repairs).	

Damaged Item & Location	Damage	Recommendations	Example Photograph
3.2. Slope in elevated concrete floor slab (Hydrotherapy, Occupational Therapy and Physiotherapy)	Differential ground settlement of approximately 40mm.	For further discussion on the remediation work required see Section 4-1. (Note: All re-levelling is to occur prior to any other permanent structural or cosmetic repairs).	
4. Pool Area			
4.1. Floor tiles around pool area	Floor tiles delaminating from floor substrate. In some instances they have also "popped" up	Appear to require a cosmetic repair only. However, when tiles are removed the concrete floor should be inspected for damage.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
4.2. Tiles within swimming pool	Crack through tiles in pool floor. The crack extends along the south end of the pool.	Appears to require a cosmetic repair only. However, when tiles are removed the concrete floor should be inspected for damage.	

4.1 DISCUSSION ON BUILDING RE-LEVELLING

The level survey, completed by Fox & Associates, has indicated that earthquake induced differential ground settlements have occurred at the Physical Medicine Block, resulting in permanent slopes in the ground floor throughout of typically 1:350. The slopes observed in the concrete ground floor slabs of the Occupational Therapy and Physiotherapy, Workshop and Gymnasium are outside the typical acceptable range for concrete construction. The ground floor surrounding the Hydrotherapy Pool is near to level in all except the southern part of the room, but is significantly lower than the remainder of the Physical Medicine Block.

Re-levelling options for Physical Medicine Block are as follows:

Occupational Therapy and Physiotherapy Areas – The ground floor framing of these areas consist of an elevated precast concrete floor slab with insitu topping supported by concrete sub-floor walls, partial basement walls and service tunnel walls on shallow strip footings. The level survey has indicated a high point in the middle of the northern wing, above the partial basement and service tunnels. Lower levels have been noted along the southern perimeter of the building.

The two primary re-levelling options available for these areas include the use of mechanical jacking, or the use of either underpinning grout, to raise the strip footings to the elevation of the highpoint noted above the partial basement and service tunnels. There are advantages and disadvantages for each solution which extend beyond structural performance which will need to be considered by CDHB. These include continuity of operation, degree of re-levelling accuracy and the willingness of the re-levelling sub-contractor to provide a producer statement, amongst other items.

From a structural standpoint, either option is acceptable provided the use of underpinning grout or engineered resin does not create any “hard points” under the building. If “hard points” are created during the re-levelling process the potential for future differential settlements can be increased. If this were to occur it would reduce the capacity of the building going forward.

Based upon the information provided by Tonkin & Taylor the soil profile throughout the Burwood Hospital (medium dense sand overlying dense sand) lends itself to localized lifting through underpinning grout or engineered resin techniques and should not create any undesirable “hard points” as described above.

The suitability of re-levelling the building through the use of either mechanical jacking or underpinning grout (or engineered resin) will need to be verified by qualified sub-contractors in conjunction with the geotechnical consultant.

Gymnasium and Workshop - The floors of the Gymnasium and Workshop are constructed of slab on grade. The level survey has indicated a high point in the middle of the Gymnasium and a fall from west to east in the Workshop. The use of underpinning grout is the only suitable option to raise and level floor slabs in these areas.

It should be noted that the re-levelling options discussed above are not expected to increase the seismic performance of the building or reduce the potential for future differential settlements. Instead the options presented are designed to re-level the building without making the future performance of the building any worse than it was prior to the earthquakes. To improve the future performance of the building, and reduce the potential for future differential settlements, would likely require the entire footprint of the building to be either piled or the ground under all the sub-floor wall footings, service tunnels and the partial basement improved. Further geotechnical investigations would be required into the type and depth of ground improvement required.

Based up the geotechnical report provided by Tonkin & Taylor [5] the potential for future total and differential settlements at the building site varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

During the re-levelling process there is also the risk that addition damage could occur to the building linings, exterior block veneer, etc. Appropriate contingencies should be provided.

4.2 REPAIR OF CONCRETE BLOCK WALLS

The concrete block walls, infill walls and block veneers of the perimeter and interior concrete frames have been damaged and require repair and/or replacement. The damage has typically occurred at the interface of the infill walls with the concrete frame columns and/or beam elements. In a number of places stepped cracking has also occurred in the mortar joint of the infill walls.

Where cracking is around the perimeter, the infill walls could be repaired to their pre-earthquake condition by removing and re-pointing all the damaged mortar joints. If cracking is larger than 0.6 mm wide then it is possible the reinforcement crossing these cracks is also damaged and in these instances replacement of the wall is recommended unless further investigation into the reinforcement is completed.

Where step cracking is present in fully grouted concrete block walls, these need to be epoxy injected. Unless cracking is larger than 0.6 mm wide then it is possible the reinforcement crossing these cracks is also damaged and in these instances replacement of the wall is recommended unless further investigation into the reinforcement is completed.

Any damaged blocks will need to be removed and replaced.

4.3 REPAIR OF WALL BRACING

The wall linings to the interior and exterior bracing walls have been damaged in Occupational Therapy and Physiotherapy areas and requires repair. Based upon the movement observed it is also believed the wall lining fixings have been damaged throughout. We believe this has resulted in a reduction to the ongoing strength and stiffness of all the bracing walls. In order to reinstate the pre-earthquake strength and stiffness to the bracing walls, the repair recommendation is to remove all cracked or damaged sections of the wall linings (Plywood or gypsum board) and replace them with new gypsum board sheathing or Plywood (in kind). The new gypsum board sheathing is to be fixed in accordance with GIB 'ezybrace' GS2-N specifications (or equivalent). The plywood shear wall fixings should be in accordance with the ECOPLY specifications (or equivalent).

All repairs to wall bracing are to be completed after the re-levelling and repair of the footings is complete. Refer to figure 4-1 for extent of wall repairs.

Note: The fixings of the walls to the timber framing below will need to be checked for damage and the ability to transfer the new bracing loads.

4.4 REPAIR OF CEILING ROOF DIAPHRAGMS

Similarly to the wall linings, the ceiling roof diaphragm and its fixings have been damaged and require repair. The repair recommendation is to remove any cracked or damaged sections of ceiling lining and replace with new plywood sheathing fixed in accordance with the ECOPLY specifications.

All repairs to the ceiling diaphragms are to be completed after the re-levelling and repair of the footings.

The primary lateral load resisting system of the Physical Medicine Block consists of a combination of cantilevered concrete columns in the Gymnasium and Hydrotherapy Pool, along with a combination of block and plywood braced walls throughout the remainder of the block. A number of the concrete columns are infilled between with concrete block walls. Lateral loads are distributed to the vertical bracing elements by the plywood roof diaphragms.

As noted in Section 2, Pre-Earthquake Building Condition, and Section 3, Post-Earthquake Building Condition, the lateral load resisting capacity of each section of the building has been assessed as a percentage of the loads imposed by the Design Basis Earthquake (DBE).

The assessed capacity of the building, in its pre-earthquake undamaged state, is specifically outlined in Section 2.4. All sections of the building have been assessed at a capacity above 33% DBE.

Provided the permanent repairs works noted in Section 4 are completed, the assessed capacity of the building will be returned to the pre-earthquake capacity above 33% DBE.

Strengthening works to increase the seismic performance of the building, and increase the assessed capacity of the building above 67% DBE have been included in Section 5.1. If strengthening of the building is to be considered this is the minimum target we would recommend.

5.1 STRENGTHENING WORKS TO ACHIEVE 67%DBE

The work involved to bring the assessed capacity of the building above 67% DBE is as follows (As per concept issued 5th March 2014):

- **Occupational Therapy and Physiotherapy –** Has been assessed at 50% DBE. It is limited by the existing fixings of the ply braced walls.

  In order to strengthen the building to 67% DBE it is recommended that the existing ply bracing be re-nailed in accordance with current practice. See Figure 5-1 for the location and extent of walls to be relined.

- **Gymnasium –** Has been assessed at 35% DBE. It is limited by the concrete moment frames in the gymnasium in the east-west direction.

  On the south side of the building, two new concrete walls should be placed within the moment frame as shown in SK01 and SK02. These walls will require reinforcing of D12 bars at 200mm crs in each direction of each face. These bars will need to be drilled and epoxied into the existing concrete frame on all sides. On the north side of the building, two walls are also required. Here the existing timber walls or block infill (depending on which bays are chosen) will need to be removed first.

  A more detailed assessment of the foundation beams supporting the cantilever columns has been completed which indicates that no strengthening is required to achieve 67% DBE.

A combination of a more detailed assessment and additional information from site investigations has shown that no strengthening is required to the north-south plywood walls to achieve 67% DBE.

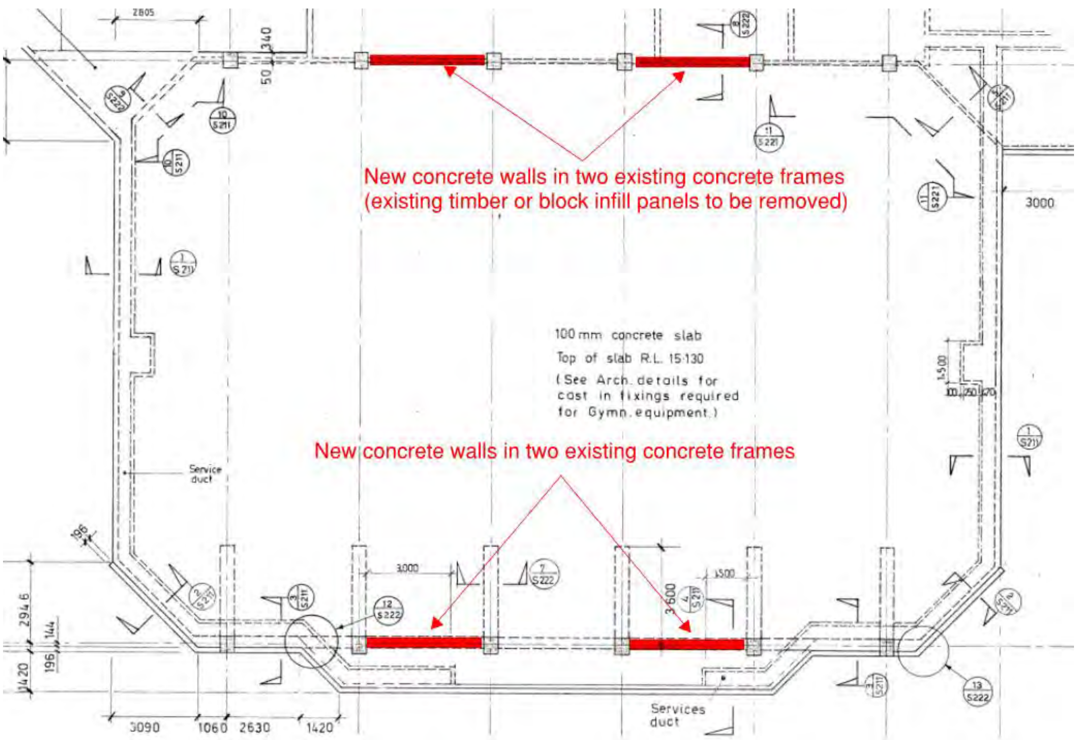


Figure 5-1: Gymnasium Floor Plan - Strengthening to Achieve 67% DBE

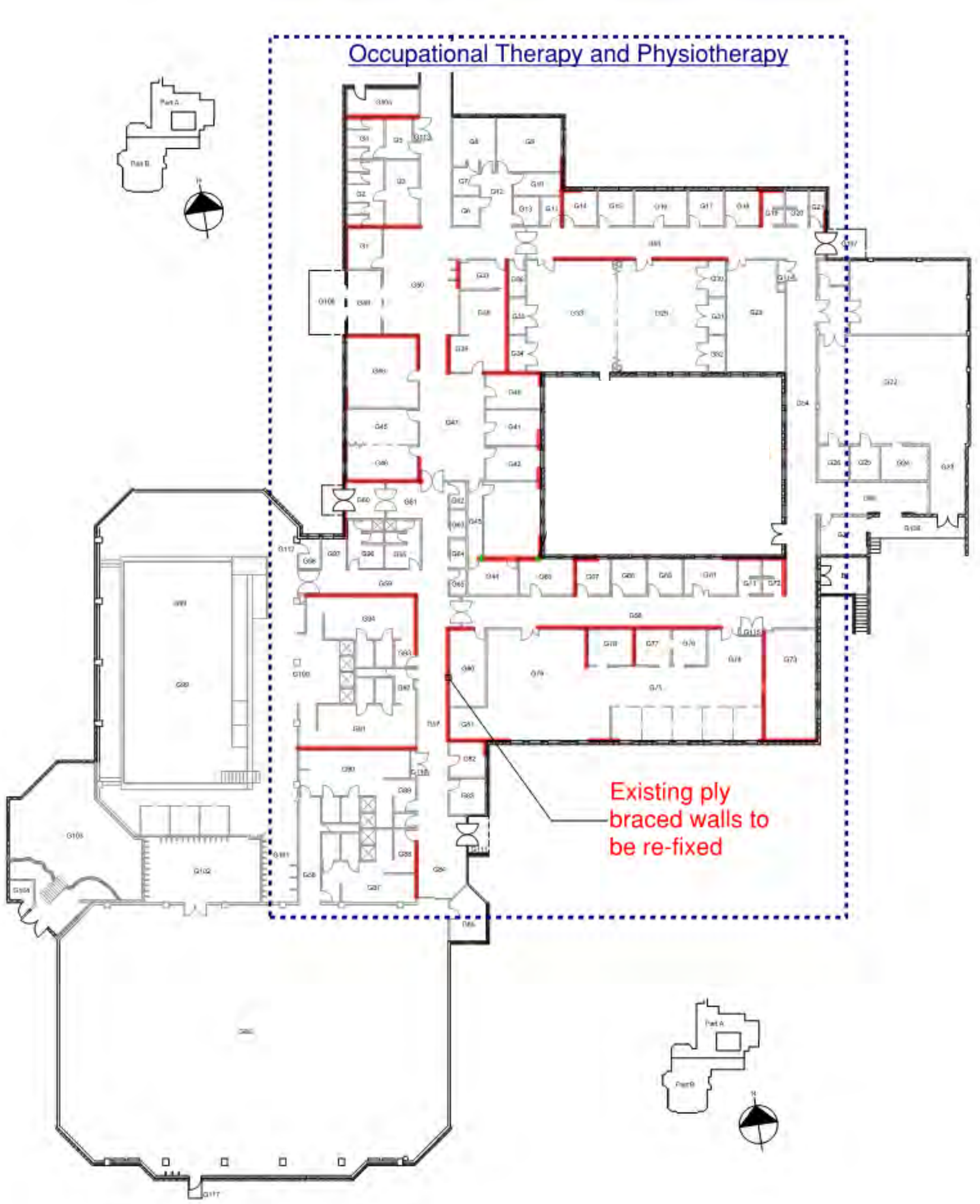


Figure 5-2: Occupational Therapy and Physiotherapy - Strengthening to Achieve 67% DBE

5.2 STRENGTHENING WORKS TO ACHIEVE 100% IL3 DBE

Refer to the 100% IL3 Strengthening Concept for the strengthening required to achieve 100% IL3 DBE.

6. REFERENCES

1. *Burwood Hospital – Detailed Seismic Assessment Report – Base Report*, Holmes Consulting Group, November 2011.
2. *Burwood Hospital – Detailed Seismic Assessment Report – Earthquake Repair Specification*, Holmes Consulting Group, July 2011.
3. B*urwood Hospital Additions - Department of Physical Medicine,* Original Architectural drawings, Cutter Pickmere Douglas Architects, March 1981
4. B*urwood Hospital Department of Physical Med Structural Dwgs,* Original Structural drawings, Frederick Sheppard and Partners, May 1981
5. *Burwood Hospital Post Earthquake Geotechnical Assessment*, Tonkin and Taylor Ltd., June 2011.
6. *Burwood Elevation Survey – Revision F*, Fox & Associates, April 2012
7. *Structural Design Actions Part 5: Earthquake Actions – New Zealand, NZS 1170.5:2004*, Standards New Zealand, 2004.
8. *Compliance Document for New Zealand Building Code - Clause B1 – Structure, Amendment 10 (Canterbury)*, Department of Building and Housing, Wellington, 19 May 2011.
9. *Code of Practice for General Structural Design and Design Loadings for Buildings*, NZS4203:, Standards New Zealand, 1976
10. *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes, NZSEE-2006,* New Zealand Society for Earthquake Engineering, 2006
11. CD*HB – Burwood Hospital Campus – Rapid Visual Inspection: 106186.03 Site Report 1,* Holmes Consulting Group, February 2011
12. *CDHB – Burwood Hospital – Rapid Seismic Assessment Post June 13th Earthquakes: 106186.03 Site Report 4,* Holmes Consulting Group, 14 June 2011
13. *CDHB – Burwood Hospital – Rapid Seismic Assessment Post June 13th Earthquakes: 106186.03 Site Report 6,* Holmes Consulting Group, 16 June 2011
14. *CDHB – Burwood Hospital – Post Earthquake Rapid Structural Assessment following 2nd January 5.5 Magnitude Earthquakes, 106186.03 Site Report 9*, Holmes Consulting Group, 9 January 2012
15. *CDHB – Burwood Hospital – Ceiling Tiles Review, 106186.03 Site Report 12*, Holmes Consulting Group, - 31 January 2012

APPENDIX A

Record of Observations

APPENDIX A – RECORD OF OBSERVATIONS & REPAIRS

Inspection date: 14 March 2012

KEY	
N	No repair required
Y	Repair required
F	Further investigation required
C	Repair complete

Pool Building:

Level	Building Element	Location	Observations	Repair Required	Repair	Photo Reference
G	Lining to walls & ceilings	G99	Cracking at junction of linings	**Y**	Aesthetic Repair to finishes, repair specification by others	P1080243 & P1080244
G		G99	Cracking of linings around columns	**Y**	Aesthetic Repair to finishes, repair specification by others	P1080244
G		G101	Cracking to linings over lintel to from opening to G101	**Y**	Aesthetic Repair to finishes, repair specification by others	n/a
G	Ceiling space	G99	No visible damage	**N**	No repair required	

Gymnasium Building:

G	Lining to walls & ceilings	G86	Cracking of lining around penetrations for lights	**Y**	Aesthetic Repair to finishes, repair specification by others	P1080258
G		G86	Cracking of ceiling linings propogating out from the corner of the ceiling tiles	**Y**	Aesthetic Repair to finishes, repair specification by others	P1080264
G			Cracking at junction of linings	**Y**	Aesthetic Repair to finishes, repair specification by others	P1080265
G			Cracking to linings over lintel to from opening to G86 (junction of linings)	**Y**	Aesthetic Repair to finishes, repair specification by others	P1080259

Level	Building Element	Location	Observations	Repair Required	Repair	Photo Reference
G	Floor Slab	Entrance to G86	Settlement of Gym slab on grade relative to the adjacent suspended floor slab to the main corridor. Approx 5-10mm.	**Y**		n/a
G	Columns	Northern row of columns	Horizontal hairline cracking to columns, particularly the central 4 columns. Average 4 cracks spaced across the first 2m of the columns. (Cracks observed 4 sides of column where possible to view all sides)	**Y**	Epoxy inject crack in accordance with HCG specification	P1080259

Remainder of Building (Internal):

G	Lining to walls & ceilings	Entry	Cracking of linings over hallway lintel	**Y**	Aesthetic Repair to finishes, repair specification by others	P1080281
G		Hallways	No visible damage	**N**	No repair required	n/a
G		Workshop	No visible damage	**N**	No repair required	n/a
G		Courtyard	No visible damage	**N**	No repair required	n/a

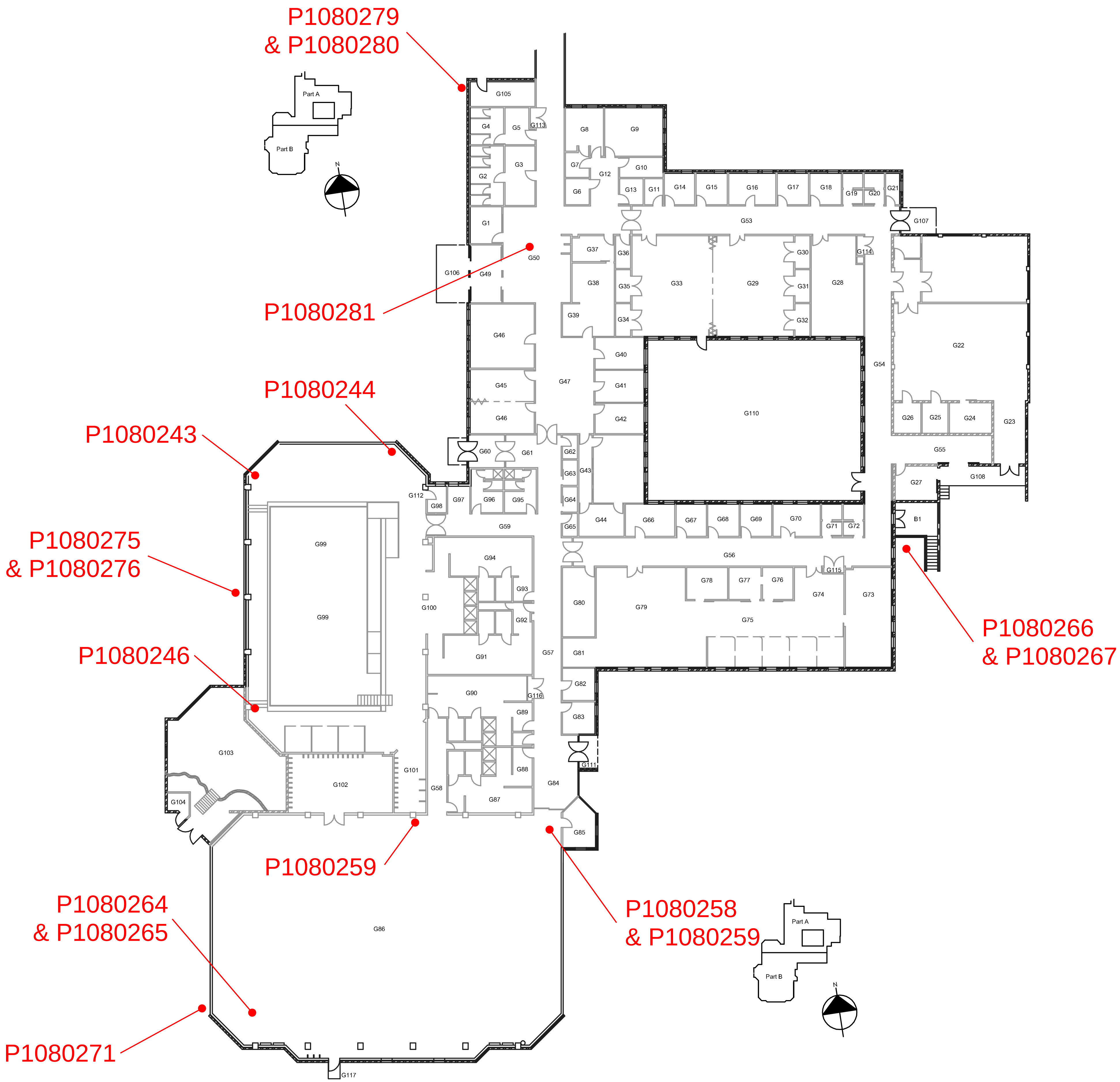
Level	Building Element	Location	Observations	Repair Required	Repair	Photo Reference

External:

G	External Walls and Columns	Pool building - Central Column	Vertical cracks and seperation between block wall and column	**Y**	Epoxy inject crack in accordance with HCG specification	P1080275 & P1080276
G		Gymnasium - End Column	Minor vertical cracking and seperation between block wall and column	**Y**	Aesthetic Repair to finishes, repair specification by others	P1080268
G		Gymnasium end wall	Minor vertical cracking and seperation between the end timber wall and external block wall	**Y**	Aesthetic Repair to finishes, repair specification by others	P1080271
G	External Slab	Adjacent Loading Dock	Settlement of bitumised area adjacent retaining wall causing minor cracking to the lower mortar joints.	**Y**	Epoxy inject crack in accordance with HCG specification	P1080266 & P1080267
G		Adjacent Admin Area	Settlement of the slab adjacent the corner of the building causing cracking to the lower mortar joints (& vertically through one block).	**Y**	Epoxy inject crack in accordance with HCG specification	P1080279 & P1080280

APPENDIX B

Location Labelling



106186.56 Burwood Hospital Physical Medicine DSA Interim Report_Rev 2_26July2012

AMENDMENT	DRAWN	CHECKED	DATE
MASTER DRAWING CREATED FROM 04000003 AND CDHB BORDER ADDED	BJT		16-06-09

BURWOOD HOSPITAL
PHYSICAL MEDICINE, GROUND FLOOR
MASTER FLOOR PLAN

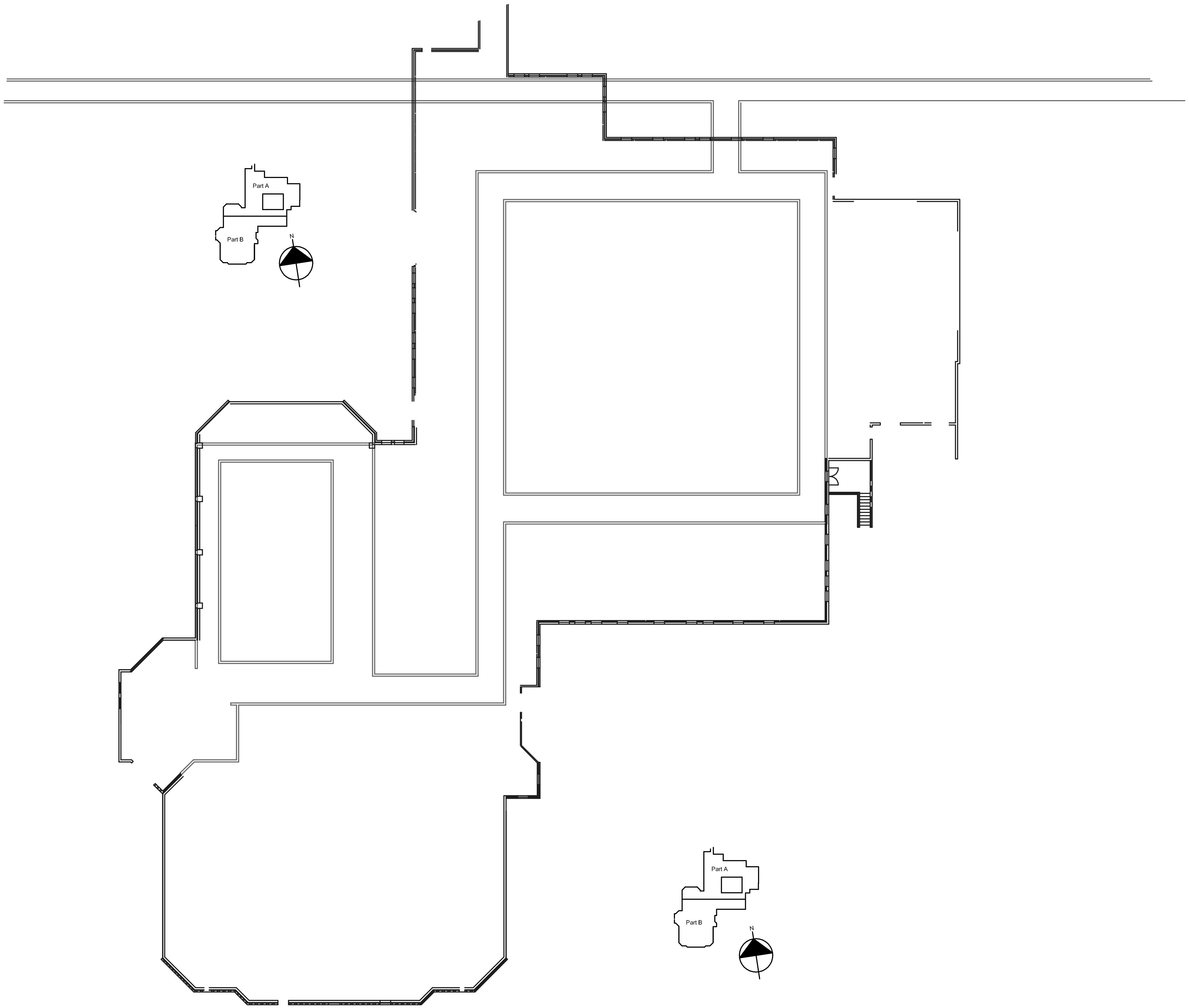
Canterbury
District Health Board
Te Poari Hauora ō Waitaha

DESIGNED:
DRAWN: BJT
CHECKED:
DATE: 16-06-09

DWG No. 04062200
SCALE 1:200 AT A1
REVISED 16-06-09

XREFED DRAWING/S:

Maintenance and Engineering Department - Christchurch Hospital



AMENDMENT	DRAWN	CHECKED	DATE
MASTER DRAWING CREATED FROM 04000003 AND CDHB BORDER ADDED	BJT		16-06-09

BURWOOD HOSPITAL
PHYSICAL MEDICINE, BASEMENT
MASTER FLOOR PLAN

Canterbury
District Health Board
Te Poari Hauora ō Waitaha

DESIGNED:
DRAWN: BJT
CHECKED:
DATE: 16-06-09

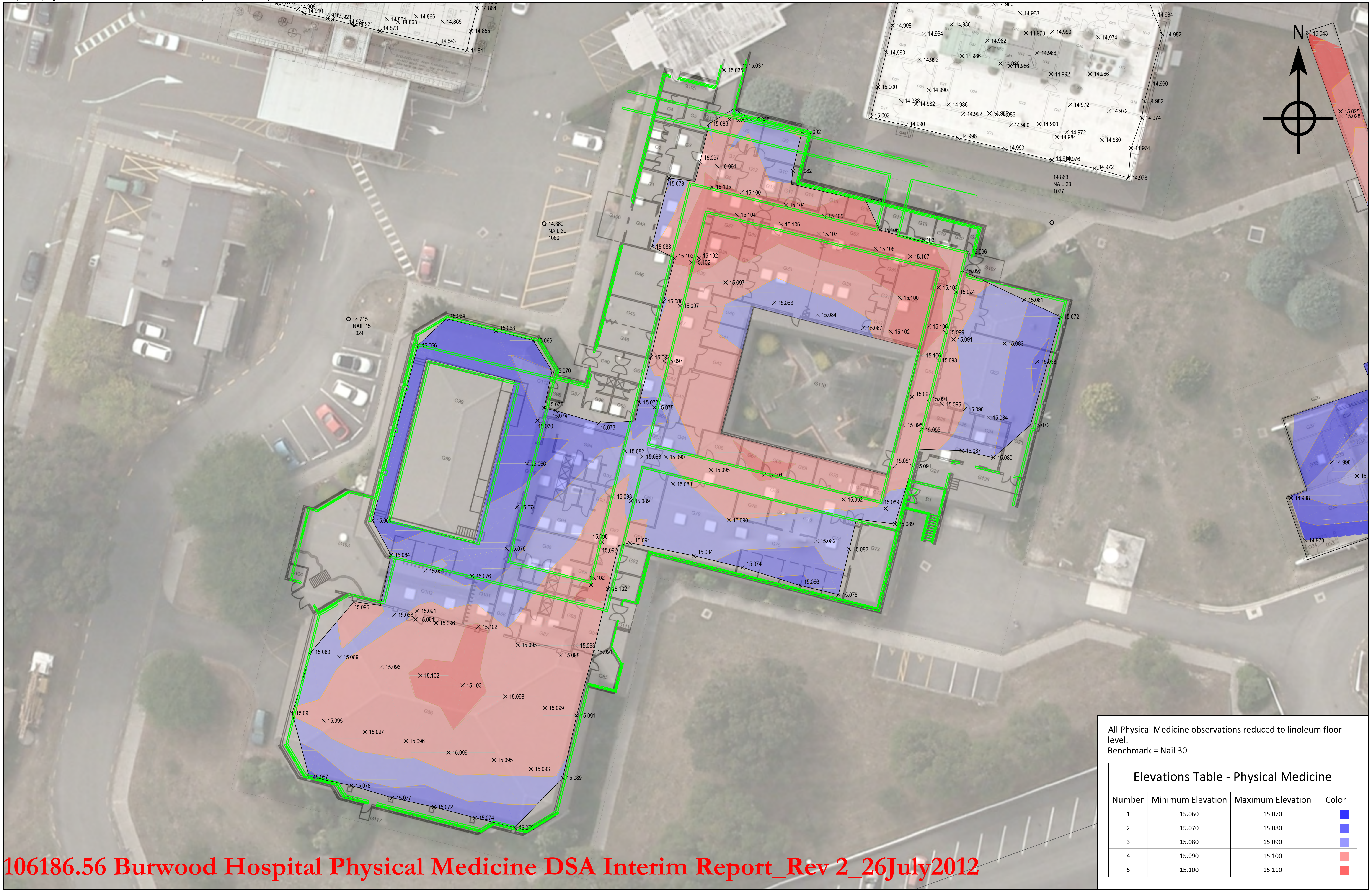
DWG No. 04061010
SCALE 1:100 AT A1
REVISED 16-06-09

XREFED DRAWING/S:

Maintenance and Engineering Department - Christchurch Hospital

APPENDIX C

Level Survey



All Physical Medicine observations reduced to linoleum floor level.
Benchmark = Nail 30

Elevations Table - Physical Medicine

Number	Minimum Elevation	Maximum Elevation	Color
1	15.060	15.070	
2	15.070	15.080	
3	15.080	15.090	
4	15.090	15.100	
5	15.100	15.110	

106186.56 Burwood Hospital Physical Medicine DSA Interim Report_Rev 2_26July2012

No.	Revision	Approved	Date
G			
F	Beacon House, BSU Hostel, Chapel, Tapper and Milner added	MJM	18/04/2012
E	Audit observations post Dec 23 2011 (not included in this Revision)	MJM	01/02/2012
D	Wards 7 & 8, Surgical Block, Melrose Chairs & Corridor added	MJM	02/12/2011
C	Phys Med, Parafed, Food Services, BIRS, Boilerhouse, EDC added	MJM	24/11/2011
B	Nurses Hostel & Champion Centre, SOU elevations added	MJM	18/11/2011
A	Administration and Allan Bean Centre elevations added	MJM	31/10/2011

032005

Scale		1:200
Reduced	A3	1:400
Designed		----
Drawn		DHP
Checked		MJM
Date		18/04/2012

Burwood Elevations Survey
Physical Medicine

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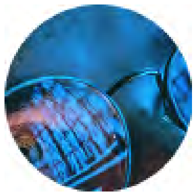
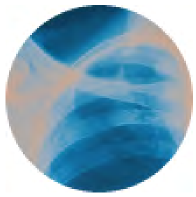
Job No. 2462C
Sheet No. 7 of 19
Rev. F

HolmesConsultingGroup



DETAILED SEISMIC ASSESSMENT REPORT

STRUCTURAL AND CIVIL ENGINEERS



BURWOOD HOSPITAL CAMPUS

REPORT 13 – SPINAL INJURIES UNIT

PREPARED FOR

CANTERBURY DISTRICT HEALTH BOARD

106186.26

INTERIM REPORT REV 3 – 20 NOV 2012

BURWWOD HOSPITAL CAMPUS – INTERIM DETAILED SEISMIC ASSESSMENT REPORT

REPORT 13 – SPINAL INJURIES UNIT

Prepared For:
CANTERBURY DISTRICT HEALTH BOARD

Date: 20 Nov 2012
Project No: 106186.26
Revision No: 3

Prepared By:

Eric McDonnell
SENIOR PROJECT ENGINEER

Updated By:

Peter Grange
STRUCTURAL ENGINEER

Reviewed By:

Jenny Fisher
PROJECT DIRECTOR

Holmes Consulting Group LP
Christchurch Office

REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
3/05/12	1	Interim results of quantitative assessment (Phase 3) for discussion (some on site investigations still to be completed)
25/06/12	2	Updated report to include Section 3.7, Further Investigations Required
20/11/13	3	Updated capacities after strengthening work

CONTENTS

APPENDICES

TABLES

FIGURES

Holmes Consulting Group has been engaged by Canterbury District Health Board to complete a full structural review of the Burwood Hospital Campus following the Lyttelton Earthquake. A series of reports have been completed as part of this. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The individual building reports, like this one for the Spinal Injuries Unit, should be read in conjunction with the base report and refer to the repair specification.

This report identifies the structural damage observed to date for the Spinal Injuries Unit as a result of the series of Earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The report summarises the effects of the damage on the lateral load capacity of the building, and provides structural repairs for the damage identified. The general form of the building, along with its capacity relative to current code levels, has been included for the buildings pre-earthquake undamaged state and post-earthquake state. Recommendations for improving the seismic performance of the building have also been identified.

The Spinal Injuries Unit was designed in 1975 and constructed in the period there after. The building is primarily a single storey structure with a central two storey plant over a partial basement. The majority of the single storey portion of the building is timber framed with the lateral bracing provided by gypsum board bracing walls. In the north-west corner of the building the hexagonal shaped dining and day rooms are formed by a series of steel portal frames. The central plant structure consists of a timber and steel framed roof over concrete walls and insitu floors below. The roof consists of tray metal roofing throughout while the exterior walls are typically clad in 150mm thick reinforced concrete block veneer.

The ground floor is formed by Unispan precast floor planks with a 75mm wire mesh reinforced topping slab which span to concrete subfloor walls below. The walls are in turn supported by continuous reinforced concrete footings which are founded approximately 1000mm below the adjacent grade. Below the precast ground floor is a 900mm high crawl space and a series of 2.4 metre deep service ducts.

The information available for the review included: the original 1975 architectural drawings [3], the original structural drawings [4], 2002 seismic strengthening and refurbishment drawings, a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [5], along with a level survey of the building completed by Fox & Associates [6].

The majority of the Spinal Injuries Unit appears to have performed relatively well considering the age of the building and the seismic actions experienced at the site. The most severe damage has occurred at the east end of the building where liquefaction induced differential settlement of 152mm has occurred over a length of approximately 9 metres (1:60 slope in the ground floor slab). Associated damage has occurred to the concrete sub-floor walls and the timber framed superstructure above. The damage to the superstructure in this portion of the building is typified by the separations of wall and ceiling framing and the cracking and warping of floor, wall and ceiling linings. Movement has also been noted at the connection of the precast floor planks to eastern most exterior sub-floor walls.

Additional damage has been noted to the gypsum wall and ceiling linings throughout. The damage is typified by cracks off the corner of door and window openings, at the ceiling line, and at wall and ceiling board joints. Cracks have also been noted to the concrete block veneer.

Earthquake induced differential settlements have been noted in other parts of the structure resulting in typical slopes in the ground floor slab on the order of 1:300. Movement and damage has also been observed to the heavy plaster ceiling tile assembly which occurs in the corridors, offices and other miscellaneous locations. One of these tiles dislodged and fell shortly after the earthquakes which occurred on the 23rd December 2011.

While some amount of damage has likely occurred in all the significant events noted, it is believed that the majority of the damage observed, occurred as a result of the 22nd February event.

Further observations of the earthquake damage observed have been included in the body of this report.

Based upon a review of the drawings available, and the site investigations completed, the primary lateral forced resisting elements of the building were assessed in their pre-earthquake undamaged state. The assessment has been updated to include the ceiling strengthening that was completed during 2012.

For the purposes of this assessment the Spinal Injuries Unit has been considered to be an Importance Level 2 building (IL2). The ceiling has been strengthened by replacing ceiling tiles with a contiguous diaphragm, such that the timber framed portion of the building has been assessed to have a capacity to resist approximately 85% of the demand required by the current loading code Design Basis Earthquake (DBE) in the north-south direction and approximately 55% DBE in the east-west direction. The limiting factor on the %DBE is the capacity of the exterior bracing along the north wall.

The steel portal framed portion of the building has been assessed to have a pre-earthquake capacity to resist approximately 75% DBE requirements for strength and approximately 60% DBE for deflections. In turn the central concrete plant structure has been assessed at 100% DBE in the north-south and east-west directions.

If the building were to be assessed as an Importance Level 3 building the capacity would drop to approximately 40% DBE for the timber portion of the building, 60% DBE for strength and 45% DBE for deflection for the steel portal framed portion of the building, and 95% DBE for the concrete portion of the building.

The reduction in the lateral capacity of the building due to the earthquake damage observed is hard to quantify. As noted, the primary damage to the structure is to the sheet clad timber bracing walls and the differential settlement at the east end of the building. Although there is some reduction in strength of the bracing walls due to the damage noted, the primarily affect is to the ongoing stiffness of the building. The reduced stiffness will result in larger lateral displacements during future seismic events and additional damage to interior linings and building contents, including the heavy ceiling tile assembly. There will also be some lost capacity as a result of the differential settlement noted in the east end of the building, but again this is difficult to quantify.

While the differential settlement noted for the rest of the building is less severe, the settlements noted will have resulted in some reduction to the capacity of the building, along with limiting the ability of the building to absorb future differential settlements before severe distress to the structure occurs. In addition, while the typical slopes in the ground floor slab (outside the east end) are within the typical acceptable range for standard occupancy buildings, CDHB may still wish to pursue re-levelling of the entire structure due to the nature of the patient group occupying the building and ongoing serviceability concerns.

The minimum repairs required to reinstate the building to its pre-earthquake undamaged condition have been included in Section 4. This includes the re-levelling of the east end of the building, the repair and re-fixing of the wall and ceiling linings.

In addition to the repairs, recommended strengthening concepts to increase the seismic performance of the building and bring the assessed capacity above 67% DBE have been included in section 5.

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 and/or strengthening of the building have been completed.

1. INTRODUCTION

Holmes Consulting Group has been engaged by Canterbury District Health Board to complete a full structural review of the Burwood Hospital Campus following the Lyttelton Earthquake. A series of reports have been completed as part of this. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The individual building reports, like this one for the Spinal Injuries Unit, should be read in conjunction with the base report and refer to the repair specification.

The Burwood Hospital Campus base report covers the purpose and scope of the structural review. The current statutory requirements relevant to earthquake damaged buildings are outlined and the level of shaking experienced at the site estimated. Earthquake induced ground settlement damage across the campus is also discussed. The repair specification has been prepared to include repair details for typical damage observed in buildings on the Burwood Hospital Campus and is referred to as required in the specific building reports.

1.1 SCOPE OF WORK

This report is on the Spinal Injuries Unit, at Burwood Hospital, Mairehau Rd, Christchurch. The report identifies the general form of the structure, along with the gravity and lateral load resisting systems. Each component of the structural system was reviewed based upon the information available and any potential Critical Structural Weaknesses (CSW's) were noted.

The report also identifies the structural damage observed to date as a result of the series of Earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake, in particular, subjected the building to strong ground motions which significantly exceeded the current code loading demand for buildings of this nature.

The capacity of the Spinal Injuries Unit has been assessed relative to current code loading in the buildings pre-earthquake undamaged state and in its post-earthquake damaged state. The post-earthquake assessment summarizes the effects of the damage identified on both the gravity and lateral load resisting elements. Repair options to restore the buildings capacity to pre-earthquake levels for strength, durability and stiffness have been included. The repair options aim to maintain the buildings utility. Where required, strengthening options have also been provided.

1.2 LIMITATIONS

Findings presented as a part of this project are for the sole use of the Canterbury District Health Board, 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 professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practising in this field at this time. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

Conclusions relate to the structural performance of the building under earthquake loads. We have not assessed the live load capacity of the floors, nor have we assessed the performance of non-structural components or building contents under earthquake loads.

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. Because all of the structure has not been available for detailed inspection or evaluation, this report is limited to those elements available and engineering judgement as to the likely condition of unseen elements. 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. PRE-EARTHQUAKE BUILDING CONDITION

This section discusses the form and capacity of the building prior to the Darfield Earthquake.

The information available for the review included: the original 1975 architectural drawings [3], the original structural drawings [4], 2002 seismic strengthening and refurbishment drawings, a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [5], along with a level survey of the building completed by Fox & Associates [6].

2.1 BUILDING FORM

The Spinal Injuries Unit at the Burwood Hospital campus was designed in 1975 by Cutter, Pick mere, Douglas and Partners Architects and constructed in the period there after. The original structural design was completed by Frederick Sheppard and Partners Consulting Engineers.

The building is primarily a single storey structure with a central two storey plant over a partial basement. The majority of the single storey portion of the building is timber framed with the lateral bracing provided by gypsum board bracing walls. In the north-west corner of the building the hexagonal shaped dining and day rooms are formed by a series of steel portal frames. The central plant structure is formed by timber roof framing and concrete floors and walls below. The roof consists of tray metal decking throughout while the exterior walls are clad in 150mm thick reinforced concrete block veneer.

The ground floor is formed by Unispan precast floor planks with a 75mm wire mesh reinforced topping slab which span to concrete sub-floor walls. The walls are in turn supported by continuous footings which are founded approximately 1000mm below the adjacent grade. Below the precast ground floor are a 900mm high crawl space and a series of deeper service ducts.

Figure 2-1: Spinal Injuries Unit – North End

The building as a whole consists of three primary sections. The first is the main single storey timber framed portion of the building, which covers the majority of the building footprint. The second is a two storey central plant structure over a basement. The third section consists of two hexagon shaped vaulted rooms at the northwest corner of the building which are formed by a series of steel portal frames. A ground floor plan, architectural roof plan and structural roof plan noting the outline of the various building sections have been included in figure 2-2, figure 2-3 and figure 2-4.

Figure 2-2: Spinal Injuries Unit – Ground Floor Plan

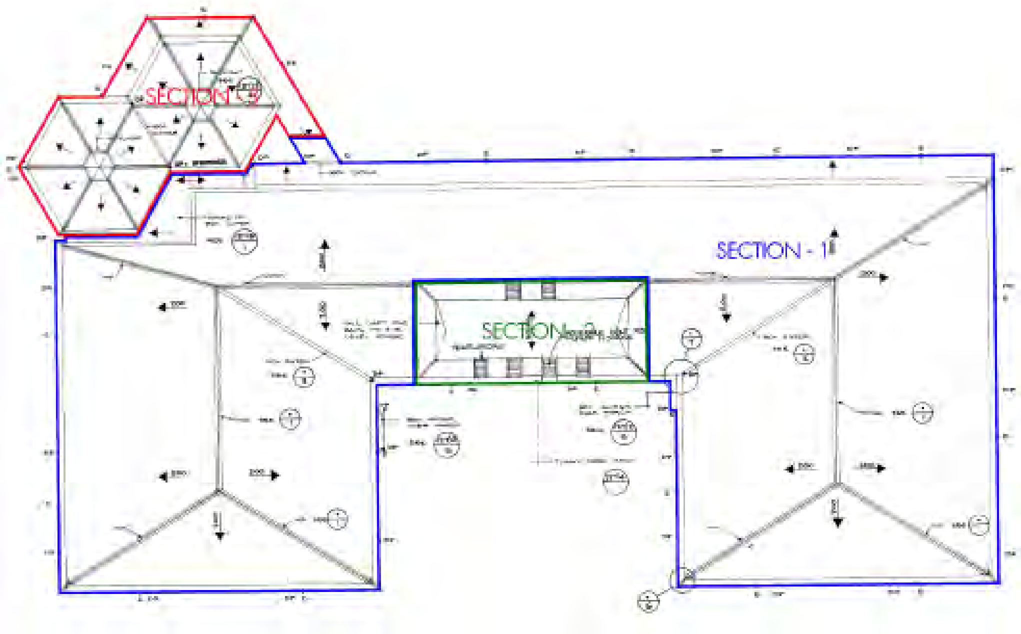


Figure 2-3: Spinal Injuries Unit – Architectural Roof Plan

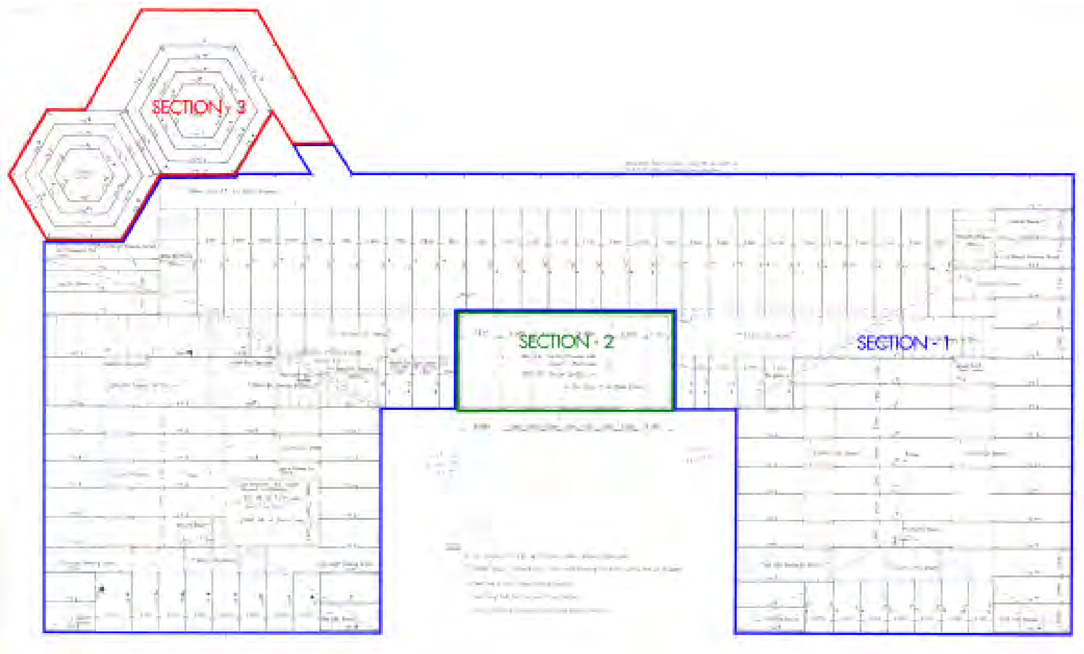


Figure 2-4: Spinal Injuries Unit – Structural Roof Plan

As noted above the ground floor under Sections 1 and 3 of the building are supported by a concrete topped precast floor system which spans between concrete sub-floor walls. The concrete sub-floor walls form a series of service tunnels and crawl spaces which connect into the basement of the central plant structure. The location of the sub-floor walls and the orientation of the precast planks are shown in figure 2-5 and figure 2-6 below.

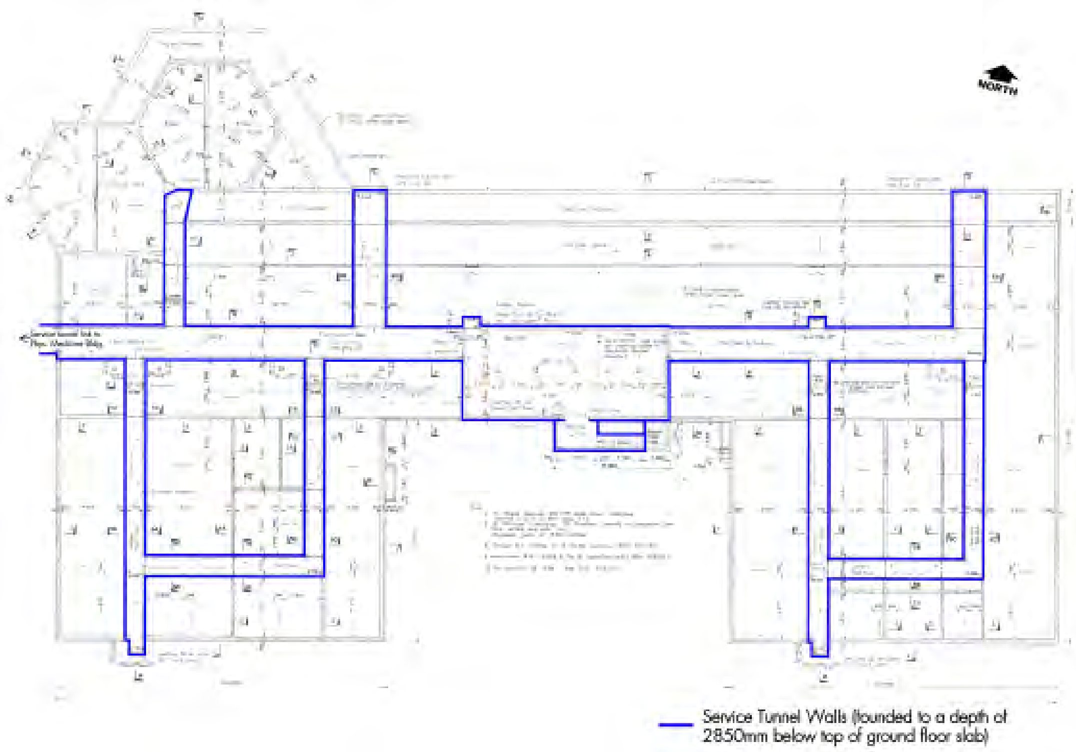


Figure 2-5: Spinal Injuries Unit – Basement/Foundation Plan

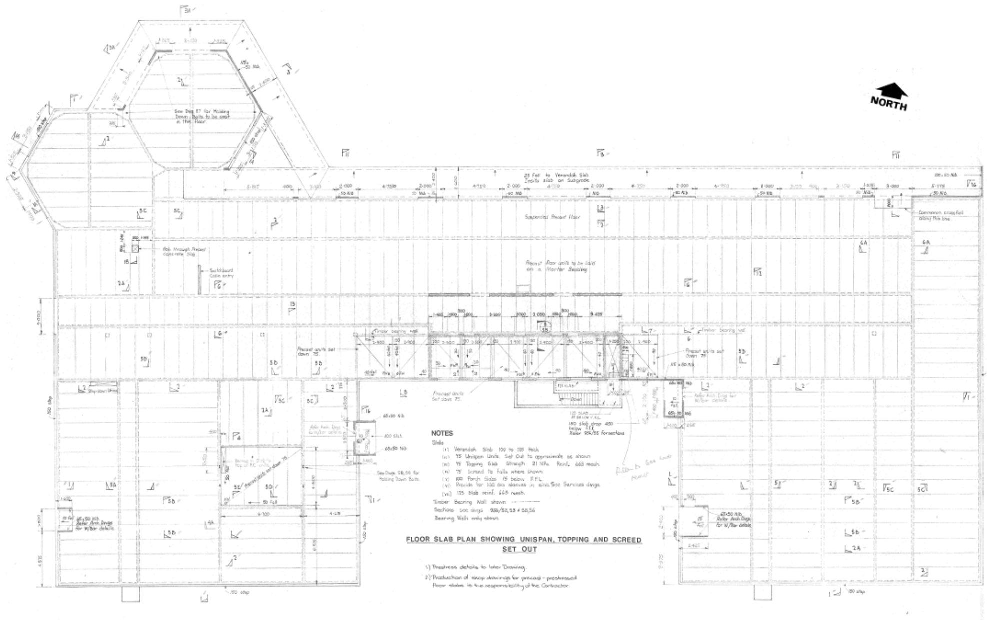


Figure 2-6: Spinal Injuries Unit – Ground Floor Precast Slab Plan

Section 1 – Timber Framed: The roof assembly for the main single storey timber framed portion of the structure typically consists of tray metal decking, building paper and chicken wire over timber roof purlins which span between timber roof trusses. The roof trusses in turn span between internal and external load bearing timber stud walls.

The ceilings originally consisted of a combination of gypsum board sheathing and a heavy plaster acoustical tile assembly. The heavy tiles have been replaced during 2012 with gypsum board sheathing. In general, timber ceiling joists are either hung from the roof trusses above or span between interior corridor walls. The gypsum board is fixed to timber battens, which are in turn fixed to the ceiling joists above.

The external timber stud walls are lined on the inside face with gypsum wallboard sheathing and the exterior face is clad with 90mm and 150mm thick reinforced block veneer. The internal walls are clad on each face with gypsum board sheathing, which typical extends up to the ceiling line. At fire separations the gypsum wallboard extends all the way up to the underside of the metal deck roofing. Both internal and external stud walls are fixed to the elevated precast floor system below. For typical sections through the timber framed portion of the building see figure 2-7.

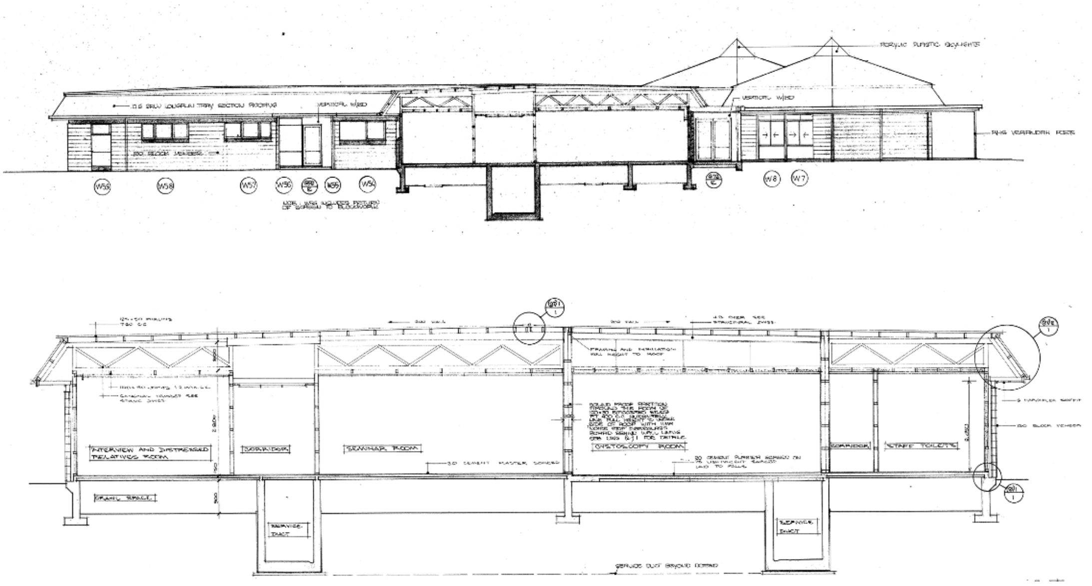


Figure 2-7: Spinal Injuries Unit – Building Section

Section 2 – Concrete Framed: The central concrete plant structure consists of a basement level and two additional storeys above grade. The roof of the plant structure consists of tray metal decking over timber purlins which span to steel portal frames which form the vaulted space of the plant structure. The steel portal frames bear on the external reinforced concrete walls which extend down to the basement level. The first floor is formed by a reinforced insitu slab while the ground floor is formed by Unispan precast units with a 75mm reinforced wire mesh topping. The interior and exterior concrete walls are supported on a tanked reinforced concrete mat slab. For a section through the plant structure see figure 2-8.

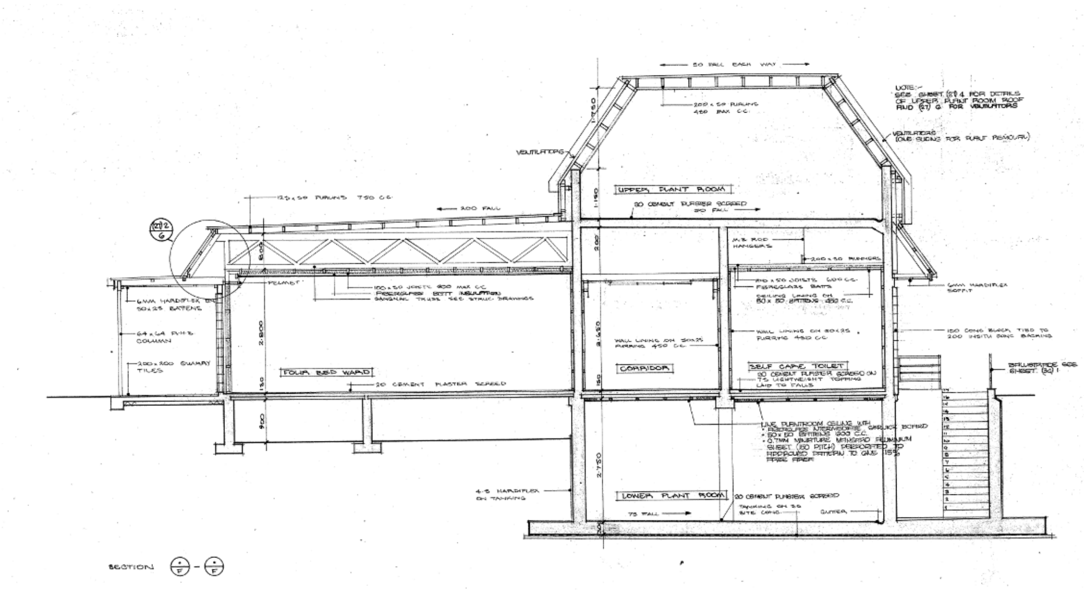


Figure 2-8: Spinal Injuries Unit – Plant Building Section

Section 3 – Steel Framed: The steel framed portion of the building consists of a series of steel portal frames which form the hexagonal shaped Dining and Day Rooms. Each leg of the steel portals frames into a central steel ring beam which forms the central skylight. The steel portals are topped by timber ceiling framing, timber trusses, timber purlins and battens which support tray metal decking. As with the remainder of the building, the ground floor is formed by an elevated precast floor system which spans to concrete sub-floor walls below. For a section through the Day Room see figure 2-9 below.

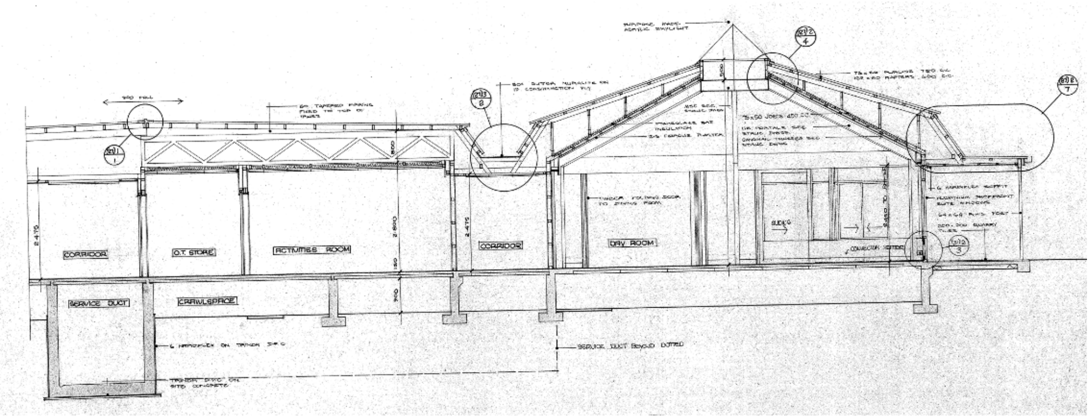


Figure 2-9: Spinal Injuries Unit – Plant Building Section

2.2 LATERAL LOAD RESISTING SYSTEMS

Section 1 – Timber Framed: The lateral load resisting system for the timber framed portion of the building (Section 1) consists of timber stud bracing walls clad in gypsum board sheathing. As noted above, the ceiling was strengthened during 2012 and is now a contiguous gypsum board ceilings so can act as a ceiling diaphragm to evenly distribute loads to the internal and external bracing walls.

Section 2 – Concrete Framed: The lateral load resisting system for the concrete framed portion of the building (Section 2) consists of reinforced concrete shear walls. At the roof level a gypsum clad ceiling acts as a flexible roof diaphragm. At the first floor a rigid diaphragm is formed by the reinforced insitu slab while at the ground floor the diaphragm is formed by the precast floor units and reinforced topping slab.

Section 3 – Steel Framed: At the steel framed portion of the building, the lateral load resisting system is formed by the steel portal frames and the gypsum board ceiling diaphragms.

At the ground floor level the precast floor units, along with the reinforced topping slab, act as a rigid diaphragm to distribute lateral loads to the concrete sub-floor walls and partial basement below.

The lateral load resisting system below the ground floor level is significantly stiffer than the timber or the steel framed portions of the superstructure above. As a result these portions of the superstructure have been treated as being de-coupled from the concrete sub-floor above for the purpose of this evaluation.

2.3 PRE-EARTHQUAKE BUILDING CAPACITY – DIRECT CODE COMPARISON

The building capacity under earthquake actions discussed in this section is compared to the capacity that a similar building would be designed to today. A new building would be designed to the *Structural Design Actions Standard, Part 5: Earthquake Actions – New Zealand*, NZS 1170.5:2004[9] and incorporating the amendments made to this standard as a result of the Lyttelton Earthquake as outlined in the Amendment 10 of the Building Code [9]. The implications of the recent amendments are discussed more fully in the Burwood Hospital Campus Base Report however, for a building of this type the amendments essentially result in an increase to the design loads of 36 % when compared to pre-earthquake design levels.

When the building was originally designed in 1975, the loading standard at the time was the *New Zealand Standard Model Building Bylaw – Chapter 8, Basic Design Loads*, NZSS 1900:1965 [12]. When these By-Laws were written, neither the seismology of the different areas within New Zealand, or the impact this could have on buildings was as well understood as it is today. Along with an increase in the seismic demands required by the change in the loading code over this period, the seismic detailing requirements have also progressed significantly resulting in more ductile and better performing buildings.

The current code requires a new building to be designed for an earthquake, known as the Design Basis Earthquake (DBE), which is based upon the buildings physical location, local soil conditions, building type, fundamental period and importance level.

The original structural drawings for the building are available, but the structural calculations and specifications are not, so the exact design and loading assumptions originally made are unknown. For the purposes of this report seismic loading assumptions have been made based on a detailed review of the drawings available and physical observations of the building.

Based upon building occupancy, the Spinal Injuries Unit has been classified as an Importance Level 2 building in accordance with NZS 1170:2004 [9] The associated return period of the DBE is 500 years, with a risk factor for design of R = 1.0. The sub soil for the site is taken as Soil Type D, which is consistent with the findings of a post-earthquake geotechnical investigation [5].

As the Spinal Injuries Unit contains patient facilities, the building has also been assessed at an increased importance level (IL3). The associated return period of the DBE, for an Importance Level 3 building is 1000 years, with a risk factor for design of R = 1.3.

Based upon the period of construction, and the detailing of the lateral load resisting elements, the concrete portion of the building has been concluded to have nominal ductility, and as such the reinforced concrete walls have been assigned a ductility factor of μ=1.25. The steel framed portion of the building is believed to have limited ductility and has been assigned a ductility factor of μ=2.00.

A comparison between the Design Basis Earthquake of NZSS 1900:1965 and NZS 1170:2004 for the site is plotted below. Based upon a fundamental building period below 0.50 seconds, the seismic demands required by the loading code have increased on the concrete and steel portion of the structure by approximately 560% and 300% respectively since 1975.

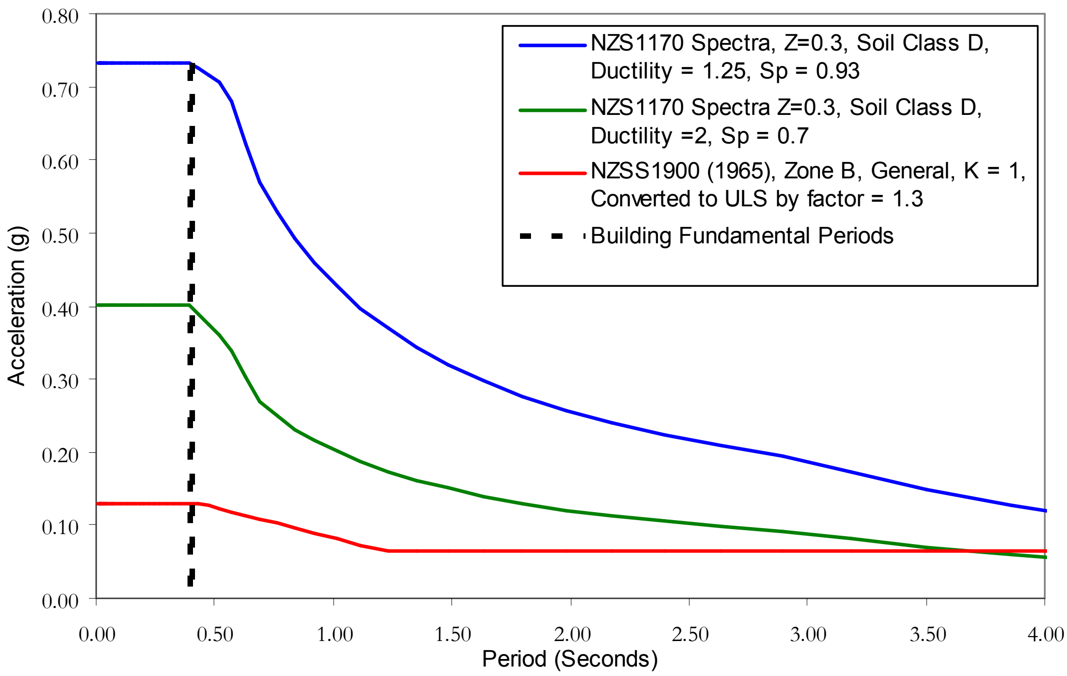


Figure 2-10: Comparison of Design Codes

In 2002 a seismic upgrade to the timber framed portion of the building was completed to the New Zealand Standard *Timber Framed Buildings*, NZS 3604:2001. At that time the assessed capacity of the bracing walls was at 60% of the requirements of the loading code in the north-south direction and 70% of the requirements in the east-west direction. Since this time an updated New Zealand Standard *Timber Framed Buildings*, NZS 3604:2011[13] has been published. The updated standard incorporates amendments made to the earthquake codes as a result of the Lyttelton Earthquake, as outlined in the Amendment 10 of the Building Code [9]. The implications of the recent amendments essentially results in an increase to the design loads of approximately 67 % when compared to pre-earthquake, NZS3604:2001, design levels.

2.4 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 estimated 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.

Following the Lyttelton earthquake, a geotechnical report was conducted by Tonkin & Taylor, titled "Burwood Hospital Post Earthquake Geotechnical Assessment", dated June 2011 [5]. This report has been used to aid in the evaluation of the site conditions and the likely effect of the ground on the buildings past and future performance. The soil parameters described in the geotechnical report complete by Tonkin and Taylor have been used for the evaluation of the buildings existing foundation system.

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 [17]. 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. These are described in more detail in the Burwood Hospital Base report [1] and include strength governed elements such as short columns and deflection governed elements such as floor and stair elements with inadequate support seating.

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 the strength and deflection requirements.

As previously noted, the timber and steel framed portions of the building have been treated as being de-coupled from the ground floor slab and concrete sub-floor walls below. The concrete and steel sections of the building have been evaluated using NZS 1170:2004, while the timber framed portion has been evaluated using the bracing requirements of NZS 3604:2011.

Since the ceiling has been strengthened by replacing tiles with a contiguous diaphragm, the limiting factor for the building is the capacity of the exterior bracing along the north wall.

For the purpose of this evaluation of the timber frame portion of the structure several assumptions also had to be made in regards to the existing timber building properties. Specifically, the existing bracing capacities of interior and exterior walls are of primary concern. The expected strength values for these elements were taken from NZSEE 2006, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes* [17] and ASCE 41-06, *Seismic Rehabilitation of Existing Buildings* [18]. These values could be further refined through destructive investigations of the existing materials. The assumed diaphragm and shear wall expected strength values are as follows:

- Exterior Walls: Timber framed stud walls with gypsum wallboard cladding on the interior face. Expected strength = 1.5kN/m (30BU/m) with ductility, $\mu = 3.3$

- Interior Walls: Timber framed stud walls with gypsum wallboard cladding on each face. Expected strength = 3.0kN/m (60BU/m) with ductility, $\mu = 3.3$

- Ceiling Diaphragm: Timber ceiling joists with direct fixed gypsum wallboard. Expected strength = 1.5kN/m (30BU/m) with ductility, $\mu = 3.3$

The bracing requirements in NZS 3604:2011 assume a ductility factor, $\mu = 3.5$ for the bracing walls and diaphragms. To account for the less ductile existing walls outlined above, the wall bracing demands from NZS 3604:2011 have been factored up proportionally as required in our analysis. Values for the bracing supplied by the reinforced concrete sub-floors walls have been taken from NZS 3604:2011.

The foundations have been assessed at an ultimate bearing capacity of 150kPa, for factored loads, as per recommendations provided by Tonkin and Taylor.

A summary of the %DBE for each primary element has been noted in Table 2-1, Table 2-2, Table 2-3 and Table 2-4 below.

Building Element	%DBE (IL2)	Comments
Ceiling Diaphragm – N-S E-W	85% 70%	The heavy ceiling tiles have been replaced with gypsum board ceilings so there is a contiguous ceiling diaphragm throughout the building
Wall Bracing – N-S E-W	100% 55%	Limited by capacity of exterior bracing walls

Table 2-1: Section 1 – Seismic Assessment % DBE

Building Element	%DBE (IL2)	Comments
Roof Ceiling Diaphragm – N-S E-W	100% 100%	
First Floor Diaphragm – N-S E-W	100% 100%	
Ground Floor Diaphragm – N-S E-W	100% 100%	
Concrete Shear walls – N-S E-W	100% 100%	
Foundations – N-S E-W	100% 100%	

Table 2-2: Section 2 – Seismic Assessment % DBE

Building Element	%DBE (IL2)	Comments
Ceiling Diaphragm – N-S ……………………….E-W	100% 100%	
Steel Portal Frames – N-S E-W	60% 60%	Limited by drift (75% of strength requirements)

Table 2-3: Section 3 – Seismic Assessment % DBE

Building Element	%DBE (IL2)	Comments
Ground Floor Diaphragm – N-S ……………… …….E-W	100% 100%	
Sub-floor bracing – N-S E-W	100% 100%	
Foundations – N-S E-W	100% 100%	

Table 2-4: Sub-floor – Seismic Assessment % DBE

For the purposes of this assessment the Spinal Injuries Unit has been considered to be an Importance Level 2 building (IL2). If the building were to be assessed for an increased importance factor, IL3, the seismic demand imposed by the DBE would increase by 30% (R=1.3) and as such the assessed capacities would be reduced proportionally. The result would be an assessed capacity of approximately 40% DBE for the timber portion of the building, 60% DBE for strength and 45% DBE for deflection for the steel portal framed portion of the building, and 95% DBE for the concrete portion of the building.

A review of the drawings available and site observations revealed no obvious Critical Structural Weaknesses (CSW's).

Methodology to improve the seismic performance of the buildings and provide strengthening to achieve 67% DBE have been included in Section 5.

3. POST-EARTHQUAKE BUILDING CONDITION

This section covers the structural damage sustained by the Spinal Injuries Unit, and its effect on the buildings capacity to resist seismic loads, as a result of the series of earthquakes which includes the Darfield Earthquake that struck at 4:36am on 4th September, 2010, the Lyttelton Earthquake that struck at 12:51pm on the 22nd February 2011, the June Earthquake that struck at 2.20pm on the 13th of June 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake subjected the building to strong ground motions which significantly exceed the full design earthquake load for buildings of this nature and appears to have caused the majority of the earthquake damage observed, after the Darfield event.

3.1 THE LYTTELTON EARTHQUAKE

The fundamental period of the building is estimated to be approximately 0.4 seconds. 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 strong motion data presented in the Base Report, it appears the Lyttelton Earthquake produced ground shaking intensities of approximately 60-120% of the current Ultimate Limit State design spectra for an Importance Level 2 building.

It should be noted that the Lyttelton Earthquake was very short in terms of strong shaking produced with the strong motion only lasting for duration of 5-7 seconds. Rupture of the Alpine Fault is expected to contain 50 to 60 seconds of strong motion.

3.2 PRELIMINARY INVESTIGATIONS

Preliminary investigations were carried out to identify 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
- review of available structural engineering construction documentation
- damage observed during 'Rapid' Structural Assessments following the Lyttelton Earthquake, June 13th aftershocks and subsequent events

In conjunction with a review of the structural drawings for the building the following areas were identified for potential damage:

- movement or damage to structure associated with ground movement and/or settlement

- cracking and joint failure of concrete sub-floor walls, service tunnels and foundations
- general distress to the steel portal frame, including beam-column joint welds
- cracking in concrete shear walls or floor diaphragms
- distress at connection of timber roof and ceiling framing to concrete walls
- connections of timber roof framing to exterior timber stud walls
- distress and cracking of gypsum clad bracing walls and ceilings
- signs of distress at connection of interior and exterior stud walls to precast floor system below
- distress and cracking of reinforced concrete block veneer and connection to timber framing above
- signs of distress at interfaces between different sections of the building

Rapid Level 2 assessments were carried out on the 24th February 2011[22] and on the 14th [23] and 15th June 2011 [24] following the June 13th earthquakes. Two additional Rapid Visual Structural Assessment was conducted on 24th December 2011 [25] and 5th January 2012 [26], following the 23rd December 2011 and 4th January 2012 events. These structural observations involved a complete walk around the exterior and throughout the interior of the building. The following primary areas of damage were identified from the damage assessments:

- significant settlement at the east end of the building, including visible separation of the surrounding soils from the exterior concrete sub-floor walls
- separation of wall and ceiling intersections at east end of the building
- cracking and warping of wall and ceiling linings throughout
- visible movement and cracks noted in heavy plaster acoustical tile assembly
- minor localized cracking to block veneer
- separation at the interface between the exterior block veneer and the linings on the underside of the exterior roof overhang
- hairline cracks in basement and service tunnel walls
- separation between isolated concrete piers and ground floor slab above
- movement and distress at door and window joinery
- extensive differential ground settlement surrounding the building resulting in distress of exterior walkways and site elements

A review of the above information on the building type and preliminary observations highlighted this building as requiring a more detailed inspection. The aim of the detailed inspections was to determine the full extent of the damage caused to the building, particularly those elements identified for potential damage above. These areas were targeted to identify if damage had occurred, and to what extent the damage had reduce the capacity of the buildings lateral load resisting system to resist future seismic events.

3.3 DETAILED STRUCTUAL OBSERVATIONS

Further detailed inspections and structural explorations (including removal of finishes) have been carried out following the initial assessments to ascertain the full extent of structural damage. The majority of the detailed structural observations were completed on the 20th October 2011, with additional trips made to the site on 31st January 2012, 9th March 2012 and 14 March 2012. The visit on the 31st January 2012 was to specifically review the heavy ceiling tiles after one of them dislodged and fell on the evening of the 18th January 2012 [26].

A full record of these observations can be found in Appendix A, with reference plans describing the location labelling used found in Appendix B. A full photographic record of the observations is available electronically on request. The detailed structural observation identified the following additional damage to those items noted in the initial rapid assessments:

- visible ground fissures and evidence of liquefaction in the crawl space below the east end of the building
- damage to ceiling and roof framing connections at the east end of the building
- damage to floor linings
- evidence of pounding at the interface between the veranda and the steel framed walkway
- differential settlement of 20mm at access ramp
- opening of existing crack over lintel at basement level of the central plant structure
- differential settlement and associated cracks in the foundations at corridor link on west end of building
- additional movement and hairline cracking noted in heavy ceiling tile assembly
- additional cracking and distress to wall and ceiling linings
- warping of tray metal deck roofing at east end of building

3.4 GEOTECHNICAL REVIEW

A review of the ground damage and conditions was carried out by Tonkin & Taylor for the Burwood Hospital Campus. A subsequent report titled *Burwood Hospital Post Earthquake Geotechnical Assessment* was issued in June 2011 [11]. The geotechnical review concluded that the settlement and damage to building foundations and slabs on the Burwood Hospital Campus was likely due to the liquefaction of underlying soil layers. It is believed that excessive pore water pressures have been dissipated and that further settlement is not expected to occur, unless another significant event were to occur.

It is estimated that the building has settled a total of 110mm – 200mm overall with a differential settlement of approximately 175mm noted across the elevated ground floor slab. The most severe settlement has occurred at the east end of the building where visible evidence of ground fissures and liquefaction have been noted in the crawl space below the slab.

Figure 3-1: Evidence of Ground Fissures and Liquefaction

Based up the geotechnical report provided by Tonkin & Taylor [11] the potential for future total and differential settlements at the building site varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

3.5 LEVEL SURVEY & VERTICALITY STUDY

A detailed survey of the ground floor levels in the Spinal Injuries Unit was conducted by Fox & Associates and issued on 31stth October, 2011 [6]. The survey indicates a differential settlement of approximately 175mm, with the most significant differential settlements occurring at the east end of the building. The worst case permanent slope in the slab on grade, based upon this survey, is a drop of approximately 152mm over a 9 metre length resulting in a slope in the ground floor slab of approximately 1.7% or 1:60. This slope is well outside the typical acceptable range and could be remediated through localised lifting of the structure using mechanical or grout injection techniques. A discussion on how to reinstate the east end of the building has been included in Section 3.1.

The differential settlement noted for the rest of the building are less severe, typically resulting in slopes on the order of 1:300 in the elevated ground floor slab. While the typical slopes (outside the east end) are within the typical acceptable range for standard occupancy buildings, CDHB may still wish to pursue re-levelling of the entire structure due to the nature of the patient group occupying the building, and ongoing serviceability concerns.

An additional survey following the earthquakes on the 23rd December 2011 and the 2nd January 2012 was completed on the 1st February 2012 and noted additional settlements of up to 7mm. For the extent of the differential settlement noted see the level survey included in Appendix C.

3.6 SUMMARY OF BUILDING DAMAGE

The following is a summary of the observations made for the building, and our conclusions as to their condition and seismic load resisting capacity. These observations do not specifically distinguish between damage caused by the Darfield Earthquake, the Lyttelton Earthquake or any significant aftershocks, such as those that occurred on the 13th June 2011, 23rd December 2011 or 2nd January 2012. Despite not being able to specifically distinguish when individual damage observed occurred, it is believed that the majority of the damaged can be linked to the February 22nd event.

The majority of the Spinal Injuries Unit appears to have performed relatively well considering the age of the building and the seismic actions experienced at the site. The most severe damage has occurred at the east end of the building where liquefaction induced differential settlement of 152mm has occurred over a length of approximately 9 metres (1:60 slope in the ground floor slab). Associated damage has occurred to the concrete sub-floor walls and the timber framed superstructure above. The damage to the superstructure in this portion of the building is typified by the separations of wall and ceiling framing and the cracking and warping of floor, wall and ceiling linings.

Other than the concentration of damage at the east end of the building, the majority of the damage noted has been limited to wall and ceiling linings and movement to the heavy plaster ceiling tile assembly. Our observations suggest that the building would have undergone a limited number of full cycles of primarily elastic deformation. The short duration of the strong ground motion recorded and the damaged observed would support this hypothesis. A summary of the building damage observed can be typified as follows:

- **Differential Ground Settlement** – As previously noted the majority of the damage noted to date appears to be associated with the liquefaction induced differential ground settlement, which has been concentrated at the east end of the building. This has resulted in damage and distress to the timber superstructure above.

  Surprisingly, the associated damage observed to date to the concrete sub-floor walls and footings has been limited to hairline cracks. Movement has been noted at the connection of the precast floor planks to eastern most exterior sub-floor walls but otherwise no specific damage has been noted.

  In the crawl space under the ground floor slab there are a number of isolated concrete piles which have separated from the underside of the concrete slab above. The purpose of the piles is unknown and the loss of these isolated support points does not appear to be causing any distress to the ground floor slab above.

- **Distress to Wall and Ceiling Finishes** – Cracking, warping and general distress has been noted to the wall and ceiling linings throughout. The cracking in the gypsum board wall and ceiling linings has typically occurred off the corners of door and window openings, along existing wallboard joints and at the interface between the top of the wall and the ceiling finishes. The warping of the finishes has typically occurred at wall intersections.

  At the east end of the building, in addition to the typical damage noted to the wall and ceiling linings, complete separation of wall and ceiling framing has occurred. Associated damage to the framing connections above the ceiling line has been noted.

- **Heavy Acoustical Ceiling Tile Assembly** – In general, the heavy plaster acoustical ceiling tiles are supported on a series of continuous metal tracks which are directly fixed to timber ceiling framing above. Along the corridor the tiles are typically three tiles in width, with the centre tile securely fixed to the continuous metal tracks. The outer "loose" tiles are typically supported on two sides and span between the tracks. Because the tracks are directly fixed to the ceiling framing, and not suspended, the movement experience by the tracks is roughly limited to the movement experienced by the ceiling framing.

  Most of the ceiling tiles in the Spinal Injuries Unit appeared to be in relatively good condition; however hairline cracks have been noted in several of the tiles in addition to some visible gaps that have opened up between the tiles due to shaking. It is believed that the lack of a contiguous ceiling diaphragm has led to more movement of the ceiling framing, and in turn the tile assembly, than would typically be expected.

  Following the 23rd December 2011 and the 4th January 20012 earthquakes a ceiling tile did dislodge and fall in one of the offices in the southwest end of the building. It is believed the tile was either installed with minimal seating or was gradually shaken loose by the accumulation of the earthquakes over the past year and a half.

  The heavy acoustical ceiling tiles were replaced during 2012 with a contiguous Gib ceiling diaphragm.

- **Site Paving and Structures** – The differential ground settlement noted surrounding the building has resulted in damage to the stone walkways, landscaping and site structures. Pounding damage has also been noted between the veranda and the steel covered walkway at the northeast end of the building.

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. Secondary elements, such as windows and fittings, have not generally been reviewed.

3.7 ADDITIONAL INVESTIGATIONS REQUIRED

Several assumptions were made in the completion of the Pre-earthquake (undamaged state) and Post-earthquake (damaged state) Structural Assessments. Destructive exploration is required in a number of locations in order to verify these assumptions.

3.7.1 Investigations Required For Further Assessment

The areas requiring further investigation to finalize the assessments are as follows:

- Based upon the damage observed further investigations of the exterior block façade is warranted. This includes a summary of its veneers general condition, investigation of damaged mortar joints and a review of the fixings to the exterior timber stud walls. This work should be completed by a qualified Mason and should include any repair recommendations. See items 2.11, 2.12, 3.2, 3.3 and 3.5 in Table 4-1.

The exterior block veneer was reviewed by SA Thelning Brick and Blocklayer in September 2012. This is summarised as follows:

The block veneer is 90mm thick with a 50mm cavity and ties at 450mm horizontally and 600mm vertically. The blockwork is still in very good condition and seems structurally sound. There has been movement around the windows which needs to be resealed. The block window sills have moved and should be ground out and resealed. Mortar joints have cracked throughout and should be ground out and repointed. The east side sills are missing and are to be supplied and installed by others. The brick ties are in overall very good condition. The ties are all tight and intact with the sub framing beyond. No wire mesh to the mortar joints was found.

- Note: Corridor link structure is to be reported separately.

3.7.2 Investigations to be Completed During Building Repairs

- Re-inspection of building will be required upon completion of any re-levelling works, to determine if any additional damage has occurred.

- Check existing timber stud wall framing and fixings to concrete slabs below where new wall linings are to be installed.

3.8 POST-EARTHQUAKE BUILDING CAPACITY

Based upon our investigations to date, we do not consider the Spinal Injuries Unit to have any significant reduction to the overall gravity load resistance of the structure. Localized damage to the gravity load resistance system has occurred but only in the lightly loaded timber framed portion of the building. Nor does the damage noted to date appear to have any significant reduction to the lateral load capacity of the concrete and steel portions of the building (Sections 2 & 3).

At the timber framed portion of the building (Section 1) the damage observed to the gypsum board linings of the bracing walls will have resulted in a reduction in lateral load capacity, although the actual reduction in strength is difficult to quantify. While there has been some reduction in strength, according to the Department of Building and Housings, *Revised Guidance on Repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence* [20], the primarily result of the damage noted will be a reduction in the stiffness of the wall bracing. Based upon the movement observed in the building a similar reduction in stiffness can be expected to the sections of gypsum clad ceiling linings.

The differential settlement observed in the building will also have resulted in a reduction in the overall lateral load resisting capacity of the building. The reduction in capacity will be the greatest at the east end of the building where distress to the timber framed superstructure has occurred. Separation of the wall and ceiling framing has been noted in this area, including damage to ceiling framing connections. A review of the damage noted does not raise any immediate concerns regarding the gravity support of the structure in this area, although repairs associated with the damage will be required.

While the differential settlement noted for the rest of the building is less severe, the settlements noted will have resulted in some reduction to the capacity of the building, along with limiting the ability of the building to absorb future differential settlements before severe distress to the structure occurs.

The damage observed will require repair to restore the strength, stiffness, durability and performance of the individual structural components. The differential settlement noted at the east end of the building will need to be addressed by demolishing and re-building this section of the building, or through re-levelling, to restore the serviceability of the building. The repair work is outlined in Section 4. Following the recommended repair of the structural damage, the lateral load resisting performance of the structure will be restored to approximately pre-earthquakes levels (see Section 2.4).

4. OBSERVED DAMAGE AND REQUIRED RAPAIRS

4.1 PRIMARY DAMAGE OBSERVED AND REPAIRS REQUIRED

This section covers the damaged noted during our detailed assessment of the building. Note that our observations have been restricted to structural aspects of the building 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.

Table 4-1 provides a photographic summary of the observed damage and typical repairs required for the Spinal Injuries Unit. Table 4-1 should be read in conjunction with Appendix A – Record of Observation and Appendix B – Reference Plans. The Repair Specification [2] referred to in Table 4-1 has been issued separately.

In general, the aim of the repair work indicated in this section is to restore the structure to its pre-earthquake state, as far as practicable, while maintaining the utility of the building. The repairs presented attempt to address the loss of strength, stiffness and durability of the structural elements due to the damage noted.

It should be noted that more damage may be identified during the repair works and (if required) additional repair details will be specified accordingly.

Please note that all repair works are to be completed after the building has been re-levelled to a satisfactory condition as further damage to the wall and ceiling linings can be expected during the re-levelling process.

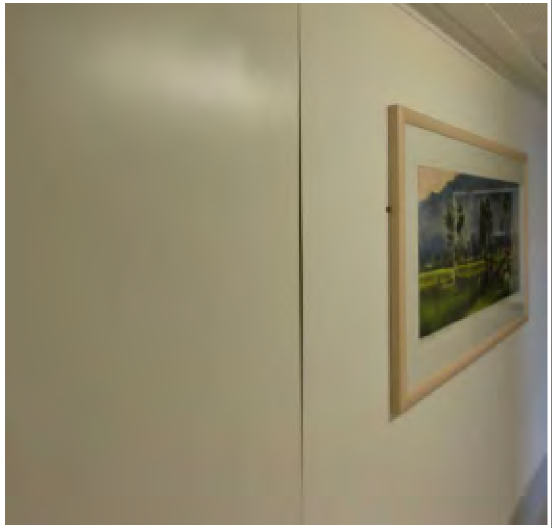
Further recommendations for improvement to the buildings seismic performance, and to achieve a minimum capacity of 67% DBE have been included in Section 5.

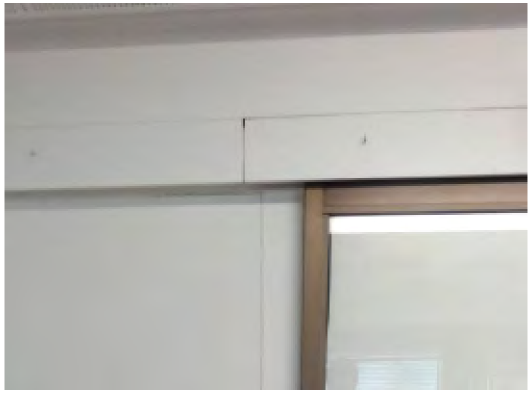
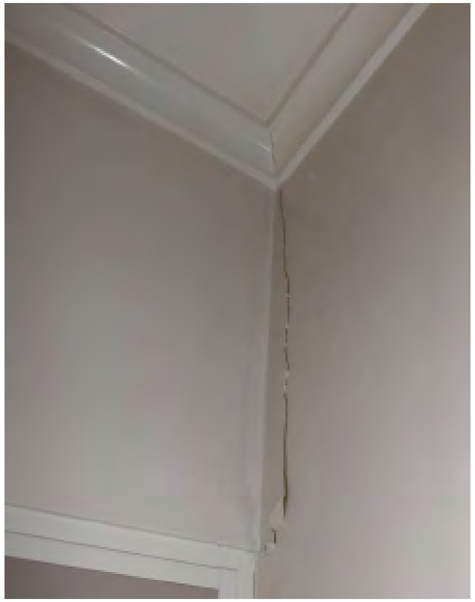
Table 4-1: Photographs of observed damage and repairs required

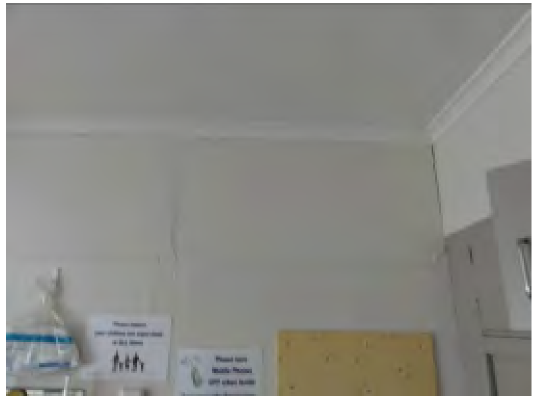
Damaged Item & Location	Damage	Recommendations	Example Photograph
1. Partial basement, concrete service tunnels, sub-floor walls and foundations			
1.1. Differential ground settlement	Differential ground settlement of approximately 175mm resulting in a worst case slope in the ground floor slab of approximately 1.7% (1:60)	The differential settlements noted at the east end of the building will need to be addressed by either demolishing and rebuilding this section of building, or through re-levelling. For further discussion on the remediation work required see Section 4-2. (Note: All re-levelling is to occur prior to any other structural or cosmetic repairs).	
1.2. Movement of soils surrounding sub-floor walls	Visible ground fissures in the crawl space below the ground floor slab and separation of soils from the exterior concrete sub-floor walls	Once re-levelling of the east end of the building has been completed re-compact soil surrounding the exterior sub-floor walls and in the crawl space. For an image of the ground fissures see Figure 3-1.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
1.3. Foundation at interface with link corridor	Vertical crack (10mm) at interface with link corridor on western end of the building.	*Further evaluation of the link corridor structure is required*	
1.4. Precast ground floor framing	Separation noted at joint between Unispan precast floor units (aligns with interface with link corridor above)	*Further evaluation of the link corridor structure is required*	
1.5. Service tunnel slab on grade	Separation noted at joint in service tunnel slab on grade (aligns with interface with link corridor above)	*Further evaluation of the link corridor structure is required. Further evaluation of damage to waterproofing membrane required.*	

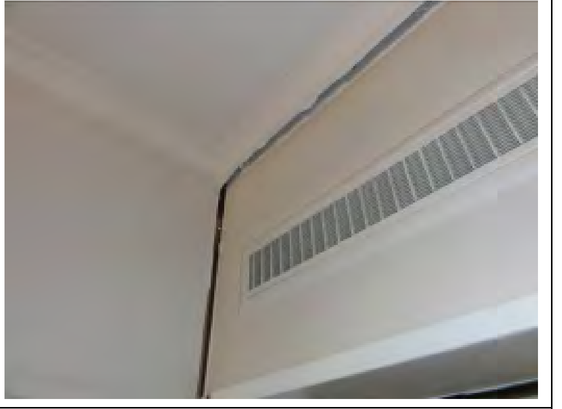
Damaged Item & Location	Damage	Recommendations	Example Photograph
1.6. Service tunnels and concrete sub-floor walls	Minor cracking noted to concrete service tunnel and sub-floor walls	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [3]. For cracks greater than 1mm, HCG to confirm the integrity of the existing reinforcement of the wall. If reinforcement is damaged, an engineered repair will be required.	
1.7. Ground floor slab to sub-floor walls	Movement has been noted interface of Unispan precast floor units and the exterior concrete sub-floor wall on the east end of the building	*Re-inspect joint following completion of re-levelling*	

Damaged Item & Location	Damage	Recommendations	Example Photograph
1.8. Isolated interior concrete piles	In general, the isolated interior concrete piles have settled and separated from the ground floor slab above.	The purpose of the isolated interior piles is unknown and separation noted with the slab above does not appear to have affected the gravity load carrying capacity of the slab.	
2. Timber Framed Structure			
2.1. Interior Wall Linings	Popping of wall linings at existing joint locations. Typical throughout.	Replace damaged wall boards with new gypsum board sheets. All wall boards to remain are to be re-fixed as per Section 4.3. For additional strengthening options, see Section 5.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
2.2. Interior Wall Linings	Separation of wall linings at existing joint locations. Typical Throughout	Replace damaged wall boards with new gypsum board sheets. All wall boards to remain are to be re-fixed as per Section 4.3. For additional strengthening options, see Section 5.	
2.3. Interior Wall Linings	Cracking of wallboard sheathing off corner of door and window openings. Typical throughout.	Replace damaged wall boards with new gypsum board sheets. All wall boards to remain are to be re-fixed as per Section 4.3. For additional strengthening options, see Section 5.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
2.4. Interior Wall and Ceiling Linings	Cracking of wallboard sheathing at existing joint location. Additional cracking noted at interface between wallboard and ceiling linings. Typical throughout.	Replace damaged wall boards with new gypsum board sheets. All wall and ceiling boards to remain are to be re-fixed as per Section 4.3 and Section 4.4. For additional strengthening options, see Section 5.	
2.5. Ceiling Linings	Cracking noted along ceiling at interface of wall and ceiling linings. Typical throughout.	Replace damaged ceiling boards with new gypsum board sheets. All ceiling boards to remain are to be re-fixed as per Section 4.4. For additional strengthening options, see Section 5.	
2.6. Ceiling Linings	Cracking noted along existing ceiling board joints. Typical throughout.	Replace damaged ceiling boards with new gypsum board sheets. All ceiling boards to remain are to be re-fixed as per Section 4.4. For additional strengthening options, see Section 5.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
2.7. Damage to timber walls and ceiling framing at east end of the building	At the east end of the building, the differential ground settlement noted has resulted in the separation of wall and ceiling framing.	Once the building has been re-levelled the existing timber wall and ceiling framing will need to be re-fixed together. Any damaged wall or ceiling boards will also be required to be replaced with new gypsum board sheets. All undamaged wall and ceiling boards to remain are to be re-fixed as per Section 4.4. For additional strengthening options, see Section 5.	
2.8. Damage to timber walls and ceiling framing at east end of the building	At the east end of the building, the differential ground settlement noted has resulted in the separation of wall and ceiling framing.	Once the building has been re-levelled the existing timber wall and ceiling framing will need to be re-fixed together. Any damaged wall or ceiling boards will also be required to be replaced with new gypsum board sheets. All undamaged wall and ceiling boards to remain are to be re-fixed as per Section 4.4. For additional strengthening options, see Section 5.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
2.9. Damage to timber walls and ceiling framing at east end of the building	At the east end of the building, the differential ground settlement noted has resulted in the separation of wall and ceiling framing.	Once the building has been re-levelled the existing timber wall and ceiling framing will need to be re-fixed together. Any damaged wall or ceiling boards will also be required to be replaced with new gypsum board sheets. All undamaged wall and ceiling boards to remain are to be re-fixed as per Section 4.4. For additional strengthening options, see Section 5.	
2.10. Heavy plaster ceiling tile assembly	Movement and separation noted in joints of ceiling tiles, in addition to hairline cracking.	Replace existing heavy tile assembly with a new hard lid gypsum board ceiling. See Section 4.4 for additional information. 20/11/13 Existing heavy tile ceilings were replaced during 2012 with gypsum board ceilings.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
2.11. Typical Exterior Soffit	Separation of exterior soffit linings noted at interface with the exterior block veneer.	At all exterior soffit linings re-fix as per Section 4.4. For additional strengthening options, see Section 5. *Further investigation of connection between the block veneer and the ceiling framing is required.* 20/11/13 As outlined in Section 3.7, the veneer is tied to the timber wall framing at 450mm vertical crs and 600mm horizontal crs. No specific reference to the connection between the block and ceiling framing is made.	
2.12. Exterior Veranda Soffit (north face of building)	Cracking noted in exterior soffit lining of corner of wall piers, at interface with block veneer and along existing joints.	Replace existing exterior soffit lining with new fibre cement board or exterior grade plywood sheathing. For further information on the recommended repairs see the discussion included in Section 4.4. *In conjunction with the repairs an investigation of connection between the block veneer and the ceiling framing is required.*	

Damaged Item & Location	Damage	Recommendations	Example Photograph
3. Exterior Block Veneer			
3.1. Exterior Block Veneer at South End of Partial Basement	Cracking noted in toe of concrete block veneer, in addition to the opening of an existing crack in concrete lintel over the door opening (in the plane of the block veneer)	Locally replace damaged block veneer. At concrete lintel break away a section of the lintel and re-cast.	 

Damaged Item & Location	Damage	Recommendations	Example Photograph
3.2. Block Veneer Intersections	Vertical crack at intersections of exterior block veneer	Repair of seal to be specified by others. *Further investigation required to verify connection between sections of block veneer.*	
3.3. Interface of Block Veneer and exterior stud walls	Vertical crack at interface between exterior block veneer and stud wall framing noted.	Repair of seal to be specified by others. *Further investigation required to verify connection between block veneer and exterior timber stud wall.* 20/11/13 As outlined in Section 3.7, the veneer is tied to the timber wall framing at 450mm vertical crs and 600mm horizontal crs.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
3.4. Block Veneer to Bottom of Window Opening	Cracking noted at the interface between the top of the block veneer and the underside of the window opening.	Remove loose and damaged mortar and re-point joint.	
3.5. Base of Block Veneer to Exterior Concrete Sub-floor Walls	Cracking noted at the interface between the base of the block veneer and the concrete sub-floor walls.	Epoxy inject cracks in the sub-floor walls between 0.2mm and 1mm, in accordance with HCG specification. Note: repair of damage to exterior tiles to be specified by others. *Further investigation of the bond between the block veneer and the concrete sub-floor walls is required.*	

Damaged Item & Location	Damage	Recommendations	Example Photograph
4. Interior Floor Finishes & Miscellaneous Non-Structural Items			
4.1. Floor Finishes	Damage to threshold and flooring noted at west corridor entrance to Spinal Injuries Unit.	Repair specifications to be provided by other. *Further evaluation of the link corridor structure is required*	
4.2. Floor Finishes	Cracking of floor finishes of wall intersections	Repair specifications to be provided by other.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
4.3. Floor Finishes	Warping of floor finishes at base of walls. Typical at east end of the building	Repair specifications to be provided by other.	
4.4. Damage to door and window frames	The differential ground settlement noted at the east end of the building has warped and damaged several door and window frames.	Repair specifications to be provided by others.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
5. Exterior Pathways and Site Structures			
7.1. Exterior walkways and landscaping	Extensive differential settlement induced damage noted to exterior walkways and landscaping	Repair exterior walkways and landscaping. Repair specifications to be provided by other.	 

Damaged Item & Location	Damage	Recommendations	Example Photograph
7.2. Exterior Drains	Ground settlement induced damage to exterior drains	Repair of exterior drains required. Repair specification is to be provided by others.	
7.3. Interface of Exterior Veranda and Steel framed Covered Walkway (Northeast corner of the building)	Cracks noted to partition walls and ceiling finishes.	Provide aesthetic repairs to facia and soffit. Repair specification to be provided by others. Consideration also to be given to increasing the size of the joint between the structures. Additional investigation and evaluation of the steel covered is still required.	

4.2 DISCUSSION ON DIFFERENTIAL SETTLEMENT REMEDIATION

The level survey, completed by Fox & Associates, has indicated differential ground settlement of approximately 175mm across the length of the ground floor slab. The worst differential settlement is concentrated at the east end of the building (see Appendix C for complete level survey) and has resulted in permanent slopes in the elevated ground floor slab of up to 1.7% (1:60). This slope is well outside the typical acceptable range and will need to be addressed in order to restore the function of the building.

This can either be addressed by demolishing and reconstruction of this portion or through re-levelling. If demolition and reconstruction is chosen the east wing would be rebuilt as a whole with a seismic joint located at the interface with the portion of the building to remain.

If re-levelling is chosen the east end of the building would be proposed to be lifted up to the highest point of the building which is located roughly in the centre of the concrete framed portion of the building. For the extent of the proposed re-levelling see Figure 4-1 below. This would address the most severe slopes in the elevated ground floor slab at the east end of the building, along with most of the moderate slopes noted in the slab on the order of 1:300 (see Appendix C).

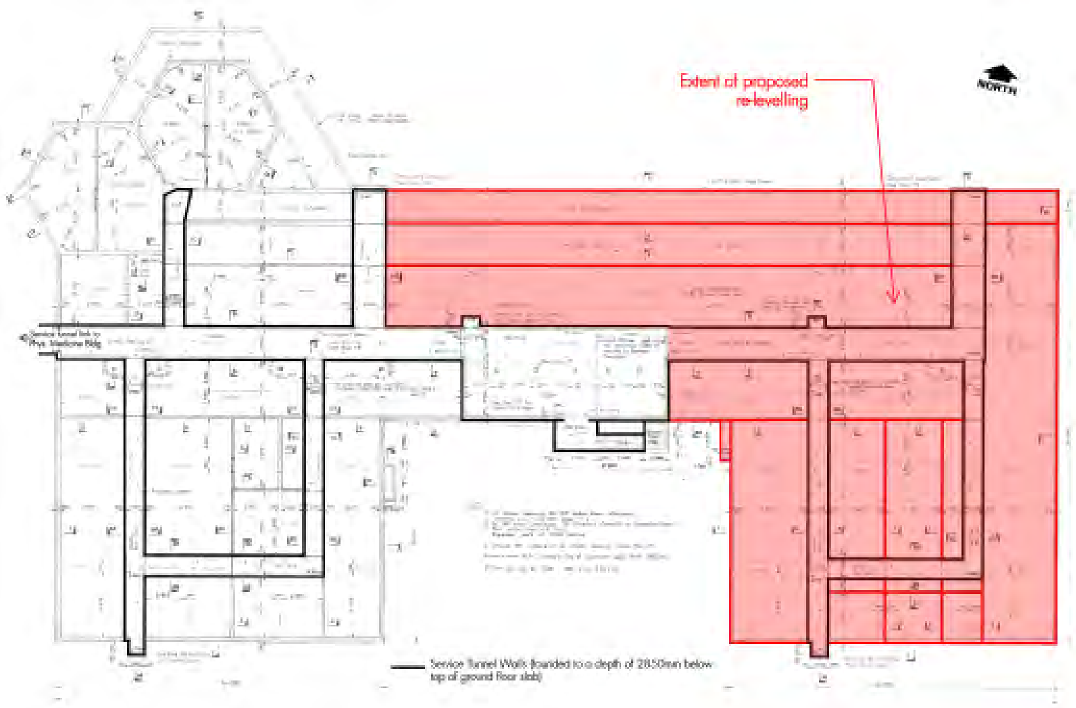


Figure 4-1: Foundation Plan – Damage Repairs

The two primary re-levelling options available include the use of mechanical jacking or the use of either underpinning grout or engineered resin. There are pro's and con's of each solution which extend beyond structural performance which will need to be considered by CDHB. These include continuity of operation, degree of re-levelling accuracy and the willingness of the re-levelling sub-contractor to provide a producer statement, amongst other items.

From a structural standpoint, either option is acceptable provided the use of underpinning grout or engineered resin does not create any "hard points" under the building. If "hard points" are created during the re-levelling process the potential for future differential settlements can be increased. If this were to occur it would reduce the capacity of the building going forward.

Based upon the information provided by Tonkin & Taylor the soil profile under the Spinal Injuries Unit (medium dense sand overlying dense sand) lends itself to localized lifting through underpinning grout or engineered resin techniques and should not create any undesirable "hard points" as described above.

The building also lends itself nicely to the use of mechanical jacking due an elevated ground floor slab and the relatively good shape of the exterior and interior concrete sub-floor walls in this area. The exterior sub-floor walls are typically roughly 1 metre in depth, heavy reinforced and well detailed, and should easily span between jacking locations placed under the sub-floor walls.

The suitability of re-levelling the building through the use of either mechanical jacking or underpinning grout (or engineered resin) will need to be verified by qualified sub-contractors in conjunction with the geotechnical consultant.

It should be noted that both options discussed above are not expected to increase the seismic performance of the building or reduce the potential for future differential settlements. Instead the options presented are designed to re-level the building without making the future performance of the building any worse than it was prior to the earthquakes. To improve the future performance of the building, and reduce the potential for future differential settlements, would likely require the entire footprint of the building to be either piled or the ground under all the sub-floor wall footings, service tunnels and the partial basement improved. *Further geotechnical investigations would be required into the type and depth of ground improvement required.*

Based up the geotechnical report provided by Tonkin & Taylor [11] the potential for future total and differential settlements at the building site varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

During the re-levelling process there is also the risk that addition damage could occur to the building linings, exterior block veneer, etc. Appropriate contingencies should be provided.

4.3 REPAIR OF TIMBER FRAMED BRACING WALLS

The wall linings to the interior and exterior bracing walls have been damaged throughout the building and require repair. While the damage to the fixings may not be obvious, based upon the movement observed, we believe there has been a reduction to the ongoing strength and stiffness of all the bracing walls. In order to reinstate the pre-earthquake strength and stiffness to the bracing walls, the repair recommendation is to remove all cracked or damaged sections of the wall linings and replace them with new gypsum board sheathing. The new gypsum board sheathing is to be fixed in accordance with GIB 'ezybrace' GS2-N specifications (or equivalent). All existing internal wall linings to remain are to be re-fixed to the existing studs in a similar manner. Any non-gypsum wall boards will need to be replaced in conjunction with these repairs. A new finish is then to be applied to all interior walls.

All repairs to wall bracing are to be completed after the re-levelling of the east end of the building has been completed.

Note: The fixings of the walls to the timber framing below will need to be checked for damage and the ability to transfer the new bracing loads.

Figure 4-2: Ground Floor Plan – Damage Repairs

4.4 REPAIR OF GYPSUM BOARD CEILINGS

Similarly to the wall linings, the existing sections of gypsum clad ceiling diaphragms and their fixings have been damaged and require repair to reinstate their pre-earthquake strength and stiffness. The repair recommendation is to remove any cracked or damaged sections of gypsum board ceiling lining and replace them with new gypsum board sheathing fixed in accordance with GIB specifications. All existing ceiling linings that to remain are to be re-fixed to existing ceiling joists in a similar manner. A new finish is then to be applied to all ceilings.

All repairs to the ceiling diaphragms are to be completed after the re-levelling of the east end of the building has been completed.

4.5 REPAIR OF EXTERIOR SOFFITS

The exterior soffits linings and their fixings have been damaged and require repair. The typical repair recommendation, to reinstate the strength and stiffness of the soffit linings, is to remove and replace any cracked or damaged sections with in kind material. Any existing linings to remain are also to be re-fixed to the soffit framing above.

The concentration of damage to the exterior soffits has occurred at the exterior veranda on the north end of the building. At this location all the soffit linings are to be removed and replace with new fibre cement board (or exterior grade plywood) fixed to the soffit framing above. As a by product, the new gypsum board soffit linings will better distribute lateral loads to the exterior wall line on the north end of the building.

Once this work is complete a new finish is to be applied to all the exterior soffits. All repairs to the soffits are to be completed after the re-levelling of the east end of the building has been completed.

5. STRENGTHENING RECOMMENDED

The Spinal Injuries Unit can be separated into three primary sections; a single storey timber framed portion (building section 1), a central two storey concrete plant structure (building section 2) and a steel portal framed portion (building section 3). As noted in Section 2, Pre-Earthquake Building Condition, and Section 3, Post-Earthquake Building Condition, the lateral load resisting capacity of each section of the building has been assessed as a percentage of the loads imposed by the Design Basis Earthquake (DBE).

The pre- and post-earthquake capacity of the concrete and steel portions of the structure have been assessed at a capacity of approximately 100% DBE and 60% DBE respectively. The structure below the ground floor has been assessed at a capacity above 100% DBE.

The original heavy acoustical ceiling tiles were replaced during 2012 with a contiguous Gib ceiling diaphragm to allow lateral loads to be more evenly distributed to the existing bracing walls. The timber framed portion of the building has an upgraded assessed capacity of approximately 85% DBE in the north-south direction and approximately 55% DBE in the east-west direction. The limiting factor on the %DBE is now the capacity of the exterior bracing along the north wall.

Additional recommended strengthening to achieve a capacity of 67% DBE, and improve the overall seismic performance of the building, have been included in sub-sections below.

5.1 STRENGTHENING WORKS TO ACHIEVE 67% DBE

Exterior Wall Bracing – Along the north end of the building the existing wall bracing provided does not meet the minimum requirements of NZS 3604:2011 [13]. In order to achieve the minimum wall bracing requirements, and increase the assessed capacity of the building, the wall bracing along this end of the building will need to be strengthened. The proposed strengthening would be to replace the inside face of the exterior wall linings with new plywood sheathing. Additional fixings to the concrete sub-floor walls would also be required in conjunction with the strengthened bracing walls. This would include additional fixings to the elevated ground floor slab and addition holdowns at either end of the wall openings. The proposed additional bracing locations have been included in Figure 5-1 below.

Steel Portal Frame – The steel portal frames have been assessed at 60% DBE for drift and at 75% DBE for strength. In order to improve the seismic performance of this section of the building the base of the columns could be fixed through the installation of additional fixings to the concrete sub-floor walls below. This would result in reducing the drift of the portal frames and as a result increase the assessed %DBE of the building.

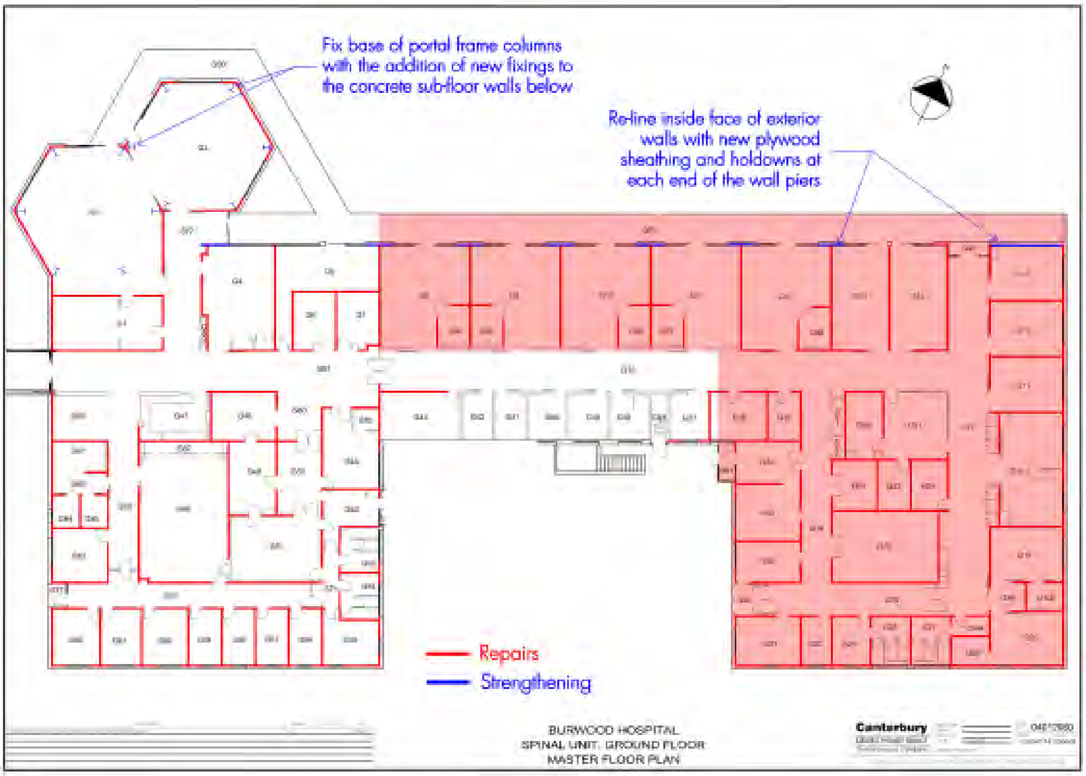


Figure 5-1: Ground Floor Plan – Strengthening Recommended

6. REFERENCES

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2. *Burwood Hospital – Detailed Seismic Assessment Report – Earthquake Repair Specification*, Holmes Consulting Group, July 2011.

3. *Additions Burwood Hospital Christchurch – Spinal Injury Unit,* Original architectural drawings, Cutter, Pickmere, Douglas and Partners Architects, October 1975.

4. *Burwood Hospital – Spinal Injury Unit,* Original structural drawings, Fredderick Sheppard and Partners Consulting Engineers, September 1975

5 *Burwood Hospital Post Earthquake Geotechnical Assessment*, Tonkin and Taylor Ltd., June 2011.

6 *Burwood Elevation Survey – Revision E*, Fox & Associates, January 2012

7 *Burwood Hospital Campus – Seismic Risk Assessment Report*, Holmes Consulting Group, April 2002

8 *Burwood Hospital Campus – 2007 Seismic Risk Assessment Update,* Holmes Consulting Group, June 2007

9 *Compliance Document for New Zealand Building Code - Clause B1 – Structure, Amendment 10 (Canterbury)*, Department of Building and Housing, Wellington, 19 May 2011.

10 *Structural Design Actions Part 5: Earthquake Actions – New Zealand, NZS 1170.5:2004*, Standards New Zealand, 2004.

11 *Code of Practice for General Structural Design and Design Loadings for Buildings*, NZS4203:1992, Standards New Zealand, 1992

12 *New Zealand Standard Model Building Bylaw – Chapter 8 Basic Design Loads*, NZSS1900:1965, New Zealand Standards Institute, 1965

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

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

18 *Seismic Rehabilitation of Existing Buildings,* ASCE 41-06, American Society of Civil Engineers, 2007

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21 *Practice Note – Design of Conventional Structural Systems Following the Canterbury Earthquakes*, SESOC, December 2011

22 *CDHB – Burwood Hospital Campus – Rapid Visual Inspection: 106186.03 Site Report 1,* Holmes Consulting Group, February 2011

23 *CDHB – Burwood Hospital – Rapid Seismic Assessment Post June 13th Earthquakes: 106186.03 Site Report 4,* Holmes Consulting Group, 14 June 2011

24 *CDHB – Burwood Hospital – Rapid Seismic Assessment Post June 13th Earthquakes: 106186.03 Site Report 5,* Holmes Consulting Group, 15 June 2011

24 *CDHB – Burwood Hospital – Post Earthquake Rapid Structural Assessment following 5.8 and 6.0 Magnitude Earthquakes, 106186.03 Site Report* 8, Holmes Consulting Group, 24 December 2011

25 *CDHB – Burwood Hospital – Post Earthquake Rapid Structural Assessment following 2nd January 5.5 Magnitude Earthquakes, 106186.03 Site Report 9*, Holmes Consulting Group, 9 January 2012

26 *CDHB – Burwood Hospital – Heavy Ceiling Tiles, 106186.03 Site Report 12*, Holmes Consulting Group, 31 January 2012

APPENDIX A

Record of Observations

APPENDIX A – RECORD OF OBSERVATIONS & REPAIRS

Inspection date: 27 April, 2011

KEY	
N	No repair required
Y	Repair required
F	Further investigation required
C	Repair complete

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	East wall	Foundations	Soil surrounding building has been compacted with building movement or subsided with liquefaction.	**Y**	Once building has been re-levelled, re-compact soil around the existing sub-floor walls	6794-6800, 340-345
G	East wing	Roof Framing	Roof framing inspected above corridor to determine if perimeter foundation movement is associated with damage to the roof structure. Trusses extending to external wall still in full bearing with girder truss, no movement or damage visible.	**N**		346-351
G	East wing	Wall	Internal cladding of wall to ceiling repaired since EQ.	**Y**	Replace damaged wall boards with new gypsum board sheets. All wall boards to remain are to be re-fixed.	-

APPENDIX A – RECORD OF OBSERVATIONS & REPAIRS

Inspection date: 20 October 2011

KEY	
N	No repair required
Y	Repair required
F	Further investigation required
C	Repair complete

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	Typical Damage	Foundations	Generally in good condition except as noted. Settlement of backfill material on East Wall is apparent.	**Y**	Once building has been re-levelled, re-compact soil around the existing sub-floor walls	-
G		Walls (TYP)	Cracking in internal partition walls at locations of joints between cladding panels. Typical throughout building especially corridors. Heavy damage to timber framed walls in East Wards noted in report.	**Y**	Replace damaged wall boards with new gypsum board sheathing. All wall boards to remain are to be re-fixed. At locations where timber wall and timber framing and/or fixings have been damaged they are to be replaced in kind	-
G		Floors (TYP)	Typically no noticeable steps in flooring except as noted. Typically no significant falls in slab able to be determined by sight, except in Eastern Ward. No significant cracking to soffit of pre-cast planks when observed from underneath	**Y**	Re-level east end of the building and replace damaged floor linings where they occur	-
G		Ceilings	Minor cracking of cornices at wall and ceiling interfaces. Typical to flat ceilings throughout wards	**Y**	Repair specification by others	-

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
S	Typical Damage	Service Tunnels	Diagonal hairline cracks to penetrations in walls typical throughout tunnels. Differential movement noted at building interface.	**Y**	Epoxy inject all cracks in the slab between 0.2mm & 1mm per the HCG specification. If cracks of greater than 1mm are observed in the walls advice HCG for addition inspection	-
S		Basement Plant	No significant damage noted	**N**		-
1		Level 1 Plant Room	No significant damage noted	**N**		-
G	G81	Concrete Foundation	Vertical crack to reinforced concrete foundation at interface between BSU and link corridor, approximately 10mm	**F**	*Further investigation of the corridor link structure is required*	899
G	G78	Ceiling	1.5mm tapered diagonal crack to sheeting.	**Y**	Replace damaged ceiling linings with new gypsum board sheathing. All ceiling linings to remain are to be re-fixed.	900
G	G78 (typ)	Cornice	Movement of joinery adjacent to doors has been re-screwed in some locations.	**Y**	Repair specification by others	901
G	G78	Ceiling and Door Jambs	Gap between walls and doors, approximately 10mm. 1.5mm tapered diagonal crack to ceiling each side of door. Stretching of ceiling joints noted adjacent to door.	**Y**	Replace damaged ceiling linings with new gypsum board sheathing. All ceiling linings to remain are to be re-fixed. At locations where timber wall and timber framing and/or fixings have been damaged they are to be replaced in kind	902, 903

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G78	Verandah Fascia Beam	'Pounding' at interface of verandah and new steel framed walkway. Flashing of verandah appears to be bent locally.	**Y**	Provide aesthetic repairs to facia and soffit. Repair specification to be provided by others. Consideration also to be given to increasing the size of the joint between the structures. Additional investigation and evaluation of the steel covered is still required.	904
G	Courtyard	Paving	Subsidence of paving adjacent to bin, verandah and walkway column foundations. 20mm crack through asphalt paving. Settlement between paving and verandah slab, approximately 50mm	**Y**	Repair specification by others	905, 906
G	East Wall	Foundation	Cracking at change in level of foundation (seen in step of block veneer). Appears to be a discontinuous pour joint. Settlement of soil adjacent to foundation noted.	**Y**	Epoxy inject all cracks in the slab between 0.2mm & 1mm per the HCG specification. If cracks of greater than 1mm are observed in the walls advice HCG for addition inspection.	907
G	Building Perimeter	Damp Proof Course (TYP)	Horizontal crack along DPC	**Y**	Re-point exterior block veneer joint as required.	908
G	East Wall	Concrete Masonry, Tiles	Horizontal crack at 3rd course. 3 dislodged tiles; dislodged tiles typical to perimeter of building.	**Y**	Repair tiles and re-point exterior block veneer as required.	908, 909
G	East Wall	Foundation Wall	Settlement of backfill material adjacent to and underneath corbel. Review of documentation shows foundation wall is detailed as retaining and the settled material is considered backfill.	**Y**	Once building has been re-levelled, re-compact soil around the existing sub-floor walls	910, 911

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G85	Pavement	Access ramp has settled differentially to entry slab. Step has formed at interface, approximately 20mm.	**Y**	Once building has been re-levelled, demolish and replace the ramp to the correct finished height	912
G	G83	Wall and Foundation	5mm diagonal crack to lintel. 1.5mm tapered diagonal crack to reinforced concrete masonry wall, the comer of one of the block has spalled. The crack propagates through the DPC and concrete step at the door.	**Y**	Locally replace damaged block veneer.	913-916
G	South Wall	Pavement	Settlement of pavement adjacent to building, approximately 30-50mm	**Y**	Repair specification by others	917
G	South Wall	Concrete Block Wall	Vertical crack in wall propagating from foundation through to the lintel. Crack occurs at an apparent reinforced concrete internal wall which forms part of the plant rooms structure.	**Y**	At concrete lintel break away a section of the lintel and re-cast.	918
G	West Wing	Window grout	Horizontal crack in grout to the underside of the window	**Y**	Re-point exterior block veneer joint as required.	919
G	Link Corridor	Wall joint	Differential settlement at wall joint between link corridor and BSU. Approximately 10mm vertical movement.	**F**	*Further investigation of the corridor link structure is required*	920
G	G81	Floor Plate	Metal plate has dislodged over joint between structures. Horizontal and vertical movements between structures appear evident. Approximately 10mm horizontal to the North Wall and 10mm Vertical to the South Wall.	**Y**	Repair adjacent floor linings and cover plate	921
G	G2	Concrete Slab	Minor 'bump' in floor slab, any cracking is concealed by vinyl floor covering	**F**	*Removal of the floor finishes is required to investigate the precast floor slab below*	922

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G72, G69, G81, G73	Internal Walls (TYP)	Cracking in internal partition walls at locations of joints between cladding panels. Typical throughout building especially corridors.	**Y**	Replace damaged wall boards with new gypsum board sheathing. All wall boards to remain are to be re-fixed.	924, 925, 926, 931
G	G87	Ceiling/Services	Services appear to have moved around relative to ceiling grid, some damage to ceiling	**Y**	Replace heavy acoustical tile assembly with a new gypsum board ceiling	927
G	G87	Floor	Minor step in floor. Step noted at joint in carpet and was covered by carpet at the time of inspection	**F**	*Removal of the floor finishes is required to investigate the precast floor slab below*	928
G	G71	Floor	Crack in vinyl floor covering	**Y**	Repair specification by others	929
G	G45	Ceiling	Crack between cornice and ceilings.	**Y**	Replace damaged ceiling linings with new gypsum board sheathing. All ceiling linings to remain are to be re-fixed.	930
G	G37	Ceiling	Crack between cornice and ceiling	**Y**	Replace damaged ceiling linings with new gypsum board sheathing. All ceiling linings to remain are to be re-fixed.	932
G	G36	Wall	Vertical crack to wall at bulkhead	**Y**	Replace damaged wall boards with new gypsum board sheathing. All wall boards to remain are to be re-fixed.	933
G	G36	Ceiling	Crack between cornice and ceiling	**Y**	Replace damaged wall and ceiling linings with new gypsum board sheathing. All ceiling linings to remain are to be re-fixed.	934
G	G36	Wall	Vertical crack to wall	**Y**	Replace damaged wall boards with new gypsum board sheathing. All wall boards to remain are to be re-fixed.	935

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G73	Ceiling	Cracks propagating from corner of wall	Y	Replace damaged ceiling linings with new gypsum board sheathing. All ceiling linings to remain are to be re-fixed.	936
G	G33	Cornice	Cornice separated form wall	Y	Replace damaged ceiling linings with new gypsum board sheathing. All ceiling linings to remain are to be re-fixed. At locations where timber wall and timber framing and/or fixings have been damaged they are to be replaced in kind	937
G	G75	Cornice	Cracking between wall, cornice and ceiling	Y	Replace damaged wall and ceiling linings with new gypsum board sheathing. All ceiling linings to remain are to be re-fixed.	938
G	G20	Wall and Cornice	Vertical crack to wall at door head. Cornice separated from wall	Y	Replace damaged wall boards with new gypsum board sheathing. All wall boards to remain are to be re-fixed. At locations where timber wall and timber framing and/or fixings have been damaged they are to be replaced in kind	939
G	G19	Door and joinery	Door jamb out of skew and will not close properly. 10mm gap in adjacent joinery	Y	Remove and replace door frame. Check timber stud and fixings for damage and replace in knid as required	940, 941
G	G18	Wall, Cornice, Joinery	Cracking to partition wall and cornices. Joinery supporting corridor has cracked at ceiling	Y	Replace damaged wall and ceiling linings with new gypsum board sheathing. All wall and ceiling linings to remain are to be re-fixed. At locations where timber wall and timber framing and/or fixings have been damaged they are to be replaced in kind	942, 943

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G17	Wall	Tapered cracking between N-S and E-W wall has been repaired with putty.	**Y**	Replace damaged wall and ceiling linings with new gypsum board sheathing. All wall and ceiling linings to remain are to be re-fixed. At locations where timber wall and timber framing and/or fixings have been damaged they are to be replaced in kind	944
G	G17	Floor	Delamination of vinyl floor covering adjacent to re-lined GIB wall	**Y**	Remove floor and wall linings and check for damage to the timber studs and fixings to the ground floor slab below. Repair specification of floor finishes to be provided by others	945
G	G17	Wall	Delamination of wall paper at corner of room	**Y**	Replace damaged wall and ceiling linings with new gypsum board sheathing. All wall and ceiling linings to remain are to be re-fixed. At locations where timber wall and timber framing and/or fixings have been damaged they are to be replaced in kind	946
G	G15	Ceiling	Movement between services and ceiling causing damage to ceiling	**Y**	Replace damaged ceiling linings with new gypsum board sheathing. All ceiling linings to remain are to be re-fixed.	947
G	G15	Wall	20mm crack between ceiling and wall. Vertical crack in wall propagating from service penetration. E-W wall has been re-lined with GIB in 2002.	**Y**	Replace damaged wall and ceiling linings with new gypsum board sheathing. All wall and ceiling linings to remain are to be re-fixed. At locations where timber wall and timber framing and/or fixings have been damaged they are to be replaced in kind	948

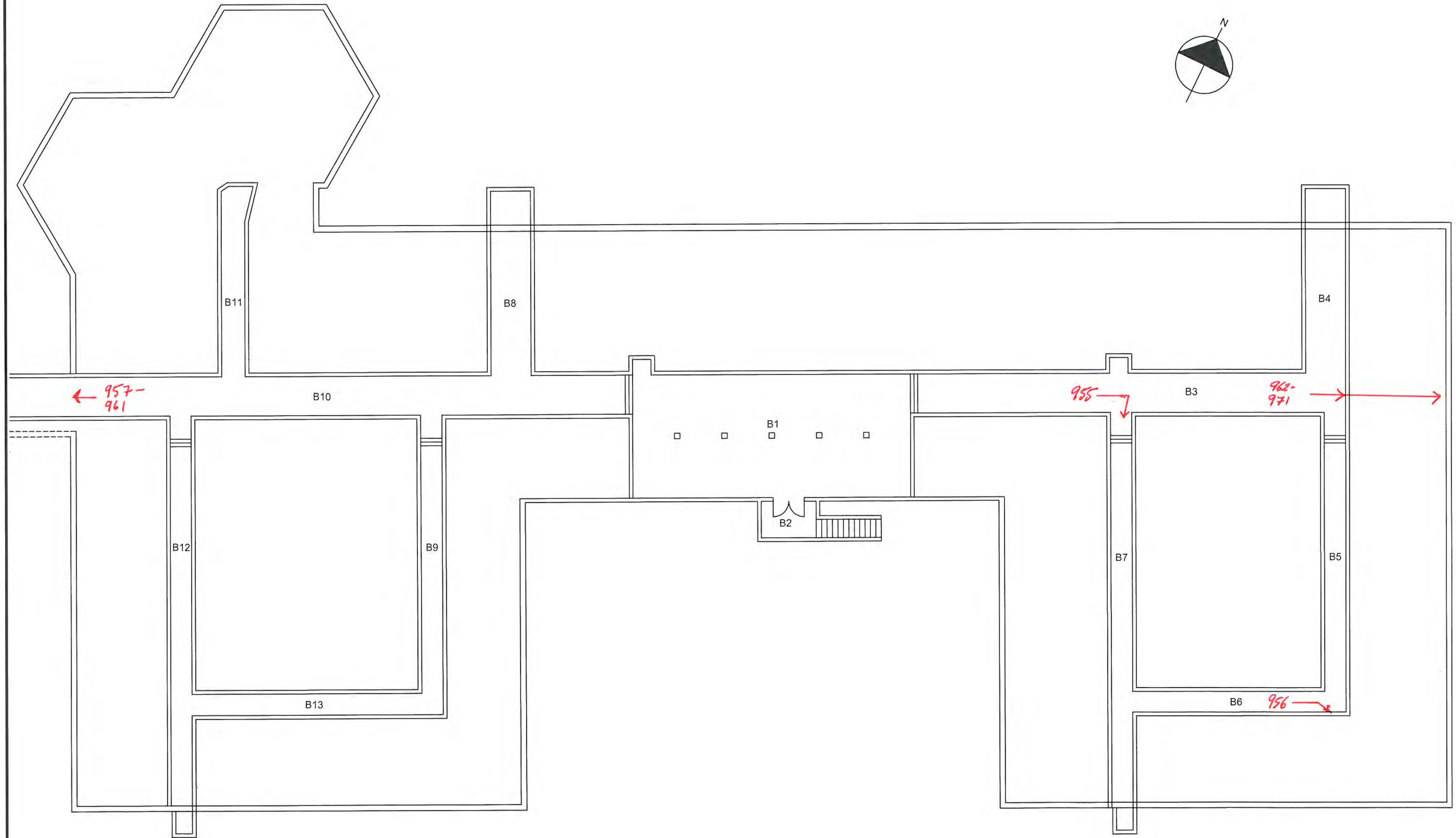
Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G15	Wall	Vertical crack in wall	**Y**	Replace damaged wall boards with new gypsum board sheathing. All wall boards to remain are to be re-fixed.	949
G	G16	Wall	20mm crack between ceiling and wall. 20mm crack between N-S and E-W wall. E-W walls were re-lined with GIB in 2002.	**Y**	Replace damaged wall and ceiling linings with new gypsum board sheathing. All wall and ceiling linings to remain are to be re-fixed. At locations where timber wall and timber framing and/or fixings have been damaged they are to be replaced in kind	950, 951, 954
G	G16	Floor	Delamination of vinyl floor covering adjacent to wall. Vertical crack in wall at location of delaminated vinyl.	**Y**	Remove floor and wall linings and check for damage to the timber studs and fixings to the ground floor slab below. Repair specification of floor finishes to be provided by others	953
G	G16	Ceiling	Movement between services and ceiling causing damage to ceiling	**Y**	Replace damaged ceiling linings with new gypsum board sheathing. All ceiling linings to remain are to be re-fixed.	952
S	B3	Lintel/Wall	Spalling of concrete from wall at bearing end of lintel supporting pre-cast concrete plank flooring. Lintel appears in sound condition	**Y**	Patch as required to reinstore concrete cover	955
S	B6	RC Retaining Wall (TYP)	Diagonal hairline cracks propagating from penetrations in walls. Typical to service tunnels	**Y**	Epoxy inject all cracks in the slab between 0.2mm & 1mm per the HCG specification. If cracks of greater than 1mm are observed in the walls advice HCG for addition inspection.	956, 957
S	B10	Construction Joint	Horizontal and Vertical Movement to service tunnel at location of interface between BSU and Link Corridor. Corresponds to damage noted in Photos: 899, 920, 921	**F**	*Further investigation of the corridor link structure is required*	958-961

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
S	B4	Backfill	Settlement of backfill away from wall (viewed from sub-floor space)	**Y**	Once building has been re-levelled, re-compact soil around the existing sub-floor walls	962, 963, 964
S	B4	Paving	Cracking at pour joint in access pavement in sub-floor. Slab appears unreinforced across joint	**N**		965, 966
S	B4	Soil	Crack in soil in sub-floor of building. Appears to continue across width of building.	**Y**	Once building has been re-levelled, re-compact soil	967-970
S	B4	Paving	Separation between retaining wall and access paving	**Y**	Repair specification by others	971
G	G69	Truss/Wall	Typical truss connection (West Wing)	**N**		973, 974
G	G19	Truss/Wall	Typical truss connection (East Wing)	**N**		975

APPENDIX B

Reference / Key Plans

PHOTO REFERENCE MAP.
JOE. J. 27.10.11



AMENDMENT	DRAWN	CHECKED	DATE
MASTER DRAWING CREATED FROM 04000003 AND CDHB BORDER ADDED	BJT		12-06-09

BURWOOD HOSPITAL
SPINAL UNIT, BASEMENT
MASTER FLOOR PLAN

Canterbury
District Health Board
Te Poari Hauora ō Waitaha

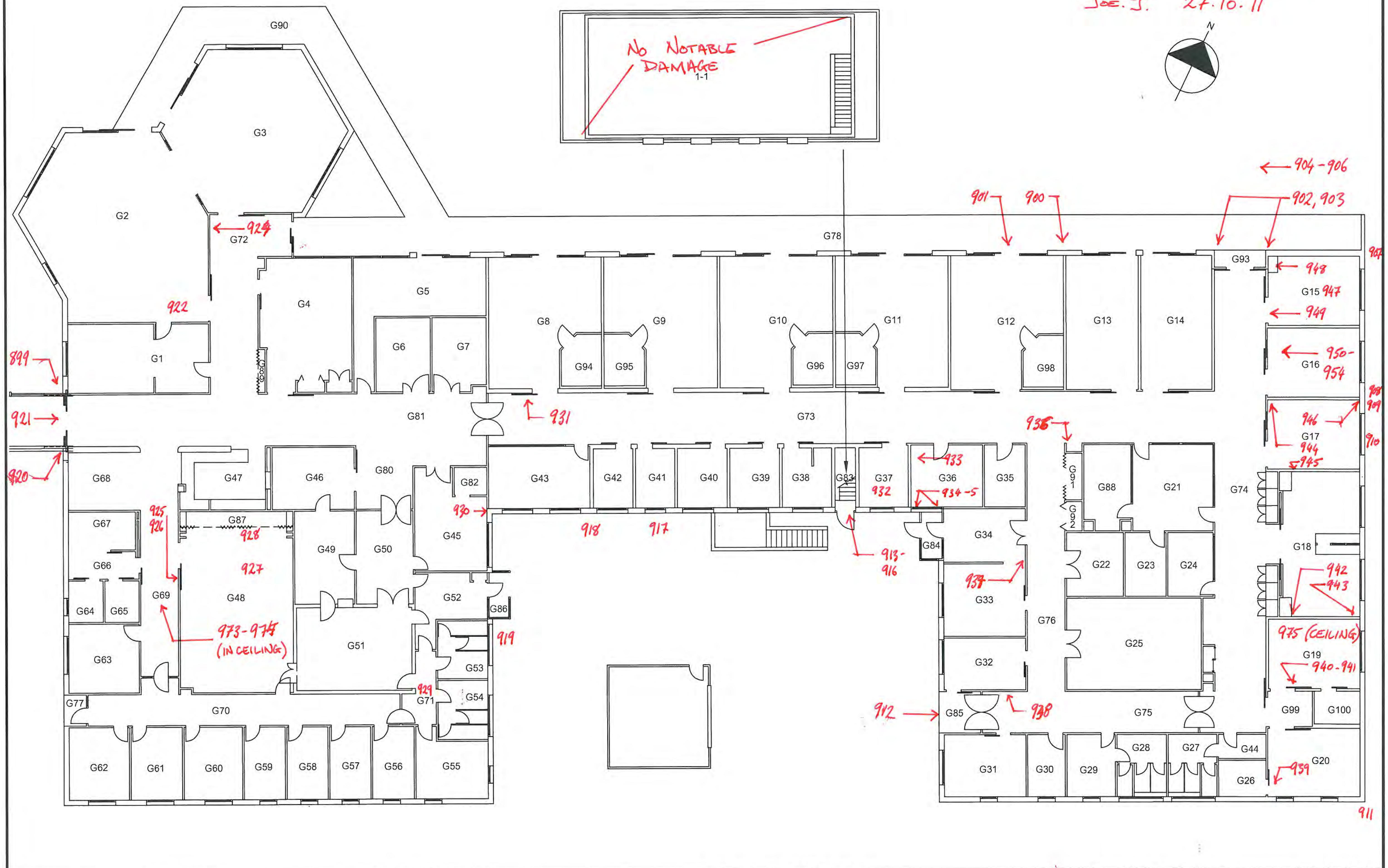
DESIGNED:
DRAWN: BJT
CHECKED:
DATE: 12-06-09

DWG No. 04011080
SCALE 1:100 AT A1
REVISED 12-06-09

XREFED DRAWING/S:

Maintenance and Engineering Department - Christchurch Hospital

PHOTO REFERENCE MAP.
JOE. J. 27.10.11



AMENDMENT	DRAWN	CHECKED	DATE
MASTER DRG CREATED FROM 04000003 AND CDHB BORDER ADDED	BJT		12-06-09

BURWOOD HOSPITAL
SPINAL UNIT, GROUND FLOOR
MASTER FLOOR PLAN

Canterbury District Health Board
Te Poari Hauora ō Waitaha

DESIGNED:
DRAWN: BJT
CHECKED:
DATE: 12-06-09
XREFED DRAWINGS:

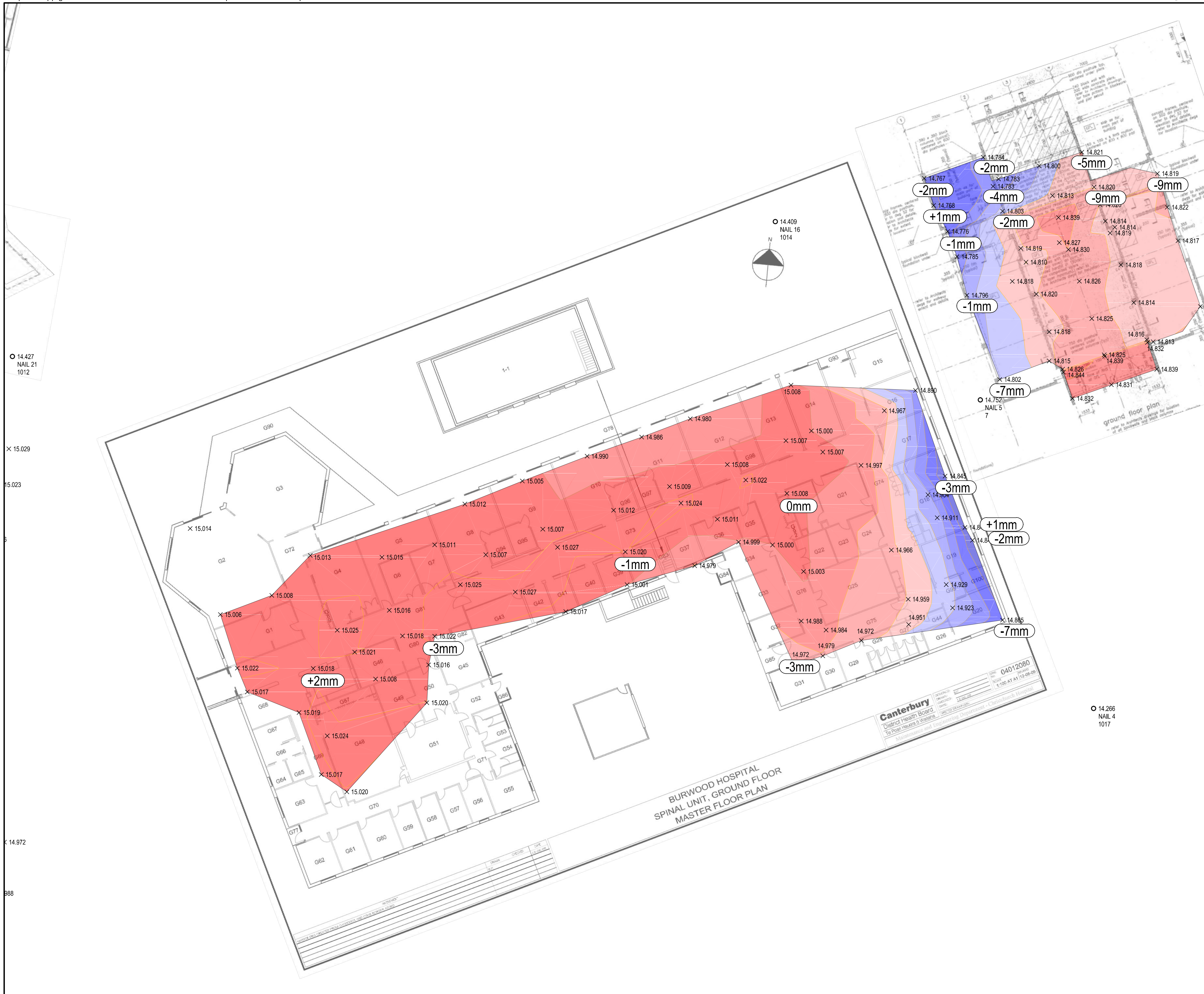
DWG No. 04012080
SCALE 1:100 AT A1
REVISED 12-06-09

Maintenance and Engineering Department - Christchurch Hospital

APPENDIX C

Level/Elevation Survey

K:\2462C CDHB Burwood\dwg\2462C 20120124 Elevations Survey Jan 2012.dwg : 01 Feb 2012 12:49 p.m. : 3 Spinal Unit And Allan Bean Centre



Spinal Unit
All levels taken to linoleum floor level with no reductions applied.
Benchmark = Nail 16

Allan Bean Centre
All levels reduced to concrete slab level.
Benchmark = Nail 5

Elevations Table - Allan Bean Centre

Number	Minimum Elevation	Maximum Elevation	Color
1	14.760	14.770	
2	14.770	14.780	
3	14.780	14.790	
4	14.790	14.800	
5	14.800	14.810	
6	14.810	14.820	
7	14.820	14.830	
8	14.830	14.840	
9	14.840	14.850	

Elevations Table - Spinal Unit

Number	Minimum Elevation	Maximum Elevation	Color
1	14.840	14.860	
2	14.860	14.880	
3	14.880	14.900	
4	14.900	14.920	
5	14.920	14.940	
6	14.940	14.960	
7	14.960	14.980	
8	14.980	15.000	
9	15.000	15.020	
10	15.020	15.040	

No.	Revision	Approved	Date
G			
F			
E	Resurvey and audit observations post Dec 23 2011	MJM	01/02/2012
D	Wards 7 & 8, Surgical Block, Melrose Chairs & Corridor added	MJM	02/12/2011
C	Phys Med, Parafed, Food Services, BIRS, Boilerhouse, EDC added	MJM	24/11/2011
B	Nurses Hostel & Champion Centre, SOU elevations added	MJM	18/11/2011
A	Administration and Allan Bean Centre elevations added	MJM	31/10/2011

Scale	1:200
Reduced	A3 1:400
Designed	----
Drawn	DHP
Checked	MJM
Date	24 January 2012

Burwood Elevations Survey
Spinal Unit and Allan Bean Centre

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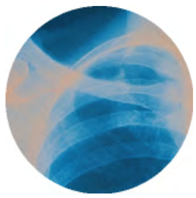
Job No. 2462C
Sheet No. 3 of 15
Rev. E

032005

HolmesConsultingGroup

DETAILED SEISMIC ASSESSMENT REPORT

STRUCTURAL AND CIVIL ENGINEERS



BURWOOD HOSPITAL CAMPUS

REPORT 15 – SURGICAL SERVICES UNIT AND SURGICAL OPERATING SUITES

PREPARED FOR

CANTERBURY DISTRICT HEALTH BOARD

106186.29

INTERIM REPORT – APRIL 2014

BURWWOD HOSPITAL CAMPUS – INTERIM DETAILED SEISMIC ASSESSMENT REPORT

REPORT 15 – SSU / SOU

Prepared For:
CANTERBURY DISTRICT HEALTH BOARD

Date: 02/04/14
Project No: 106186.29
Revision No: 2

Prepared By:

David J Herman
PROJECT ENGINEER

Reviewed By:

Richard Seville
PROJECT DIRECTOR

Updated By:

Peter Grange
STRUCTURAL ENGINEER

Holmes Consulting Group LP
Christchurch Office

REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
13/11/12	1	Interim
02/04/14	2	Updated to include further investigations completed and updated capacities

CONTENTS

APPENDICES

TABLES

FIGURES

Holmes Consulting Group has been engaged by Canterbury District Health Board to complete a full structural review of the Burwood Hospital Campus following the Lyttelton Earthquake. A series of reports have been completed as part of this. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The individual building reports, like this one for the Surgical Services Unit and Surgical Operating Units (SSU and SOU), should be read in conjunction with the base report and refer to the repair specification.

This report identifies the structural damage observed to date for SSU and SOU building as a result of the series of Earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The report summarises the effects of the damage on the lateral load capacity of the building, and provides structural repairs for the damage identified. The general form of the building, along with its capacity relative to current code levels, has been included for the buildings pre-earthquake undamaged state and post-earthquake state. Recommendations for improving the seismic performance of the building have also been identified.

The SSU and SOU building was designed in 2005 as an Importance Level 3 (IL3) building according to NZS 1170 [10] and constructed in the period there after. The building effectively consists of three seismically separate structures, the SSU Ward, the SOU Theatre, and the Link Corridor. The building is primarily a single storey structure with central two storey plant areas in the SSU Ward and SOU Theatre areas of the building considered. The Link Corridor runs along the southeast of the SSU Ward and links into the SOU Theatre.

The majority of the structure is composed of precast concrete walls or blade columns and there is a precast concrete moment frame along the western edge of the plant room central to the SOU Theatre. Walls and blade columns are supported by reinforced concrete strip footings. The roof primarily consists of light weight metal roofing with a section of plywood diaphragm along the tops of the concrete blade columns. The ground floor slab consists of insitu concrete with wire mesh reinforcement on compacted hard fill. Several service tunnels run underneath the slabs. The second storey plant room slabs consist of Interspan and Hibond precast units in the SSU Ward and of Interspan and Unispan precast units in the SOU Theatre. There is an insitu topping slab with mesh reinforcement placed over all precast slab units. Lateral loads at roof level are distributed to lateral resisting elements by the steel framing members and plywood diaphragms where present.

The information available for review included the original 2004 architectural and structural drawings [4], a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [5], along with a level survey of the building completed by Fox & Associates [6].

The SSU and SOU superstructure appears to have performed relatively well considering the likely seismic actions experienced at the site. However, significant differential settlement has occurred. There is differential settlement throughout the building, with the worst areas in the central portion of the building as well as along the Link Corridor. A differential settlement of 43 mm has occurred along a length of approximately 1 metre (1:280 slope) along the Link Corridor length.

The damage to the superstructure is typified by separations of the wall and ceiling framing, cracking and warping of floor, wall, and ceiling linings, and cracking in several precast walls in the SOU Theatre area. Roof bracing and vertical strap bracing has elongated and has slackened. Cracking was observed in the tunnel walls and exterior foundation walls. Cracking and separation in the slab on ground along construction and shrinkage joints was also noted and indicates some lateral spreading. Cracking was noted in the exterior non-structural garden walls which are also out of plumb.

While some of the damage has likely occurred in all significant events noted, it is believed that the majority of the damage observed, including the onset of damage, occurred as a result of the 22nd February event.

Further observations of the earthquake damage observed have been included in the body of this report. Further investigations are required and include investigating the condition of the connections between collector struts and the lateral force resisting elements, typically concrete walls.

Based upon a review of the drawings available, and the site investigations completed, the primary lateral forced resisting elements of the building have been assessed in their pre-earthquake undamaged state.

The SSU Ward structure was assessed, by Equivalent Static Analysis, to have a pre-earthquake capacity to resist approximately 67% of the demand required by the current loading code Design Basis Earthquake (DBE) in the east-west direction. In the north-south direction the capacity was 36% DBE and was limited by the connection detail between the RHS collector strut and the precast concrete wall panel along Grid 4. The strengthening of this connection and collector member was carried out in January 2014, increasing the overall rating of the SSU Ward to 67% DBE. The capacity is now controlled by yielding of the concrete blade column starter bars to the foundation.

The Link Corridor has been assessed to have a pre-earthquake capacity of 80% DBE in the east-west direction and 100% DBE in the north-south direction. However, the portal frames which resist loads in this direction are highly flexible. This means the area is susceptible to non-structural damage in future events.

The SOU Theatre structure has been assessed through a Nonlinear Time History Analysis to have a pre-earthquake capacity to resist approximately 67% of the demand required by the current loading code DBE in the north-south direction. The capacity is limited by the connection of the SHS collector strut along the west length of the building to the precast shear wall at the north end. The failure of this connection would remove the main lateral resisting element for the west portion of the SOU Theatre and could result in a partial collapse along the west edge. Improving this connection could increase the overall capacity of the SOU Theatre to 75% DBE. Following strengthening of the collector connection, the capacity of the structure would be limited by the yielding of roof braces and yielding in flexure of precast concrete panels.

Based on the observations to date, we do not consider the SSU and SOU building to have any reduction in gravity load resistance. We do not believe that there is any significant reduction in the strength of the lateral load resistance of the structure due to the earthquake damage observed. There are a number of areas of observed damage that have caused a reduction in overall stiffness and resilience of the building including: yielding of tension only bracing, differential settlement of the slabs and foundation, lateral movement of the Link Corridor portal frames, and insufficient displacement capacity in the seismic gap connection. The reduced stiffness will result in larger lateral displacements during future seismic events and additional damage to interior linings and building contents. Additionally, further investigations of the collector element connections is required to determine if they still maintain their pre-earthquake load transfer capacities. The pre-earthquake analysis indicated these areas control the capacity of the structures. Investigations to determine the condition of these connections are underway.

The damage observed will require repair (or replacement) to restore the strength, stiffness, and resilience of some of the individual structural components. The minimum repairs required to reinstate the building to its pre-earthquake undamaged condition have been included in Section 4. The repairs involve re-levelling the ground floor slab and/or wall foundations, repairing of all cracks in concrete slabs and walls, re-levelling of the Link Corridor portal frames, replacing cracked GIB bracing, replacing elongated roof bracing, and replacing or repairing of steel connections to concrete wall panels.

A localised risk exists along the north wall of the SOU Theatre. The wall capstones did not appear to be sufficiently fixed to the parapet and could fall in a future seismic event. These have been subsequently removed. With the increased seismic loads in Canterbury, we consider the timber framing design supporting the brick veneer to be too flexible. The brick veneer has subsequently been replaced with a lightweight brick veneer.

In addition to the repairs, recommended strengthening concepts to increase the seismic performance of the building have been included in section 5. Recommended strengthening includes stiffening the Link Corridor portal frames, stitching the concrete floor slab together, and installing a larger seismic gap between the SSU Ward and SOU Theatre.

1. INTRODUCTION

Holmes Consulting Group has been engaged by Canterbury District Health Board to complete a full structural review of the Burwood Hospital Campus following the Lyttelton Earthquake. A series of reports have been completed as part of this. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The individual building reports, like this one for the Surgical Services Unit and Surgical Operating Units (SSU and SOU), should be read in conjunction with the base report and refer to the repair specification.

The Burwood Hospital Campus base report covers the purpose and scope of the structural review. The current statutory requirements relevant to earthquake damaged buildings are outlined and the level of shaking experienced at the site estimated. Earthquake induced ground settlement damage across the campus is also discussed. The repair specification has been prepared to include repair details for typical damage observed in buildings on the Burwood Hospital Campus and is referred to as required in the specific building reports.

1.1 SCOPE OF WORK

This report is on the SSU and SOU building at Burwood Hospital, Mairehau Rd, Christchurch. The report identifies the general form of the structure, along with the gravity and lateral load resisting systems. The structural system was reviewed based upon the information available and any potential Critical Structural Weaknesses (CSW's) were investigated.

The report also identifies the structural damage observed to date as a result of the series of Earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake, in particular, subjected the building to strong ground motions which are likely to have significantly exceeded the current code loading demand for buildings of this nature.

The capacity of the SSU and SOU building has been assessed relative to current code loading in the buildings pre-earthquake undamaged state and in its post-earthquake damaged state. The post-earthquake assessment summarizes the effects of the damage identified on both the gravity and lateral load resisting elements.

Repair options to restore the buildings capacity to pre-earthquake levels for strength, durability and stiffness are provided. The repair options aim to maintain the buildings utility. Where required, strengthening options have also been provided.

1.2 LIMITATIONS

Findings presented as a part of this project are for the sole use of the Canterbury District Health Board, 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. Because all of the structure has not been available for detailed inspection or evaluation, this report is limited to those elements available and engineering judgement as to the likely condition of unseen elements. 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. PRE-EARTHQUAKE BUILDING CONDITION

This section discusses the form and capacity of the building prior to the Darfield Earthquake. The Surgical Services Unit and Surgical Operating Unit building, shown below in Figure 2-1, at the Burwood Hospital campus was designed in 2005 and constructed in the period there after. The original structural design was completed by Holmes Consulting Group. The information available for the review included the original 2005 structural drawings [4], a post-earthquake geotechnical assessment conducted for the campus by Tonkin and Taylor [5].

Figure 2-1: SSU and SOU Building, Burwood Hospital Campus

2.1 BUILDING FORM

The overall building footprint is approximately 130 m by 40 m. The building is split into three seismically separated structures: the SSU Ward to the south, the Link Corridor to the southeast, and the SOU Theatre to the north. The SSU Ward structure contains the set off Link Corridor that runs from south to north along its eastern edge and is considered separate to the SSU Ward. The SSU Ward and SOU Theatre are separated by a 50 mm seismic gap. The building is typically of single storey configuration with plant areas within the roof space and basement service tunnels. The SOU Theatre contains a central two storey plant area approximately 24 m by 20 m. The overall building footprint is show in Figure 2-2 below with the SSU Ward, Link Corridor, and SOU Theatre outlined and labelled.

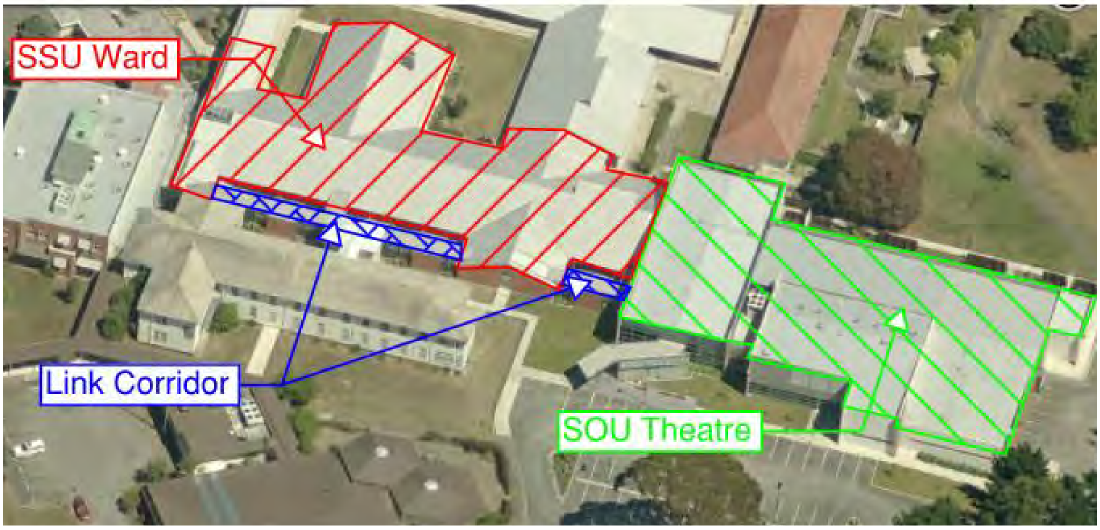


Figure 2-2: SSU and SOU Building, Burwood Hospital Campus

The building typically consists of steel roof framing with a lightweight metal decking and a combination of concrete precast panels and blade columns with steel framed and non load bearing timber stud walls. Several exterior walls have a single skin brick veneer. The plant rooms have suspended precast slab units with mesh reinforced topping slabs in both the SSU Ward and SOU Theatre.

The buildings foundations are a series of pad and strip footings founded just below ground level. The ground floor slab is constructed on compacted fill on grade and typically has edge thickenings. Basement service tunnels are approximately 2.2 m deep and are constructed from precast wall panels and an insitu concrete slab.

The interior walls are typically non-load bearing, lightweight timber partition walls with lightweight cladding. The timber stud walls on the north east wall of the SOU Theatre area are lined on the inside face with gypsum wallboard sheathing and the exterior face is clad with 70mm thick brick veneer with a 40 mm cavity between the brick and the stud walls.

To the southeast of the SSU Ward area is the Link Corridor which is formed by SHS steel portal frames in one direction, timber stud walls with GIB bracing in the other direction, and strap roof bracing to transfer lateral forces. The foundation consists of a slab on grade with edge thickenings. An exterior brick veneer is supported on the slab edges in some locations.

The roof plans of the SSU Ward and SOU Theatre have been included in Figure 2-3 and 2-4. The ground floor plans of the SSU Ward and SOU Theatre have been included in Figure 2-5 and 2-6.

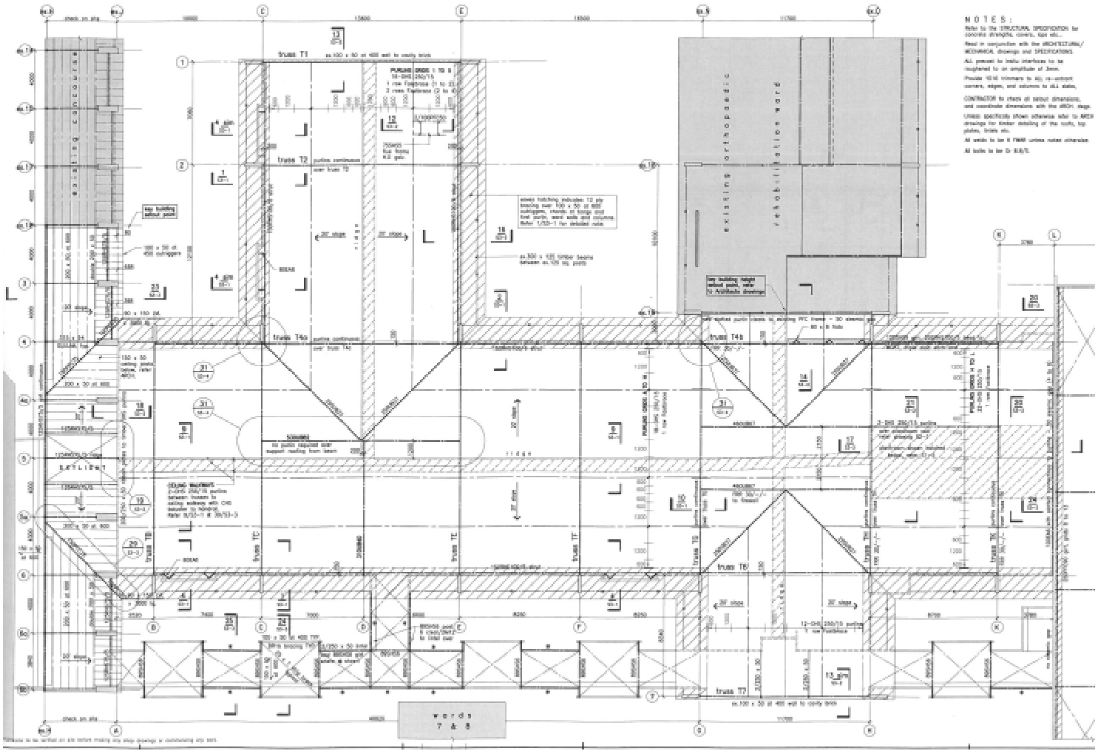


Figure 2-3: SSU Ward area roof plan

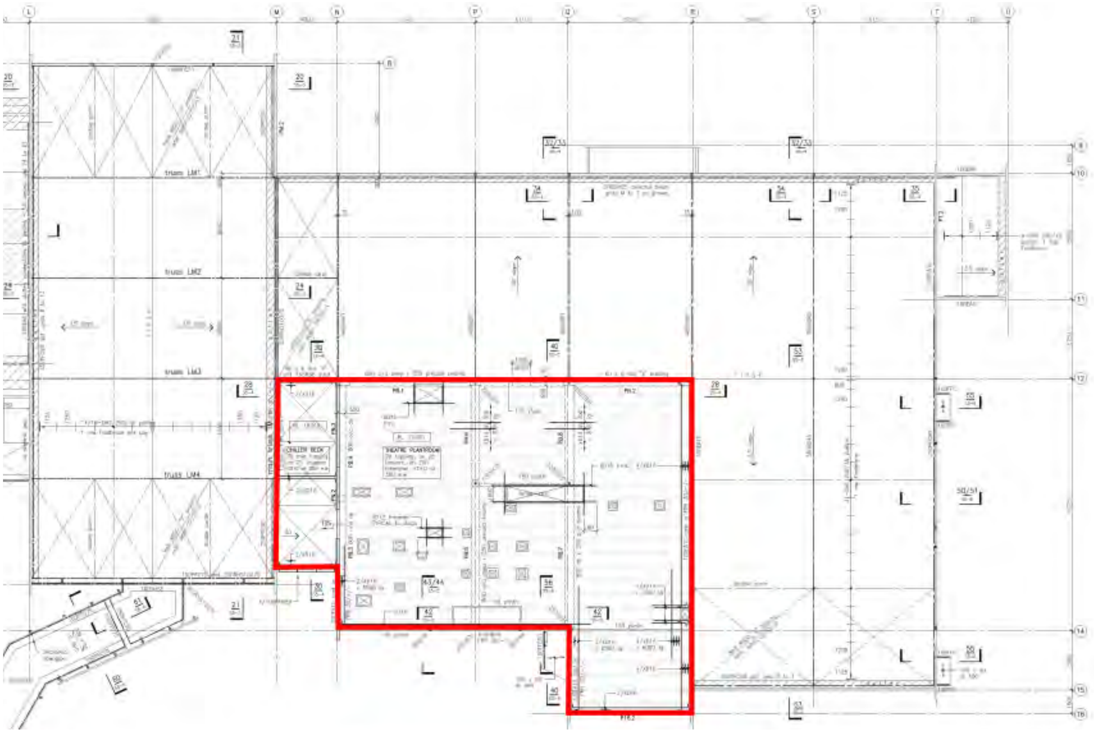


Figure 2-4: SOU Theatre area roof plan (red line shows extent of plant room floor)

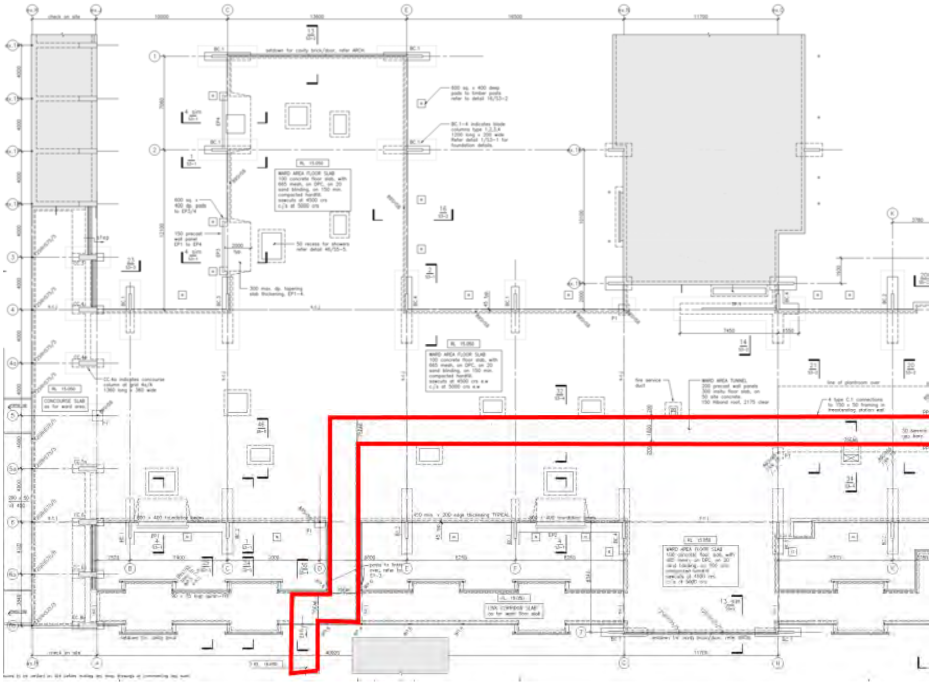


Figure 2-5: SSU Ward area ground floor plan (red lines indicate extent of service tunnels)

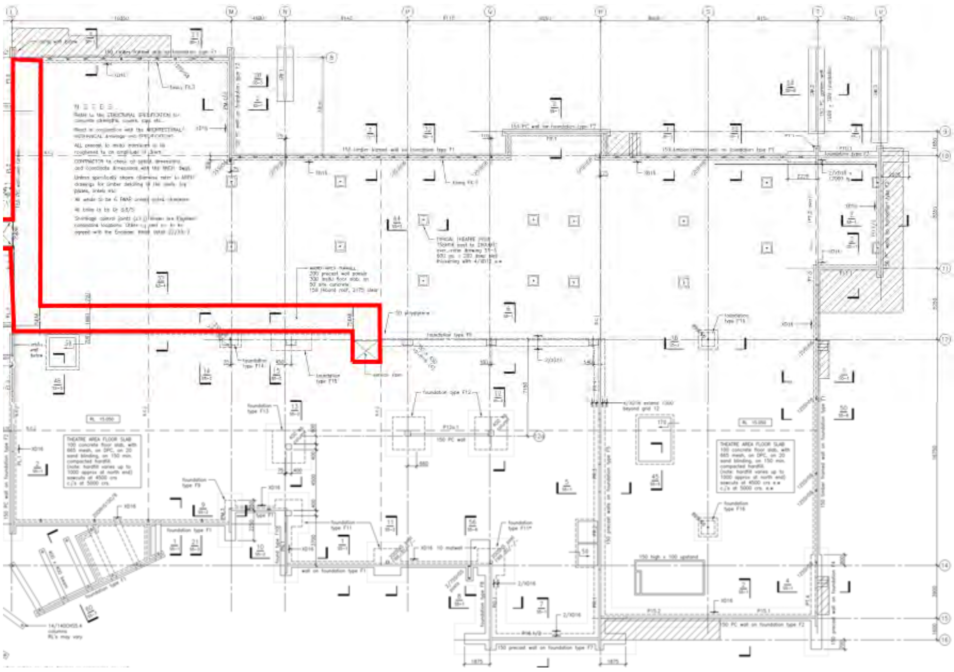


Figure 2-6: SOU Theatre area ground floor plan (red lines indicate extent of service tunnel)

2.2 LATERAL LOAD RESISTING SYSTEMS

The following subsections describe the specific seismic systems of the buildings structures, the SSU Ward, the Link Corridor, and the SOU Theatre.

2.2.1 SSU Ward

The main lateral force resisting system for the SSU Ward area consists of precast concrete wall panels and precast concrete cantilever blade columns which act as shear walls. Lateral loads at the roof level are distributed by steelwork struts and a strip of ply diaphragm. In the north-south direction the loads are distributed to precast concrete walls and cantilever precast blade columns. In the east-west direction loads are distributed to cantilever precast blade columns.

2.2.2 Link Corridor

The lateral force resisting system for the Link Corridor consists of steel SHS portal frames in the east-west direction and timber framed GIB braced walls in the north-south direction. Steel strap bracing exists along the entire corridor roof.

2.2.3 SOU Theatre

Lateral loads at the roof level are distributed by steelwork struts and tension only bracing to precast concrete wall panels in the east-west direction and to precast concrete panels and a central precast concrete moment frame along the internal edge of the plant area.

2.3 PRE-EARTHQUAKE BUILDING CAPACITY – DIRECT CODE COMPARISON

The building capacity under earthquake actions discussed in this section is compared to the capacity that a similar building would be designed to today. A new building would be designed to the *Structural Design Actions Standard, Part 5: Earthquake Actions – New Zealand*, NZS 1170.5:2004[10] and incorporating the amendments made to this standard as a result of the Lyttelton Earthquake as outlined in the Amendment 10 of the Building Code [9]. The implications of the recent amendments are discussed more fully in the Burwood Hospital Campus Base Report however, for a building of this type the amendments essentially result in an increase to the design loads of 36 % when compared to pre-earthquake design levels.

The building was designed in 2005, to the current loading standard: NZS 1170.5:2004. The current code requires a new building to be designed for an earthquake, known as the Design Basis Earthquake (DBE), which is based upon the buildings physical location, local soil conditions, building type, fundamental period and importance level.

The original structural drawings for the building are available, but the structural calculations and specifications could not be located. The exact design and loading assumptions originally made for the structure are unknown.

Based upon building occupancy, the SSU and SOU building has been classified as an Importance Level 3 (IL3) building in accordance with NZS 1170.5:2004 [10]. The associated return period of the DBE is 1000 years for a design life of 50 years, with a risk factor for design of R=1.3, typical for health facility buildings as prescribed in this code (no post-disaster or special functions). The sub soil for the entire site is taken as Soil Type D, which is consistent with the findings of post-earthquake geotechnical investigation [5].

Based upon the detailing of the lateral load resisting elements, we have assumed the following levels of ductility:

- precast concrete wall panels - nominally elastic, μ= 1.25

- precast concrete blade columns - limited ductility, μ = 2.0

- precast concrete and steel moment frames – limited ductility, μ = 3.0

A comparison between the Design Basis Earthquake of NZS 1170.5:2004 for the site prior to and after the site hazard factor, z, was increased from 0.22 to 0.33 is plotted below. Based upon a fundamental building period below 0.40 seconds, the seismic demands required by the loading code have increased on the concrete portion of the structure by approximately 36% since 2004.

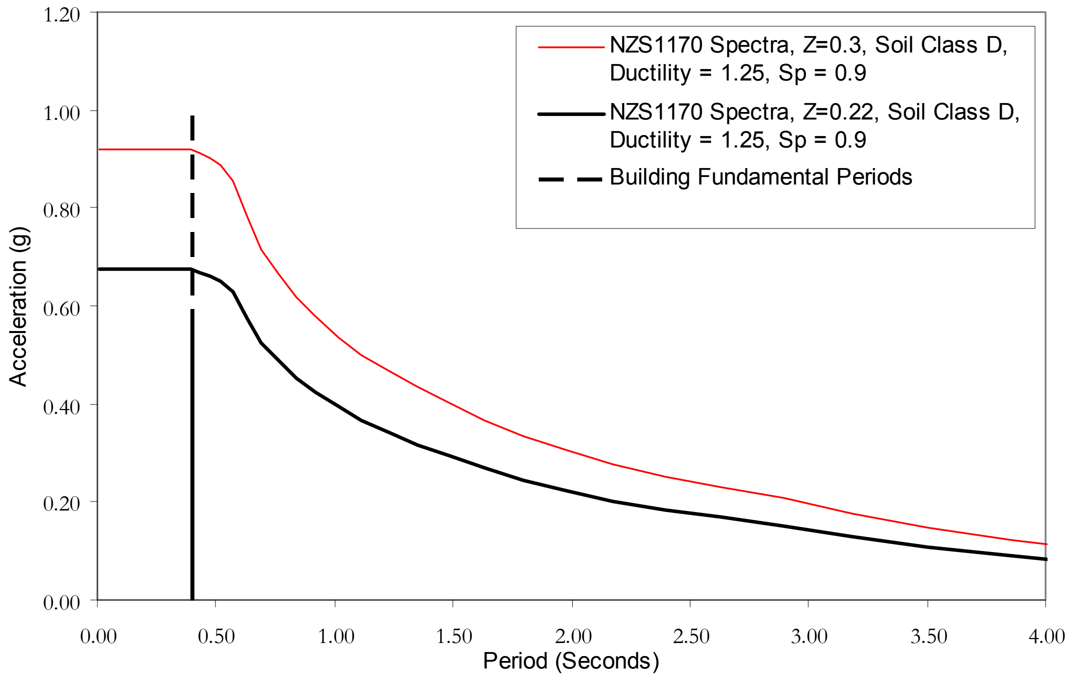


Figure 2-7: Comparison of Design Codes

2.4 PRE-EARTHQUAKE BUILDING CAPACITY – ANALYSIS METHODS

2.4.1 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 estimated 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.

Following the Lyttelton earthquake, a geotechnical report was conducted by Tonkin & Taylor, titled "Burwood Hospital Post Earthquake Geotechnical Assessment", dated June 2011 [5]. This report has been used to aid in the evaluation of the site conditions and the likely effect of the ground on the buildings past and future performance. The soil parameters described in the geotechnical report completed by Tonkin and Taylor have been used for the evaluation of the buildings existing foundation system.

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 [17]. 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. These are described in more detail in the Burwood Hospital Base report [1] and include strength governed elements such as short columns and deflection governed elements such as floor and stair elements with inadequate support seating.

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 the strength and deflection requirements.

2.4.2 Nonlinear Time History Analysis (NLTHA)

The SOU Theatre was originally assessed using an equivalent static analysis method and hand calculations. Upon completion of the static analysis, it was determined that performing a nonlinear time history analysis would more accurately assess the seismic performance of the building. NLTHA has the ability to better model the building's rocking walls and assess the effect of the structures irregular layout and numerous concrete panel penetrations.

Nonlinear Time History Analysis (NLTHA) allows a more detailed understanding of the buildings performance that extends to the likely non-linear behaviour, i.e. what happens as elements of the structure yield, following which load and deformation is redistributed around the structure. This is important because there are generally many elements of a building that may have their capacity exceeded permanently, but that in itself does not constitute the building as a whole exceeding its reliable capacity. This assessment philosophy forms the basis of what is termed performance based design.

The NLTHA provides more accurate information on when the building is likely to experience deficiencies that result in significant structural damage and would be considered as the building reaching its Ultimate Limit State (ULS) capacity and when it is likely to experience deficiencies that might lead to the onset of partial collapse, it's Collapse Limit State (CLS).

The assessment criteria used to assess the performance of the NLTHA model are based on a paper presented at the New Zealand Society of Earthquake Engineers Conference in 2012 titled "Nonlinear Analysis Acceptance Criteria for the Seismic Performance of Existing Reinforced Concrete Buildings," This paper was written by Holmes Consulting Group Engineers and aims to address the building performance objectives detailed in the Design Advisory Group Detailed Engineering Evaluation Guidelines [27]. Generally, using a NLTHA is much less conservative than a modal analysis. The specific criteria for each element (or building component) used in this assessment are described in detail in the internal NLTHA report for this building which can be provided on request.

As mentioned above, the capacities are presented as a percent of the Design Basis Earthquake (DBE) loading. The Engineering Advisory Group Draft Guidelines recommend a margin over collapse is provided [11]. Therefore, the building capacity at the onset of the CLS (which is also considered to be the onset of Critical Structural Weaknesses, or CSW) is divided by 1.5 to 1.8 to provide a factor of safety against collapse. This factor is briefly discussed in the Base Report [1].

2.5 PRE-EARTHQUAKE BUILDING CAPACITY – RESULTS

2.5.1 SSU Ward

The limiting factor for the capacity of the Ward area is the ability of the RHS strut member and strut-to-wall connections to transfer building lateral loads into the precast shear walls along grid lines 4. The connection detail of the strut-to-wall connection was at 36% of DBE. The RHS strut capacity was at 47% for combined axial and bending forces. In January 2014, these elements were strengthened to 67% DBE with a PFC collector element bolted into the existing collector and concrete shear wall. This has improved the capacity of both the strut and the concrete breakout of the connection.

Precast concrete blade column capacities were determined assuming a ductility factor of 2.0 for flexure. Shear and foundation capacities were considered for elastic loads. Panel starter bars were found to have a capacity of 67% DBE for the blade columns and 70% DBE for the precast concrete walls.

Building Element	%DBE (IL3)	Comments
Ceiling Diaphragm	NA	No discernable ceiling diaphragm; loads transferred directly through roof framing
Precast Blade Column – NS EW	67 67	Blade Columns are limited by starter bar tension capacity.
Precast Wall Panels – NS	70	Panel is limited by starter bar tension capacity.
Foundations – Overturning Sliding	100 100	
Collectors/Struts – NS EW	47 100	Strengthened to 67%. Now limited by the length of wall available to connect into.
Strut to P/C Wall Connection	36	Strengthened to 67%. Now limited by the length of wall available to connect into.

Table 2-1: SSU Ward area – Seismic Assessment % DBE

2.5.2 Link Corridor

The portal frames that compose the east-west lateral system of the link corridor were analysed assuming limited ductility ($\mu = 3.0$). The capacities of the portal frames were calculated to be 80% of DBE. However, the deflections at this level of load are higher than general design limits.

Building Element	%DBE (IL3)	Comments
GIB BR1b Bracing	100	
Steel Portal Frames Strength	80	Capacity is limited by combined compression and bending

Table 2-2: Link Corridor – Seismic Assessment % DBE

2.5.3 SOU Theatre

The SOU Theatre was designed to behave in a ductile manner with the precast concrete walls rocking and the concrete moment frame columns hinging near the base. The NLTHA model that was developed for the SOU Theatre is shown in Figures 2-8 and 2-9.

The NLTHA showed the walls rocking as designed. The model was analysed several times to determine the load level building elements begin to fail and cause a collapse limit state. This load level was then converted to an ultimate loads state (ULS) capacity by dividing the %DBE that would lead to the collapse limit state (CLS) by a factor of safety of 1.5. This determined the overall capacity of the structure.

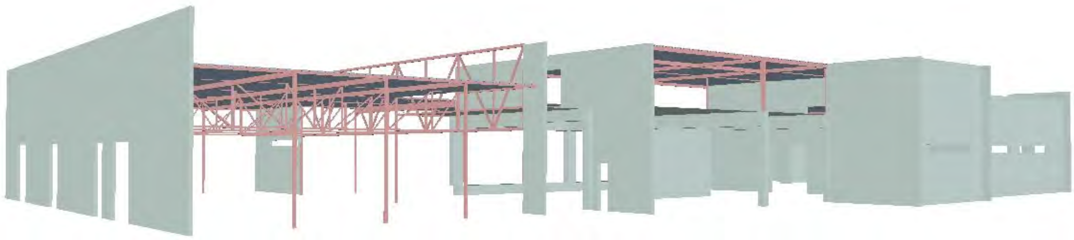


Figure 2-8: Image of NLTHA model used to analyse SOU Theatre (View from south-east)

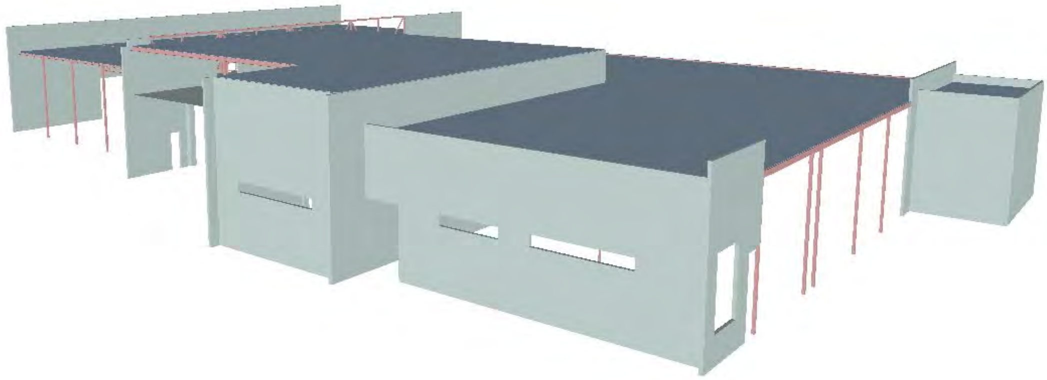


Figure 2-9: Image of NLTHA model used to analyse SOU Theatre (View from north-east)

Based on the Nonlinear Time History Analysis, the capacity of the SOU Theatre is controlled by the connection of the SHS collector connection to the shear wall at the northwest corner of the building between grid lines T/10 and U/10. The capacity of the collector connection is 67% of DBE. The capacity of the main seismic resisting elements is limited by precast concrete panel flexure which is at 75% of DBE. A summary of these results are listed in Table 2.3. All capacities are presented assuming the building is Importance Level (IL3).

Building Element	%DBE (IL3)	Comments
Onset of significant damage (equivalent to the Ultimate Limit State (ULS) performance of a new building. Excluding foundation performance.	75	Onset of yielding in tension bracing in plant room roof
Onset of deficiencies that have a high probability of leading to partial or total collapse (the Collapse Limit State (CLS)).	100	200SHS collector element exceeds capacity.
Onset of collapse deficiencies divided by factor of safety against collapse of 1.5; i.e. CLS/1.5	67	200SHS collector element capacity adjusted to provide factor of safety between ULS and CLS.

Table 2-3: SOU Theatre Area – Seismic Assessment % DBE

The capacities in the table above are a direct result of the NLTHA. Some design loads, displacements, and element capacities must be considered through separate calculations. Demands on wall panel out of plane connections were determined via hand calculations. The NLTHA also assumes a fixed base and, although the wall foundations rock, the bearing strength of the soil is not considered in the analysis and must be considered separately. Additionally, building drift along Grid L has been compared with the existing seismic gap. These issues are discussed in the following subsections.

Building Drift – From the NLTHA, building drifts at the ULS and CLS limit states were examined. At 67% of DBE (limit of the collector connection), the drift in the long direction of the structure is 51 mm. At 75% of DBE (limit of the concrete precast panels in flexure), the drift in the long direction of the structure is 61 mm. Along Grid L, a 50 mm seismic gap has been provided via slotted bolt holes in the roof purlins of the SSU Ward structure. The movement allowed for in the connection provided is exceeded by the SOU Theatre demands without considering the drift demand from the SSU Ward structure. If the movement allowance was exceeded at this connection, it is likely that failure would occur by tearing of the purlins at the bolts rather than yielding of the wall. The purlins would cantilever if support is lost.

Foundations – The SOU Theatre shear walls are primarily founded on shallow strip footings. Seismic loading of the SOU Theatre significantly increases the load applied to the soil below the foundations relative to the static load case. If the bearing pressure capacity of the soil below the foundations is exceeded, this could lead to differential settlements along each wall and would be detrimental to the performance of the building in future seismic events.

To assess the capacity of the foundations, we have compared the demands on the foundations from the NLTHA model with bearing capacities estimated from previous calculations on this site.

The NLTHA model assumes the ground below the foundations is rigid, this conservatively over-estimates the demand from the building on the foundation as no load sharing between adjacent parts of the foundation is modelled.

When the foundations bearing demands from the NLTHA model at 67% and 75% of DBE loads are compared with the design bearing pressures, several areas are observed to have demands higher than assumed design capacity and therefore would be expected to sustain residual deformations during this level of earthquake loading. Although yielding of the soils is likely to occur and cause permanent residual deformations of the soil, this is unlikely to lead to global instability; however the impact of these deformations on future building performance needs to be considered. We do not consider the performance of the SOU Theatre foundations likely to govern the performance of this building.

Out of Plane Wall Capacity - The out of plane support for the precast wall panels along Grids L, M, and 15 consist of a 250PFC spanning between supports and attached to the walls at 1500 to 1900 mm. The capacity of the support is limited by the PFC and is at 67% DBE.

The freestanding garden walls outside of the structure have a capacity of 80% DBE and are limited by the ability of the foundation to resist out of plane overturning.

2.6 SSU AND SOU PRE-EARTHQUAKE CAPACITY SUMMARY

As an Importance Level 3 building, the results of the equivalent static and nonlinear time history analyses indicate that the ultimate limit state capacity of the structures of the SSU and SOU building are as follows:

SSU Ward – 36% DBE (Strengthened to 67% in January 2014)

Link Corridor – 80% DBE

SOU Theatre – 67% DBE

The SSU Ward area is limited by the connection between the RHS strut member and the precast concrete wall panel along Grid 4. This has now been strengthened to 67% DBE. The Link Corridor is limited by the capacity of the portal frames. The SOU Theatre is limited by the connection of the SHS collector element along Grid 10 to the precast concrete wall at Grid T.

An assessment has also been made of the foundation capacity, these are not considered to be critical to the buildings performance and are not considered earthquake prone. The foundations are likely to exceed the allowable bearing pressure of the soil at levels between 45% and 67% DBE. At these load levels yielding of the soil is likely to occur and cause permanent residual deformations. However, the foundations and slabs are likely to accommodate this movement and redistribute the load along the length of the foundation. This is unlikely to lead to global instability but could the deformations could affect capacities of the building going forward.

The freestanding garden walls, external to the building have a capacity of 80% DBE and limited by the foundations' capacity to resist overturning.

A review of the drawings available and site observations revealed that the collector strut elements and connections to wall panels that account for the rating of the SSU Ward and SOU Theatre areas are considered Critical Structural Weakness (CSW's). Failure of these elements could result in a localised collapse. Further investigations are currently occurring in these areas.

Methodology to improve the seismic performance of the buildings and provide strengthening to achieve 67% DBE have been included in Section 5.

3. POST-EARTHQUAKE BUILDING CONDITION

This section covers the structural damage sustained by the SSU and SOU structures, and its effect on the buildings capacity to resist seismic loads, as a result of the series of earthquakes which includes the Darfield Earthquake that struck at 4:36am on 4th September, 2010, the Lyttelton Earthquake that struck at 12:51pm on the 22nd February 2011, the June Earthquake that struck at 2.20pm on the 13th of June 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake subjected the building to strong ground motions which significantly exceed the full design earthquake load for buildings of this nature and appears to have caused the majority of the earthquake damage observed, after the Darfield event.

3.1 THE LYTTELTON EARTHQUAKE

The fundamental period of the building is estimated to be approximately 0.4 seconds. 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 strong motion data presented in the Base Report, it appears the Lyttelton Earthquake produced ground shaking intensities of approximately 130% of the current Ultimate Limit State design spectra for an Importance Level 3 building.

It should be noted that the Lyttelton Earthquake was very short in terms of strong shaking produced with the strong motion only lasting for duration of 5-7 seconds. Rupture of the Alpine Fault is expected to contain 50 to 60 seconds of strong motion.

3.2 PRELIMINARY INVESTIGATIONS

Preliminary investigations were carried out to identify 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
- review of available structural engineering construction documentation
- damage observed during 'Rapid' Structural Assessments following the Lyttelton Earthquake, June 13th aftershocks and subsequent events

In conjunction with a review of the structural drawings, areas were identified for potential damage in the three sections of the structure considered. These areas for the SSU Ward, Link Corridor, and SOU Theatre are listed in the following subsections.

3.2.1 SSU Ward Preliminary Investigations

In conjunction with a review of the structural drawings for the SSU Ward structure, the following areas were identified for potential damage:

- movement or damage to structure associated with ground movement and/or settlement;
- cracking and joint failure of concrete sub-floor walls, service tunnels, foundations, and ground slabs;
- cracking of precast concrete blade columns and wall panels;
- distress of precast concrete blade column and wall panel connections;
- damage to brick parapets and capstones;
- distress of roof framing, bracing, and connections with concrete walls;
- signs of distress at connection of interior and exterior stud walls to floor system below
- brick veneer support connections;
- signs of distress at interfaces between different sections of the building;

3.2.2 Link Corridor Preliminary Investigations

In conjunction with a review of the structural drawings for the Link Corridor structure, the following areas were identified for potential damage:

- movement or damage to structure associated with ground movement and/or settlement;
- cracking and joint failure of ground slabs;
- general distress to steel portal frames, including beam-column joint welds
- signs of distress at connection of interior and exterior stud walls to floor system below
- signs of distress at interfaces between different sections of the building

3.2.3 SOU Theatre Preliminary Investigations

In conjunction with a review of the structural drawings for the building the following areas were identified for potential damage:

- movement or damage to structure associated with ground movement and/or settlement;

- cracking and joint failure of concrete sub-floor walls, service tunnels, foundations, and ground slabs;
- cracking of concrete moment frame;
- damage to brick parapets and capstones
- distress of roof framing and bracing and connections with concrete walls;
- signs of distress at connection of interior and exterior stud walls to floor system below
- brick veneer support connections
- signs of distress at interfaces between different sections of the building

3.3 RAPID LEVEL 2 ASSESSMENT

Rapid Level 2 assessments were carried out on the 24th February 2011[22] and on the 14th [23] and 15th June 2011 [24] following the June 13th earthquakes. These structural observations involved a complete walk around the exterior and throughout the interior of the building. The following primary areas of damage were identified from the damage assessments:

- damage to the brick veneer on the north wall (SOU Theatre)
- precast panel connections along Grid L (SSU Ward and SOU Theatre)
- cracking of exterior precast panels on Grids Q/16, R/16, T/15, and U/11 (SOU Theatre)

A review of the above information on the building type and preliminary observations highlighted this building as requiring a more detailed inspection. The aim of the detailed inspections was to determine the full extent of the damage caused to the building, particularly those elements identified for potential damage above. These areas were targeted to identify if damage had occurred, and to what extent the damage had reduce the capacity of the buildings lateral load resisting system to resist future seismic events.

3.4 DETAILED STRUCTUAL OBSERVATIONS

Further detailed inspections and structural explorations (including removal of finishes) have been carried out following the initial assessments to ascertain the full extent of structural damage. The majority of the detailed structural observations were completed on the 1st April, 10th May, 19th May, and 13th October 2011.

A full record of these observations can be found in Appendix A, with reference plans describing the location labelling used found in Appendix B. A full photographic record of the observations is available electronically on request. The detailed structural observation identified additional damage to those items noted in the initial rapid assessments which is listed in the subsections below.

3.4.1 SSU Ward Detailed Structural Observations

- visible ground fissures and evidence of liquefaction in the service tunnel below the south end of the building

- concrete cracking at floor joints below vinyl flooring
- Vertical cracks to concrete walls throughout the service tunnels.
- Differential settlement throughout building
- additional cracking and distress to wall and ceiling linings

3.4.2 Link Corridor Detailed Structural Observations

- concrete cracking at floor joints below vinyl flooring
- Differential settlement throughout building
- additional cracking and distress to wall and ceiling linings

3.4.3 SOU Theatre Detailed Structural Observations

- concrete cracking at floor joints below vinyl flooring
- vertical cracks in foundations
- concentrated damage to the partitions and structural connection elements at the location of the seismic gap (Grid L) between the Ward and Theatre
    - Diagonal cracking at the top of the precast concrete observed from the plant room on the south side
    - Damage to the web plate connections to the steel trusses on the north side of the precast panels
    - Spalling of concrete at location of precast wall connection to PFC stiffener
- Horizontal and vertical cross bracing appears to have yielded and slackened throughout the building
- Vertical cracks to concrete walls throughout the service tunnels.
- Cracking of external freestanding precast wall elements
- Displacement of precast theatre wall caps
- Leaning of freestanding garden walls
- Differential settlement throughout building
- additional cracking and distress to wall and ceiling linings

3.5 GEOTECHNICAL REVIEW

A review of the ground damage and conditions was carried out by Tonkin & Taylor for the Burwood Hospital Campus. A subsequent report titled *Burwood Hospital Post Earthquake Geotechnical Assessment* was issued in June 2011 [5]. The geotechnical review concluded that the settlement and damage to building foundations and slabs on the Burwood Hospital Campus was likely due to the liquefaction of underlying soil layers. It is believed that excessive pore water pressures have been dissipated and that further settlement is not expected to occur, unless another significant event was to occur.

Figure 3-1: Evidence of Liquefaction below slab in service tunnel

Based up the geotechnical report provided by Tonkin & Taylor [5] the potential for future total and differential settlements at the building site varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

3.6 LEVEL SURVEY & VERTICALITY STUDY

A detailed survey of the ground floor levels in the Spinal Injuries Unit was conducted by Fox & Associates and issued on 31st[th] October, 2011 [6]. The survey indicates a maximum differential ground settlement of approximately 60 mm across the footprint of the building. The differential settlement varies through the building with a worst case slope in the ground floor slab of approximately 0.45% (1:220). CDHB may still wish to pursue re-leveling of the entire structure due to the nature of the building use, and ongoing serviceability concerns. Options for further consideration for the re-levelling of the building includes localised lifting of the structure using mechanical or grout injection techniques. A discussion on how has been included in Section 4.2.

3.7 SUMMARY OF BUILDING DAMAGE

The following is a summary of the observations made for the building, and our conclusions as to their condition and seismic load resisting capacity. These observations do not specifically distinguish between damage caused by the Darfield Earthquake, the Lyttelton Earthquake or any significant aftershocks, such as those that occurred on the 13th June 2011, 23rd December 2011 or 2nd January 2012. Despite not being able to specifically distinguish when individual damage observed occurred, it is believed that the majority of the damaged can be linked to the February 22nd event.

The majority of the SSU and SOU superstructure appears to have performed relatively well considering the seismic actions experienced at the site. Observations of some structural elements of the building were limited due to difficulties in accessing the elements hidden within building finishes. The most severe damage appears to have occurred at the seismic joint between the Ward and Theatre areas and the portal frames along the Link Corridor. The bolts connecting the Ward framing to the precast concrete wall panel along Grid L show evidence of having reached the displacement limit within the slotted bolt holes during the seismic event and have caused the wall to pull away from the connections on the Theatre face. The ability of this location to support gravity loads appears unaffected. Differential settlement of the foundations and slabs is also typical throughout the building site.

The Link Corridor is mostly separate from the lateral systems of the ward and theatre areas and appears to have experienced significant displacements. The cracking in the GIB walls and concrete slabs along the Link Corridor are indicative of the lateral movement experienced by the portal frames.

A sample of the critical elements and connections has been observed for the purposes of evaluation and reporting. A summary of the building damage observed can be typified as follows:

- **Differential Ground Settlement** – As previously noted the majority of the damage noted to date appears to be associated with the liquefaction induced differential ground settlement. This has resulted in separating of the slab on grade at locations of construction joints and resulted in some distress to the timber framed interior partitions above. The damp proof course (DPC) may also have sustained damage.

  Differential ground settlement of the foundations is also likely to have occurred.

  In the tunnel space below the ground floor slab, numerous cracks were observed in the floor slab and concrete walls. Efflorescence and evidence of liquefaction were present.

- **Lateral Spreading of Concrete Floor Slabs** – Extensive spreading and cracking was observed in the concrete slab on grade. This was typically located at existing construction and shrinkage control joints. The cracking was most significant along the joints that ran east to west. See Appendix A for locations of cracking that overlap construction and shrinkage joints in the slabs. The DPC may also have sustained damage due to lateral spreading.

  Shrinkage control joints are detailed so that the wire mesh reinforcement stops on either side of the joint, and thus, no reinforcing is present to tie the sections of slab on grade together. Construction joints are detailed such that every other mesh wire is cut.

- **Landscaping Structures** – The differential ground settlement noted surrounding the building has resulted in damage to the external concrete garden walls and building perimeter foundation walls. Cracks were observed down to the ground level. Free standing garden walls were observed to be leaning vertically out of plumb.

- **Precast Wall Panels –** Several wall panels have diagonal cracking.

- **Seismic Gap at Grid L** – Building drifts in opposite directions have exceeded the drift capacity of the seismic gap. Opposing displacements from the structures on either side of the seismic gap have caused the bolts to stretch the slotted bolt holes and induce a localized rotation in the wall between the roof member connections to the wall on either side. Framing connections on the Theatre side have rotated and stretched due to the corresponding drift demands.

- **Roof Bracing** –Roof tension rod bracing and vertical strap bracing members are slack and/or buckled due to elongation during lateral movement.

- **Distress to Exterior Wall Supporting Brick** – The exterior brick façade on the north end of the Theatre area is supported by a timber framed wall. Support angles have deflected and bricks have worked loose likely due to out of plane deflections.

- **Distress to Wall and Ceiling Finishes** – Cracking and general distress has been noted to the wall and ceiling linings throughout. The cracking in the gypsum board wall and ceiling linings has typically occurred off the corners of door and window openings, along existing wallboard joints and at the interface between the top of the wall and the ceiling finishes.

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. Secondary elements, such as windows and fittings, have not generally been reviewed.

3.8 ADDITIONAL INVESTIGATIONS REQUIRED

Several assumptions were made in the completion of the Pre-earthquake (undamaged state) and Post-earthquake (damaged state) Structural Assessments. Destructive exploration is required in a number of locations in order to verify these assumptions.

3.8.1 Investigations Required For Further Assessment

The areas requiring further investigation to finalise the assessments are as follows:

- Based upon the pre-earthquake strength assessment, further investigation of the connection straps and angles between the SHS strut member and the precast concrete wall panels along Grids 4 and 6 in the Ward area is required.

  Investigations carried out on these connections during December 2012 did not reveal any significant damage so the reported values remain unchanged. Strengthening work on the connection between the SHS strut and the concrete shear wall on Gridline 4 was carried out in January 2014.

- Based upon the pre-earthquake strength assessment and observed damage, further investigation of the precast concrete wall panel along Grid 10 at the north end of the building is required, specifically to search for cracks and distress near the vertical cast in SHS member at Grid T.

  Investigations carried out on this panel during December 2012 did not reveal any significant damage so the reported values remain unchanged.

- Based upon the pre-earthquake deflection assessment and damage observed, further investigation of the Link Corridor portal frames is required, specifically the welded connections and base plate connection to slab. The portal frames and slabs should be checked for verticality.

  This is proposed to be carried out as part of the strengthening of theses portal frames, as outlined in Section 5.

3.8.2 Investigations to be Completed During Building Repairs

- Re-inspection of building will be required upon completion of any re-levelling works, to determine if any additional damage has occurred.

- Check existing timber stud wall framing and fixings to concrete slabs below where new wall linings are to be installed.

3.9 POST-EARTHQUAKE BUILDING CAPACITY

Based upon our investigations to date, we do not consider the SSU and SOU building to have any significant reduction to the overall gravity load resistance of the structure.

Diagonal cracks in precast concrete walls may have minimally decreased the capacity of the walls to resist shear. The walls along Grid L and P, however, have enough shear reinforcement to resist the DBE lateral forces provided the steel has not yielded. Additionally, stiff elements around the central plant room in the SOU structure have the ability to redistribute loads. Most of the damage noted to date does not appear to have caused any significant reduction to the lateral load capacity of the concrete and steel portions of the building (Sections 2 & 3).

The pre-earthquake capacity of the SSU Ward and SOU Theatre was governed by the connections between the steel strut and shear wall panel along grids 4 and 10. Investigations have shown no significant damage to these elements so the pre-earthquake capacity is not considered to be reduced. Strengthening work to the connection on grid 4 has been carried out to increase the capacity to 67% DBE.

At the timber framed and GIB braced portion of the Link Corridor, the damage observed to the gypsum board linings of the bracing walls will have resulted in a reduction in lateral load capacity. While there has been some reduction in strength, according to the Department of Building and Housings, *Revised Guidance on Repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence* [20], the primarily result of the damage noted will be a reduction in the stiffness of the wall bracing.

The movement noted in the slab on grade is not believed to have significantly affected the existing capacity of the building as there was no reinforcement present across the control joints prior to the earthquake. We believe the roof framing is flexible enough to have absorbed the lateral movement in both directions of the building without imposing undue stress on the base of the precast wall panels. However, the settlements noted will have resulted in some reduction to the ability of the building to absorb future differential settlements before severe distress to the structure occurs, in future seismic events.

While it is believed that the predicted movements noted for future SLS and ULS events can be absorbed without disproportionate damage or partial collapse of the building, we believe that the accumulative stress to precast elements under an additional ULS event will likely require repair or replacement of these elements. The movement predicted for the SLS event is also likely to result in the damage of the floor finishes and require future repair of the slab on grade.

In its pre-earthquake and post-earthquake condition, the SSU Ward area was assessed at 36% DBE, the Link Corridor at 80% DBE, and the SOU Theatre area at 67% DBE. Although the SSU Ward area capacity was assessed at 36% DBE, the overall lightness of the structure, the stiffness of partitions, and ability of the structure to redistribute lateral loads may explain why this area of the structure performed well under the seismic demand placed on it. In its current post-earthquake condition, the reduction in capacity due to damage is minimal and the capacities essentially remain as calculated for pre-pre-earthquake conditions. In January 2014, the Ward area was strengthened to 67% DBE:

- SSU Ward – 67% DBE
- Link Corridor – 80% DBE
- SOU Theatre – 67% DBE

Recommendations for strengthening and improving the resilience of the SSU Ward area, Link Corridor, and SOU Theatre area are discussed in Section 5.

4. OBSERVED DAMAGE AND REQUIRED RAPAIRS

4.1 PRIMARY DAMAGE OBSERVED AND REPAIRS REQUIRED

This section covers the damaged noted during our detailed assessment of the building. Note that our observations have been restricted to structural aspects of the building 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.

Table 4-1 provides a photographic summary of the observed damage and typical repairs required for the SSU Ward and SOU Theatre structures. Table 4-1 should be read in conjunction with Appendix A – Record of Observation and Appendix B – Reference Plans. The Repair Specification [2] referred to in Table 4-1 has been issued separately.

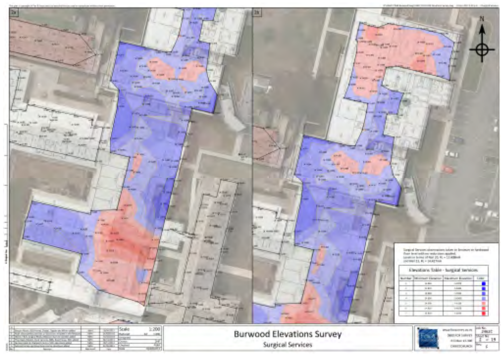
In general, the aim of the repair work indicated in this section is to restore the structure to its pre-earthquake state, as far as practicable, while maintaining the utility of the building. The repairs presented attempt to address the loss of strength, stiffness and durability of the structural elements due to the damage noted.

It should be noted that more damage may be identified during the repair works and (if required) additional repair details will be specified accordingly.

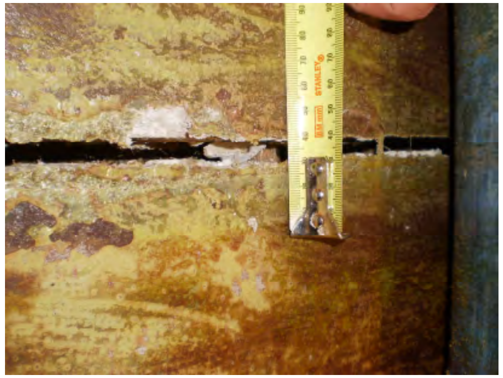
Please note that all repair works are to be completed after the building has been re-levelled to a satisfactory condition as further damage to the wall and ceiling linings can be expected during the re-levelling process.

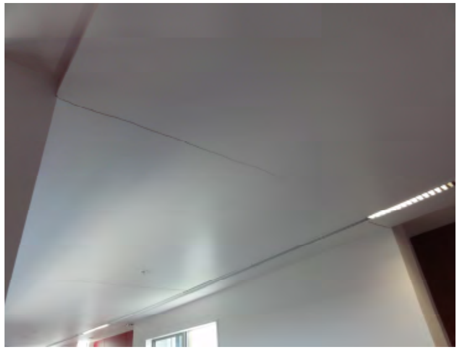
Further recommendations for improvement to the buildings seismic performance, and to achieve a minimum capacity of 67% DBE have been included in Section 5.

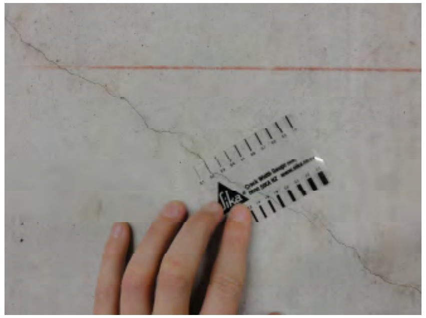
Table 4-1: Photographs of observed damage and repairs required

Damaged Item & Location	Damage	Recommendations	Example Photograph
1. Concrete service tunnels, sub-floor walls, slabs on grade and foundations			
1.1. Differential ground settlement	Differential ground settlement of approximately 60mm resulting in a worst case slope in the ground floor slab of approximately 0.35% (1:280) through the Link Corridor.	The differential settlement noted throughout the building will need to be addressed by either demolishing and rebuilding the ground floor slabs, or through re-levelling. For further discussion on the remediation work required see Section 4-2. (Note: All re-levelling is to occur prior to any other structural or cosmetic repairs).	
1.2. Service tunnel slabs	Evidence of liquefaction. All cracks appear to be old shrinkage cracks or construction joints	If Water stop is damaged, remove strip of slab and recast with new water stop.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
1.3. Service tunnel walls	Numerous cracks observed throughout tunnel walls. Most are vertical with approx 0.7 mm width. Some larger (1.4 mm width) diagonal shear cracks visible. Efflorescence was visible.	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [2]. For cracks greater than 0.5-0.6mm, HCG to confirm the integrity of the existing reinforcement of the wall. If reinforcement is damaged, an engineered repair will be required.	
1.4. Foundation Walls	Vertical cracks observed in concrete foundation walls on west side of Theatre. Cracks range from 0.7mm to 3.5mm wide and run height of visible foundation.	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [2]. For cracks greater than 0.5-0.6mm, HCG to confirm the integrity of the existing reinforcement of the wall. If reinforcement is damaged, an engineered repair will be required.	
1.5. Concrete Slab, typical	Cracks observed throughout ground slab. Most covered by vinyl. Horizontal and vertical movement apparent. Cracks straight, possible at slab joint locations. Refer to slab crack map.	Where slab cracks occur along shrinkage control and construction joints, slabs are to be reinforced across all joints, then subsequently grouted. Refer to joint and crack locations in Appendix A. All ties should be placed after re-levelling of the structure.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
1.6. Concrete Slab, Typical	Opening up of slab at shrinkage control joints and sawcut slab joints.	Where slab cracks occur along shrinkage control and construction joints, slabs are to be reinforced across all joints, then subsequently grouted. Remove strip of concrete and replace DPM or water stop and slab, specification by others. Refer to joint and crack locations in Appendix A. All ties should be placed after re-levelling of the structure.	
2. Link Corridor			
2.1. Foundations	External vertical cracks in reinforced concrete foundations. Cracks appear to correspond with cracking in concrete floor slabs and wall and ceiling partitions throughout corridor.	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [2]. For cracks greater than 0.5-0.6mm, HCG to confirm the integrity of the existing reinforcement of the wall. If reinforcement is damaged, an engineered repair will be required.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
2.2. Concrete Slabs	Cracks in floor slap. Up to 5 mm horizontal and vertical differential movement between sides. Cracks are concealed by vinyl floor covering.	Where slab cracks occur along shrinkage control and construction joints, slabs are to be reinforced across all joints, then subsequently grouted. Refer to joint and crack locations in Appendix A. All ties should be placed after re-levelling of the structure.	
2.3. Gypsum Board Ceilings and Timber Framed Walls	Vertical and tapered diagonal and straight cracks to wall partition and ceilings.	Replace all damaged ceiling and wall boards with new gypsum board sheets.	
3. Precast Elements			
3.1. Freestanding wall cracking	Tapered vertical crack through wall above foundation.	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [2]. For cracks greater than 0.5-0.6mm, HCG to confirm the integrity of the existing reinforcement of the wall. If reinforcement is damaged, an engineered repair will be required.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
3.2. Freestanding wall out of plumb	Exterior freestanding garden walls visibly out of plumb. Contractor measured ~25 mm per metre height.	Remove and replace external freestanding garden walls with foundation and reinforcement sized for 100% DBE lateral force.	
3.3. Precast Concrete Wall Panels	Typical diagonal crack though precast concrete walls	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [2]. For cracks greater than 0.5-0.6mm, HCG to confirm the integrity of the existing reinforcement of the wall. If reinforcement is damaged, an engineered repair will be required.	
3.4. Precast Concrete Wall P16 along Grid 16	Typical diagonal crack through wall running from bottom corner to central window penetration.	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [2]. For cracks greater than 0.5-0.6mm, HCG to confirm the integrity of the existing reinforcement of the wall. If reinforcement is damaged, an engineered repair will be required.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
4. Seismic Gap			
4.2 Precast Panel to PFC wall Connection	Concrete has spalled from several connections and some anchor bolts are loose.	Install new connection between PFC and wall panel. This work has already been performed. See SR07 and SR08.	
4.3 Steel DHS Purlin Connection from Theatre	Roof appears to have moved relative to the wall for the full extent of the slotting (+/-25mm) in the purlin connection.	Fabricate new connection and replace connection bolts.	
4.4. Steel DHS Purlin Connection from Ward	The ledger plate connecting the purlins to the precast wall panel appears to have pulled ~5 mm off the wall in some locations. Bolts were only installed in approximately half of the holes drilled through the ledger for this connection.	Check existing fixings, and install bolts where required.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
4.5. Truss Connections	Spalling of concrete at truss connections to the precast concrete wall at the seismic gap. Some bolts loose.	Tighten loose bolts. Install anchors where absent. Grout areas where concrete has spalled. This work has already been performed. See SR07 and SR08.	
5. Bracing			
5.1 Roof Rod Bracing	Rod bracing appears to have elongated and slackened.	Reidbar tension bracing connections do not have sufficient ductility to develop yielding in bracing rods. Reidbar rods should be replaced with new connection details and resized accordingly.	
5.2 Lateral Strap Bracing	Strap bracing appears to have elongated and slackened along Grid 12 of Plant room.	Replace elongated strap bracing.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
6. Parapet			
6.1 Parapet along Grid T	Loose bricks top of wall. Top of wall flexible when weight applied.	Replace existing brick veneer with new thin lightweight brick veneer. This work is in the process of being completed. See RFI NL-RC#0073 Response.	
6.2 Capstone along Grid T	Capstones have dislodged from top of wall.	Replace cap stones with new lightweight capstone with metal flashing. This work has already been completed. See RFI #009 Response.	

4.2 DISCUSSION ON DIFFERENTIAL SETTLEMENT REMEDIATION

The level survey, completed by Fox & Associates, has indicated differential ground settlement of approximately 60 mm across the length of the ground floor slab (see Appendix C for complete level survey), that has resulted in permanent slopes in the ground floor slab of up to 0.45% (1:220). The differential settlement is likely to have affected the foundations as well. The slope will need to be addressed in order to restore the function of the building.

The settlements affecting the foundations can be addressed by re-levelling of the foundations. The slope in the ground floor slab can be addressed either by demolishing and reconstruction of the entire floor slab or through re-levelling. If demolition and reconstruction of the ground floor slab is chosen, the entire slab would need to be reconstructed. If re-levelling is chosen, the building would be proposed to be lifted up to the highest point of the building. Both options require the re-levelling of the foundations prior to re-levelling of the slab.

The two primary re-levelling options available for the foundations and slab include the use of mechanical jacking or the use of either underpinning grout or engineered resin. There are pro's and con's of each solution which extend beyond structural performance which will need to be considered by CDHB. These include continuity of operation, degree of re-levelling accuracy and the willingness of the re-levelling sub-contractor to provide a producer statement, amongst other items.

From a structural standpoint, either option is acceptable provided the use of underpinning grout or engineered resin does not create any "hard points" under the building. If "hard points" are created during the re-levelling process the potential for future differential settlements can be increased. If this were to occur it would reduce the capacity of the building going forward.

Based upon the information provided by Tonkin & Taylor the soil profile under the SSU and SOU buildings (medium dense sand overlying dense sand) lends itself to localised lifting through underpinning grout or engineered resin techniques and should not create any undesirable "hard points" as described above.

The suitability of re-levelling the building through the use of either mechanical jacking or underpinning grout (or engineered resin) will need to be verified by qualified sub-contractors in conjunction with the geotechnical consultant.

It should be noted that both options discussed above are not expected to increase the seismic performance of the building or reduce the potential for future differential settlements. Instead the options presented are designed to re-level the building without making the future performance of the building any worse than it was prior to the earthquakes. To improve the future performance of the building, and reduce the potential for future differential settlements, would likely require the entire footprint of the building to be either piled or the ground under all the sub-floor wall footings, service tunnels and the partial basement improved. *Further geotechnical investigations would be required into the type and depth of ground improvement required.*

Based up the geotechnical report provided by Tonkin & Taylor [5] the potential for future total and differential settlements at the building site varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

During the re-levelling process there is also the risk that addition damage could occur to the building linings, exterior block veneer, etc. Appropriate contingencies should be provided.

4.3 REPAIR OF GIB BRACING ELEMENTS

The wall linings interior bracing walls along the Link Corridor have been damaged in several locations and require repair. While the damage to the fixings may not be obvious, based upon the movement observed, we believe there has been a reduction to the ongoing strength and stiffness of all the bracing walls. In order to reinstate the pre-earthquake strength and stiffness to the bracing walls, the repair recommendation is to remove all cracked or damaged sections of the wall linings and replace them with new gypsum board sheathing. The new gypsum board sheathing is to be fixed in accordance with GIB 'Braceline' BL1-H specifications (or equivalent). All existing internal wall linings to remain are to be re-fixed to the existing studs in a similar manner. Any non-gypsum wall boards will need to be replaced in conjunction with these repairs. A new finish is then to be applied to all interior walls. Note the fixings of the walls to the concrete foundations below will need to be checked for damage and the ability to transfer the new bracing loads.

4.4 REPAIR OF LINK CORRIDOR GYPSUM WALLS

Similarly to the wall linings, the existing sections of gypsum clad ceiling diaphragms and their fixings have been damaged and require repair to reinstate their pre-earthquake strength and stiffness. The repair recommendation is to remove any cracked or damaged sections of gypsum board ceiling lining and replace them with new gypsum board sheathing fixed in accordance with GIB specifications.

4.5 REPAIR OF SLAB ON GRADE CRACKS

Repair of cracks across the shrinkage control joints is required to reinstate the structural performance and durability of the slab. To prevent similar damage from occurring in a future serviceability level event and to prevent structural issues arising due to lateral spread in an ultimate limit state event, the slab will be required to be physically 'stitched' back together placing new D12 reinforcing bars across existing joints at 600 mm centres. A chase will be required to be cut in the slab in order to place the D12 bars. The chase would then be packed with high-strength non-shrink grout.

The installation of the D12 reinforcing bars will require portions of the mesh to be cut which is acceptable. Between the added D12 reinforcing bars the existing cracks are to be repaired with Sikadur 52 low viscosity crack injection epoxy.

Damage to the damp proof course (DPC) may have occurred. The DPC should be repaired at the same time as the repair of the cracks. This may required removing a strip of slab allowing for the replacement of sections of DPC or installation of water stops.

Stitching the joints together will reduce the capacity of the slab to compensate for natural volumetric changes in the concrete, however the majority of the concrete curing shrinkage has already occurred and the slab is in a temperature controlled environment and not subject to major fluctuations in temperature induced expansion or contraction.

5. STRENGTHENING RECOMMENDED

The SSU and SOU Building can be separated into three primary sections; the SSU Ward area that composes the south part of the structure; the Link Corridor along the east side of the Ward area; and the SOU Theatre area that composes the north part of the structure. As noted in Section 2, Pre-Earthquake Building Condition, and Section 3, Post-Earthquake Building Condition, the lateral load resisting capacity of each section of the building has been assessed as a percentage of the loads imposed by the Design Basis Earthquake (DBE).

The pre- and post-earthquake capacity of the SSU Ward of the structure has been assessed at a capacity of approximately 36% DBE (strengthened to 67% DBE). The Link Corridor structure has been assessed at 80% DBE. However, the flexibility of the Link Corridor exceeds the recommended drifts and the steel portal frames running in the east-west direction should be stiffened to reduce the risk of future non-structural damage. The SOU Theatre capacity has been assessed to be 67% DBE and is limited by the strength of the SHS collector connection to the resisting shear wall element. Additional recommended strengthening to achieve the drift requirements for the Link Corridor and a capacity of 67% DBE for the SSU Ward area, and to improve the overall seismic performance of the building have been included in sub-sections below.

5.1 STRENGTHENING WORKS TO ACHIEVE 67%DBE

Recommendations described below improve the strength of the structures to 67% of DBE. Where new elements are suggested, it is recommended that they be designed to achieve 100% DBE where possible.

5.1.1 SSU Ward

Collector Struts and Connections – The collector strut member and connection, specifically the bottom chord member of Truss T4b along Grid 4, should be upgraded to meet 67% DBE. In addition a new connection to the shear wall TP.1 that does not place any additional bending forces on the top of the wall is required.

This work was carried out in January 2014, the capacity is now 67% DBE.

5.1.2 Link Corridor

Steel Portal Frame – The steel portal frames have been assessed at 40% DBE for drift and at 80% DBE for strength. We recommend that the portal frames are stiffened to reduce the risk of future non-structural damage. This could be achieved by installing stiffer frames around the existing frames. In order to improve the seismic performance of the Link Corridor, stiffer steel portal frames and larger foundations could be installed outside of the existing frames to support them. These new frames could be demined for the full seismic demand on this system.

A concept scheme for this strengthening work was issued in October 2013 and is in Appendix D.

5.1.3 SOU Theatre

Seismic Gap – As evidenced by the damage observed at the seismic gap along Grid Line L, the gap is insufficient to account for the opposing drift demand at the interface between the SSU Ward and SOU Theatre. To increase the drift capacity of the seismic gap, a separate concrete wall should be installed to provide a minimum 100 mm seismic gap to the south of Grid Line L to fully separate the SSU Ward and SOU Theatre and provide sufficient drift compatibility.

Roof Bracing Members –Existing Reidbar bracing that has yielded should be replaced with new bracing that allows for a ductile failure of the systems. Current Reidbar bracing connection details do not allow for this.

6. REFERENCES

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2. *Burwood Hospital – Detailed Seismic Assessment Report – Earthquake Repair Specification*, Holmes Consulting Group, July 2011.

3. *Burwood Hospital Stage II Surgical Services Unit*, Original architectural drawings, Shepard and Rout Architects LTD, September 2005.

4. *Burwood Hospital Stage II Surgical Services Unit,* Original structural drawings, Holmes Consulting Group, December 2005.

5 *Burwood Hospital Post Earthquake Geotechnical Assessment*, Tonkin and Taylor Ltd., June 2011.

6 *Burwood Elevation Survey – Revision F*, Fox & Associates, April 2012

7 *Burwood Hospital Campus – Seismic Risk Assessment Report*, Holmes Consulting Group, April 2002

8 *Burwood Hospital Campus – 2007 Seismic Risk Assessment Update,* Holmes Consulting Group, June 2007

9 *Compliance Document for New Zealand Building Code - Clause B1 – Structure, Amendment 10 (Canterbury)*, Department of Building and Housing, Wellington, 19 May 2011.

10 *Structural Design Actions Part 5: Earthquake Actions – New Zealand, NZS 1170.5:2004*, Standards New Zealand, 2004.

11 *Code of Practice for General Structural Design and Design Loadings for Buildings*, NZS4203:1992, Standards New Zealand, 1992

12 *New Zealand Standard Model Building Bylaw – Chapter 8 Basic Design Loads*, NZSS1900:1965, New Zealand Standards Institute, 1965

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

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

15 *Concrete Structures Standard, NZS 3101:2006*, Standards New Zealand, 2006

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

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

18 *Seismic Rehabilitation of Existing Buildings,* ASCE 41-06, American Society of Civil Engineers, 2007

19 *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury – Part 2 Evaluation Procedure*, Engineering Advisory Group, July 2011

20 *Revised Guidance on Repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence*, Department of Building and Housing, November 2011

21 *Practice Note – Design of Conventional Structural Systems Following the Canterbury Earthquakes*, SESOC, December 2011

22 *CDHB – Burwood Hospital Campus – Rapid Visual Inspection: 106186.03 Site Report 1,* Holmes Consulting Group, February 2011

23 *CDHB – Burwood Hospital – Rapid Seismic Assessment Post June 13th Earthquakes: 106186.03 Site Report 4,* Holmes Consulting Group, 14 June 2011

24 *CDHB – Burwood Hospital – Rapid Seismic Assessment Post June 13th Earthquakes: 106186.03 Site Report 5,* Holmes Consulting Group, 15 June 2011

25 *CDHB – Burwood Hospital – Post Earthquake Rapid Structural Assessment following 5.8 and 6.0 Magnitude Earthquakes, 106186.03 Site Report* 8, Holmes Consulting Group, 24 December 2011

26 *CDHB – Burwood Hospital – Post Earthquake Rapid Structural Assessment following 2nd January 5.5 Magnitude Earthquakes, 106186.03 Site Report 9*, Holmes Consulting Group, 9 January 2012

27 *CDHB – Burwood Hospital – Heavy Ceiling Tiles, 106186.03 Site Report 12*, Holmes Consulting Group, 31 January 2012

28 *Nonlinear Analysis Acceptance Criteria for the Seismic Performance of Existing Reinforced Concrete Buildings,* S.J. Oliver, A.G. Boys, D.J. Marriot, New Zealand Society of Earthquake Engineering Conference 2012, Paper 042, April 2012.

APPENDIX A

Record of Observations

APPENDIX A – RECORD OF OBSERVATIONS & REPAIRS

Inspection date: May-December 2011

KEY	
N	No repair required
Y	Repair required
F	Further investigation required
C	Repair complete

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	Seismic Gap	Articulated wall joint	Differential movement of walls each side of gap has dislodged the rubber membrane from its track.	Y	Relocate membrane back into track or replace after slabs and foundations have been re-levelled. Repair specification by others.	752
G	Link Corridor	Concrete Floor Slab	Crack in ground floor slab, up to 5mm horizontal and vertical differential movement between sides. Crack is concealed by vinyl floor covering.	F	Remove finishes to expose full length of crack in concrete slab. Refer to Section 4 and HCG Repair Specification.	744
G	External Wall of Link Corridor	Concrete Foundation	Vertical cracks in reinforced concrete foundations. Cracks appear to correspond with cracking in concrete floor slabs and wall and ceiling partitions throughout the corridor.	Y	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [2]. For cracks greater than 0.5-0.6mm, HCG to confirm integrity of existing reinforcing steel. Refer to HCG specification. Repairs to be carried out after re-levelling of slabs and foundations.	773, 774, 775
G	West side theatres	Concrete Foundation walls	Vertical cracks observed in concrete foundation walls. Smaller cracks approx 0.7 mm width. Largest approx 3.5 mm width. Cracks run height of visible foundation.	F	Cracks > 0.5-0.6mm require further investigation to confirm the integrity of the steel reinforcement. Refer to HCG specification.	1235-1238

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	Wards	Concrete Slab	A number of cracks observed through ground slab. Most covered by vinyl. Horizontal and vertical movement apparent. Cracks straight, possible at slab joint locations. Refer slab crack map.	F	Remove finishes to expose full length of crack in concrete slab. Epoxy inject all cracks in concrete slab >0.2mm as per the HCG Repair Specification [2]. For cracks greater than 0.5-0.6mm, HCG to confirm integrity of existing reinforcing steel. Refer to Section 4 and HCG specification.	1158, 1159, 1160, 1161, 1165
G	Theatres	Concrete Slab	A number of cracks observed through ground slab. Most covered by vinyl. Horizontal and vertical movement apparent. Cracks straight, possible at slab joint locations. Refer slab crack map.	F	Remove finishes to expose full length of crack in concrete slab. Epoxy inject all cracks in concrete slab >0.2mm as per the HCG Repair Specification [2]. For cracks greater than 0.5-0.6mm, HCG to confirm integrity of existing reinforcing steel. Refer to Section 4 and HCG specification.	1162, 1163, 1166-1175
G	Grid C5	Concrete Slab	9mm horizontal crack at shrinkage control joint. Reinforcing bar across gap has been wrapped in tape to allow for movement.	Y	Refer to Section 4 and HCG Repair Specification.	111214(1-2)
G	Grid N5	Concrete Slab	0.2mm horizontal crack at shrinkage control joint. We could not review the reinforcing bar.	Y	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [2]. For cracks greater than 0.5-0.6mm, HCG to confirm integrity of existing reinforcing steel. Refer to Section 4 and HCG specification.	111214(3-4)
G	Grid O5	Concrete Slab	0.5mm horizontal crack at sawcut joint. Joint does not appear on original structural documentation.	Y	Cracks > 0.5-0.6mm require further investigation to confirm the integrity of the steel reinforcement. Refer to Section 4 and HCG specification.	111214(5-6)
G	Grid ex.J6b	Concrete Slab	10mm horizontal crack with 4mm vertical displacement at sawcut joint. Reinforcing bar appears cut or fractured across joint.	F	Refer to Section 4 and HCG Repair Specification.	111214(18-31)

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	Grid ex.J6a	Concrete Slab	6mm horizontal crack at sawcut joint. 0.75mm diagonal crack propagates from the sawcut joint and underneath vinyl. We could not review the full extent of the crack due to the vinyl covering.	Y	Remove finishes to expose full length of crack in concrete slab. Epoxy inject all cracks in concrete slab >0.2mm as per the HCG Repair Specification [2]. For cracks greater than 0.5-0.6mm, HCG to confirm integrity of existing reinforcing steel. Refer to Section 4 and HCG specification.	111214(18-31)
G	Grid E4	Concrete Slab	15mm horizontal crack at shrinkage control joint. Reinforcing bar across gap has been wrapped in tape to allow for movement.	Y	Repair per Sectoin 4 and HCG Repair Specification.	1059
G	Grid L6a	Concrete Slab and Seismic Gap	15mm horizontal crack at seismic gap. Membrane has been turned up to provide bond breaker at this location. We could not review the integrity of the membrane at this location. The reinforcing bars appeared intact across joint.	Y	DPC/DPM repair specification by others. Refer to Section 4 and HCG Repair Specification.	111214(7-9)
G	Grid L4	Concrete Slab and Seismic Gap	10mm horizontal crack with 8mm vertical displacement at seismic gap. Membrane has been turned up to provide bond breaker at this location. We could not review the integrity of the membrane at this location. The reinforcing bars appeared intact however visibly deformed across the joint.	Y	DPC/DPM repair specification by others. Refer to Section 4 and HCG Repair Specification.	111214(10-18)
G	Seismic Gap	Floor, Walls and Ceiling	Horizontal and vertical movements to concrete floor slab at joint observed in vinyl floor covering. Series of cracks in wall and ceiling partitions measuring up to 5mm. There is no articulated wall joint at this location.	Y	Repair specifications by others.	749, 750, 751
G	Doorway bulkhead	GIB and Timber Framed Corridor Wall	Vertical and tapered diagonal cracks to wall partition.	Y	Repair and refix as required by GIB. Specification by others.	742, 743

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	Link Corridor	GIB Ceiling and Walls	2mm tapered crack and 5mm straight crack in ceiling of link corridor. Corresponds with crack in concrete slab and (Photo 744) and crack in external concrete foundation (Photo 775).	Y	Repair and refix as required by GIB. Specification by others.	745, 746
G	Ceiling	HVAC vent restraint	Vents appear to lack any seismic restraints.	F	Review by others.	759
G	North Elevation	Parapet	Loose bricks top of wall. Top of wall flexible when weight applied. Structure above ceiling inside of wall also observed, not damage visible.	C	Remove capstones and replace with metal flashing. Brace Parapets for out of plane loading. Replace existing brick veneer with new light weight brick veneer. Refer to RFI NL-RC#0073 and RFI#009 Response.	87, 88, 91-106, 111.
G	Seismic Gap of Ward Area Plant Room	Plywood bracing/ Ceiling grid	Bending of 'top-hat' section at connection to the purlin. General Purpose light gauge steel bracket does not appear adequate to transfer lateral loads from the plywood diaphragm to the precast wall panel. This connection should be analysed to confirm.	Y	Repair specification by others.	5861-5863
G	Grid L, panel PL.4, south side	Precast Concrete panel	Diagonal cracking at top of wall. Observed post Sept 2010 earthquake. No increase in cracking. Crack width approx 0.4 mm.	Y	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [2] after slab and foundations have been re-levelled.	1274-1276
G	Free-Standing wall Grid N	Precast concrete wall	No damage observed.	N		5880-5882
G	Free-Standing wall Grid T+U	Precast concrete wall	Walls visibly out of plumb. Contractor used spirit level to determine the wall was out of plumb ~25mm per metre.	Y	Remove and replace wall.	
G	Free-Standing wall Grid T	Precast concrete wall	3mm crack between precast concrete wall and foundation at interface of bedding.	F	Cracks > 0.5-0.6mm require further investigation to confirm the integrity of the steel reinforcement. Refer to HCG specification.	5871-5873

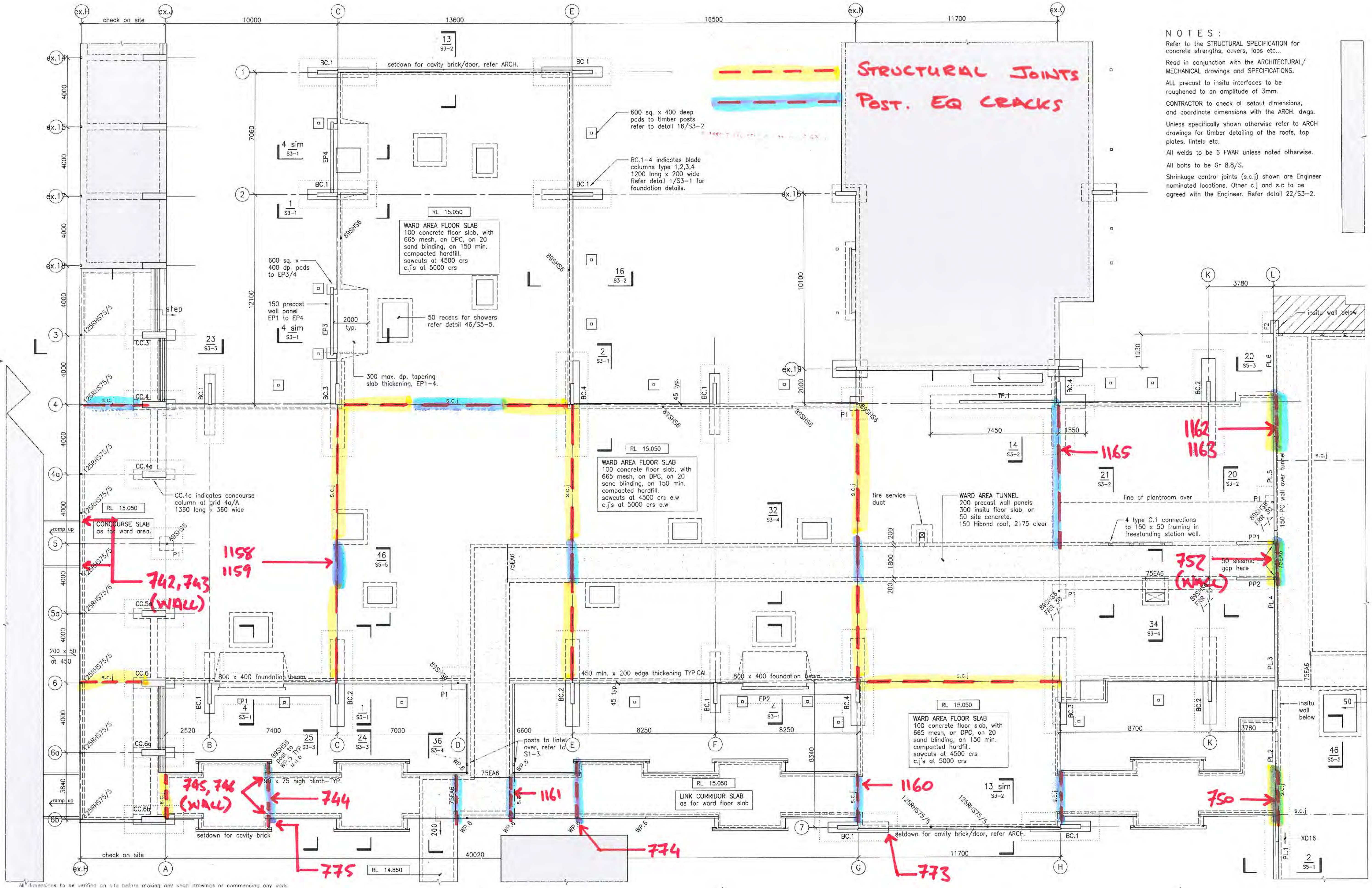
Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	Free-Standing wall Grid U	Precast concrete wall	Tapered vertical crack at centre of wall approximately 1500mm high from the top of foundation. Crack penetrates through the full depth of the wall, 0.4mm wide on the south side, 0.2mm on the north side of the wall.	Y	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [2]. For cracks greater than 0.5-0.6mm, HCG to confirm integrity of existing reinforcing steel. Refer to HCG specification.	5871-5877
G	Free-Standing wall Grid U	Precast concrete wall	No visible crack at the connection of the wall to the foundation.	N		5878-5879
G	Plant Room	Pre-Cast Concrete Wall	Series of diagonal cracks in pre-cast concrete wall adjacent to seismic gap. Worst crack in series is 0.2mm wide. Cracks propagate through the width of the wall (Photo 757 is taken from the other side of the wall.)	Y	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [2]. For cracks greater than 0.5-0.6mm, HCG to confirm integrity of existing reinforcing steel. Refer to HCG specification.	747, 748, 757
G	External Wall	Pre-Cast Concrete Wall	0.2mm tapered diagonal cracks to external wall panel.	Y	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [2]. For cracks greater than 0.5-0.6mm, HCG to confirm integrity of existing reinforcing steel. Refer to HCG specification.	768, 769
G	Grid L, panels PL.4 & PL.5-east	Precast panel to PFC wall stiffener connection	Connections identified as damaged post Sept 2010 re-inspected. Concrete has spalled from two connections. No new damage apparent. Connections above PFC inspected. No spalling of concrete.	C	Replace connection between PFC and wall. Work has been completed per SR07 and SR08.	1282, 1283, 031J, 441J
G	Grid L, PL.5-west	Precast panel to PFC wall stiffener connection	No damage observed.	N		1281
G	Grid L	Precast panel to steel truss connection	No damage observed.	N		1278-1280
G	Theatres plant room	Roof bracing	Rod bracing has slackened.	Y	Replace elongated rod bracing with new rods.	1293

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	Throughout	Service tunnel ground slabs	Evidence of liquefaction. All cracks appear to be old shrinkage cracks.	F	Refer to specification by others for repair of DBC/DPMs. Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [2]. Cracks greater than 0.5-0.6mm require further investigation to confirm the integrity of the steel reinforcement. Refer to HCG Repair Specification.	1299, 1300, 1301
G	Throughout	Service tunnel walls	Numerous cracks observed throughout tunnels. Most vertical approx 0.7 mm width. Some larger (1.4 mm width) diagonal shear cracks visible. Efflorescence already visible.	F	Refer to specification by others for repair of DBC/DPMs. Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [2]. Cracks greater than 0.5-0.6mm require further investigation to confirm the integrity of the steel reinforcement. Refer to HCG Repair Specification.	1295-1298, 1302-1307
G	Seismic Gap of Ward Area Plant Room	Steel DHS Purlin Connection	Roof appears to have moved relative to the wall for the full extent of the slotting (+/- 25mm) in the purlin connection.	N		5855-5863
G	Seismic Gap of Ward Area Plant Room	Steel DHS Purlin Connection	The ledger plate connecting the purlins to the precast wall panel appears to have pulled ~5mm off the wall in some locations. Bolts were only installed in approximately half of the holes which were drilled through the ledger for this connection.	F	Ensure integrity of exisitng bolts. Install new bolts in existing holes and/or replace damaged bolts. Refer to Section 5.	5858
G	Roof void adjacent to seismic gap	Truss and Stiffener Connections	Spalling of concrete at truss and stiffener connections to the pre-cast concrete wall at the seismic gap. Extent of damage appears to be consistent with that noted after the February 22 earthquake.	Y	Repair spalling per HCG Repair Specification.	753-756

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
G	Theatres plant room, west wall	Vertical cross bracing	Flat plate bracing has elongated and buckled.	Y	Replace elongated plate bracing.	1284, 1285
G	Theatres plant room, west wall	Vertical cross bracing	Flat plate bracing has elongated and buckled (refer Photos: 1284 and 1285 of previous inspection). No apparent change since previous observations post Feb 22 earthquake.	Y	Replace elongated plate bracing.	761

APPENDIX B

Reference / Key Plans



NOTES:

Refer to the STRUCTURAL SPECIFICATION for concrete strengths, covers, laps etc...

Read in conjunction with the ARCHITECTURAL/MECHANICAL drawings and SPECIFICATIONS.

ALL precast to insitu interfaces to be roughened to an amplitude of 3mm.

CONTRACTOR to check all setout dimensions, and coordinate dimensions with the ARCH. dwgs.

Unless specifically shown otherwise refer to ARCH drawings for timber detailing of the roofs, top plates, lintels etc.

All welds to be 6 FWAR unless noted otherwise.

All bolts to be Gr 8.8/S.

Shrinkage control joints (s.c.j) shown are Engineer nominated locations. Other c.j and s.c to be agreed with the Engineer. Refer detail 22/S3-2.

All dimensions to be verified on site before making any shop drawings or commencing any work.

Canterbury District Health Board
Te Poari Hauora ō Waitaha

HolmesConsultingGroup
STRUCTURAL AND CIVIL ENGINEERS
61 Cambridge Terrace Christchurch
Telephone : 366-3366
Facsimile : 379-2169

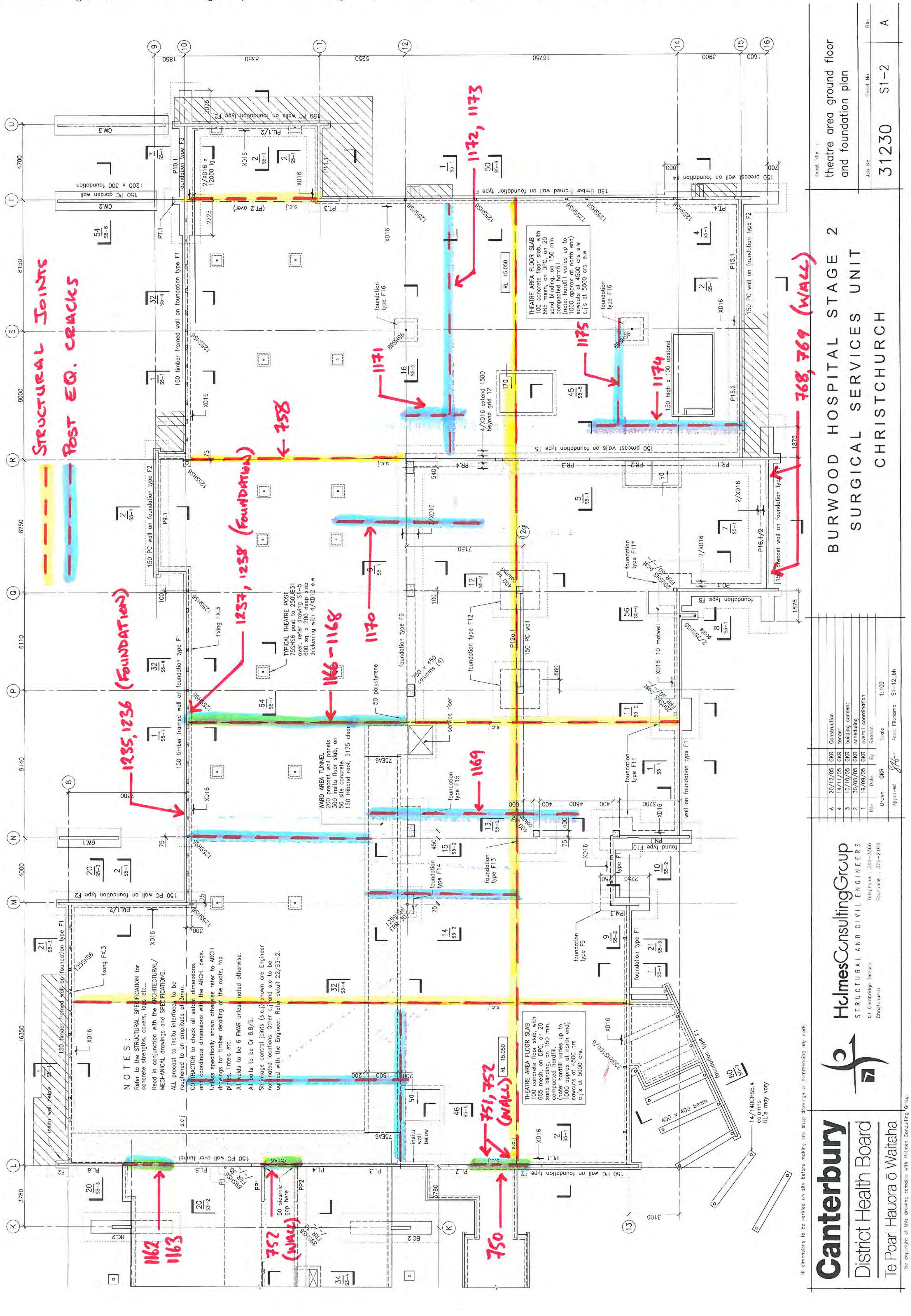
Rev	Date	By	Reason
A	20/12/05	GKR	Construction
4	14/11/05	GKR	tender
3	10/10/05	GKR	building consent
2	30/09/05	GKR	scheduling
1	19/08/05	GKR	ward co-ordination

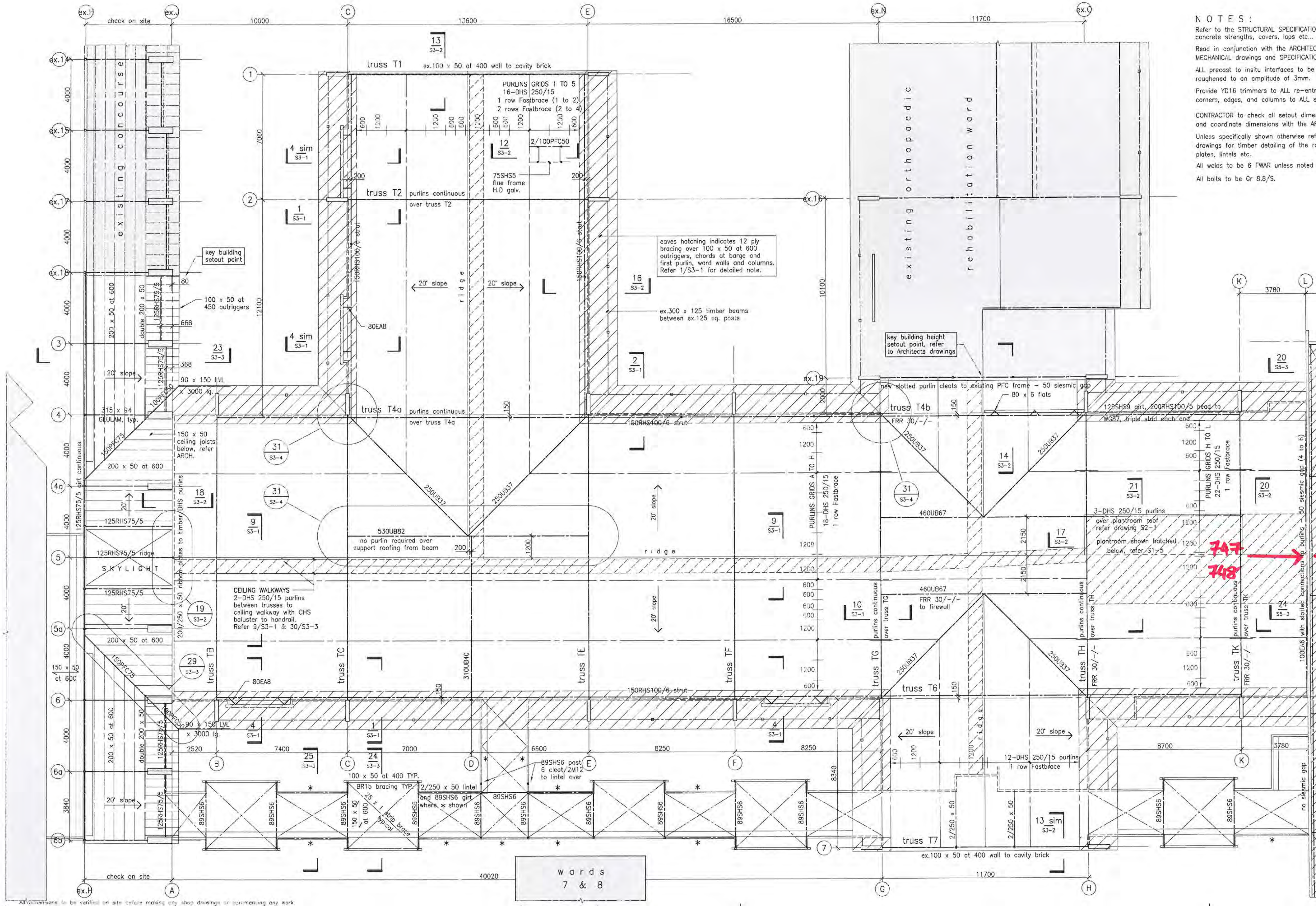
Drawn GKR Scale 1:100
Approved Acad Filename S1-12_bh

BURWOOD HOSPITAL STAGE 2
SURGICAL SERVICES UNIT
CHRISTCHURCH

Sheet Title: ward area ground floor and foundation plan

Job No	Sheet No	Rev
31230	S1-1	A


STRUCTURAL JOINTS
POST EQ. CRACKS
1235, 1236 (FOUNDATION)
1237, 1238 (FOUNDATION)
758
1166-1168
1170
1171
1172, 1173
1174
1175
1169
768, 769 (WALL)
751, 752 (WALL)
752 (WALL)
750
1162 1163
THEATRE AREA FLOOR SLAB
TYPICAL THEATRE POST
WARD AREA TUNNEL
NOTES:
BURWOOD HOSPITAL STAGE 2
SURGICAL SERVICES UNIT
CHRISTCHURCH
theatre area ground floor and foundation plan
31230
S1-2
A
HolmesConsultingGroup
STRUCTURAL AND CIVIL ENGINEERS
Canterbury District Health Board
Te Poari Hauora ō Waitaha



All dimensions to be verified on site before making any shop drawings or commencing any work.

N O T E S :

Refer to the STRUCTURAL SPECIFICATION for concrete strengths, covers, laps etc...

Read in conjunction with the ARCHITECTURAL/ MECHANICAL drawings and SPECIFICATIONS.

ALL precast to insitu interfaces to be roughened to an amplitude of 3mm.

Provide YD16 trimmers to ALL re-entrant corners, edges, and columns to ALL slabs.

CONTRACTOR to check all setout dimensions, and coordinate dimensions with the ARCH. dwgs.

Unless specifically shown otherwise refer to ARCH drawings for timber detailing of the roofs, top plates, lintels etc.

All welds to be 6 FWAR unless noted otherwise.

All bolts to be Gr 8.8/S.

Canterbury
District Health Board
Te Poari Hauora ō Waitaha

HolmesConsultingGroup
STRUCTURAL AND CIVIL ENGINEERS
61 Cambridge Terrace
Christchurch
Telephone : 366-3266
Facsimile : 379-2169

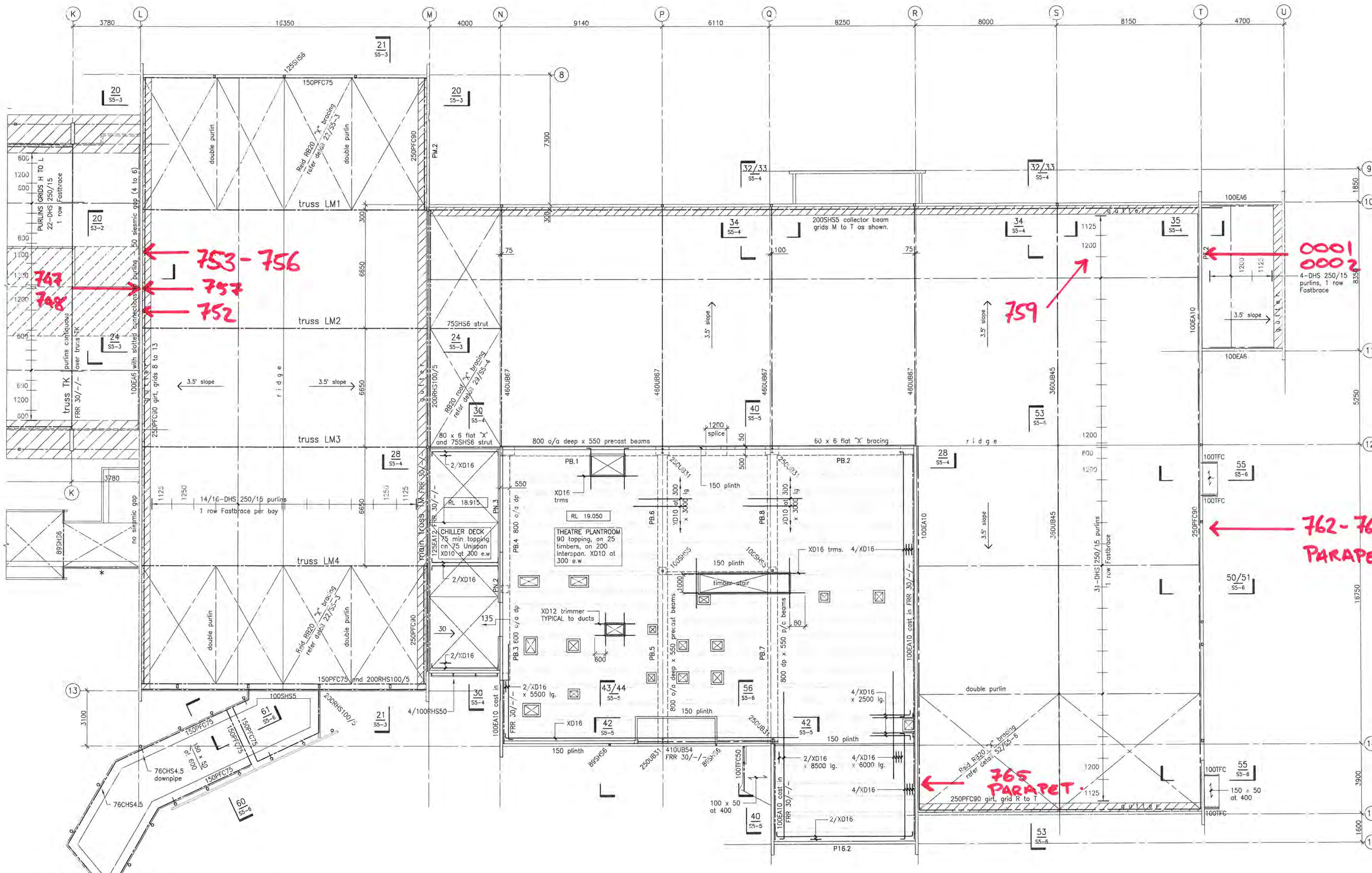
Rev	Date	By	Reason
A	20/12/05	GKR	Construction
4	14/11/05	GKR	tender
3	10/10/05	GKR	building consent
2	30/09/05	GKR	scheduling
1	19/08/05	GKR	ward co-ordination

Drawn GKR Scale 1:100
Approved [signature] Acad Filename S1-34_bh

BURWOOD HOSPITAL STAGE 2
SURGICAL SERVICES UNIT
CHRISTCHURCH

Sheet Title : ward area roof framing plan - south.

Job No	Sheet No	Rev
31230	S1-3	A



All dimensions to be verified on site before making any shop drawings or commencing any work.

Canterbury
District Health Board
Te Poari Hauora ō Waitaha

HolmesConsultingGroup
STRUCTURAL AND CIVIL ENGINEERS
61 Cambridge Terrace
Christchurch
Telephone : 366-3366
Facsimile : 379-2169

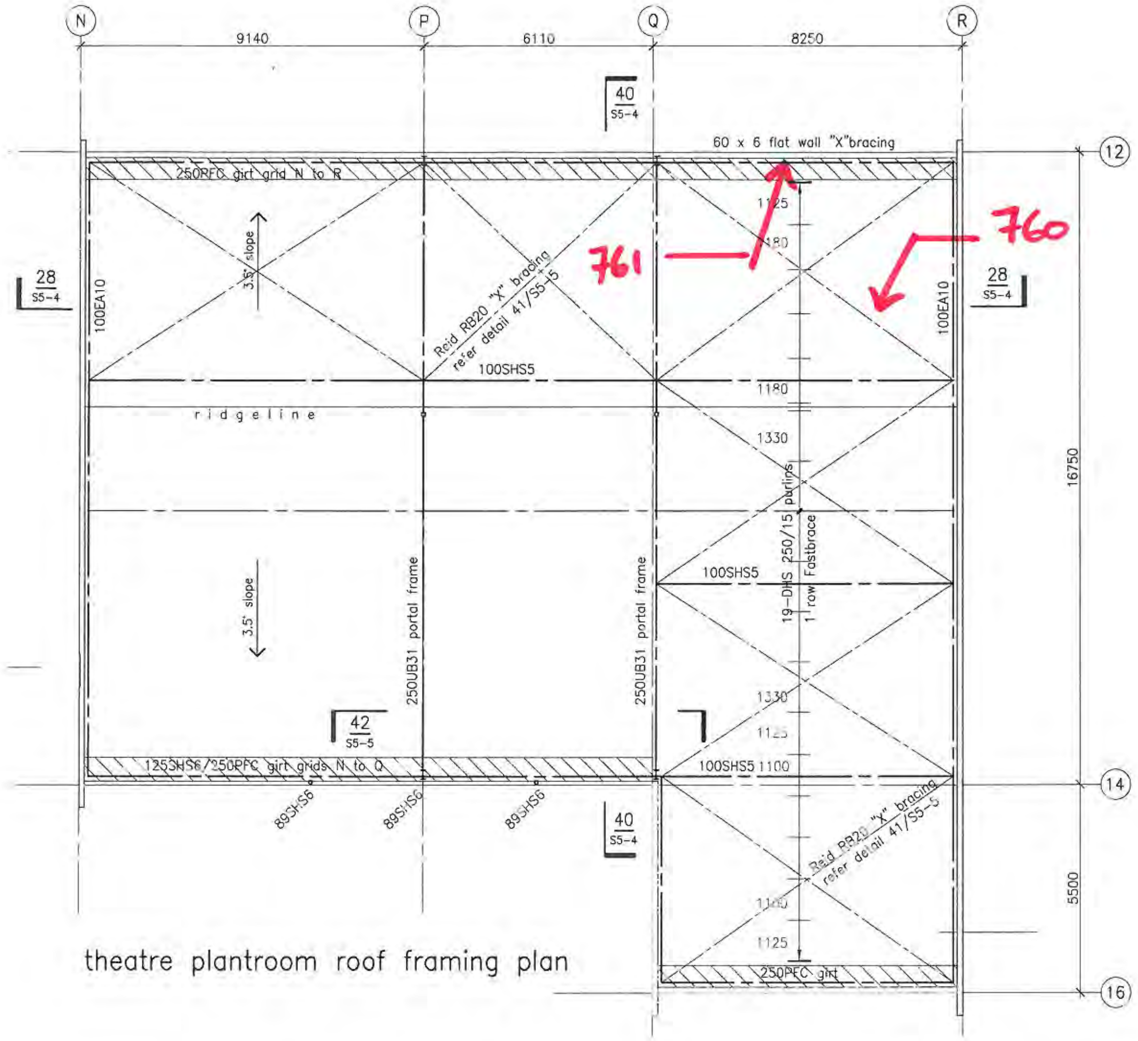
Rev	Date	By	Reason
A	20/12/05	GKR	Construction
4	14/11/05	GKR	tender
3	10/10/05	GKR	building consent
2	30/09/05	GKR	scheduling
1	19/09/05	GKR	overall coordination

Drawn GKR Scale 1:100
Approved Acad Filename S1-34_bh

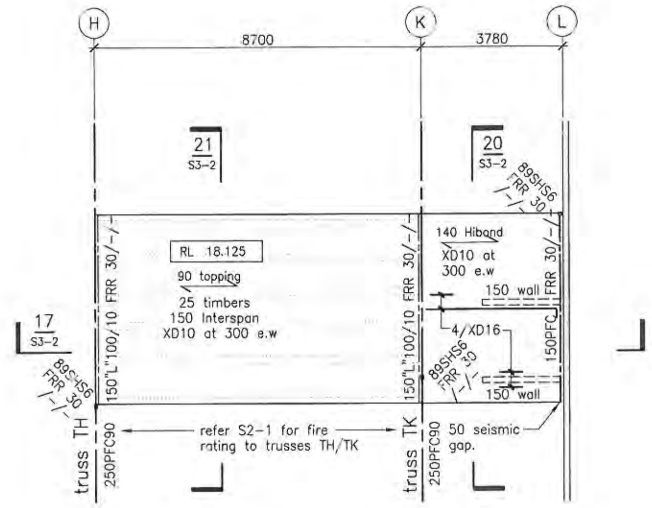
BURWOOD HOSPITAL STAGE 2
SURGICAL SERVICES UNIT
CHRISTCHURCH

Sheet Title : theatre area roof framing and plantroom floor plans – north.

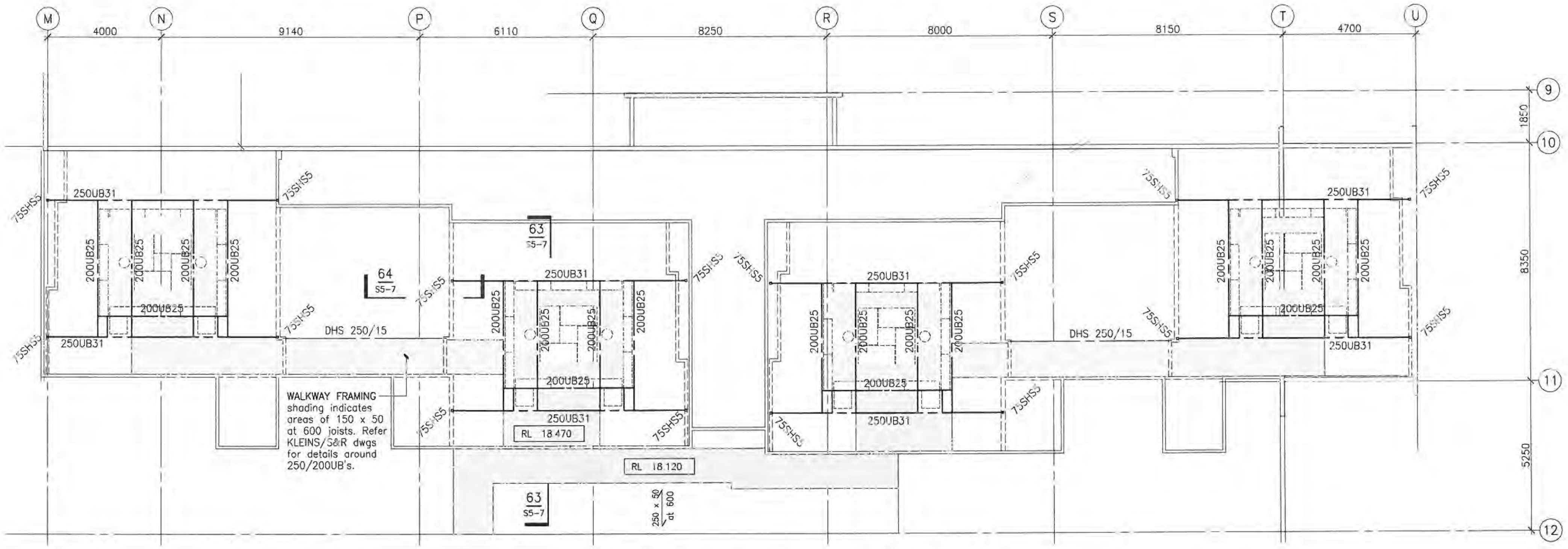
Job No	Sheet No	Rev
31230	S1-4	A



theatre plantroom roof framing plan



ward plantroom floor framing plan



theatre ceiling plan showing pendant/ceiling support

note: allow brackets for pendant supports

All dimensions to be verified on site before making any shop drawings or commencing any work.

Canterbury District Health Board
Te Poari Hauora ō Waitaha

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STRUCTURAL AND CIVIL ENGINEERS
81 Cambridge Terrace Christchurch
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Rev	Date	By	Revision
A	20/12/05	GKR	Construction
5	14/11/05	GKR	tender
4	10/10/05	GKR	building consent
3	30/09/05	GKR	scheduling
2	19/09/05	GKR	overall coordination
1	19/08/05	GKR	ward co-ordination

Drawn GKR Scale 1:100
Approved Acad Filename S1-5_bh

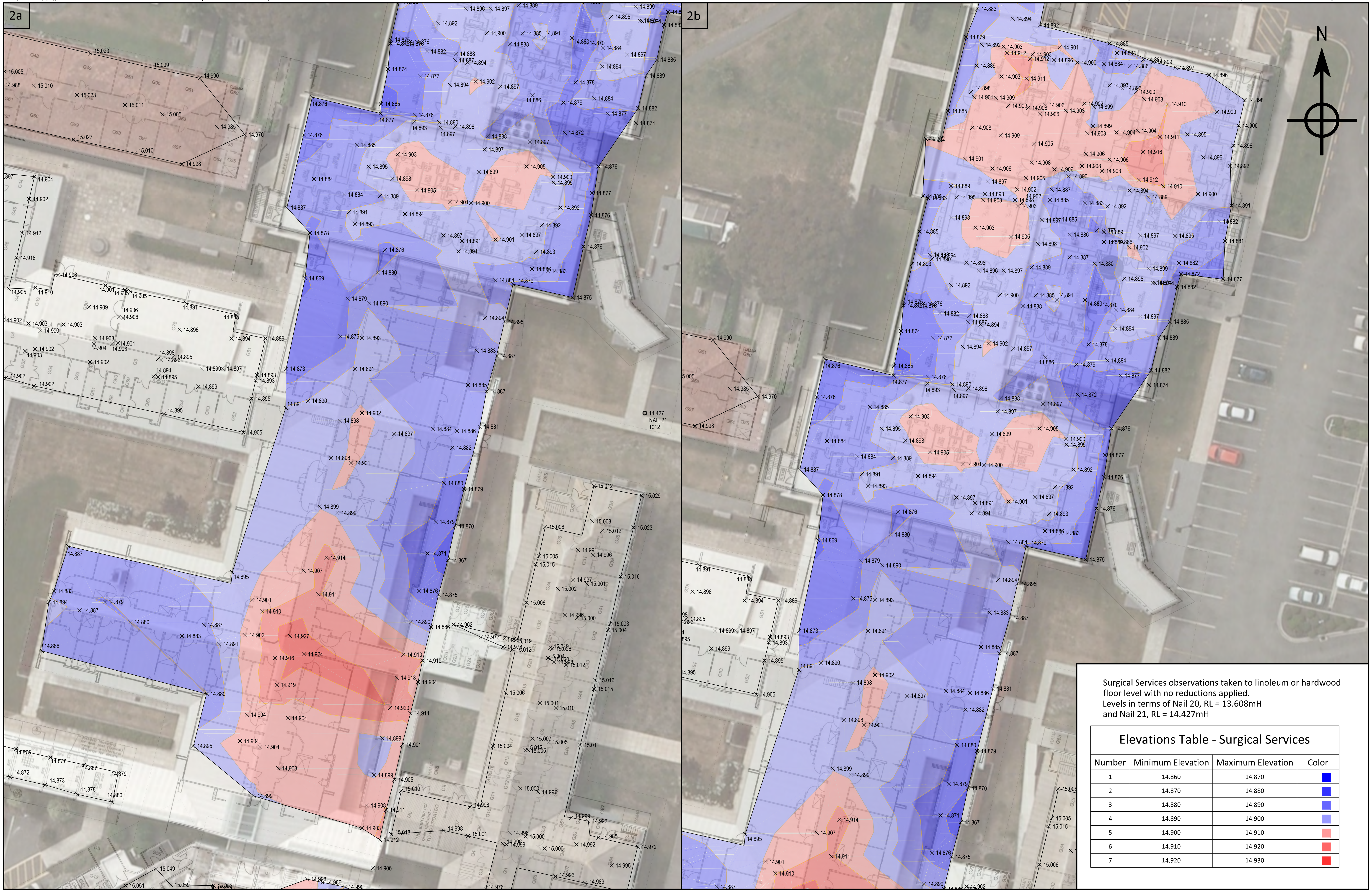
BURWOOD HOSPITAL STAGE 2
SURGICAL SERVICES UNIT
CHRISTCHURCH

Sheet Title: ward plant/theatre plantroom roof and theatre ceiling framing plans

Job No	Sheet No	Rev
31230	S1-5	A

APPENDIX C

Survey of Levels



Surgical Services observations taken to linoleum or hardwood floor level with no reductions applied.
Levels in terms of Nail 20, RL = 13.608mH
and Nail 21, RL = 14.427mH

Elevations Table - Surgical Services

Number	Minimum Elevation	Maximum Elevation	Color
1	14.860	14.870	
2	14.870	14.880	
3	14.880	14.890	
4	14.890	14.900	
5	14.900	14.910	
6	14.910	14.920	
7	14.920	14.930	

No.	Revision	Approved	Date
G	Med Records, Pain Mgmt, Admin B, and Surgical infill added	MJM	18/06/2012
F	Beacon House, BSU Hostel, Chapel, Tapper and Milner added	MJM	18/04/2012
E	Audit observations post Dec 23 2011 (not included in this Revision)	MJM	01/02/2012
D	Wards 7 & 8, Surgical Block, Melrose Chairs & Corridor added	MJM	02/12/2011
C	Phys Med, Parafed, Food Services, BIRS, Boilerhouse, EDC added	MJM	24/11/2011
B	Nurses Hostel & Champion Centre, SOU elevations added	MJM	18/11/2011
A	Administration and Allan Bean Centre elevations added	MJM	31/10/2011

Scale		1:200
Reduced	A3	1:400
Designed		----
Drawn		DHP
Checked		MJM
Date		18/06/2012

Burwood Elevations Survey

Surgical Services

SURVEYORS
Fox & Associates
URBAN DESIGNERS
LAND DEVELOPMENT
CONSULTANTS

www.foxsurvey.co.nz
0800 FOX SURVEY
P.O.Box 13-390
CHRISTCHURCH

Job No. 2462C
Sheet No. 2 of 20
Rev. G

A1 Original Size

032005

APPENDIX D

Link Corridor Strengthening

Corridor Portals – Drift Control

The corridor section of the building has been assessed at 80% DBE for strength but only 40% DBE for drift. This means that the corridor is likely to experience an undesirable level of displacement during shaking and non-structural items such as wall linings may be damaged during smaller earthquakes.

One option to bring the drift capacity up to 67% DBE is to add two 89x6 SHS sections which would be welded across the frame, approximately 350mm below the top SHS (estimated to be just above ceiling level), as shown in Figure 1 below. These should be welded all round using a 6mm fillet.

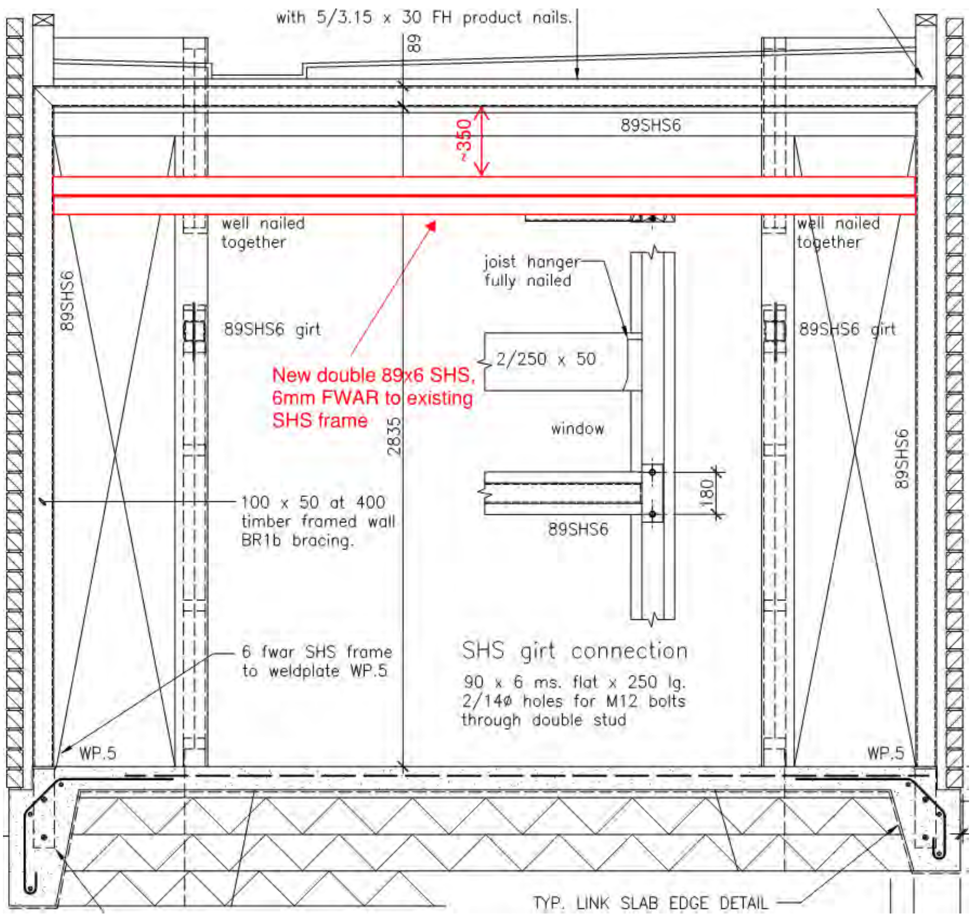
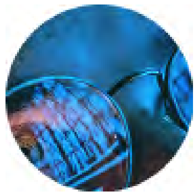


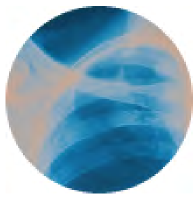
Figure 1: SHS Bracing Member to Reduce Drift

An assessment of the services present in the ceiling space should be carried out before further development of this option.



DETAILED SEISMIC ASSESSMENT REPORT

STRUCTURAL AND CIVIL ENGINEERS



BURWOOD HOSPITAL CAMPUS

REPORT 14 – SURGICAL BLOCK

PREPARED FOR

CANTERBURY DISTRICT HEALTH BOARD

106186.57

4 MARCH 2014

HolmesConsultingGroup

BURWOOD HOSPTITAL CAMPUS – DETAILED SEISMIC ASSESSMENT REPORT

REPORT 14 – SURGICAL BLOCK

Prepared For:
CANTERBURY DISTRICT HEALTH BOARD

Date: 4 March 2014
Project No: 106186.57
Revision No: 3

Prepared By:

Peter Grange
PROJECT ENGINEER

Reviewed By:

Jenny Fisher
PROJECT DIRECTOR

Holmes Consulting Group LP
Christchurch Office

REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
23/11/12	1	Interim results of quantitative assessment (Phase 3) for discussion.
19/07/13	2	Updated for rocking wall analysis.
04/03/14	3	Updated following alterations and including further investigations

CONTENTS

APPENDICES

TABLES Page

FIGURES PAGE

Holmes Consulting Group has been engaged by Canterbury District Health Board (CDHB) to complete a full structural review of the Burwood Campus following the Lyttelton Earthquake. A series of reports have been compiled as part of this process. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The specific building reports, like this one for the Surgical Block, should be read in conjunction with the base report, and refer to the repair specification.

This report identifies the structural damage sustained by the Surgical Block as a result of the series of Earthquakes, including the Darfield Earthquake that struck at 4:36am on the 4th September 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February 2011; the June Earthquake that struck at 2.20pm on the 13th of June 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The report summarises the effects of the damage on the lateral load capacity of the building, and provides structural repairs for the damage identified. The general form of the building, along with its capacity relative to current code levels, has been included for the buildings pre-earthquake undamaged state and post-earthquake state. Recommendations to increase the strength of the building to greater than 67% current code capacity have also been summarized.

The Surgical Block was constructed in 1959 and currently houses operating theatres on the ground floor level and treatment rooms on the first floor. Below the ground floor slab there are several service tunnels and a partial basement plant room. There are also timber framed additions to the building constructed in 1988. These include an additional single storey corridor added to the south end of the building, a small single storey extension to the east side of the building and an extension to the tank room on the roof. The northern edge of the building abuts the Birthing and Minor Procedures Unit, where a small seismic gap is present. In 2013, further minor alterations were carried out with the addition of two new doorways on the ground floor, cut from the existing in situ concrete walls.

The primary structural elements of the original building are constructed of almost entirely of reinforced concrete. This includes insitu roof and floor slabs, along with insitu interior and exterior walls. In general the roof and floor slabs are two way flat slabs supported by the concrete walls and a grid work of reinforced concrete beams below. The ground floor slab is supported by a combination of concrete sub-floor walls, service tunnel and partial basement walls below, which are in turn supported by shallow strip footings. The reinforcing in walls and slabs consists of smooth round reinforcing bars. This includes short laps (~380mm) of the vertical reinforcing at the base of the ground floor and first floor walls.

A 115mm brick veneer covers the exterior of the building while internal partition walls consist of 115mm thick unreinforced brickwork caped with an insitu concrete beam which extends just above the ceiling line. Above the roof slab there is a small tank room which has a light weight metal roof over timber roof and wall framing.

The information available for the review included: the original 1958 architectural and structural drawings [3,4], a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [5], along with a level survey of the building completed by Fox & Associates [6].

The majority of the Surgical Block appears to have performed relatively well considering the age of the building and the seismic actions experienced at the site. The damage to the building is typified by cracking to the concrete walls, interior brick partition walls and minor cracking to the roof and floor slabs. The damage to the walls is concentrated at door and window openings and at lintel spans over corridors. Additional damage has been noted to the exterior brick façade, dropped ceilings and to the linings of the tank room. Minor differential ground settlements have been noted, but the resulting permanent slopes in the ground floor are within the typical acceptable range for buildings of this type of construction.

It is believed that the majority of the damage observed, including the onset of damage, occurred as a result of the 22nd February event. Further observations of the earthquake damage observed have been included in the body of this report.

For the purposes of this assessment the Surgical Block has been considered to be an Importance Level 3 building (IL3).

Based upon a review of the drawings available, and the site investigations completed, the primary lateral force resisting elements of the Surgical Block were assessed in their pre-earthquake undamaged state. The assessed capacity of the main building, relative to the demand imposed by the current loading code Design Basis Earthquake (DBE), is estimated to be approximately 67% DBE in both the north-south and east-west directions. The limiting factor is the capacity of the reinforced concrete ground floor walls, in particular the ability of the short reinforcing laps to develop resisting moment at the base of the walls.

The tank room on the roof has been assessed at a capacity of approximately 35% DBE in the north-south direction and 75% DBE in the east-west direction. This capacity is limited by the span of the roof diaphragm and the observed nailing pattern of the timber framed wall linings.

The internal brick partition walls at the first floor levels have been assessed at a capacity of 15%, limited by flexural capacity in face loading. The ground floor brick partitions were secured with timber studs during July and August 2013, increasing their capacity from 15% to 67% DBE.

While the damage observed will require repair to restore the strength, stiffness, durability and performance of the individual structural components, the overall reduction in lateral load resisting capacity of the building is expected to be relatively minor. This is because the analysis completed has accounted for the likelihood of debonding to occur at the base of the ground floor walls under the ULS design basis earthquake (analysed for rocking at the base of the walls).

Based upon the extent of the damage observed to the exterior brick veneer, a detailed investigation by a qualified Mason has been carried out. This report by S A Thelning has identified the most affected areas of the veneer. Movement has occurred in a few localised areas, particularly at corners and around parapets. Repairs have since been carried out.

The minimum repairs required to reinstate the building to its pre-earthquake undamaged condition, including the replacement/securing of the brick partition walls, have been included in Section 4. In addition to the repairs of the building, recommended strengthening concepts to increase the seismic capacity of the tank room to above 67% DBE have been included in Section 5.

Strengthening of the concrete shear wall elements of the building would be extremely difficult and is not believed to be economical.

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 Canterbury District Health Board (CDHB) to complete a full structural review of the Burwood Hospital Campus following the Lyttelton Earthquake. A series of reports have been completed as part of this. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The individual building reports, like this one, should be read in conjunction with the base report and refer to the repair specification.

The Burwood base report covers the purpose and scope of the structural review. The current statutory requirements relevant to earthquake damaged buildings are outlined and the level of shaking experienced at the site estimated. Earthquake induced ground settlement damage across the campus is also discussed. The repair specification has been prepared to include repair details for typical damage observed in buildings on the Burwood Hospital Campus and is referred to as required in the specific building reports.

1.1 SCOPE OF WORK

This report is on the Surgical Block building located at the Canterbury District Health Board (CDHB) Burwood Hospital Campus, approximately 7 km north-east of downtown Christchurch. The report identifies the general form of the structure, along with the gravity and lateral load resisting systems. Each component of the structural system was reviewed based upon the information available and any potential Critical Structural Weaknesses (CSW's) were noted.

The report also identifies the structural damage observed to date as a result of the series of Earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February 2011; the June Earthquake that struck at 2.20pm on the 13th of June 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake, in particular, subjected the building to strong ground motions which significantly exceeded the current code loading demand for buildings of this nature.

The capacity of the Surgical Block has been assessed relative to current code loading in the buildings pre-earthquake undamaged state and in its post-earthquake damaged state. The post-earthquake assessment summarizes the effects of the damage identified on both the gravity and lateral load resisting elements. Repair options to restore the buildings capacity to pre-earthquake levels for strength, durability and stiffness have been included. The repair options aim to maintain the buildings utility. Where required, strengthening options have also been provided.

1.2 LIMITATIONS

Findings presented as a part of this project are for the sole use of the Canterbury District Health Board, 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. Because all of the structure has not been available for detailed inspection or evaluation, this report is limited to those elements available and engineering judgement as to the likely condition of unseen elements. 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. PRE-EARTHQUAKE BUILDING CONDITION

This section discusses the form and capacity of the building prior to the Darfield Earthquake.

The information available for the review included: the original 1958 architectural and structural drawings by Manson Seward & Stanton [3,4], a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [5], along with a level survey of the building completed by Fox & Associates [6].

2.1 BUILDING FORM

The Surgical Block is a two-storey concrete structure, constructed in 1959, and currently houses operating theatres on the ground floor level and treatment rooms on the first floor. Below the ground floor slab there are several service tunnels and a partial basement plant room. There are also timber framed additions to the building constructed in 1988. These include an additional single storey corridor added to the south end of the building, a small single storey extension to the east side of the building and an extension to the tank room on the roof. The northern edge of the building abuts the Birthing and Minor Procedures Unit, where a small seismic gap is present.

Figure 2-1: Surgical Block – South Elevation

2.1.1 Original 1959 Construction

The primary structural elements of the original building are constructed almost entirely of reinforced concrete. This includes insitu roof and floor slabs, along with insitu interior and exterior concrete walls.

In general the roof and floor slabs are two way flat slabs supported by reinforced concrete walls and a grid work of reinforced concrete beams below. The thickness of the roof and floor slabs varies between 4.5" (114mm) and 6.5" (165mm) depending on the span between the supporting elements. The width of the supporting concrete beams varies between 8" (203mm) to 16" (406mm). The total depth of the beams (including the depth of the roof and floor slabs) varies between 16" (406mm) to 24" (609mm).

At the first floor level the reinforced concrete walls are typically 7" (178mm) thick, with a smaller number of walls 8" (203mm) thick. The first floor walls land directly over concrete walls or concrete beams spanning between sections of concrete wall below. At the ground floor level the reinforced concrete walls are more numerous in number than at the first floor level and are typically 8" (203mm) in thickness. These walls extend below the ground floor level to form the partial basement, service tunnel and sub-floor walls below.

The sub-floor walls form a 600mm crawl space below the ground floor walls, while the space formed by the service tunnels and partial basement is approximately 1800mm in height. In general the walls below the ground floor level are supported by continuous reinforced concrete strip footings which vary in width from 18" (457mm) to 36" (914mm).

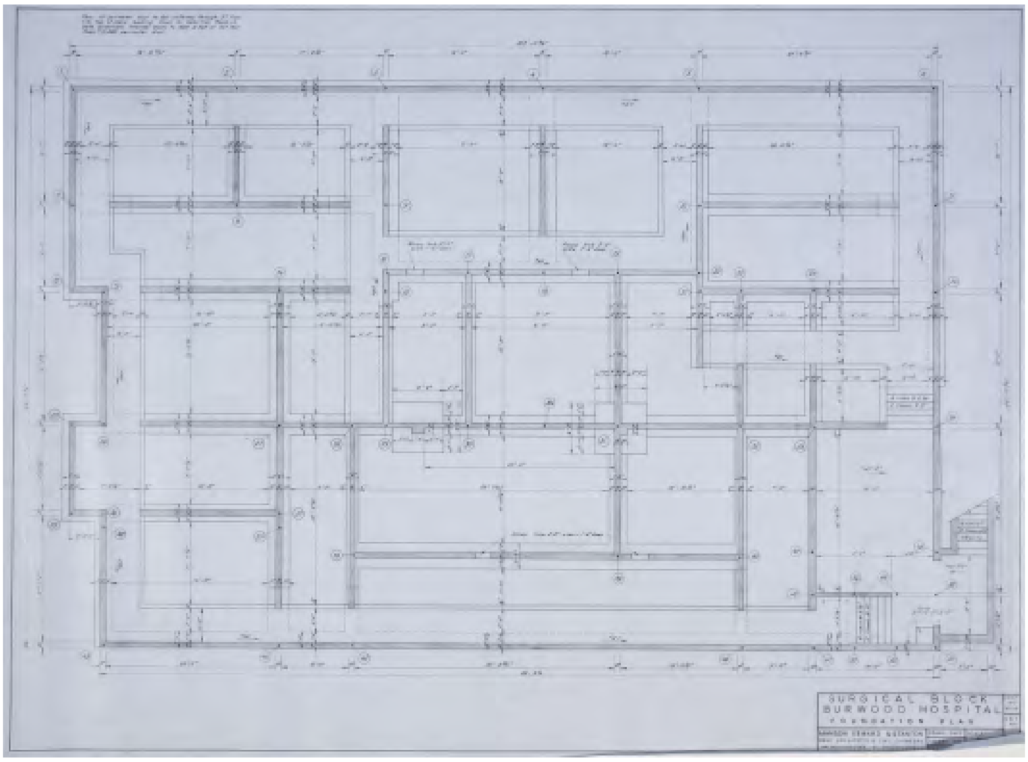


Figure 2-2: Surgical Block – Original Foundation Plan

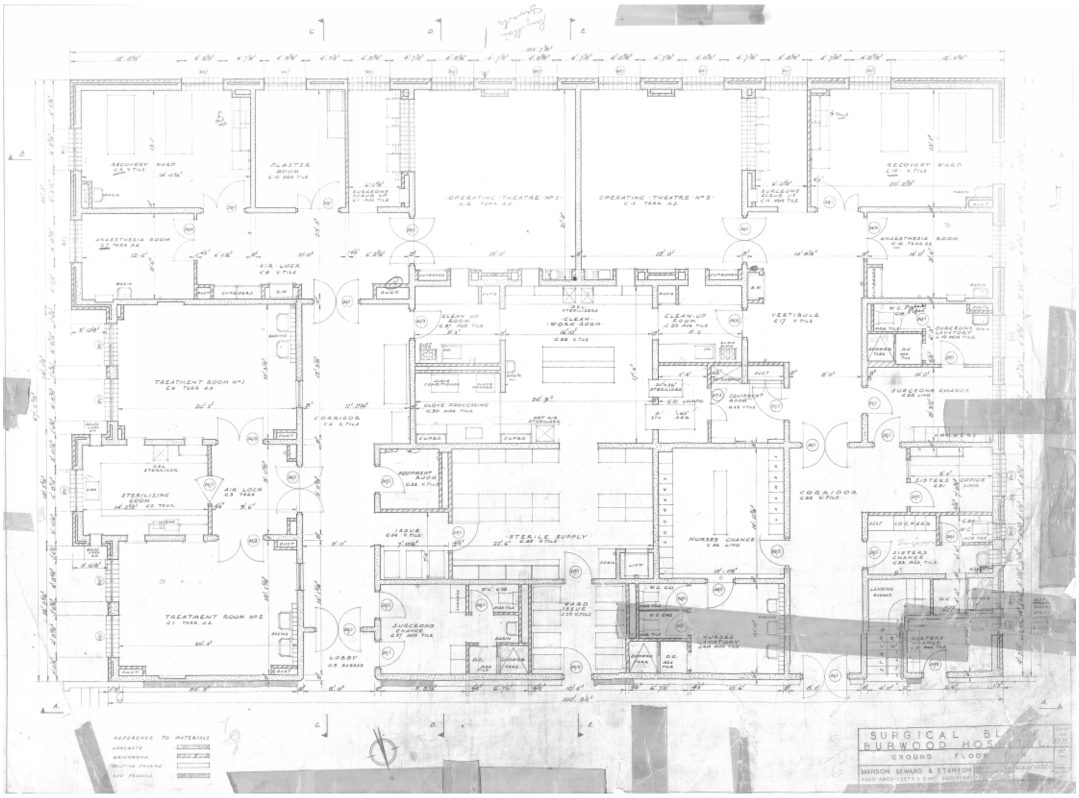


Figure 2-3: Surgical Block – Original Ground Floor Plan

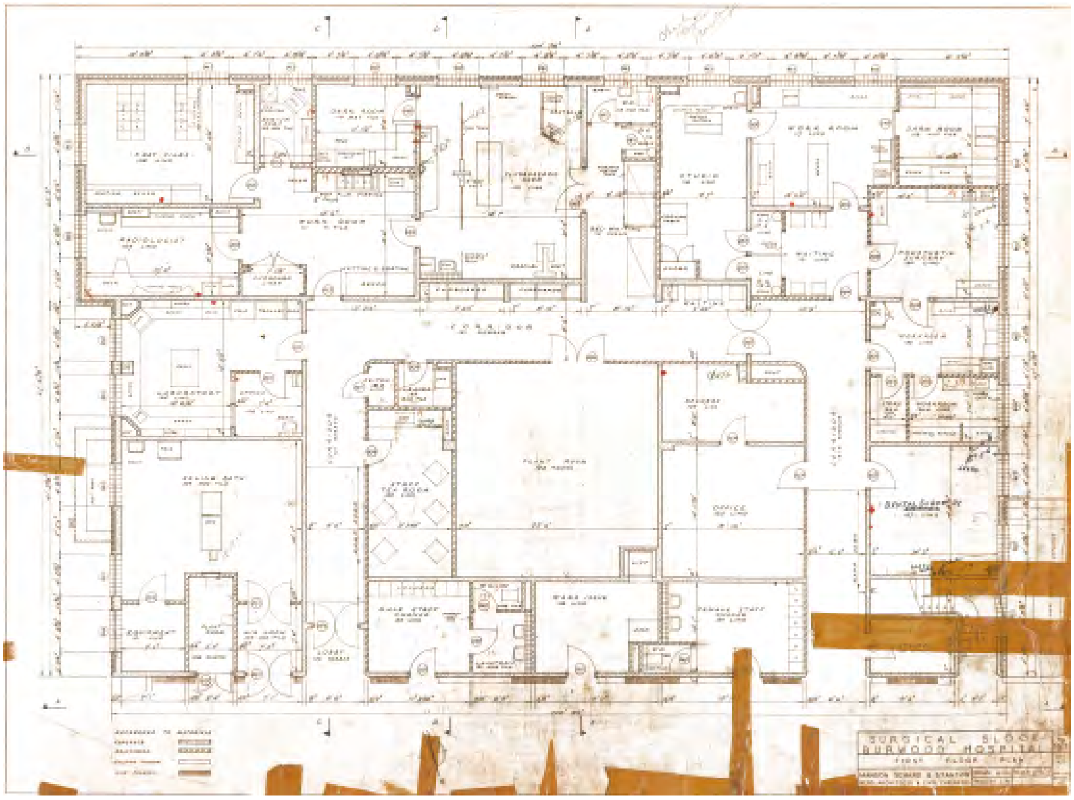


Figure 2-4: Surgical Block – Original First Floor Plan

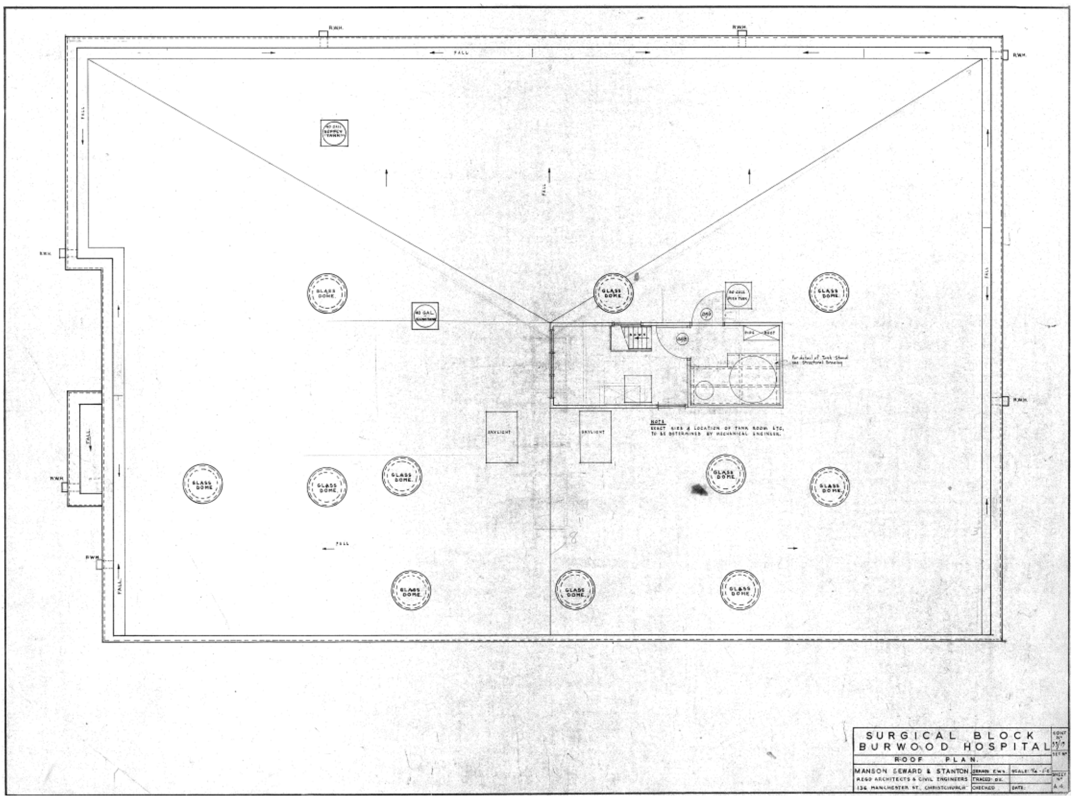


Figure 2-5: Surgical Block – Original Roof Plan

At the roof level, a 6" (152mm) thick reinforced concrete parapet, approximately 44" (1100mm) in height, extends around the southern, eastern and western extents of the building. The exterior walls of the building, including the parapet, are clad in an exterior skin of 4-1/2" (114mm) thick brick veneer with a 2-1/2" (64mm) cavity.

In addition to the interior concrete walls, there are also a number of non-load bearing 4-1/2" (114mm) thick brick partition walls with an insitu concrete capping beam. These walls typically extend just above the finished ceiling level.

The ceilings at the ground floor and first floor levels are a mixture of timber framed and steel channel supported ceilings. The ceilings are clad with plaster board linings. In several areas, including the corridors, the plasterboard ceilings have built in acoustic tiles.

Both the interior faces of the concrete and brickwork walls are typically rendered with either gypsum plaster or ceramic tile. The gypsum plaster is approximately ¾" (19mm) thick on each face.

At the first floor level there are locations where a timber floor, approximately 200mm in depth, has been added over the concrete floor slab.

The tank room, on the roof of the structure, is timber framed with a light-weight sheet metal roof. The exterior walls of the tank room are clad in vertical weatherboard, while the interior walls and ceiling are lined with plaster board linings.

Typical building elevations and sections through the building are included in Figures 2-6, 2-7, 2-8 & 2-9 below.

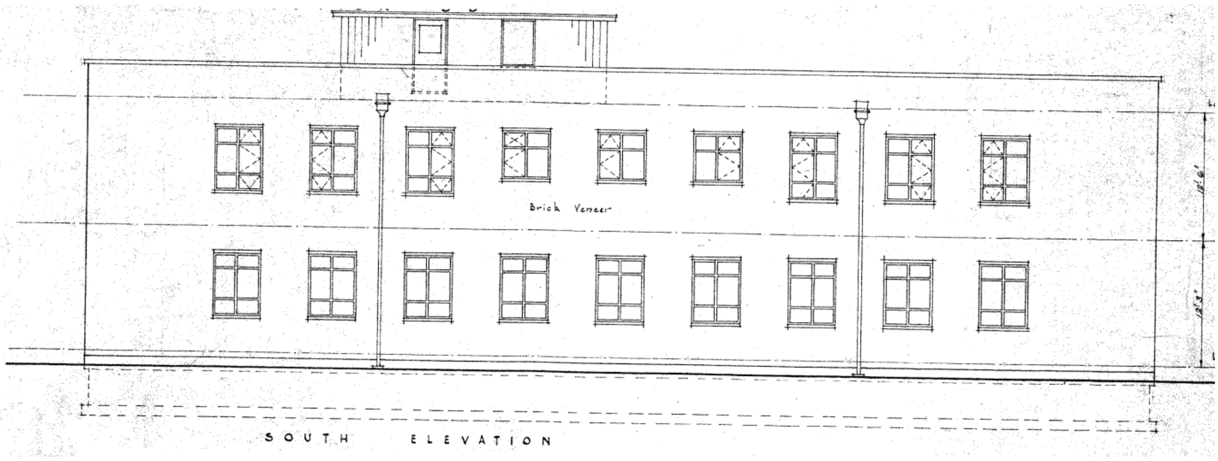


Figure 2-6: Surgical Block- South Elevation

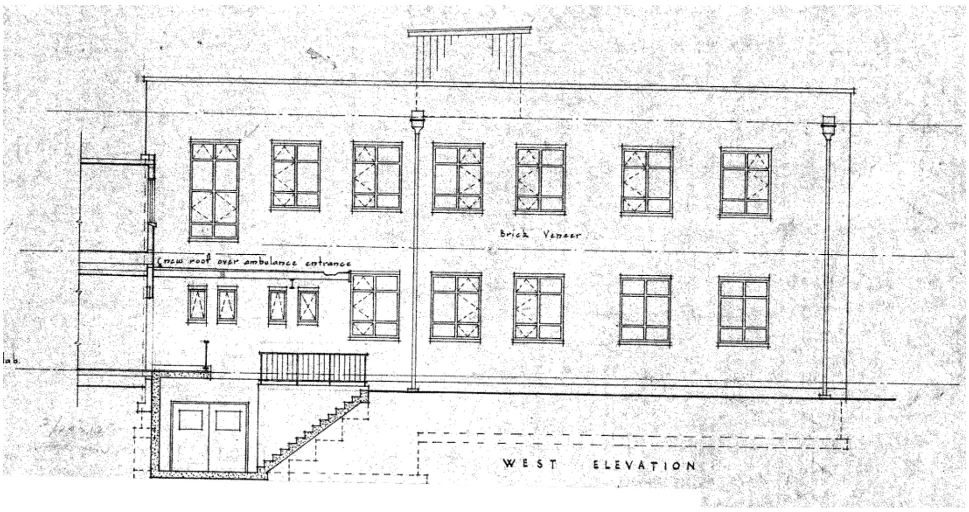


Figure 2-7: Surgical Block- West Elevation

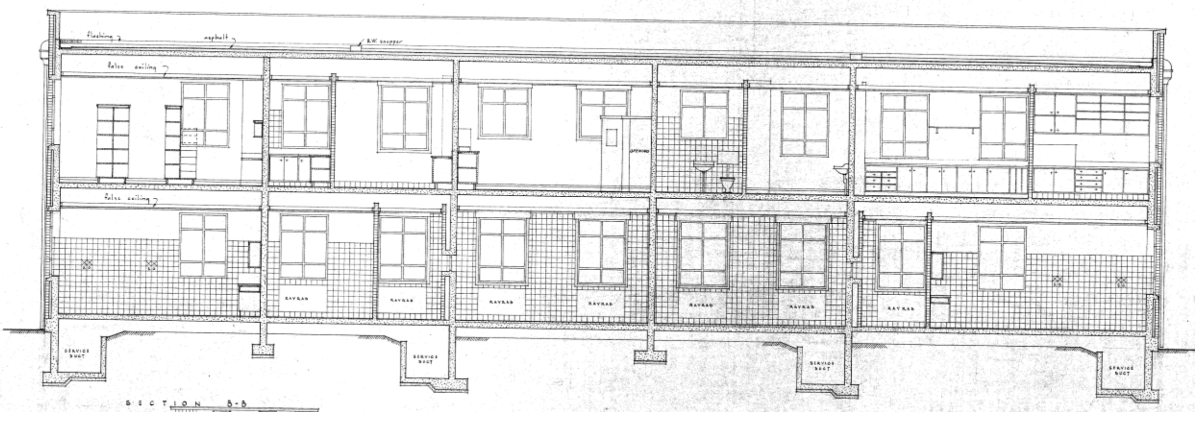


Figure 2-8: Surgical Block- East-West Building Section

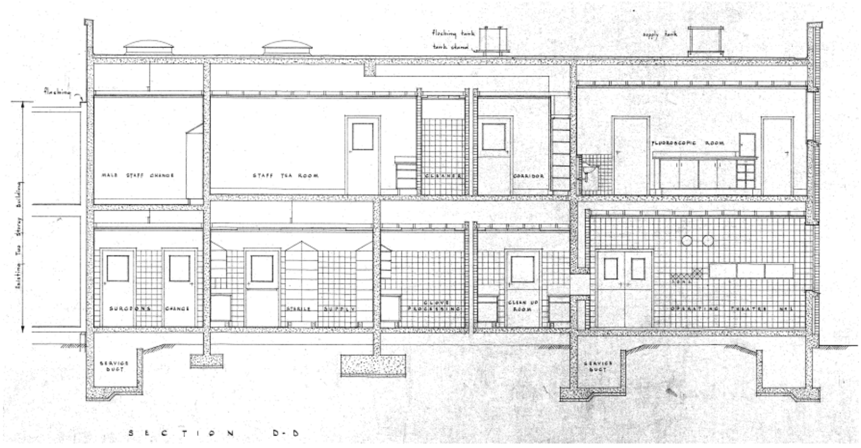


Figure 2-9: Surgical Block- North-South Building Section

2.1.2 1988 Additions

In the late 1980's several timber framed additions were added to the building. These include an additional single storey corridor added to the south end of the building, a small single storey extension to the east side of the building and an extension to the tank room on the roof.

At the ground floor level the additions consist of light-weight metal roofing over a timber framed roof and walls below. The exterior face of the walls are clad in a brick veneer while the interior face is clad in gypsum board linings. The ceiling is also timber framed with gypsum board linings. The ground floor is an elevated timber framed floor spanning between exterior concrete sub-floor walls and footings.

The addition to the tank room at the roof level consists of light-weight metal roofing over a timber framed roof and walls below. The exterior face of the walls are clad with vertically orientated weatherboard while the interior face is clad in "Hardie Board" linings. The ceiling is also clad in "Hardie Board" linings.

The additions at the foundation, ground floor and roof levels are illustrated in Figures 2-10, 2-11 and 2-12 respectively. Figure 2-11 also includes the extent of the partial basement and service tunnels of the original 1959 construction.

2.1.3 2013 Alterations

In 2013, alterations were carried out on the ground floor of the building. As well as the timber stud securing of the brick partition walls, this work included the addition of two new doorways. These doorways were created by cutting out sections of the existing in situ concrete walls. To engage the exiting steel reinforcing, the door was overcut by 100mm and small steel plates were welded onto the end of the bars and grouted in. Many of the concrete walls were timber lined for aesthetic purposes.

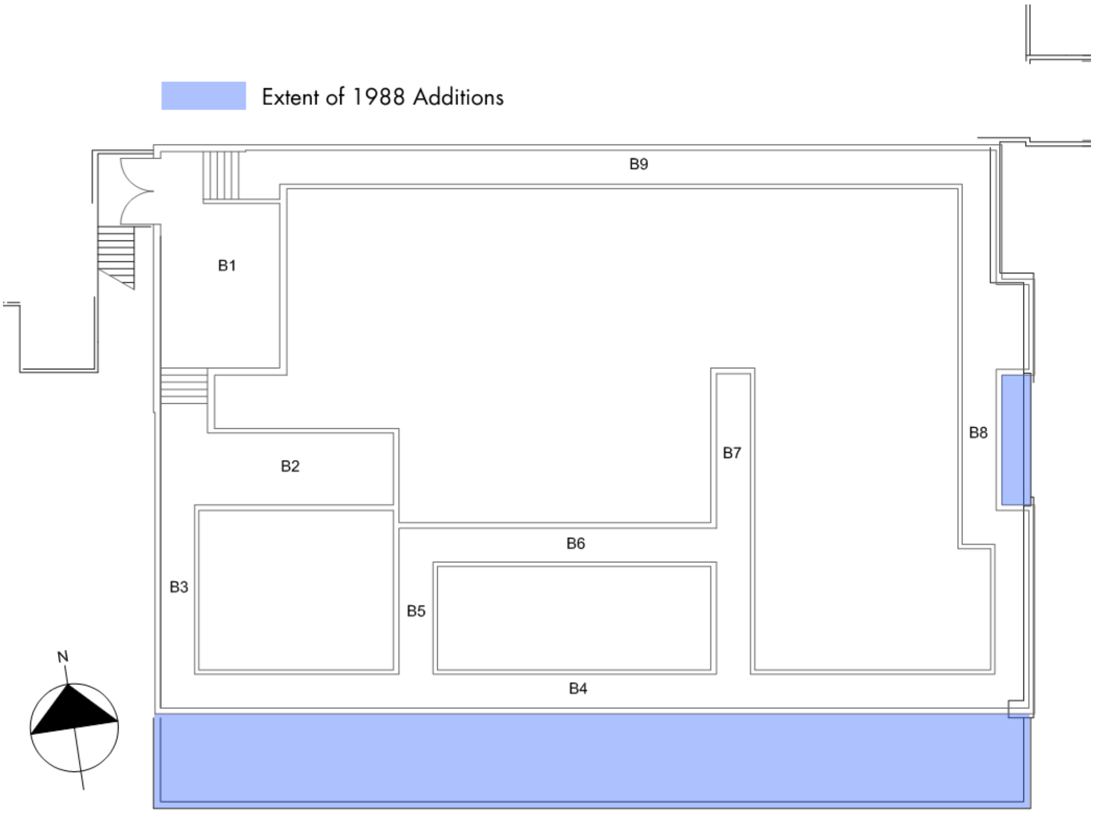


Figure 2-10: Surgical Block – Foundation Plan
(including 1988 Additions)

Figure 2-11: Surgical Block – Ground Floor Plan
(including 1988 Additions)

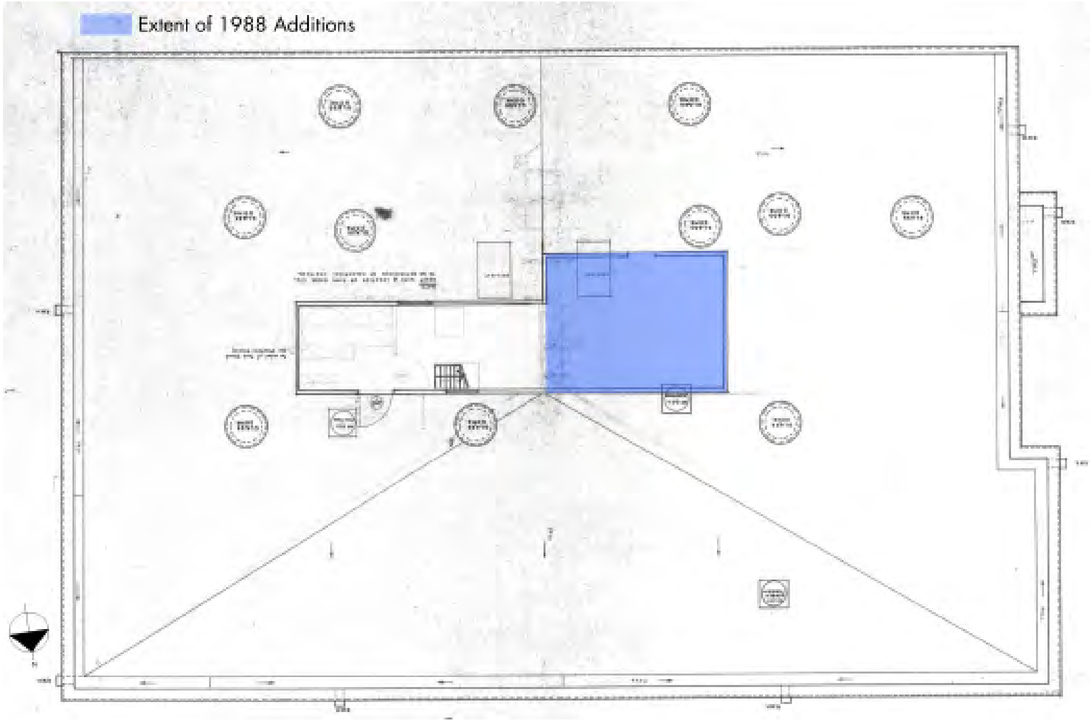


Figure 2-12: Surgical Block – Roof Plan
(including 1988 Additions)

2.2 LATERAL LOAD RESISTING SYSTEMS

2.2.1 Original 1959 Construction

The primary lateral load resisting system for the Surgical Block consists of reinforced concrete shear walls and a reinforced concrete roof, first floor and ground floor slab. The roof and floor slabs act as rigid diaphragms to distribute lateral loads to the concrete walls below. In general, the concrete shear wall bracing lines at the first floor level align with the bracing lines at ground floor level below. Likewise the wall lines at the ground floor level align with the sub-floor, service tunnel or partial basement wall lines below, and are all founded on continuous reinforced concrete strip footings. In a number of locations the first level shear walls are partially supported by concrete transfer beams which span between the ends of concrete shear walls below.

Typical slab and wall elevation details are shown below in Figures 2-13 & 2-14.

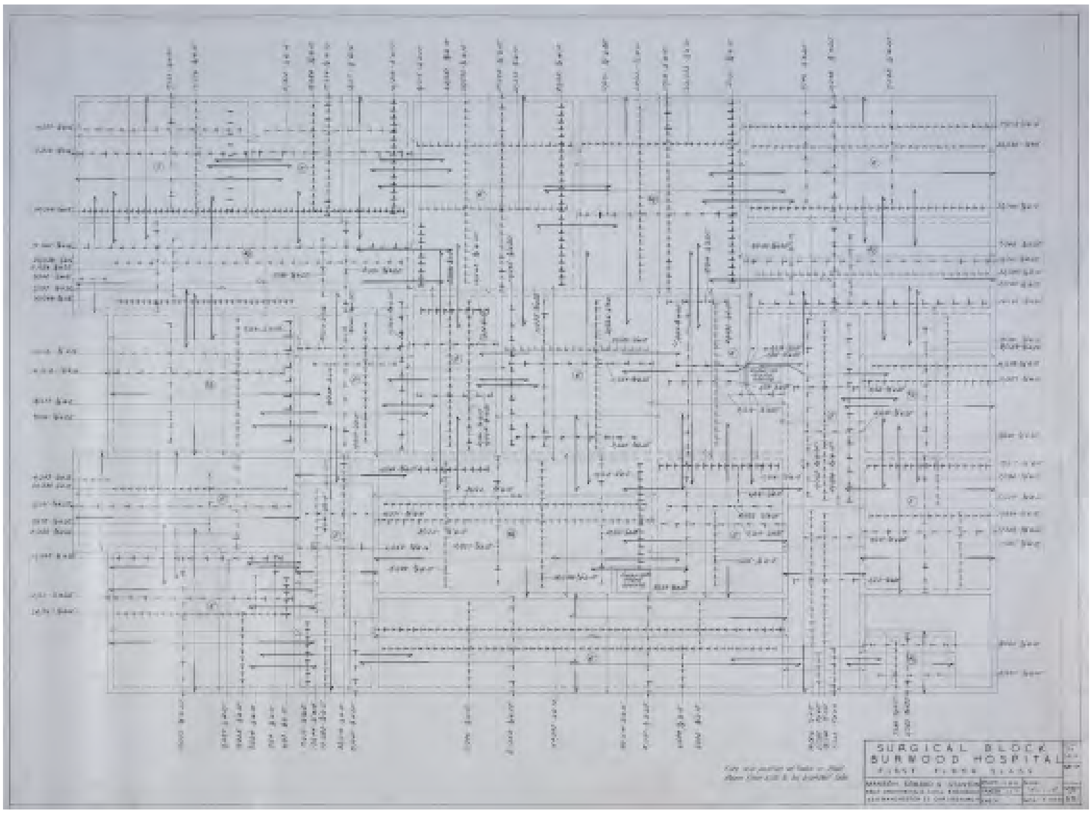


Figure 2-13: Surgical Block – Original Slab Plan

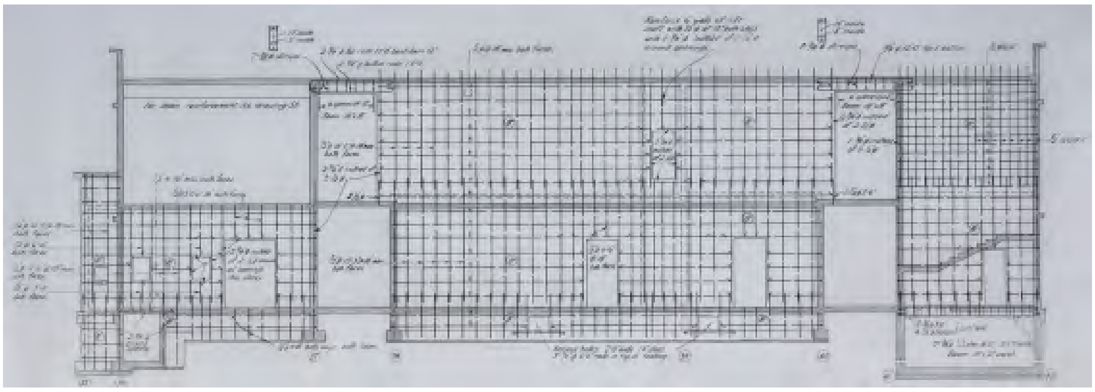


Figure 2-14: Surgical Block – Original Wall Reinforcing Elevation

The reinforcement for the insitu elements consist of smooth round mild steel bars. In general, hooks have been provided at the ends of the horizontal steel reinforcement, placed in the slabs, beams and walls. For the vertical reinforcement in the walls short straight laps have been provide just above the slab level at the ground and first floor levels. At the roof level the reinforcement hooks into the roof slab or continues on into the pararpet.

Destructive testing was completed to determine indicative lap lengths for the vertical reinforcement as this information could not be ascertained off of the existing drawings. The investigations revealed a straight contact lap of approximately 380mm (likely a specified 12” lap).

Figure 2-15: Vertical Wall Reinforcing Lap

The interior brick partition loads do not provide in-plane resistance to lateral loads. As previously noted they extend just above the dropped ceiling line. The ceilings provide lateral restraint at the top of the walls for out-of-plane or face loading.

At the tank room on the roof, bracing is provided by the "Hardie Board" linings of the timber framed walls and ceilings.

2.2.2 1988 Addition

At the corridor addition on the south end of the building, lateral forces in the north-south direction are resisted by the original concrete construction of the Surgical Block. In the east-west direction lateral loads are resisted by a combination of the exterior timber framed bracing walls of the addition and the concrete shear walls of the original Surgical Block.

The small addition on the east side of the building is entirely supported by the original Surgical Block construction.

2.3 PRE-EARTHQUAKE BUILDING CAPACITY – DIRECT CODE COMPARISON

The building capacity under earthquake actions discussed in this section is compared to the capacity that a similar building would be designed to today. A new building would be designed to the *Structural Design Actions Standard, Part 5: Earthquake Actions – New Zealand*, NZS 1170.5:2004 [10] and incorporating the amendments made to this standard as a result of the Lyttelton Earthquake as outlined in the Amendment 10 of the Building Code [9]. The implications of the recent amendments are discussed more fully in the Burwood Hospital Campus Base Report, however, for a building of this type the amendments essentially result in an increase to the design loads of 36 % when compared to pre-earthquake design levels.

The original structural drawings for the building are available, but the structural calculations and specifications are not, so the exact design and loading assumptions originally made are unknown. For the purposes of this report seismic loading assumptions have been made based on a detailed review of the drawings available and physical observations of the building.

When the building was originally designed in the 1950s, the loading standard at the time was the *New Zealand Standard Model Building Bylaw – Part IV, Basic Design Loads To Be Used And Methods Of Application*, NZSS 95: 1955 [12]. When these By-Laws were written, neither the seismology of the different areas within New Zealand, or the impact this could have on buildings was as well understood as it is today. Along with an increase in the seismic demands required by the change in the loading code over this period, the seismic detailing requirements have also progressed significantly resulting in more ductile and better performing buildings.

The current seismic loading code, NZS 1170.5, requires a new building to be designed for an earthquake, known as the Design Basis Earthquake (DBE), which is based upon the buildings physical location, local soil conditions, building type, fundamental period and importance level.

The Surgical Block is not regarded as an essential hospital facility by the CDHB and has been classified as an Importance Level 3 building in accordance with NZS 1170:2004 [10] The associated return period of the DBE is 1000 years, with a risk factor for design of R = 1.3 (no post-disaster or special function). The sub soil for the site is taken as Soil Type D, which is consistent with the findings of a post-earthquake geotechnical investigation [5].

Based upon the period of construction, and the detailing of the lateral load resisting elements, the concrete portion of the building has been assessed as a rocking wall system. As such a ductility factor of μ=2.0 has been used for the purpose of this comparison.

A comparison between the Design Basis Earthquake of NZSS 95: 1955 and NZS 1170:2004 for the site and type of construction are plotted below. Based upon a fundamental building period below 0.50 seconds, the seismic demands required by the loading code have increased on the structure by approximately 600% since 1955.

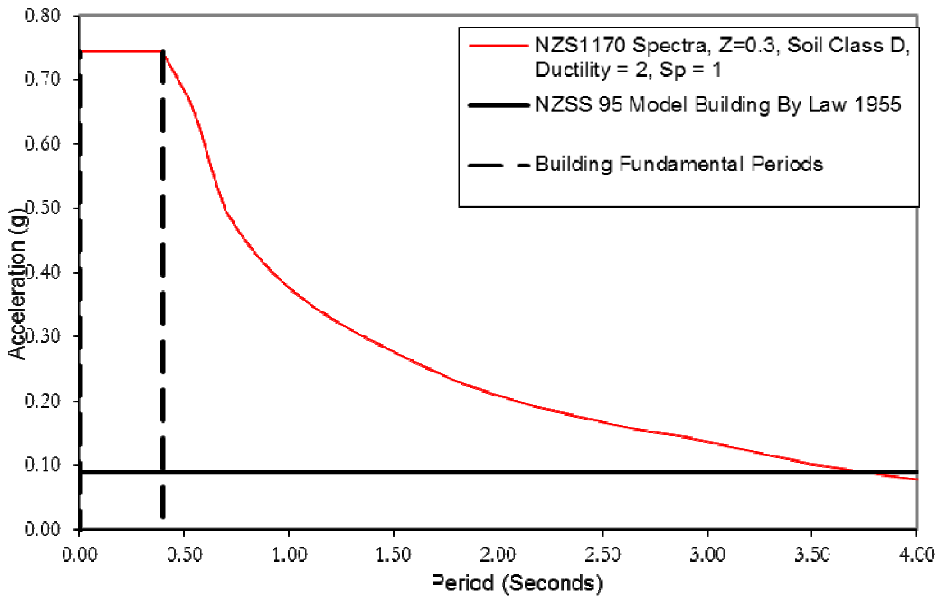


Figure 2-16: Comparison of Design Codes

2.4 EQUIVALENT STATIC ANALYSIS TO NZS1170.5 (2004)

In addition to the direct code comparison provided above, an equivalent static analysis to NZS1170.5: 2004 [10] has been carried out to gain a better understanding of the buildings estimated 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.

Following the Lyttelton earthquake, a geotechnical report was conducted by Tonkin & Taylor, titled "Burwood Hospital Post Earthquake Geotechnical Assessment", dated June 2011 [5]. This report has been used to aid in the evaluation of the site conditions and the likely effect of the ground on the buildings past and future performance. The soil parameters described in the geotechnical report completed by Tonkin and Taylor have been used for the evaluation of the buildings existing foundation system.

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 [17] and the requirements of NZS 1170:2004. 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. These are described in more detail in the Burwood Hospital Base report [1] and include strength governed elements such as short columns and deflection governed elements such as floor and stair elements with inadequate support seating.

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 (DBE), and have been expressed as a %DBE. This includes checks for both the strength and deflection requirements.

The structural analysis program, ETABS, by Computer Structures, Inc was used in the aid of the equivalent static analysis of the Surgical Block.

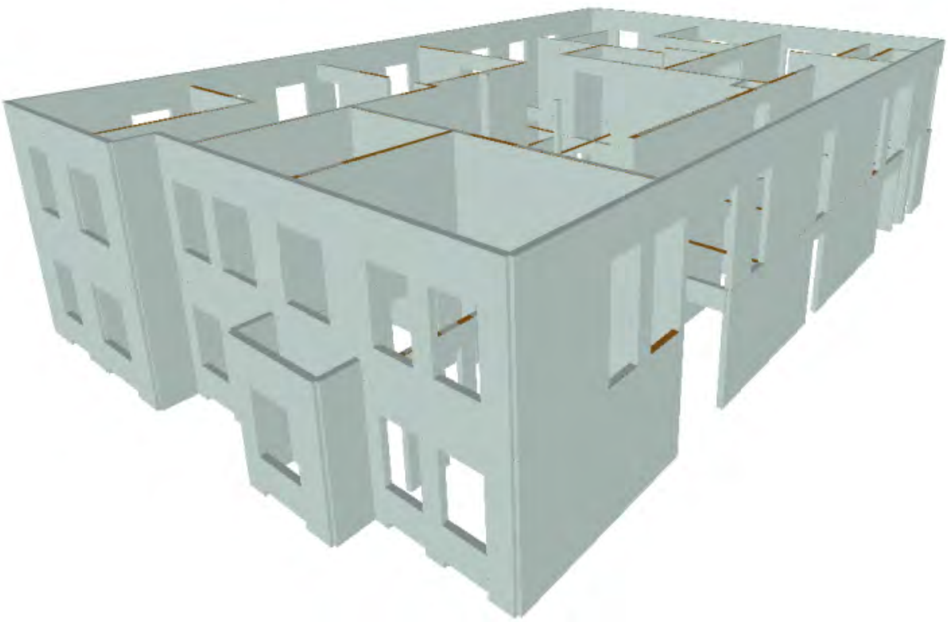


Figure 2-17: 3D Showtime Image of ETABS Model

For the purpose of this evaluation several assumptions had to be made in regards to the existing material properties of the building. This included the assumed strength of the reinforced concrete walls (25 MPa) and the assumed grade of the smooth round reinforcing bars (33 ksi or 227 MPa).

The foundations have been assessed at an ultimate bearing capacity of 150kPa, for factored loads, as per recommendations provided by Tonkin and Taylor [5].

Based upon this information, an initial equivalent static analysis was completed on the building. For an assumed ductility, $\mu = 1.0$, the short reinforcing bar laps at the base of the ground floor walls were found to fail in tension for several walls in each direction at approximately 40% DBE. This is based upon a reduced lap capacity as per the recommendations provided in ASCE 41-06, *Seismic Rehabilitation of Existing Buildings* [18].

For a more realistic estimate of the current capacity, the surgical block was then reanalysed with a rocking wall mechanism. This means that the walls have been considered to rock as their capacity is reached, and return under their own weight and the weight of concrete diaphragm above. This method of analysis is appropriate because of the global stability provided by large number of concrete walls in both directions.

The rocking wall mechanism is based around a ductility value of two, μ=2.0, which means that some damage should be expected before the full capacity is realised. Due to the construction of the building, this damage will be mostly in the form of cracking along the bottom of the walls and some spalling of concrete at the ends of the walls. As the natural period of the building is very short because of the quantity and stiffness of the walls, the expected deflections are very low, even during a ULS event.

This analysis has increased the likely capacity of the building to around 50% DBE. This is considered to be a lower bound estimate which is somewhat conservative as contribution from the round steel bars as a tie down for the walls is completely neglected. If the steel in the walls does form sufficient bond with the concrete, the likely capacity of the building would be over 100% DBE. It should be noted however, that the lap of the bars is shorter than the required length for a deformed bar, therefore it is unlikely that these will have sufficient bond. We believe that the real capacity of the building is likely to be somewhere between the two and that the building is very likely to perform at or above 67% DBE with some redundancy.

A summary of the %DBE for each primary element, assuming rocking at the base of the ground floor walls, has been noted in Tables 2-1, 2-2 & 2-3.

Building Element	%DBE (IL3)	Comments
Roof concrete slab – N-S ……………………E-W	100% 100%	
First floor shear walls – N-S ………………………..E-W	67% 75%	Limited by shear capacity of walls in North-South direction
First floor concrete slab – N-S …………………….….E-W	100% 100%	
Ground floor shear walls – N-S …………………….…...E-W	67% 67%	Limited by resisting moment that can be developed at base of walls
Brick partition walls: ground floor	67%	Capacity of URM walls was 15% DBE before ground floor walls were secured with timber studs in Jul/Aug 2013
Brick partition walls: first floor	15%	Limited by flexural capacity under face loading

Table 2-1: Superstructure – Seismic Assessment %DBE

Building Element	%DBE (IL3)	Comments
Ground floor concrete slab – N-S …………………………….E-W	100% 100%	
Sub-floor walls – N-S ……………….....E-W	90% 90%	
Foundations – N-S E-W	100% 100%	

Table 2-2: Sub-floor – Seismic Assessment %DBE

Building Element	%DBE (IL3)	Comments
Timber framed walls – N-S ………………………E-W	40% 100%	Capacity limited by nail spacing. Capacity of bracing material in N-S otherwise = 50%
Roof diaphragm – N-S …………………..E-W	35% 75%	Capacity limited by material strength and span of diaphragm

Table 2-3: Roof Tank Room – Seismic Assessment %DBE

A review of the drawings available and site observations revealed no obvious Critical Structural Weaknesses (CSW's) that would be expected to lead to premature collapse of the building. The short laps of the smooth reinforcing bars have been accounted for in the assessed capacities noted in the tables above.

3. POST EARTHQUAKE BUILDING CONDITION

This section covers the structural damage sustained by Surgical Block at Burwood Hospital Campus as a result of the series of earthquakes that includes the Darfield Earthquake that struck at 4:36am on 4th September 2010 and the Lyttelton Earthquake that struck at 12:51pm on the 22nd February 2011, the June Earthquake that struck at 2.20pm on the 13th of June 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake subjected the building to strong ground motions which likely exceeded the full design earthquake load for buildings of this nature and appears to have caused the bulk of the earthquake damage observed after the initial Darfield event.

3.1 THE LYTTELTON EARTHQUAKE

The fundamental period of the building is estimated to be approximately 0.4 seconds. 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 strong motion data presented in the Base Report, it appears the Lyttelton Earthquake produced ground shaking intensities of approximately 60-120% of the current Ultimate Limit State design spectra for an Importance Level 2 building.

It should be noted that the Lyttelton Earthquake was very short in terms of strong shaking produced with the strong motion only lasting for duration of 5-7 seconds. Rupture of the Alpine Fault is expected to contain 50 to 60 seconds of strong motion.

3.2 PRELIMINARY INVESTIGATIONS

Preliminary investigations were carried out to identify areas of the building likely to be subject to damage, and therefore requiring specific attention during the detailed assessment. The areas identified for detailed investigation have been selected based on:

- typical damage expected for buildings of this form
- review of available structural engineering construction documentation
- damage observed during 'Rapid' Structural Assessments following the Lyttelton Earthquake, June 13th aftershocks and subsequent events

In conjunction with a review of the available drawings for the building the following areas were identified for potential damage:

- movement or damage to structure associated with ground movement and/or settlement

- cracking and joint failure of concrete sub-floor walls, service tunnels and foundations
- cracking in concrete shear walls or floor diaphragms
- signs of distress in external brick veneer
- distress and cracking of plaster ceiling linings
- signs of distress at the interface between the Surgical Block and adjacent Birthing and Minor Procedures Unit
- damage at interface between original construction and 1988 timber framed additions
- damage to lightweight roof tank room

Rapid Level 2 assessments were carried out on the 24th February 2011[22] and on the 14th [23] and 15th June 2011 [24] following the June 13th earthquakes. Two additional Rapid Visual Structural Assessment was conducted on 24th December 2011 [25] and 5th January 2012 [26], following the 23rd December 2011 and 4th January 2012 events. These structural observations involved a complete walk around the exterior and throughout the interior of the building. The following primary areas of damage were identified from the damage assessments:

- external cracking in brick veneer at the south eastern and south western corners
- cracking of plaster render on concrete walls
- vertical and diagonal cracking of corners of door openings

A review of the above information on the building type and preliminary observations highlighted this building as requiring a more detailed inspection. The aim of the detailed inspections was to determine the full extent of the damage caused to the building, particularly those elements identified for potential damage above. These areas were targeted to identify if damage had occurred, and to what extent the damage had reduce the capacity of the buildings lateral load resisting system to resist future seismic events.

3.3 DETAILED STRUCTUAL OBSERVATIONS

Further detailed inspections and structural explorations have been carried out following the initial assessments to ascertain the full extent of structural damage. The majority of the detailed structural observations were completed on the 24th of April 2012, with additional observations completed on the 3rd May 2012 to inspect the partial basement and service tunnels, and on the 9th May 2012 to review the first floor plant room and roof tank room.

A full record of these observations can be found in Appendix A, with reference plans describing the location labelling used found in Appendix B. A full photographic record of the observations is available electronically on request. The detailed structural observation identified the following additional damage to those items noted in the initial rapid assessments:

- additional cracking to concrete shear walls at both the ground floor and first floor level
- cracking to the internal brick partition walls at both the ground and first floor levels
- cracking to the concrete service tunnel walls

- cracking to floor and roof slab finishes
- damage to the timber framed and plaster lined dome shaped skylights in the roof plane
- additional cracking in the external brick veneer, primarily adjacent to window openings on the eastern elevation
- damage and cracking to plaster linings and other non-structural elements including cornices, skirting etc.

3.4 GEOTECHNICAL REVIEW

A review of the ground damage and conditions was carried out by Tonkin & Taylor for the Burwood Hospital Campus. A subsequent report titled *Burwood Hospital Post Earthquake Geotechnical Assessment* was issued in June 2011 [5]. The geotechnical review concluded that the settlement and damage to building foundations and slabs on the Burwood Hospital Campus was likely due to the liquefaction of underlying soil layers. It is believed that excessive pore water pressures have been dissipated and that further settlement is not expected, unless another significant event were to occur.

Based on this report and from our detailed damage observations both internally and externally it does not appear that the overall stability of the Surgical Block has been affected by earthquake induced settlement.

Based on the geotechnical report provided by Tonkin & Taylor [5] the potential for future total and differential settlements at the building site varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

3.5 LEVEL SURVEY

A detailed survey of the ground floor levels in the Surgical Block was conducted by Fox & Associates and issued on 31st[th] October 2011 [6]. An additional survey following the earthquakes on the 23[rd] December 2011 and the 2[nd] January 2012 was completed on 1[st] February 2012. For the extent of the differential settlement noted see the level survey included in Appendix C.

The following is a summary of the differential settlements and resulting slopes in the ground floor of the building:

Original 1959 Construction – The level survey completed for the elevated concrete floor slab of the original 1959 construction indicates high points in the slab over the partial basement and services tunnels below. It is believed that this is a result of the partial basement and service tunnels being founded in deeper and stiffer material than the surrounding foundation elements. The maximum height differential in the concrete floor slab was measured to be 22mm over the length of the original 1959 construction. The worst case slope noted in the elevated slab is a drop of 14mm over a length of 5.5m (0.25% or 1:400).

1988 Additions – The level survey completed for the elevated timber floors of the corridor addition on the south end of the building indicate a maximum elevation change of 28mm over the length of the south corridor. The worst case slope noted in the floor framing is a drop of 12mm over a 4.1m length (0.29% or 1:340). This is within the typical acceptable range for residential timber framed construction.

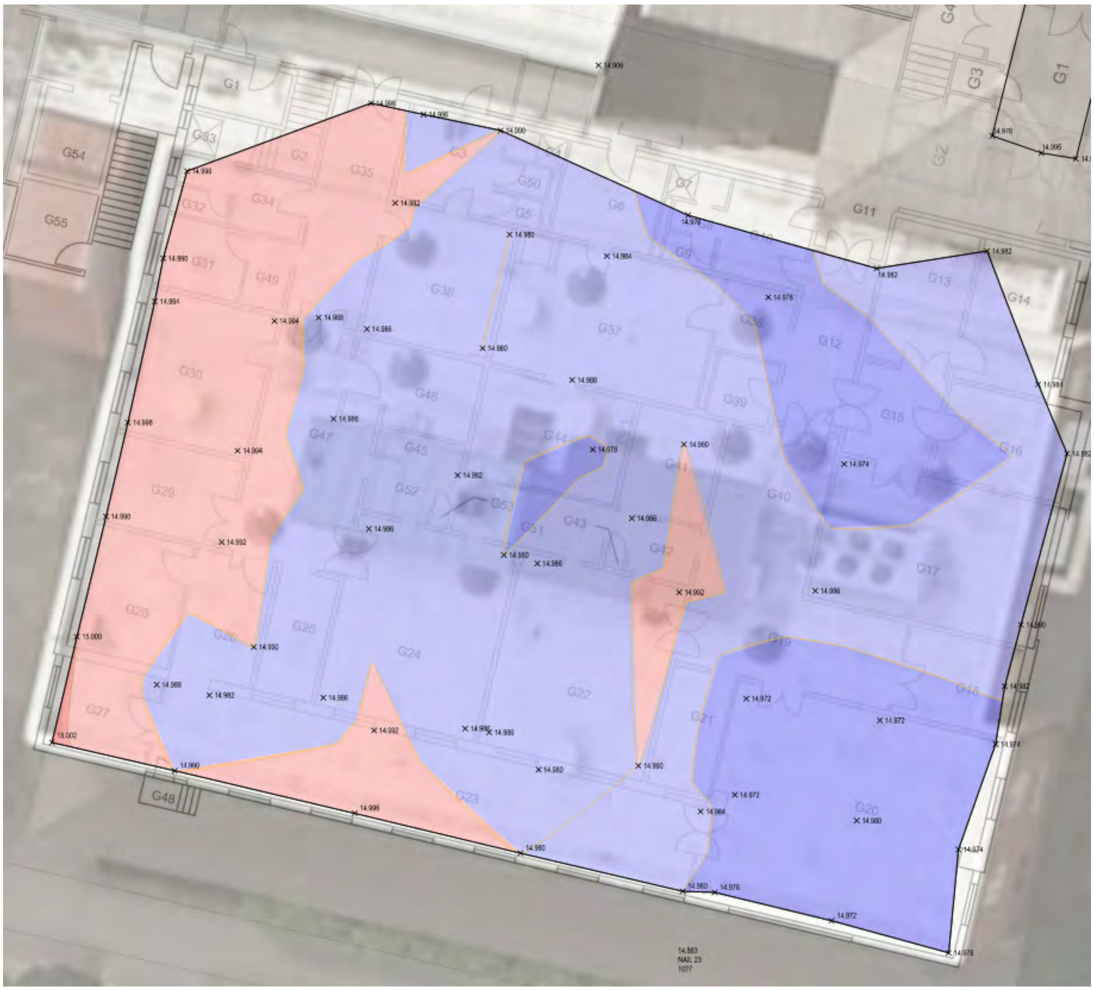


Figure 3-1: Level Survey

3.6 SUMMARY OF BUILDING DAMAGE

The following is a summary of the observations made for the building, and our conclusions as to their condition and seismic load resisting capacity. These observations do not specifically distinguish between damage caused by the Darfield Earthquake, the Lyttelton Earthquake, or any significant aftershocks thereafter, such as those that occurred on 13th June 2011, 23rd December 2011 and 2nd January 2012. Despite not being able to specifically distinguish when individual building damage observed occurred, it is believed that the majority of the damaged, or at least the onset of damage, can be linked to the February 22nd event.

The majority of the Surgical Block appears to have performed well considering the age of the building and the seismic actions experienced at the site. The damage to the superstructure is typified by cracking to the reinforced concrete walls and unreinforced brick partition walls, particularly at doors and windows openings, and minor cracking to the roof and floor slabs. This includes cracking to the basement service tunnel walls. Damage has also been noted the exterior brick veneer, interior plaster linings and other non-structural items. A summary of the typical damage observed is as follows:

- **Differential Ground Settlement** – Minor differential settlement noted at the original concrete portion of the building and at the timber framed additions. Resulting slopes in the ground floor are 0.25% (1:400) and 0.29% (1:340) respectively.

- **Cracking to Concrete Sub-floor, Service Tunnel and Partial Basement Walls** – Typical cracking noted in reinforced concrete service tunnel walls, up to 0.4 to 0.5mm in width. One outlying crack has been measure at 1.4mm.

- **Cracking to Concrete Ground and First Floor Walls** – Horizontal, vertical and diagonal cracking has been noted to the reinforced concrete ground floor and first floor walls, particularly off of the corner of window and door openings. Generally these cracks are no larger than 0.2mm in width and are expressed through the plaster or tile finishes of the walls. *Removal of finishes in sample locations will be required to understand the extent of the cracks propagation through the concrete walls.*

- **Cracking to Concrete Roof and Floor Slabs** – Minor cracking has been noted in the reinforced concrete roof and floor slabs. Some cracking has also been noted at the concrete wall / slab interface. *In general roof and floor finishes have not been removed to determine the full extent of cracking. Removal of finishes in localized areas is recommended.*

- **Cracking Interior Brick Partition Walls -** A number of cracks have been observed in the 4-1/2" (114mm) thick unreinforced brick partition walls at both the ground floor and first floor level, which are expressed through the wall finishes. Generally these are horizontal cracks that occur at the mid-height of the walls, or as diagonal cracks off of door openings. The cracks are typically no larger than 0.2mm width.

- **Damage to external brick veneer skin -** Cracking has been observed to the external 4-1/2" (114mm) thick brick veneer. The cracks have been noted in the mortar joints and propagating through the bricks themselves. The worst damage to the veneer occurs at the south-eastern and south-western corners of the building where cracking of the skin extends the full height of the building.

- **Damage to roof tank room –** Damage to the "Hardie Board" wall and ceiling lining fixings was observed. General aging and weather-related damage was also noted to the exterior weatherboard linings.

- **Separation at seismic joint –** Separation was noted at the seismic joint between the Surgical Block and the Birthing and Minor Procedures Unit. This was particularly evident where the first floor stair landing.

- **Damage to non-structural items -** Damage has been observed throughout the Surgical Block to non-structural items including plaster wall linings, ceilings, cornices and skirting elements. This is generally in the form of minor cracking or separation from adjacent elements, and does not represent a hazard or overall loss of strength to the structure. This includes damage noted to a dome light above corridor C17 on the first floor.

Table 4-1 provides a photographic summary of the typical damage observed. A full record of our detailed observations and repairs required can be found in Appendix A.

3.7 ADDITIONAL INVESTIGATIONS REQUIRED

Further investigations are required in order to understand the full extent of damage to the Surgical Block. An exhaustive survey of the full extent of cracking to the building has yet to be completed due to the presence of thick wall and floor finishes, along with a dropped ceiling in most areas. The additional investigations have been divided into investigations that should be completed as a priority for further assessment and investigations that can take place as the repairs are undertaken. In general, the additional investigations require the removal of wall and floor finishes.

3.7.1 Investigations Required for Further Assessment

- Based upon the damage observed, further investigations of the exterior brick façade is required. This includes a summary of its general condition and the fixings to the exterior concrete walls. This work should be completed by a qualified Mason and may require the local removal of the brick veneer.

  A brickwork inspection of the building was carried out by S A Thelning Brick & Blocklayer Ltd on 8th December 2011 [29]. The majority of the brickwork is considered to be sound with brick ties performing well. Movement has occurred in a few localised areas, particularly at corners and around parapets. Veneer walls have since been rebuilt in these areas.

3.7.2 Investigations to be Completed During Building Repair

- Locally remove the roof and floor finishes in the areas where the worst cracking has been noted in order to understand the full extent of the damage to the slabs beneath. Likewise further investigation is required to see if the cracks propagate to the underside of the slab. This will require the local removal of ceiling finishes. If additional damage is revealed it is likely extensive removal of ceiling, roof and floor finishes will be required throughout the building. If cracks greater than 0.5-0.6mm are noted further testing to determine if the reinforcing bars have been strain hardened may be required.

  During investigations and fit out, no cracks greater than 0.5mm to slabs were observed.

- Locally remove wall finishes over the interior and exterior concrete walls where the worst case cracking has been noted. The purpose is to understand the extent of the cracking to the concrete walls beneath. If additional damage is revealed it is likely extensive removal of wall and ceiling finishes will be required throughout the building. If cracks greater than 0.5-0.6mm (or horizontal cracks believed to have closed) are noted further testing may be require to determine if the reinforcing bars have been debonded or been strain hardened.

  During investigations and fit out of the ground floor level, no cracks greater than 0.5mm to walls were observed.

- Exposed surface of isolated concrete beams. The purpose is to understand the extent of the cracking to the concrete walls beneath. If additional damage is revealed it is likely extensive removal of ceiling finishes will be required throughout the building. If cracks greater than 0.5-0.6mm are noted further testing to determine if the reinforcing bars have been strain hardened may be required.

  During investigations and fit out, no cracks greater than 0.5mm to beams were observed.

- Additional field measurements and testing to verify concrete floor slab thicknesses and reinforcing assumed.

  The floor slab has a minimum thickness of 150mm as indicated by the drawings. It appears to be slightly thicker in some areas where it is likely a levelling screed was used. The reinforcing is generally 12mm bars at 300mm crs, though this slightly varies in some areas. This confirms the assumptions made in the analysis.

- Check adequacy of existing tank room wall fixings to the concrete roof slab below.

  The bottom plates of the timber tank room are connected into concrete nib walls cast integrally with the roof slab. The fixings appear to be cast into the nib wall. No evidence of movement between the concrete roof and timber tank room has been noted. The fixings are considered adequate for the existing bracing capacity but will most likely require upgrading if the bracing walls are to be improved.

3.8 POST-EARTHQUAKE BUILDING CAPACITY

Based upon our investigations to date, we do not consider the Surgical Block to have any notable reduction to the overall gravity load resistance of the structure.

Cracking has been observed to the concrete lateral load resisting elements, including the concrete shear walls, at all levels, and the concrete floor and roof diaphragms. It is possible that the cracking noted has also caused strain hardening and/or debonding of the smooth reinforcing bars, particularly at the base of the ground floor shear walls.

While the damage observed will require repair to restore the strength, stiffness, durability and performance of the individual structural components, the overall reduction in lateral load resisting capacity of the building is expected to be relatively minor. This is because the analysis completed has accounted for the likelihood of debonding to occur at the base of the ground floor walls under the ULS design basis earthquake (analysed for rocking at the base of the walls).

The repair work required is outlined in Section 4. Following the recommended repair of the structural damage, the lateral load resisting performance of the structure will be restored to approximately pre-earthquakes levels (see Section 2.4).

In its pre-earthquake and post-earthquake state the primary lateral load resisting elements of the Surgical Block have been assessed to have a capacity of 67% DBE, and as such the building is not considered to be "Earthquake Prone." While the interior brick partition walls are not part of the main lateral load resisting system, they have been assessed at 15% DBE for face loading and thus considered "earthquake prone" elements. The cracks noted in the walls as a result of the earthquakes have further reduced their capacity. They pose a risk to building occupants and as a result it is our recommendation that the walls be removed or shored as soon as practical. The interior brick partition walls on the ground level were secured to 67% DBE during 2013 using timber studs.

It should be noted that when compared to the loading code prior to the earthquakes, the brick partition walls would have been assessed at approximately 20% DBE, and thus would have been considered "earthquake prone" elements prior to the earthquakes. Amendment 10 [9], which was put into place following the Lyttleton Earthquake, essentially resulted in an increase to the design loads of 36 % when compared to pre-earthquake design levels.

In addition to the minimum repairs of the building, recommended strengthening concepts to increase the seismic capacity of the tank room to above 67% DBE have been included in Section 5.

4. OBSERVED DAMAGE & REPAIRS REQUIRED

4.1 TYPICAL DAMAGE & REPAIRS REQUIRED

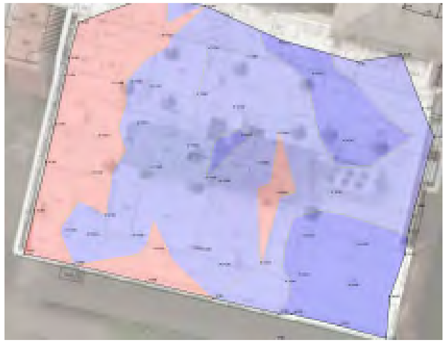
This section covers the damage noted during our detailed assessment of the building. Note that our observations have been restricted to structural aspects of the building only. Waterproofing elements, electrical and mechanical equipment, fire protection and safety systems, service connections, water supplies and sanitary fittings have not been inspected or reviewed, and secondary elements such as windows and fittings have not generally been reviewed.

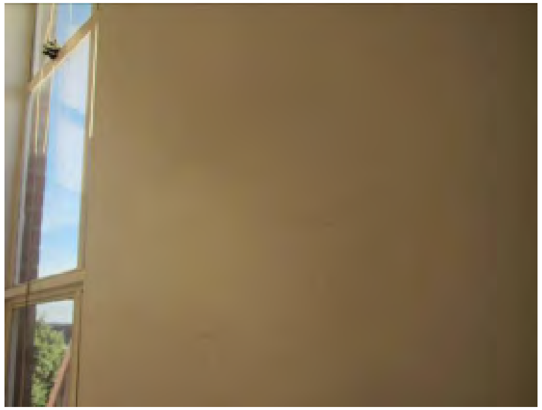
Table 4-1 provides a photographic summary of the observed typical damage and typical repairs required for the Surgical Block. Table 4-1 should be read in conjunction with Appendix A – Record of Observation and Appendix B – Reference Plans which provide the complete extent of the observed damage. The Repair Specification [2] referred to in Table 4-1 has been issued separately.

In general, the aim of the repair work indicated in this section is to restore the structure to its pre-earthquake state, as far as practicable, while maintaining the utility of the building. The repairs presented attempt to address the loss of strength, stiffness and durability of the structural elements due to the damage noted. Recommendations for strengthening works for the Tank Room, to achieve 67% DBE, are included in Section 5.

It should be noted that more damage may be identified during the repair works and (if required) additional repair details will be specified accordingly.

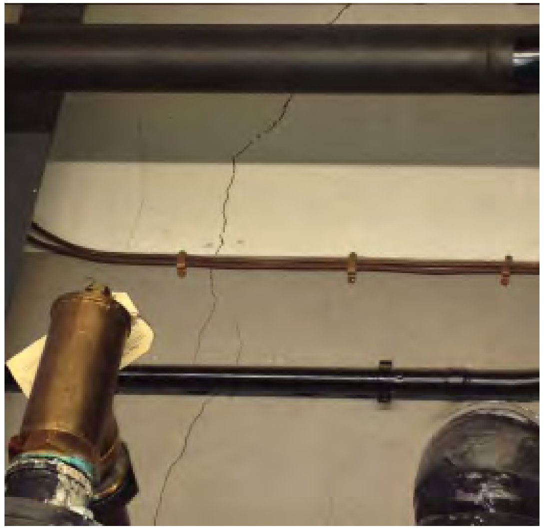
Table 4-1: Photographs of Observed Typical Damage and Repairs Required

Damaged Item & Location	Damage	Recommendations	Example Photograph
1. Differential Ground Settlement			
1.1 Permanent slopes in ground floor framing	Differential ground settlement resulting in a worst case slope in the concrete ground floor slab of approximately 0.25% (1:400) and in the elevated timber floor of approximately 0.29% (1:340)	For further discussion on potential remediation see Section 4-2. (Note: All re-levelling is to occur prior to any other permanent structural or cosmetic repairs).	
2. Foundations and Sub-Floor Walls			
2.1 Sub-floor, service tunnel and partial basement walls	Cracking in concrete walls (typically less than 0.5mm)	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [3]. For cracks greater than 0.5-0.6mm, HCG to confirm integrity of existing reinforcing steel. See Section 4.3 for additional discussion.	

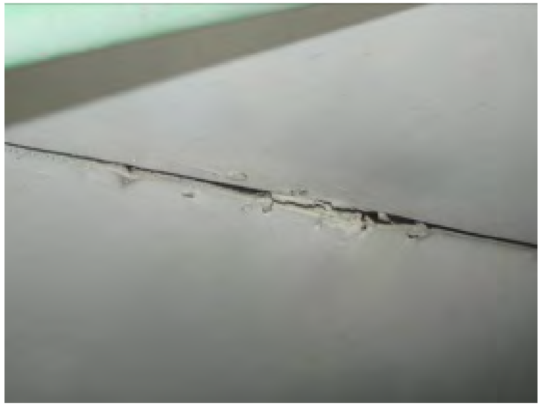
Damaged Item & Location	Damage	Recommendations	Example Photograph
3. Ground & First Floor Reinforced Concrete Walls			
3.1 Concrete shear walls- ground floor and first floor	Vertical, diagonal and horizontal cracking in concrete shear walls, up to 0.3mm in width.	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [3]. For cracks greater than 0.5-0.6mm, HCG to confirm integrity of existing reinforcing steel. See Section 4.3 for additional discussion.	
3.2 Concrete shear walls – at door and window openings.	Cracking around door and window openings.	See item 3.1.	

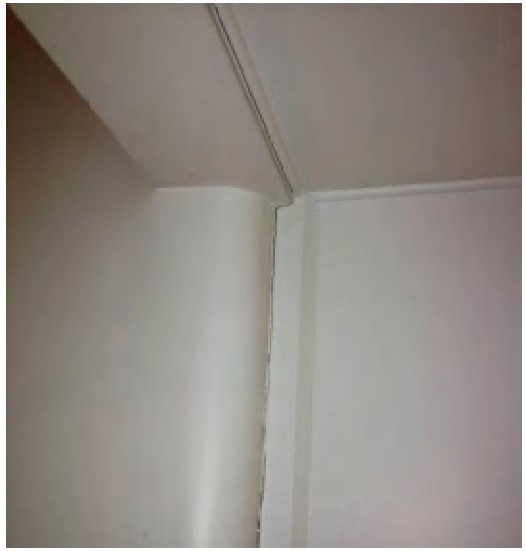
Damaged Item & Location	Damage	Recommendations	Example Photograph
4. Concrete Roof and Floor Slabs			
4.1 Concrete floor slabs – ground and first floor levels	Cracking in floor slab, particularly at restrained corners	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [3]. For cracks greater than 0.5-0.6mm, HCG to confirm integrity of existing reinforcing steel. See Section 4.3 for additional discussion.	 

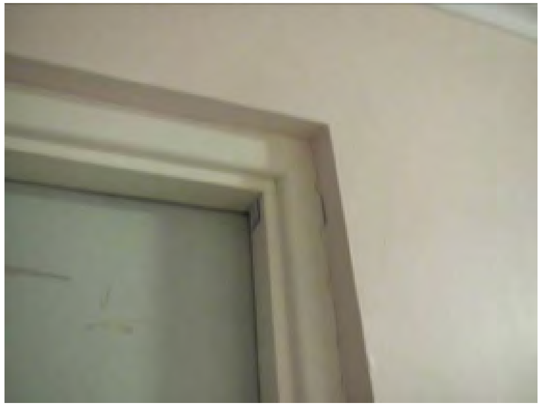
Damaged Item & Location	Damage	Recommendations	Example Photograph
4.2 Concrete roof slab	Cracking in roof slab, particularly around tank room area.	See item 4.1.	
5. Concrete Parapet Walls			
5.1 Concrete parapet walls	Cracking in parapet walls (typically vertical cracking)	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [3]. For cracks greater than 1mm, HCG to confirm the integrity of the existing reinforcement of the wall. If reinforcement is damaged, an engineered repair will be required.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
6. Interior Brick Partition Walls			
6.1 Interior brick partition walls	Cracking in brick walls and overlays	Demolish and replace damaged brick partition walls with new light weight stud walls clad in gypsum board ceiling. See Section 4.4 for additional discussion. Jul/Aug 2013: Ground floor brick partitions were secured by the addition of timber studs.	 

7. Interior Dropped Ceiling			
7.1 Dropped plasterboard ceilings	Cracking to dropped ceiling linings	The dropped ceilings provide lateral restraint for the top of the brick partition walls. Repair and/or replace as part of the internal brick partition wall repairs. See Section 4.5 for additional discussion.	
7.2 Dome skylight	Cracking in plaster lining over timber framing.	See item 7.1	

Damaged Item & Location	Damage	Recommendations	Example Photograph
8. Roof tank room			
8.1 Hardie Board Wall and Ceiling Linings	Tank room wall and ceiling lining has been damage, particularly at nail fixing locations.	Re-fix existing "Hardie Board" wall and ceiling linings.	
9. Brick Veneer			
9.1 Brick veneer	Cracking off corners of door and window openings	*Further investigation required by a qualified Mason to determine the full extent of damage and repair required.* A brickwork inspection of the building was carried out by S A Thelning Brick & Blocklayer Ltd on 8th December 2011 [29]. The majority of the brickwork is considered to be sound with brick ties performing well. Movement has occurred in a few localised areas, particularly at corners and around parapets. Veneer walls have since been rebuilt in these areas.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
9.2 Brick veneer	Full height cracking of veneer at corners.	*Further investigation required by a qualified Mason to determine the full extent of damage and repair required.* A brickwork inspection of the building was carried out by S A Thelning Brick & Blocklayer Ltd on 8th December 2011 [29]. The majority of the brickwork is considered to be sound with brick ties performing well. Movement has occurred in a few localised areas, particularly at corners and around parapets. Veneer walls have since been rebuilt in these areas.	
10. Seismic Joint			
10.1 Seismic joint at interface with Birthing and Minor Procedures Unit	Separation in wall and ceiling finishes	Repair existing wall and ceiling framing and finishes. Consideration is to be given to increasing seismic joint separation in order to prevent future damage.	

Damaged Item & Location	Damage	Recommendations	Example Photograph
11. Miscellaneous items			
11.1 Skirting/timber framing elements	Minor cracking and damage to door frames, timber skirting elements etc. throughout.	Aesthetic repair by others.	

4.2 DISCUSSION ON BUILDING RE-LEVELLING

The level survey, completed by Fox & Associates, has indicated that earthquake induced differential ground settlements have occurred at the Surgical Block building resulting in permanent slopes in the ground floor. The maximum slope noted in the elevated concrete ground floor slab was 0.25% or 1:400. The maximum slope noted in the elevated timber floor of the 1988 addition was 0.29% or 1:340. Both slopes are within the typical acceptable range for concrete and timber construction.

Besides the slopes noted in the ground floor framing, the differential settlements observed will have resulted in some reduction in the capacity of the building, along with a reduction in the buildings ability to undergo future differential settlements before the onset of more severe damage.

Given the complexity of the foundation system under the main portion of the building (sub-floor, service tunnel and partial basement walls) remediation of the floor levels of the main portion of the building through either the use of mechanical jacking or grout injection is impractical. Any additional leveling would likely be best achieved through the use of self-levelling compound. At the timber framed additions (southern corridor) re-levelling could be achieved by disconnecting the floor framing, jacking it up to level and re-fixing the framing to the existing foundations.

During any re-levelling process there is a risk that addition damage could occur to the buildings linings, exterior brick veneer, etc. and appropriate contingencies should be provided.

A discussion on re-levelling on a campus wide basis is also included in the Burwood Hospital campus base report. This includes a study on the effect of re-levelling individual buildings on the serviceability of the hospital campus as a whole.

4.3 REPAIR OF CONCRETE ELEMENTS

Cracking has been observed throughout the Surgical Block to the reinforced concrete walls as discussed in Section 3. This includes horizontal and vertical cracking along with diagonal cracking off the corners of window and doors openings. Cracking has also been observed to the underside of the ground floor slab and to the top of the roof slab. The majority of the cracks observed to the concrete elements appear to be approximately 0.2mm in width or less, although some larger cracks have been noted in the concrete sub-floor walls (~0.5mm) and the ground and first floor level walls (~0.3mm).

The cracking noted, to the concrete walls in particular, indicates that the round bar laps have potentially debonded or the reinforcing steel has been strain hardened. No detailed investigations have been carried out, however for buildings of this age, the walls were typically constructed with horizontal reinforcing with hooked ends and vertical reinforcing with short straight laps. The limited original structural drawings available coincide with this assumption.

Based on the results of the testing of the reinforcing steel at Riverside Hospital and 235 Antigua Street, a vertical or diagonal crack exceeding 0.5-0.6mm in width would indicate that a significant level of strain hardening is likely to have occurred. The width of a horizontal crack is not an indication of the extent of strain hardening or debonding as the gravity loads close the cracks. The results of the testing completed to date in other buildings indicates that debonding or strain hardening is likely to have occurred where diagonal cracks extend to near the base of the wall or where there are horizontal cracks.

In general, based upon the size and extent of the cracking noted we do not believe debonding or strain hardening of the reinforcing steel has occurred on a large scale. It appears as though the strength and stiffness of the majority of the cracked concrete elements can be restored to approximately pre-earthquake levels through epoxy injection of the cracks. Testing of the reinforcing steel at the worst case horizontal cracks and at any vertical or diagonal cracks greater than 0.5-0.6mm will be required to ensure either debonding or strain hardening has not occurred at these locations. If debonding or strain hardening has occurred the individual elements will be required to be demolished and reconstructed.

During further investigations and fit-out of the building in July-October 2013, no cracks greater than 0.5mm were observed.

4.4 BRICK PARTITION WALLS

Cracking has been observed to the internal brick partition walls throughout the Surgical Block. As outlined in Table 2-1, these walls had an assessed pre-earthquake capacity of 15% DBE, making them "Earthquake Prone." The recommended repair is to demolish and replace these walls with new light-weight partition walls. These walls could be constructed of either timber framing, or cold-formed framing, and clad on either side with gypsum board wall linings.

Alternatively timber framing can be used to encapsulate the brick walls such that the timber walls restrain the seismic weight of the brick walls.

All brick walls on the ground floor level have been either removed or secured with timber studs to 67% DBE.

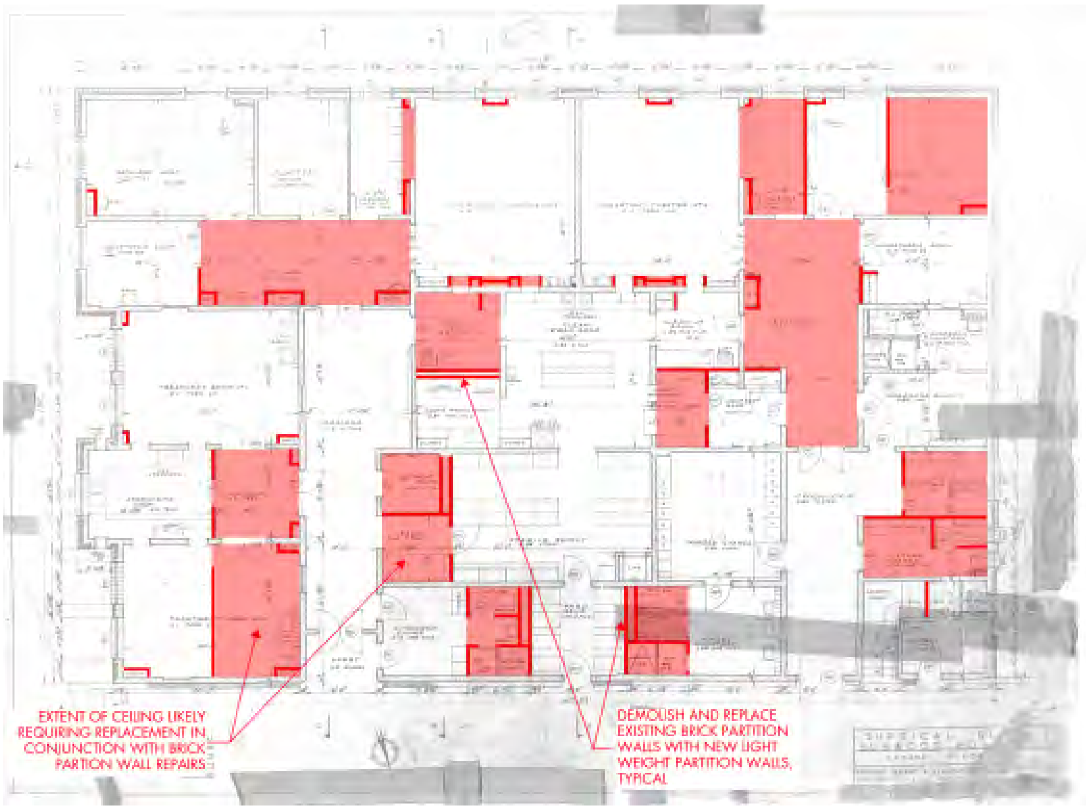


Figure 4-1: Ground Floor Plan – Required Repairs

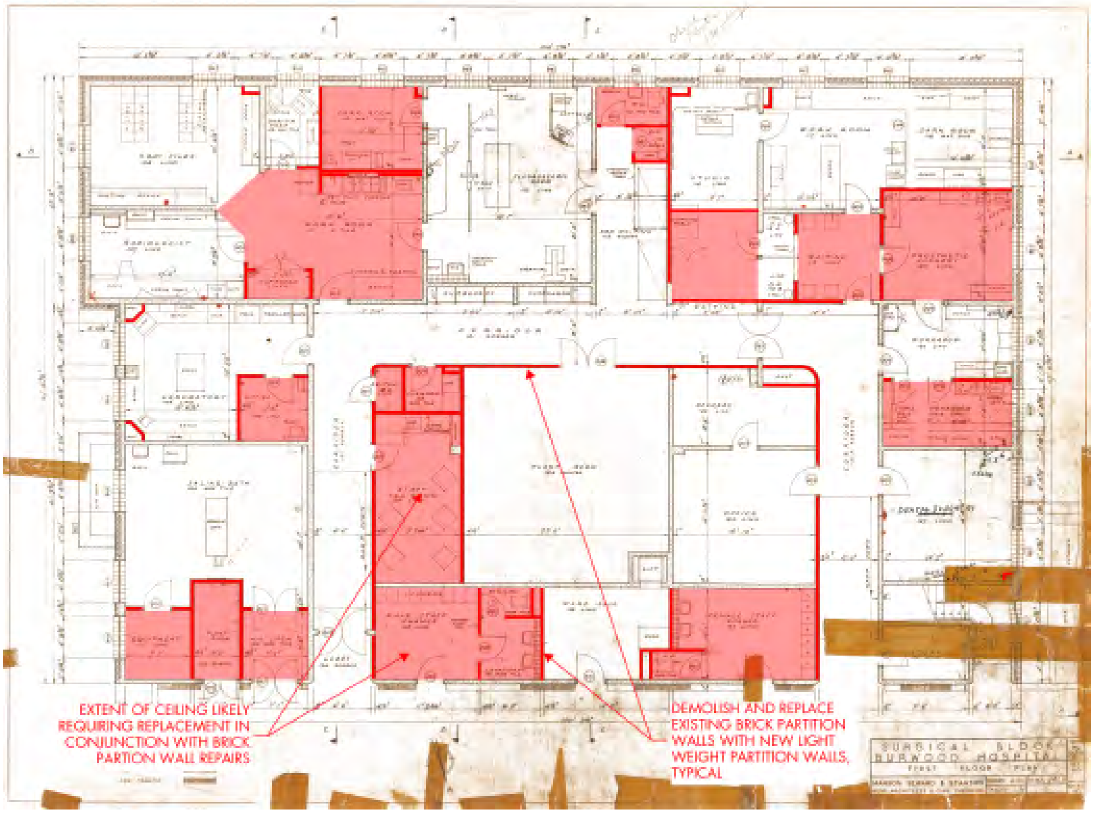


Figure 4-2: First Floor Plan – Required Repairs

4.5 DROPPED CEILING FRAMING

At the ground floor and first floor levels, the dropped ceiling framing currently provides lateral restraint to the top of the internal brick partition walls. In isolated locations the ceiling has been damaged and requires repair. In addition, at least some of the ceiling linings will be required to be removed as part of the brick partition wall repairs/removal.

4.6 REPAIR OF EXTERNAL BRICK VENEER

The external brick veneer of the Surgical Block has been damaged and requires repair. Based upon the extent of the damage observed, a more detailed investigation has been carried out by a qualified Mason, to identify the full extent of the damage, along with the recommended repairs. The investigation also included a review of the existing fixings of the brick veneer to the exterior concrete walls.

In response to this review, brick repairs to the most affected areas of the veneer of the building have been carried out during August 2013.

4.7 REPAIR OF TANK ROOM WALL AND CEILING LININGS

The wall and ceiling linings of the tank room have been damaged in locations and require repair, primarily at the Hardie Board lining fixings. Based upon the movement observed it is believed the wall and ceiling lining fixings have been damaged throughout. This has resulted in a reduction to the ongoing strength and stiffness of all the bracing walls and the ceiling diaphragm. In order to reinstate the pre-earthquake strength and stiffness to these elements, the repair recommendation is to remove all cracked or damaged sections of the wall linings and replace them with in kind material. All existing internal wall and ceiling linings to remain are to be re-fixed to the existing timber framing.

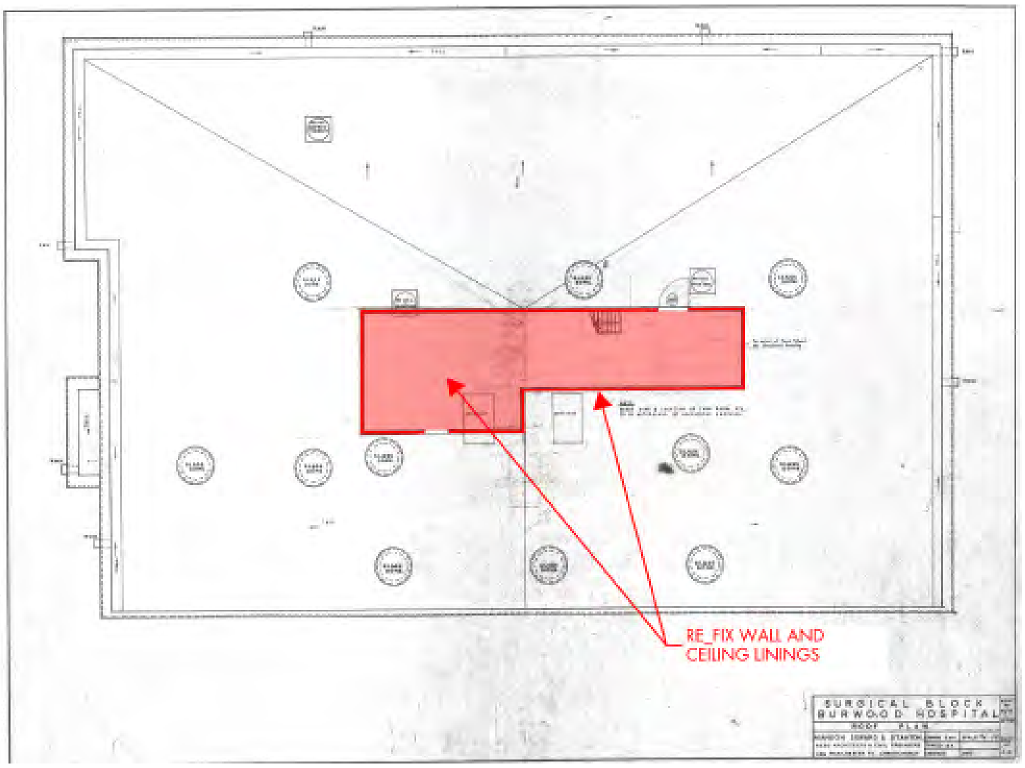


Figure 4-3: Roof Plan – Required Repairs

5. STRENGTHENING RECOMMENDED

The primary lateral load resisting structure of the Surgical Block consists of reinforced concrete shear walls and rigid concrete slab diaphragms. As discussed in Section 2, the lateral load resisting capacity of the building has been assessed as a percentage of the loads imposed by the Design Basis Earthquake (DBE). The assessed capacity of the building, in its pre-earthquake undamaged state, is specifically outlined in Section 2.4.

In the north-south and east-west directions the capacity of the main portion of the building has been assessed at approximately 67% DBE, and is governed by the capacity of the concrete walls at the ground floor level. In particular this is due to the short laps of the smooth reinforcing bars at the base of the concrete shear walls, which limit the moment resisting capacity of the walls.

Provided the repairs specified in Section 4 are implemented, including the replacement or encapsulation of the heavy brick partition walls, the seismic capacity of the building will be restored and even slightly increased due to a reduction in seismic mass.

Strengthening of the concrete shear walls elements of the building would be extensive and difficult. It is unlikely that any minor strengthening works are going to change the behaviour of the main building as the high stiffness will prevent any additional sections from engaging unless they are cast integrally with the existing walls. For this reason, the risk that the gravity support will be lost in the building following a significant event is low.

5.1 TANK ROOM - STRENGTHENING WORKS TO ACHIEVE 67% DBE

Additional Wall Bracing – The pre-earthquake and post-earthquake repair assessed capacities of the roof diaphragm and wall bracing of the tank room on the roof are approximately 40% DBE and 50% DBE respectively. In order to bring the assessed capacity above 67% it is recommended that new wall bracing be added in the north-south direction. The additional bracing could be added internally or externally with the use of steel braces fixed to the roof framing and the concrete slab below. In addition to providing additional wall bracing, the length of the ceiling diaphragm would be reduced and thus increasing its assessed capacity as well. See Figure 5-1 for the additional proposed bracing locations.

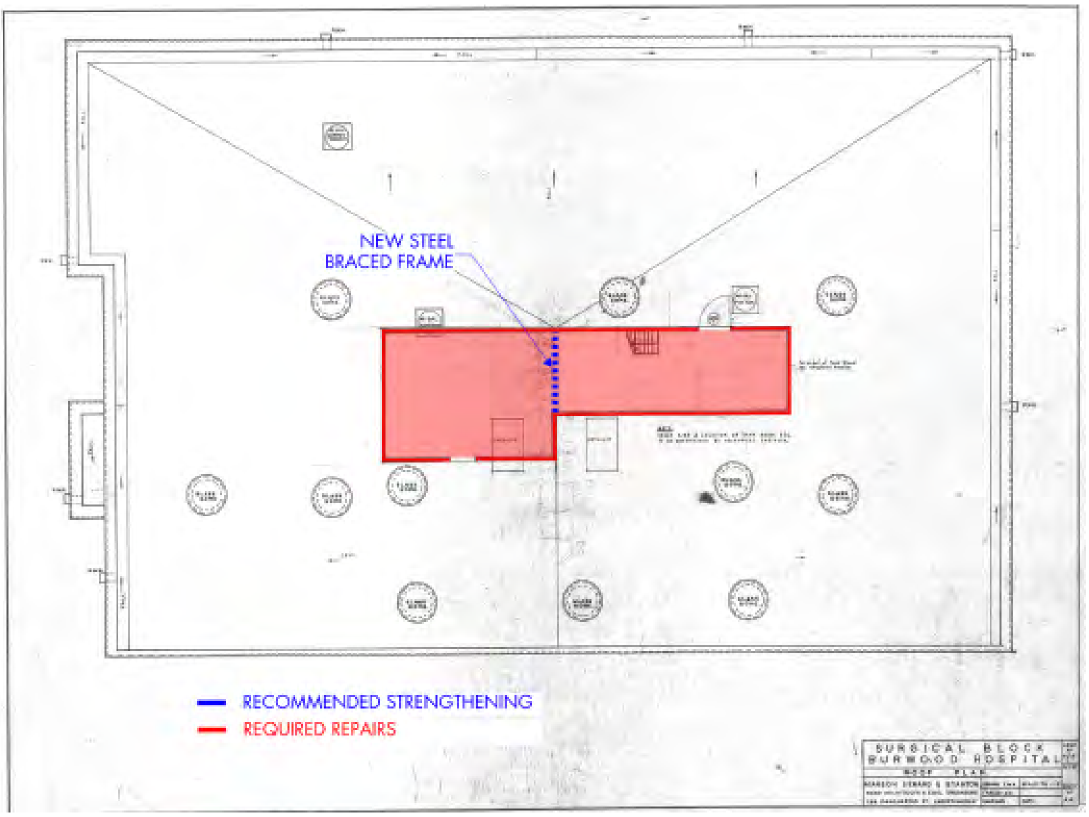


Figure 5-1: Tank Room – Strengthening Recommended

6. REFERENCES

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23 *CDHB – Burwood Hospital – Rapid Seismic Assessment Post June 13th Earthquakes: 106186.03 Site Report 4,* Holmes Consulting Group, 14 June 2011

24 *CDHB – Burwood Hospital – Rapid Seismic Assessment Post June 13th Earthquakes: 106186.03 Site Report 5,* Holmes Consulting Group, 15 June 2011

25 *CDHB – Burwood Hospital – Post Earthquake Rapid Structural Assessment following 5.8 and 6.0 Magnitude Earthquakes, 106186.03 Site Report* 8, Holmes Consulting Group, 24 December 2011

26 *CDHB – Burwood Hospital – Post Earthquake Rapid Structural Assessment following 2nd January 5.5 Magnitude Earthquakes, 106186.03 Site Report 9*, Holmes Consulting Group, 9 January 2012

27 *Burwood Hospital Christchurch – Survey of Existing Buildings – Surgical Block,* Cutter Pickmere Douglas - Architects, 1976

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APPENDIX A

Record of Observations

APPENDIX A – RECORD OF OBSERVATIONS & REPAIRS

Inspection date: 24th April 2012, 3rd May 2012, 9th May 2012

KEY	
N	No repair required
Y	Repair required
F	Further investigation required
C	Repair complete

Level	Room Number	Building Element	Observations	Repair Required	Repair	Key Plan Reference
G	G35	Concrete shear wall	Diagonal crack in concrete wall above door opening.	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	G01
G	G36	Concrete shear wall	Crack along length of wall at mid height approximately 0.2mm	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	G02
G	G35	Concrete shear wall	Vertical crack in concrete wall above door opening.	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	G03
G	G47	Concrete shear wall	Vertical crack in concrete wall above door opening.	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	G04
G	G24/G22	Concrete shear wall	Critical wall tiled. No damage observed.	**N/A**	-	G05
G	G46	Concrete slab/wall	No damage observed above ceiling to slab/wall including connection	**N/A**	-	G06
G	G19	Concrete shear wall	0.3mm crack in shear wall observed above ceiling level.	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	G07

Level	Room Number	Building Element	Observations	Repair Required	Repair	Key Plan Reference
G	G40	Concrete slab/wall	No damage observed above ceiling to slab/wall including connection	**N/A**	-	G08
G	G12	Concrete slab/wall	No damage observed above ceiling to slab/wall including connection	**N/A**	-	G09
G	G40	Concrete shear wall	Cracking at mid height of wall.	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	G10
G	External	Brick veneer	Cracking in brick veneer above window at ground floor level	**Y**	Remove damaged brick veneer and replace.	G11
G	External	Brick veneer	Cracking in brick veneer above window.	**Y**	Remove damaged brick veneer and replace.	G12
G	External	Brick veneer	Cracking in brick veneer at SE extent of building. Cracking extends full height of building down to foundations. One to two brick widths wide.	**Y**	Remove damaged brick veneer and replace. Adequacy of veneer ties confirmed.	G13
G	External	Brick veneer	Cracking in brick veneer at SW extent of building. Cracking extends full height of building down to foundations. One to two brick widths wide.	**Y**	Remove damaged brick veneer and replace. Adequacy of veneer ties confirmed.	G14
G	G23	1988 Extension roof fixings	No damage observed to 1988 roof truss fixing - original structure	**N/A**	-	G15
G	External	Brick veneer ties	Brick veneer ties observed at south east corner of original structure. Appear to be at approximately 500mm centres	**N/A**	-	G16
1	1-2	Concrete shear wall	Multiple horizontal cracks 0.2mm in wall adjacent window in stair well	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	101

Level	Room Number	Building Element	Observations	Repair Required	Repair	Key Plan Reference
1	1-2	Concrete shear wall	Cracking through width of wall. .2mm-.3mm horizontal cracking in wall above door opening. Vertical crack at wall junction	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	102
1	1-2	Wall	Damage at to wall linings at interface between Surgical Block and CRRU Building	**N**	Non-structural damage. Repair for aeathetic reasons only.	103
1	1-3	Brick partition wall	Multiple horizontal cracks in wall	**Y**	Recommend all brick partition walls demolished and replaced with timber walls.	104
1	1-3	Concrete shear wall	Vertical and horizontal crack in wall, approx. 0.2mm	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	105
1	1-3	Concrete shear wall	Horizontal crack along length of wall approx. 0.2mm	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	106
1	1-3	Concrete shear wall	Vertical crack above door.	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	107
1	1-3	Brick partition wall	Diagonal cracking approximately 0.2mm top to bottom of wall	**Y**	Recommend all brick partition walls demolished and replaced with timber walls.	108
1	1-29	Brick partition wall	Vertical & horizontal cracking in brickwork	**Y**	Recommend all brick partition walls demolished and replaced with timber walls.	109
1	1-29	Concrete shear wall	.2mm cracking through width of wall.	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	110
1	1-31	Concrete shear wall	Diagonal crack from corner of door opening.	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	111
1	1-28	Concrete shear wall	Vertical crack in wall above door opening approx. 0.2mm	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	112

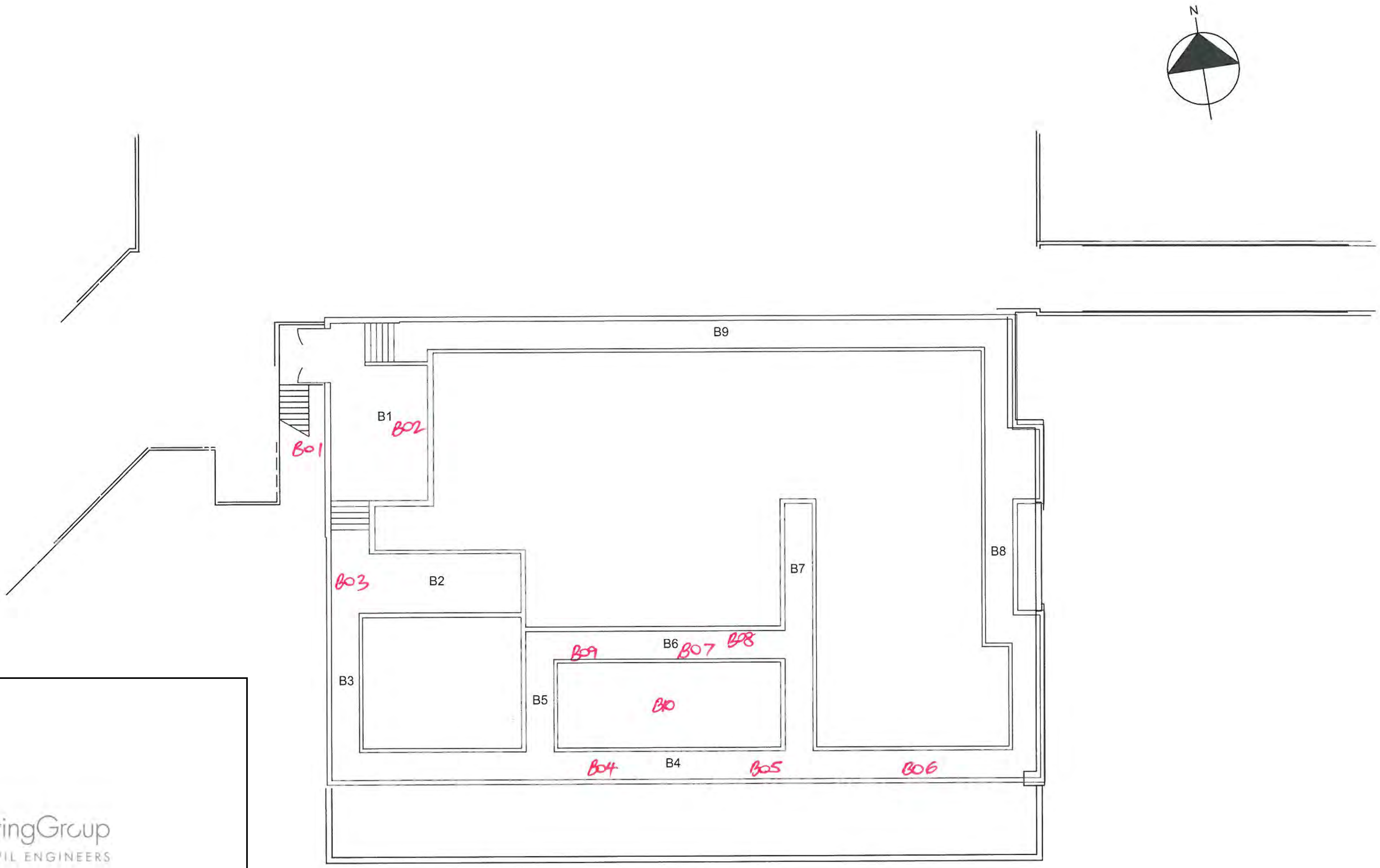
Level	Room Number	Building Element	Observations	Repair Required	Repair	Key Plan Reference
1	1-29	Concrete shear wall	Vertical/diagonal crack in wall above door opening (as in 112 from other side of door)	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	113
1	1-30	Concrete shear wall	Damage to concrete wall at duct location- 'pounding' damage.	**Y**	Inject cracks > 0.2mm, patch wall in conjunction with the HCG Repair Specification.	114
1	1-25	Concrete shear wall	Horizontal cracking in wall panel aligning with the bottom of the window	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	115
1	1-27	Concrete shear wall	Horizontal cracking in wall panel aligning with the bottom of the window	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	116
1	1-30	Concrete shear wall	Horizontal cracking in wall panel aligning with the bottom of the window	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	117
1	1-33	Concrete shear wall	Cracking in tiles from bottom corner of window- likely cracking in concrete wall under	**F**	If found, epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	118
1	1-33	First floor concrete slab	Cracked floor tiles- possible cracking of floor slab under.	**F**	If found, epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	119
1	1-17	Concrete shear wall	Multiple vertical/diagonal cracking in wall top-bottom approx 0.3mm.	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	120
1	1-17	Brick partition wall	Multiple horizontal cracking at mid-height of wall	**Y**	Recommend all brick partition walls demolished and replaced with timber walls.	121
1	1-17	Dome light	Cracking in plaster lining of dome light framing.	**N**	Non-structural damage. Remove damaged plaster lining and replace with new Gib lining.	122
1	1-17	Dome light	Cracking in plaster lining of dome light framing.	**N**	Non-structural damage. Remove damaged plaster lining and replace with new Gib lining.	123
1	1-29	Timber skirting	Separation of timber skirting from concrete wall.	**N**	Non-structural damage. Repair for aeathetic reasons only. TYPICAL	124

Level	Room Number	Building Element	Observations	Repair Required	Repair	Key Plan Reference
1	1-36	Plaster ceiling/wall	Cracking in plaster at wall/ceiling junction	**N**	Non-structural damage. Repair for aeathetic reasons only. TYPICAL	125
1	1-12	Concrete shear wall	Vertical crack top-bottom of wall up to 0.7mm	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	126
1	1-12	Concrete shear wall	Horizontal crack in wall 0.2mm	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	127
1	1-12	First floor concrete slab	No damage observed to exposed first floor slab in plant room.	**N/A**	-	128
1	1-12	Concrete beam/shear wall.	Possible crack at beam-wall connection. Further investigation required. Note no other damage observed to roof concrete beams and soffit of roof slab.	**F**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	129
1	External	Brick veneer	Cracking in brick veneer above window.	**Y**	Remove damaged brick veneer and replace.	130
B	Basement stairs	Basement concrete wall	Horizontal crack 0.3mm at mid height of basement wall adjacent plant room entry	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	B01
B	B1	Basement concrete wall	Vertical crack top to bottom of concrete wall 0.4mm	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	B02
B	B2	Basement concrete wall	Vertical crack in wall top-bottom approx. 0.2mm	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	B03
B	B4	Basement concrete wall	Cracking in wall maximum 0.3mm.	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	B04

Level	Room Number	Building Element	Observations	Repair Required	Repair	Key Plan Reference
B	B4	Basement concrete wall	Vertical crack in concrete wall. Possibly existing at pour joint.	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	B05
B	B4	Basement concrete wall	Vertical crack in wall maximum 0.5mm	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	B06
B	B6	Basement concrete retaining wall	Vertical crack in wall up to 1.4mm wide. Possibly existing.	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	B07
B	B6	Basement concrete wall	Multiple diagonal cracks in bottom of wall.	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	B08
B	B6	Ground floor slab soffit	0.2mm crack in soffit of ground floor slab adjacent sub floor wall	**Y,F**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	B09
B	B4/B6	Sub floor concrete wall	0.2mm crack both sides of wall	**Y,F**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	B10
R	Roof tank room	Ceiling lining	Damage to existing roof lining- torn at edge of sheets.	**Y**	Replace existing roof lining.	R01
R	Roof tank room	Ceiling lining	Nails fixing sheets to roof structure are missing.	**Y**	Replace existing roof lining.	R02
R	Roof	Roof concrete slab	No damage observed to roof concrete slab	**N/A**	-	R03
R	Roof tank room	External cladding	Damage observed to external weatherboard cladding	**N**	Non-structural damage. Repair for aesthetic purposes only.	R04
R	Roof	Parapet	Long horizontal crack in brick render on north elevation.	**Y**	Epoxy inject all cracks > 0.2mm in conjunction with the HCG Repair Specification.	R05

APPENDIX B

Reference / Key Plans



HolmesConsultingGroup
STRUCTURAL AND CIVIL ENGINEERS

Project Name:	*Burwood Surgical Block*
Project Number:	*106186.57*
Sketches By:	*PJG*
Date:	*Mar 2014*
Sketch Number:	*Appendix B - SK01*

BASEMENT - KEY PLAN

APPENDIX B PAGE 1

AMENDMENT	DRAWN	CHECKED	DATE
MASTER DRAWING CREATED FROM 04000003 AND CDHB BORDER ADDED	BJT		15-06-09

BURWOOD HOSPITAL
SURGICAL BLOCK, BASEMENT
MASTER FLOOR PLAN

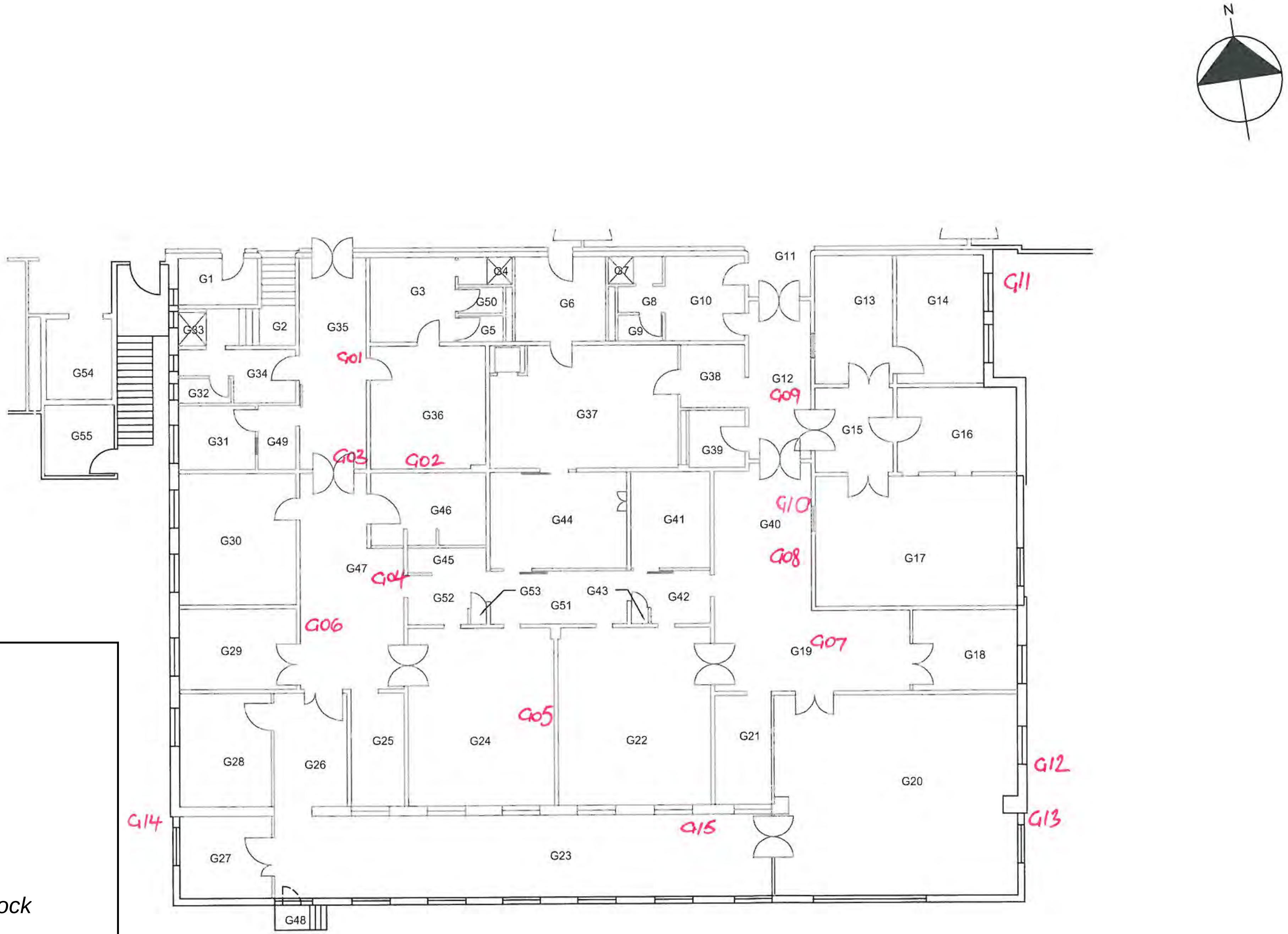
Canterbury
District Health Board
Te Poari Hauora ō Waitaha

DESIGNED:
DRAWN: BJT
CHECKED:
DATE: 15-06-09

DWG No. 04041010
SCALE 1:100 AT A1
REVISED 15-06-09

XREFED DRAWING/S:

Maintenance and Engineering Department - Christchurch Hospital



HolmesConsultingGroup

STRUCTURAL AND CIVIL ENGINEERS

Project Name:	*Burwood Surgical Block*
Project Number:	*106186.57*
Sketches By:	*PJG*
Date:	*Mar 2014*
Sketch Number:	*Appendix B - SK02*

GROUND FLOOR - KEY PLAN

APPENDIX B PAGE 2

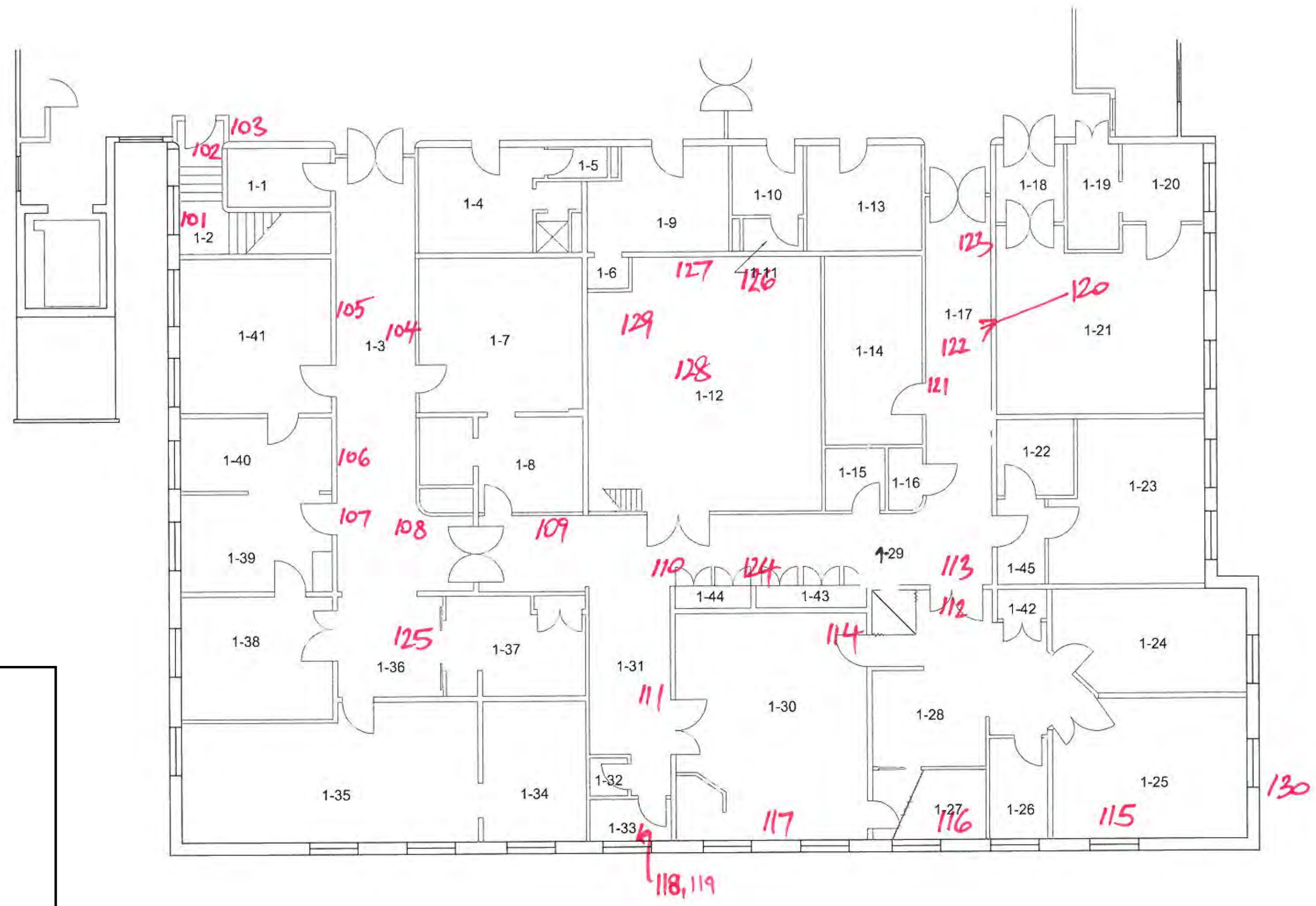
AMENDMENT	DRAWN	CHECKED	DATE
MASTER DRAWING CREATED FROM 04000003 AND CDHB BORDER ADDED	BJT		15-06-09

BURWOOD HOSPITAL
SURGICAL BLOCK, GROUND FLOOR
MASTER FLOOR PLAN

Canterbury District Health Board
Te Poari Hauora o Waitaha

DESIGNED:
DRAWN: BJT
CHECKED:
DATE: 15-06-09
DWG No. 04042010
SCALE 1:100 AT A1
REVISED 15-06-09
REFERED DRAWINGS:

Maintenance and Engineering Department - Christchurch Hospital



Project Name: Burwood Surgical Block

Project Number: 106186.57

Sketches By: PJG

Date: Mar 2014

Sketch Number: Appendix B - SK03

FIRST FLOOR - KEY PLAN

APPENDIX B PAGE 3

AMENDMENT	DRAWN	CHECKED	DATE
MASTER DRAWING CREATED FROM 04000004 AND CDHB BORDER ADDED	BJT		15-06-09

BURWOOD HOSPITAL
SURGICAL BLOCK, FIRST FLOOR
MASTER FLOOR PLAN

Canterbury
District Health Board
Te Poari Hauora o Waitaha

DESIGNED:
DRAWN: BJT
CHECKED:
DATE: 15-06-09

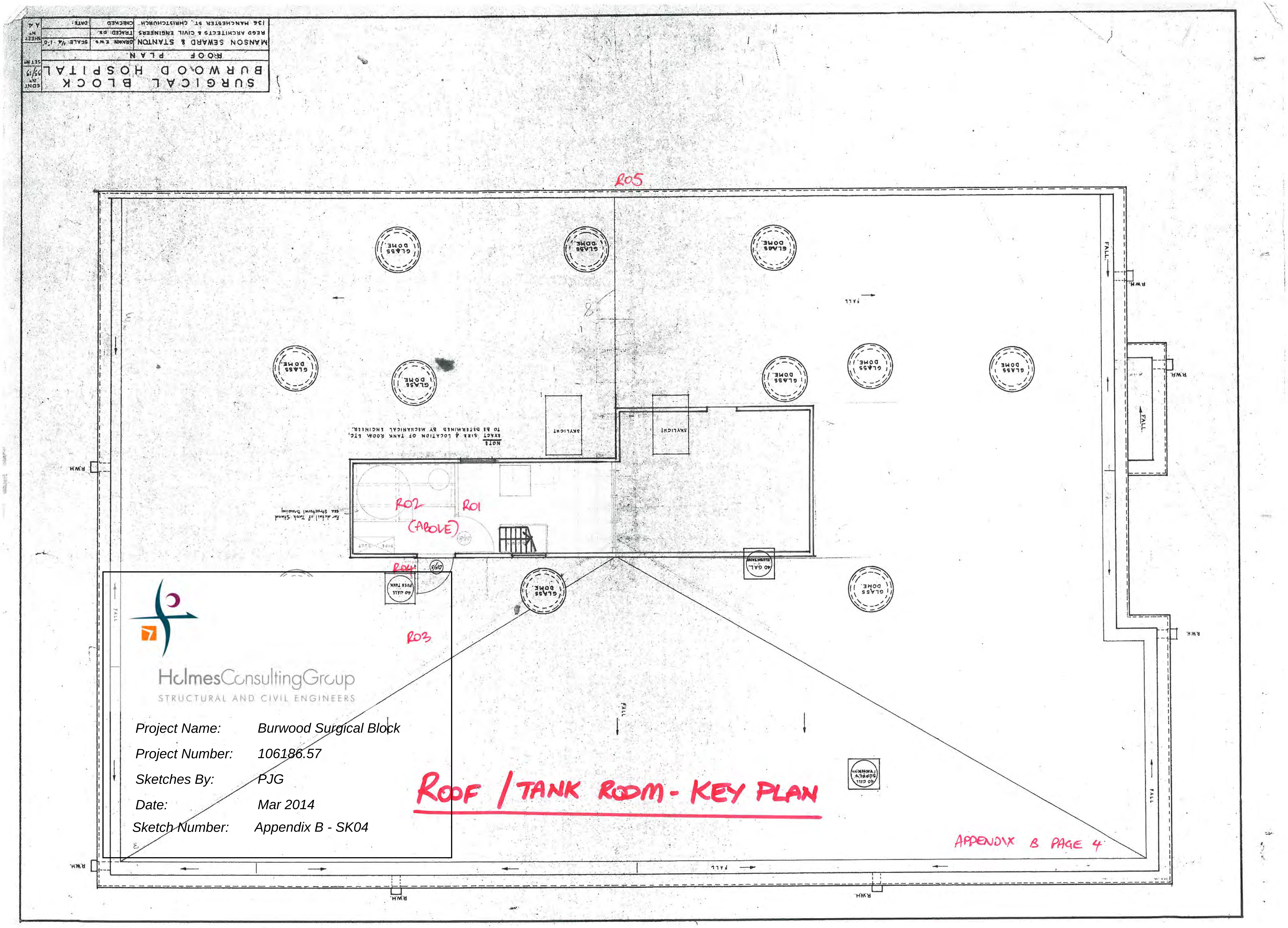
DWG No. 04043010

SCALE 1:100 AT A1

REVISED 15-06-09

XREFED DRAWING(S):

Maintenance and Engineering Department - Christchurch Hospital

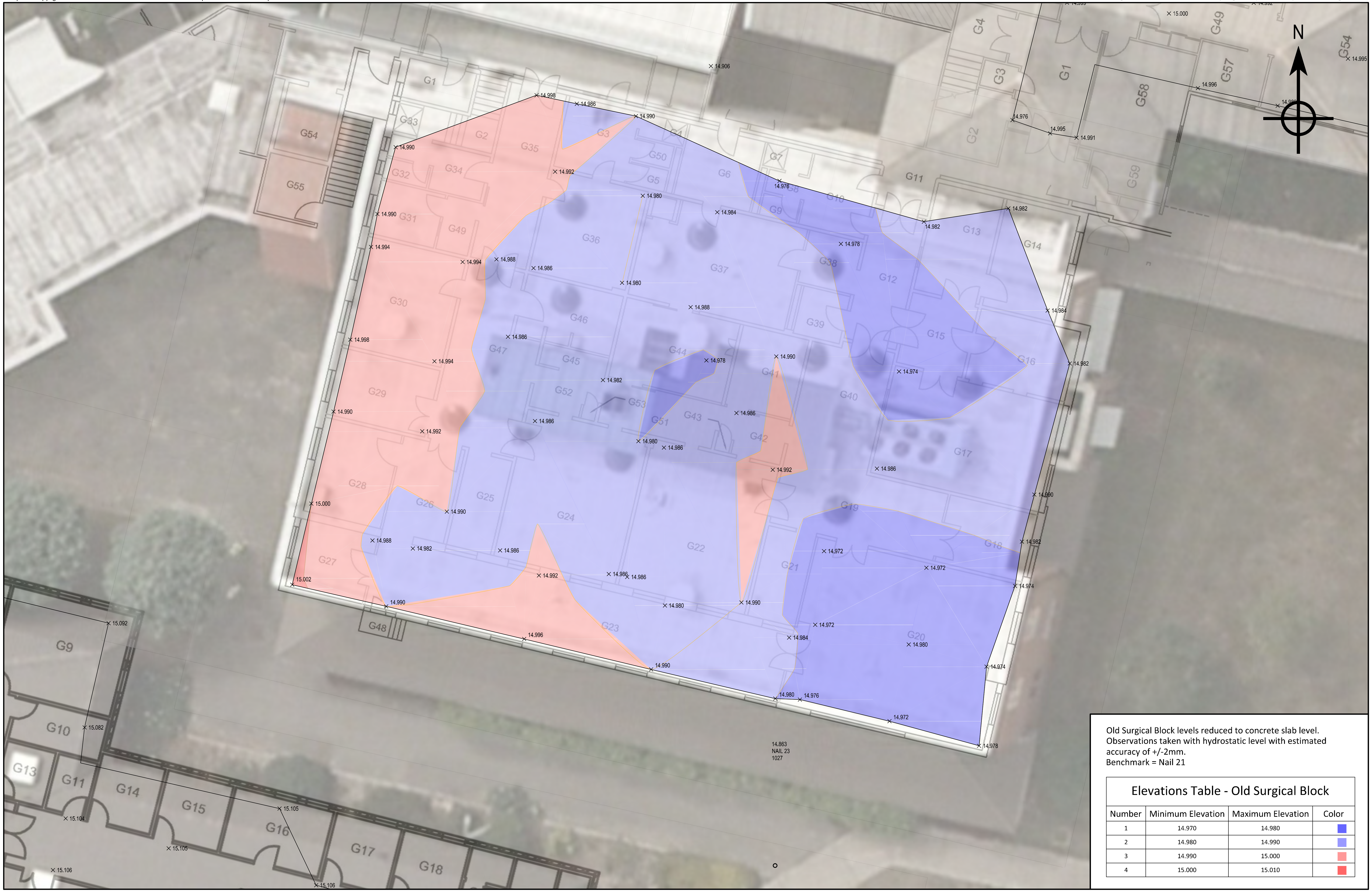


HolmesConsultingGroup
STRUCTURAL AND CIVIL ENGINEERS

Project Name:	Burwood Surgical Block
Project Number:	106186.57
Sketches By:	PJG
Date:	Mar 2014
Sketch Number:	Appendix B - SK04

APPENDIX C

Survey of Levels



Old Surgical Block levels reduced to concrete slab level.
Observations taken with hydrostatic level with estimated accuracy of +/-2mm.
Benchmark = Nail 21

Elevations Table - Old Surgical Block

Number	Minimum Elevation	Maximum Elevation	Color
1	14.970	14.980	
2	14.980	14.990	
3	14.990	15.000	
4	15.000	15.010	

No.	Revision	Approved	Date
G			
F	Beacon House, BSU Hostel, Chapel, Tapper and Milner added	MJM	18/04/2012
E	Audit observations post Dec 23 2011 (not included in this Revision)	MJM	01/02/2012
D	Wards 7 & 8, Surgical Block, Melrose Chairs & Corridor added	MJM	02/12/2011
C	Phys Med, Parafed, Food Services, BIRS, Boilerhouse, EDC added	MJM	24/11/2011
B	Nurses Hostel & Champion Centre, SOU elevations added	MJM	18/11/2011
A	Administration and Allan Bean Centre elevations added	MJM	31/10/2011

Scale		1:75
Reduced	A3	1:150
Designed		----
Drawn		DHP
Checked		MJM
Date		18/04/2012

Burwood Elevations Survey

Old Surgical Block

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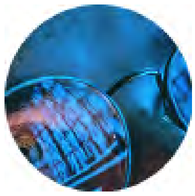
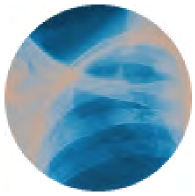
Job No. 2462C
Sheet No. 14 of 19
Rev. F

032005



DETAILED SEISMIC ASSESSMENT REPORT

STRUCTURAL AND CIVIL ENGINEERS



BURWOOD HOSPITAL CAMPUS

REPORT 24 – MAORI HEALTH

PREPARED FOR

CANTERBURY DISTRICT HEALTH BOARD

106186.77

INTERIM REPORT REV 3 – 9 AUGUST 2013

BURWOOD HOSPITAL CAMPUS – INTERIM DETAILED SEISMIC ASSESSMENT REPORT

REPORT 24 – MAORI HEALTH UNIT

Prepared For:
CANTERBURY DISTRICT HEALTH BOARD

Date: 9 August 2013
Project No: 106186.77
Revision No: 3

Prepared By:

Peter Grange
STRUCTURAL ENGINEER

Reviewed By:

Eric McDonnell
SENIOR PROJECT ENGINEER

Reviewed By:

Jenny Fisher
PROJECT DIRECTOR

Holmes Consulting Group LP
Christchurch Office

REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
18/07/12	1	Interim report for review
28/01/13	2	Updated to include additional investigations completed and added Jenny Fisher signature to title page
31/08/13	3	Updated to include lift shaft assessment

CONTENTS

APPENDICES

TABLES Page

FIGURES Page

Holmes Consulting Group has been engaged by Canterbury District Health Board (CDHB) to complete a full structural review of the Burwood Hospital Campus following the Lyttelton Earthquake. A series of reports have been compiled as part of this process. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The specific building reports, like this one on the Maori Health/Quantity/Clinical Skills Unit (Maori Health Unit for short), should be read in conjunction with the base report, and refer to the repair specification.

This report identifies the structural damage sustained by the Maori Health Unit (formerly known as the Administration building) as a result of the series of Earthquakes, including the Darfield Earthquake that struck at 4:36am on 4th September, 2010, the Lyttelton Earthquake that struck at 12.51 pm on the 22nd of February, 2011, the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The report summarises the effects of the damage on the lateral load capacity of the building, and provides structural repairs for the damage identified. The general form of the building, along with its capacity relative to current code levels, has been included for the buildings pre-earthquake undamaged state and post-earthquake state.

The Maori Health Unit was designed in 1961 and constructed in the period thereafter. The building is primarily a single storey structure with a lightweight flat bituminous roof over plywood sheathing and timber roof purlins. The roof framing is supported by interior and exterior timber framed stud walls below. The ground floor consists of an elevated timber framed floor over perimeter concrete sub-floor walls and isolated interior concrete piles. There is also a two-storey standalone steel lift shaft, added at a later date, which services the adjacent Surgical Block and Birthing Unit buildings.

The information available for the review included: the original architectural drawings [3], the 1976 Burwood Hospital Survey of Existing Buildings [4], a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [5], along with a level survey of the building completed by Fox & Associates [6].

The Maori Health Unit appears to have performed as would be expected for a building of this type and age. The bulk of structural damage is typified by minor cracking of the concrete sub-floor walls at areas of reduced section (vents) and cracking of the linings on the timber framed walls and ceilings. Differential ground settlement has also been noted across the building, resulting in a worst case slope in the ground floor framing of approximately 40mm over a 10m length (1:250 or 0.4%).

While the permanent slopes noted in the ground floor are within the typical range for residential timber framed construction, they may be deemed unacceptable by CDHB based upon the buildings function. If the slopes are deemed unacceptable, re-levelling of the ground floor could be achieved through the use of mechanical jacking. A discussion on re-levelling options is included in Section 4.

It is believed that the majority of the damage observed, including the onset of damage, occurred as a result of the 22nd February event. Further observations of the damage observed have been included in the body of the report.

Based upon a review of the drawings available and site investigations completed, the primary lateral force resisting elements of the Maori Health Unit were assessed in their pre-earthquake undamaged state. The assessed capacity of the building relative to the demand imposed by the current Design Basis Earthquake (DBE) is 70 % and 67% DBE in the N-S and E-W directions respectively. This is limited by the low capacities of some of the wall linings. The roof diaphragm has been assessed at 100 % in the E-W direction and 90 % in the N-S direction. This is limited by the conservative assumption of the nail spacing so is likely to be closer to 100 %. Both the ground floor diaphragm and the sub-floor walls are at 100 % DBE. However, the connection between the walls and the foundation walls has been assessed at only 45 % DBE, limiting the overall capacity of the structural system. The lift shaft has been assessed at 100% DBE.

For the purposes of this assessment the CDHB Maori Health Unit has been considered to be an Importance Level 2 building (IL2). If the building were to be assessed for an increased importance factor, IL3, the seismic demand would increase by 30% (R=1.3) and as such, the assessed capacities would be reduced proportionally.

The reduction in the lateral bracing capacity of the building due to the earthquake damage is difficult to quantify. The damage noted to the timber framed walls and the ceiling diaphragms may have resulted in some reduction in strength, although the primary affect is to the ongoing stiffness of the building. The reduced stiffness will result in larger future displacements during seismic events and additional risk of damage to interior linings and building contents.

The minimum repairs to reinstate the building to its approximate pre-earthquake undamaged condition are included in Section 4. This includes repair of the damaged concrete sub-floor walls along with the repair of damaged wall linings. Further to this, a brief strengthening scheme is provided in Section 5 to improve the capacity of the system beyond 67 % DBE. This involves installing extra bolts between the bottom-plate of the timber walls and the concrete foundation walls.

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 Canterbury District Health Board (CDHB) to complete a full structural review of the Burwood Hospital Campus following the Lyttelton Earthquake. A series of reports have been completed as part of this. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The individual building reports, like this one, should be read in conjunction with the base report and refer to the repair specification.

The Burwood Hospital Campus base report covers the purpose and scope of the structural review. The current statutory requirements relevant to earthquake damaged buildings are outlined and the level of shaking experienced at the site estimated. Earthquake induced ground settlement damage across the campus is also discussed. The repair specification has been prepared to include repair details for typical damage observed in buildings on the Burwood Hospital Campus and is referred to as required in the specific building reports.

1.1 SCOPE OF WORK

This report is on the Maori Health Unit/Quality/Clinical Skills Unit (Maori Health Unit for short) located at the CDHB Burwood Hospital Campus, Approximately 7 km north-east of downtown Christchurch. The building was previously known as the Administration building until these functions were located elsewhere on campus. The report identifies the general form of the structure, along with the gravity and lateral load resisting systems. Each component of the structural system was reviewed based upon the information available and any potential Critical Structural Weaknesses (CSW's) were noted.

The report also identifies the structural damage observed to date as a result of the series of Earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake, in particular, subjected the building to strong ground motions which significantly exceeded the current code loading demand for buildings of this nature.

The capacity of Maori Health Unit has been assessed relative to current code loading in the buildings pre-earthquake undamaged state and in its post-earthquake damaged state. The post-earthquake assessment summarises the effects of the damage identified on both the gravity and lateral load resisting elements. Repair options to restore the capacity of the building to pre-earthquake levels for strength, durability and stiffness have been included. The repair options aim to maintain the buildings utility. Where required, strengthening options have also been provided.

1.2 LIMITATIONS

Findings presented as a part of this project are for the sole use of the Canterbury District Health Board, 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. Because all of the structure has not been available for detailed inspection or evaluation, this report is limited to those elements available and engineering judgement as to the likely condition of unseen elements. 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. PRE-EARTHQUAKE BUILDING CONDITION

This section discusses the form and capacity of the building prior to the Darfield Earthquake.

2.1 BUILDING FORM

The Maori Health Unit (formerly known as the Administration building), is a single storey timber framed building designed in 1961 and constructed in the period thereafter. The building incorporates a corridor on the north end of the building which links to the Physical Medicine building, Surgical Orthopaedic Unit, Birthing Unit and Surgical Block.

Some alterations have been made to the building over its lifespan including the 2002 conversion of the original west-facing entrance of the building into an ambulance bay. There is also a two-storey standalone steel lift shaft, servicing the neighbouring buildings, which appears to have been added at a later date.

Figure 2-1: North face of the Maori Health Building

Figure 2-2: Lift Shaft on East Side of Building

The information available for the review included: the original architectural drawing [3], the 1976 Burwood Survey [4], a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [5], along with a level survey of the building completed by Fox & Associates [6].

The Maori Health Unit is primarily a rectangular single storey structure, approximately 29 m by 14 m in plan, with a lightweight flat bituminous roof over plywood sheathing and timber roof purlins. The roof framing is supported by interior and exterior timber framed stud walls below. The ground floor consists of an elevated timber framed floor over timber bearers, perimeter concrete sub-floor walls and isolated interior concrete piles. The timber bearers are bolted into the top of perimeter concrete sub-floor walls and connected to the interior piles with metal wire fixings.

The various claddings of the external walls include vertical weatherboard, plywood sheathing and lightweight timber panels. The internal walls claddings consist of fibrous plasterboard, plywood sheathings, partial height timber panelling and diagonal board sheathing. The ceilings of the building consist of fibrous plasterboard linings.

A plan view of the current layout of the Maori Health Unit, indicating the locations of the various wall materials, is shown in Figure 2-3. The exterior south elevation of the building, along with a typical cross-section through the building is shown in Figures 2-4 and 2-5.

The two storey lift shaft, shown in Figure 2-6 is steel framed with lightweight steel external cladding.

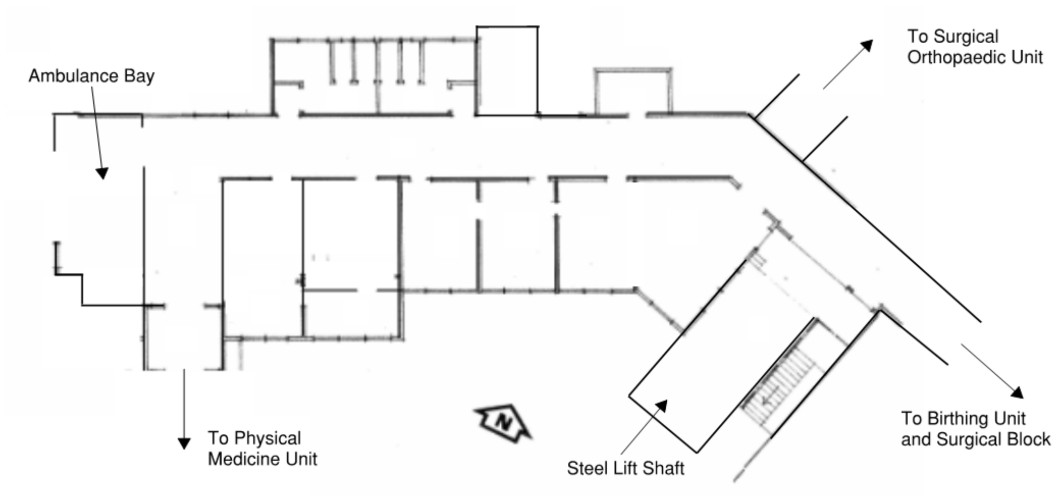


Figure 2-3: Maori Health Unit - Ground Floor Plan

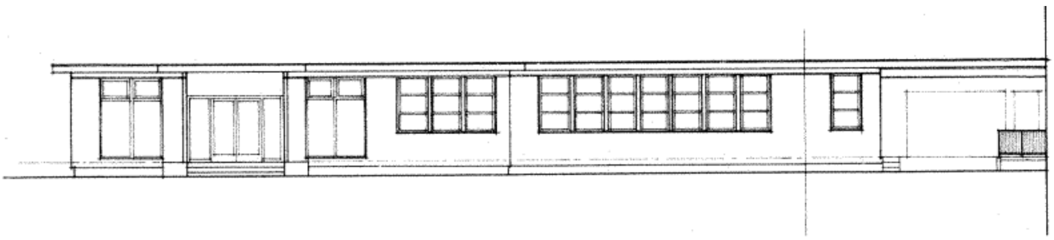


Figure 2-4: Maori Health Unit - South Elevation

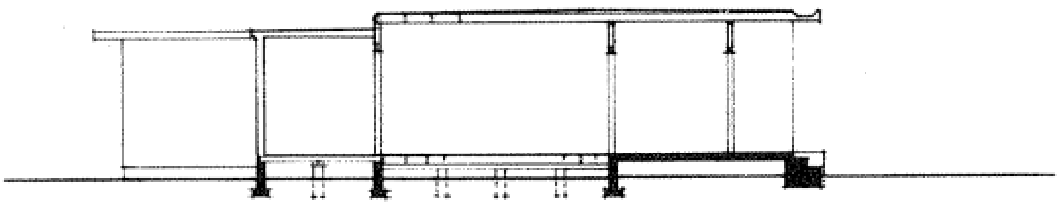


Figure 2-5: Maori Health Unit – Typical Cross-Section

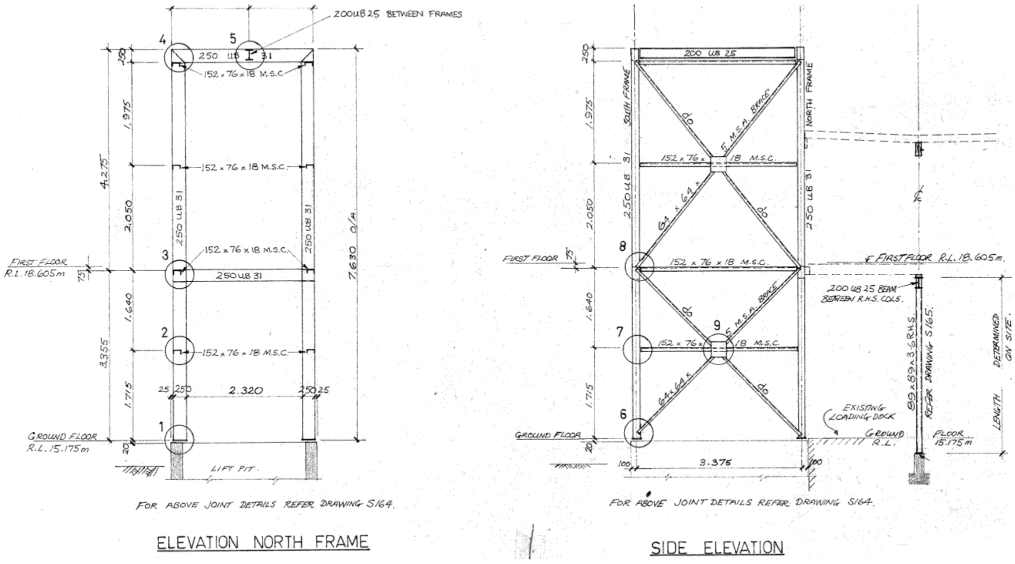


Figure 2-6: Maori Health Unit - Lift Shaft

2.2 GRAVITY LOAD SYSTEMS

The lightweight roof consists of bituminous fabric and plywood supported by timber roof purlins which span between the internal and external timber framed stud walls below. The external walls are founded on reinforced concrete sub-floor walls and strip footings. The internal walls and the timber floor structure are supported by timber joists and bearers on a grid of isolated concrete piles and an additional line of sub-floor wall running the length of the building as highlighted in Figure 2-7.

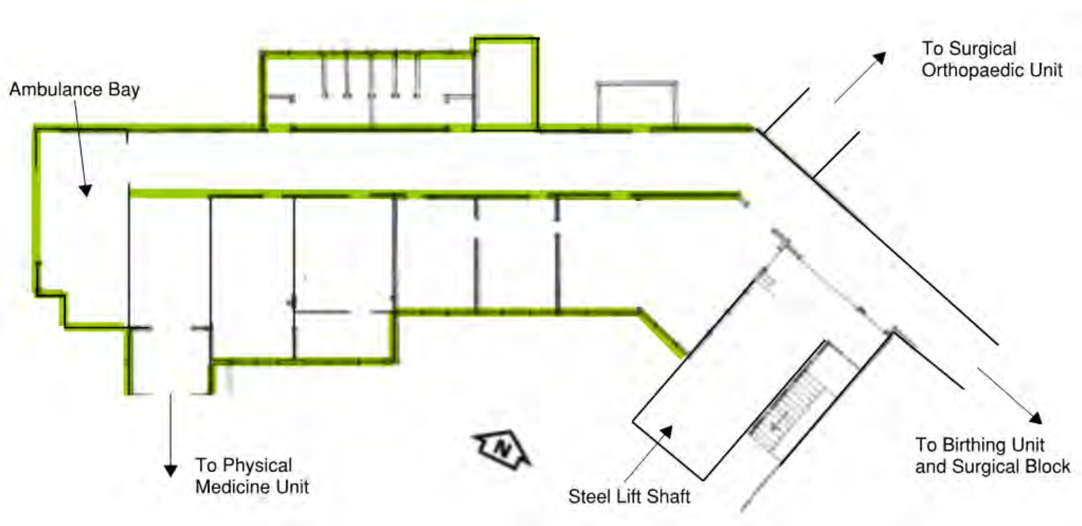


Figure 2-7: Maori Health Unit – Foundation Plan: Sub-floor walls

2.3 LATERAL LOAD RESISTING SYSTEMS

The primary lateral load resisting system for the superstructure of the Maori Health Unit consists of timber framed bracing walls and a plywood lined roof diaphragm. As previously noted the internal and external walls have a mixture of lining materials. In general, only the primary lining material was considered along any single bracing line due to stiffness incompatibility between the lining materials.

At the ground floor level, the floor diaphragm consists of straight board timber sheathing which transfers seismic loads to the perimeter concrete sub-floor walls below.

The lift shaft acts as a moment frame in the E-W direction, while truss action is developed in the N-S direction through the steel cross-braces. The frame is bolted down to transfer the load directly into the concrete foundation.

2.4 PRE-EARTHQUAKE BUILDING CAPACITY – DIRECT CODE COMPARISON

The Maori Health Unit was designed in 1961 to a predecessor of the current New Zealand Building Code; which was likely the *New Zealand Standard Model Building By-Law for Light Timber Framed Construction,* NZSS95:1955 [7]. When these By-Laws were written, neither the seismology of the different areas within New Zealand, or the impact this could have on buildings was as well understood as it is today. As a result, the bracing requirements for timber framed buildings built at this time were minimal.

The structure is not regarded as an essential hospital facility by the CDHB and has therefore classified as an Importance Level 2 (IL2) building in accordance with NZS 1170:2004 [8], with a design life of 50 years for performance and durability of elements. This assumption will need to be verified by CDHB.

If the same building were to be built today it would likely be designed to *New Zealand Standard Timber Framed Buildings*, NZS 3604:2011[9], which is consistent with timber framed, IL2, buildings of this size. . This standard incorporates amendments made to the loading codes as a result of the Lyttelton Earthquake, as outlined in Amendment 10 of the Building Code [10]. The implications of the recent amendments are discussed more fully in the Burwood Hospital Campus Base Report however, for a building of this type, the amendments essentially result in an increase to the design loads of approximately 67 % when compared to pre-earthquake, NZS3604:2001, design levels. The actual percentage difference between the bracing required in current standard and the 1955 By-Laws is several times larger.

2.5 EARTHQUAKE ANALYSIS TO NZS3604 (2011)

In addition to the direct code comparison provided above, a seismic analysis to NZS 3604:2011[9] has been carried out to gain a better understanding of the estimated capacity of the building when compared to current loading standards. The steel lift shaft capacity was assessed elastically using NZS1170.5: 2004 [18]. This analysis was carried out based upon the limited information available and site observations.

Following the Lyttelton earthquake, a geotechnical report was conducted by Tonkin & Taylor, titled "Burwood Hospital Post Earthquake Geotechnical Assessment", dated June 2011 [5]. This report has been used to aid in the evaluation of the site conditions and the likely effect of the ground on the buildings past and future performance. The soil parameters described in the report have also been used for the evaluation of the existing foundation system of the building.

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 [11] and the requirements of NZS 1170:2004. 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. These are described in more detail in the Burwood Hospital Base report [1].

Based upon the analysis performed, a relative capacity of each primary structural element has been presented as percentage of the demand imposed by the current Design Basis Earthquake (% DBE). The Design Basis Earthquake, as defined in NZS 1170:2004, is dependent on a buildings physical location, local soil conditions, building type, fundamental period and importance level.

For the purpose of this evaluation several assumptions also had to be made in regards to the existing building properties. The expected strength values for these elements were taken from NZSEE 2006, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes* [11] and ASCE 41-06, *Seismic Rehabilitation of Existing Buildings* [12]. The ground floor diaphragm values, taken from NZSEE 2006 publication, were divided by 1.5 to account for built in over-strength. The assumed values for the existing bracing elements could be further refined through destructive investigations of the existing materials and their associated fixings. The assumed diaphragm and shear wall expected strength values used in the assessment of the building are as follows:

- Internal Walls: Unblocked timber framed stud walls with various linings of plasterboard, plywood sheathing or partial height timber panelling. Expected Strength = 1.5 kN/m (30BU/m) with a ductility, μ = 3.3.

- External Walls: Unblocked timber framed stud walls with exterior linings of various materials and interior fibrous plasterboard linings with diagonal board sheathing in between. Expected Strength = 9.1 kN/m (182 BU/m) with a ductility, μ = 1.4.

- Roof Diaphragm: Plywood sheathing over timber roof purlins. Expected strength = 6.0 kN/m (120BU/m) with ductility μ = 3.5 with nails at 300 mm cntrs (conservative)

- Ground Floor Diaphragm: Straight tongue and groove sheathing over timber floor joists. Expected strength = 2.8 kN/m (60BU/m) with ductility μ = 3.5

- Reinforced Concrete Sub-floor Walls: Expected strength = 15 kN/m (300BU/m)

- Steel framing of lift shaft: Expected strength = 300MPa

The bracing requirements in NZS 3604:2011 assume a ductility factor, μ = 3.5 for the bracing walls and diaphragms. To account for the less ductile existing walls outlined above, the wall bracing demands from NZS 3604:2011 have been factored up proportionally as required in our analysis. Values for the bracing supplied by the reinforced concrete sub-floor walls were taken from NZS 3604:2011.

The foundations have been assessed at an ultimate bearing capacity of 150kPa, for factored loads, as per recommendations provided by Tonkin and Taylor [5].

A summary of the capacity of each primary lateral element as a percentage of the demand imposed by the Design Basis Earthquake (DBE) have been noted in Table 2-1 below.

Table 2-1: Summary of %DBE – Main Building

Building Element	%DBE (IL2)	Comments
Ground Floor Walls - N-S E-W	70% 67%	Limited by the length of bracing wall and low capacity of some wall lining materials
Roof Diaphragm – N-S E-W	90% 100%	Limited by the conservative assumption on the nail spacing
Ground Floor Diaphragm – N-S: E-W	100% 100%	
Sub-Floor Walls and Footings - N-S E-W	100% 100%	
Wall to floor bolted connections	45 %	Limited by bolt spacing of 1.5 m
Lift Shaft	100%	

Due to the lightweight nature of the building, along with a robust roof diaphragm and numerous and well dispersed bracing walls, nearly all aspects of the building superstructure have been assessed above 67 % DBE and many over 100 %. Likewise, the ground floor diaphragm and the concrete sub-floor walls and footings have been assessed above 100% DBE. However, due the relatively large spacing between the bolts connecting the external walls to the foundation walls, the capacity of the system is reduced to 45 %.

If the building were to be assessed for an increased importance factor, IL3, the seismic demand would increase by 30% (R=1.3) and as such, the assessed capacities would be reduced proportionally.

A review of the drawings available, along with site observations, revealed no obvious critical structural weaknesses (CSW's).

3. POST-EARTHQUAKE BUILDING CONDITION

This section covers the structural damage sustained by the Maori Health/Old Admin Building as a result of the series of earthquakes which includes the Darfield Earthquake that struck at 4:36am on 4th September, 2010, the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011 and its effect on the capacity of the building to resist seismic loads. The Lyttelton Earthquake subjected the building to strong ground motions which significantly exceed the full design earthquake load for buildings of this nature and appears to have caused the majority of the earthquake damage observed.

3.1 THE LYTTELTON EARTHQUAKE

The Fundamental Period of the building is estimated to be between 0.2 and 0.4 seconds. 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 strong motion data presented in the Base Report, it appears the Lyttelton Earthquake produced ground shaking intensities of approximately 60-120% of the current Ultimate Limit State design spectra for an Importance Level 2 building of nominal ductility.

It should be noted that the Lyttelton Earthquake was very short in terms of strong shaking produced with the strong motion only lasting for duration of 5-7 seconds. Rupture of an alpine fault is expected to contain 50 to 60 seconds of strong motion.

3.2 PRELIMINARY INVESTIGATIONS

Preliminary investigations were carried out to identify 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
- Review of available documentation
- Damage observed during 'Rapid' Structural Assessments following the Lyttelton Earthquake, June 13th, 2011 aftershocks, December 23rd, 2011 and January 2nd, Earthquakes

Following a review of the documentation available, including previous work associated with this building [4] the following areas were identified for potential damage:

- Connections of timber framing to foundation elements
- Damage at timber roof framing to stud wall connections
- Cracking to linings of timber framed walls
- Distress to timber framed floor and roof diaphragms
- Cracking in continuous concrete footings due to earthquake induced differential ground settlement
- Displacement of ground around perimeter of building

Rapid Level 2 Assessments were carried out on the 24th February 2011[13] and on 14th June 2011 [14]. An additional Visual Structural Assessment was completed on the 9th January, 2012 following the 23rd December 2011 and 2nd Jan 2012 events [15]. These structural observations involved a complete walk around the exterior and throughout the interior of the building. The following primary areas of damage were identified from the damage assessments:

- Cracking in external footings, primarily in areas of reduced section (vents)
- Cracking to internal and external wall linings and ceiling finishes

A review of the above information on the building type and preliminary observation highlighted this building as requiring a detailed inspection. The aim of the detailed inspection was to determine the cause and full extent of damage to the building, particularly the elements identified for potential damage above. These items were targeted to identify if damage had occurred and to what extent the damage had reduced the capacity of the buildings lateral load resisting system to withstand future seismic events.

3.3 DETAILED OBSERVATIONS

Further detailed inspections and structural explorations have been carried out following the initial assessments to ascertain the full extent of structural damage to the building. The detailed structural observations were completed between 7 May and 18 May, 2012. A full record of these observations can be found in Appendix A, with reference plans describing the location labelling used found in Appendix B. A full photographic record of the observations is available electronically on request. The detailed structural observation identified the following additional damage to those items noted in the initial rapid assessments:

- Additional occurrences of cracking to external linings
- Additional occurrences of cracking to internal linings

3.4 GEOTECHNICAL REVIEW

A review of the ground damage and conditions was carried out by Tonkin & Taylor for the Burwood Hospital Campus. A subsequent report titled *Burwood Hospital Post Earthquake Geotechnical Assessment* was issued in June 2011 [5]. The geotechnical review concluded that the settlement and damage to building foundations and slabs on the Burwood Hospital Campus was likely due to the liquefaction of underlying soil layers. It is believed that excessive pore water pressures have been dissipated and that further settlement is not expected to occur, unless another significant event was to occur.

Based on the geotechnical report provided by Tonkin & Taylor [5] the potential for future total and differential settlements at the building site varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

3.5 LEVEL SURVEY

A detailed survey of the ground floor levels in the Administration Building was conducted by Fox & Associates and issued on 18th June, 2012 [6]. The survey indicates differential ground settlement at each end of the building, resulting in a worst case drop in the elevated ground floor framing of approximately 40 mm over a 10 m length (1:250 or 0.4%).

While the permanent slopes noted in the ground floor are within the typical range for residential timber framed construction, they may be deemed unacceptable by CDHB based upon the building function. If the slopes are deemed unacceptable, re-levelling of the ground floor could be achieved through the use of mechanical jacking. A discussion on re-levelling options is included in Section 4.2.

For the extent of the differential settlement noted to the building see Figure 3-1 below and the level survey included in Appendix C.

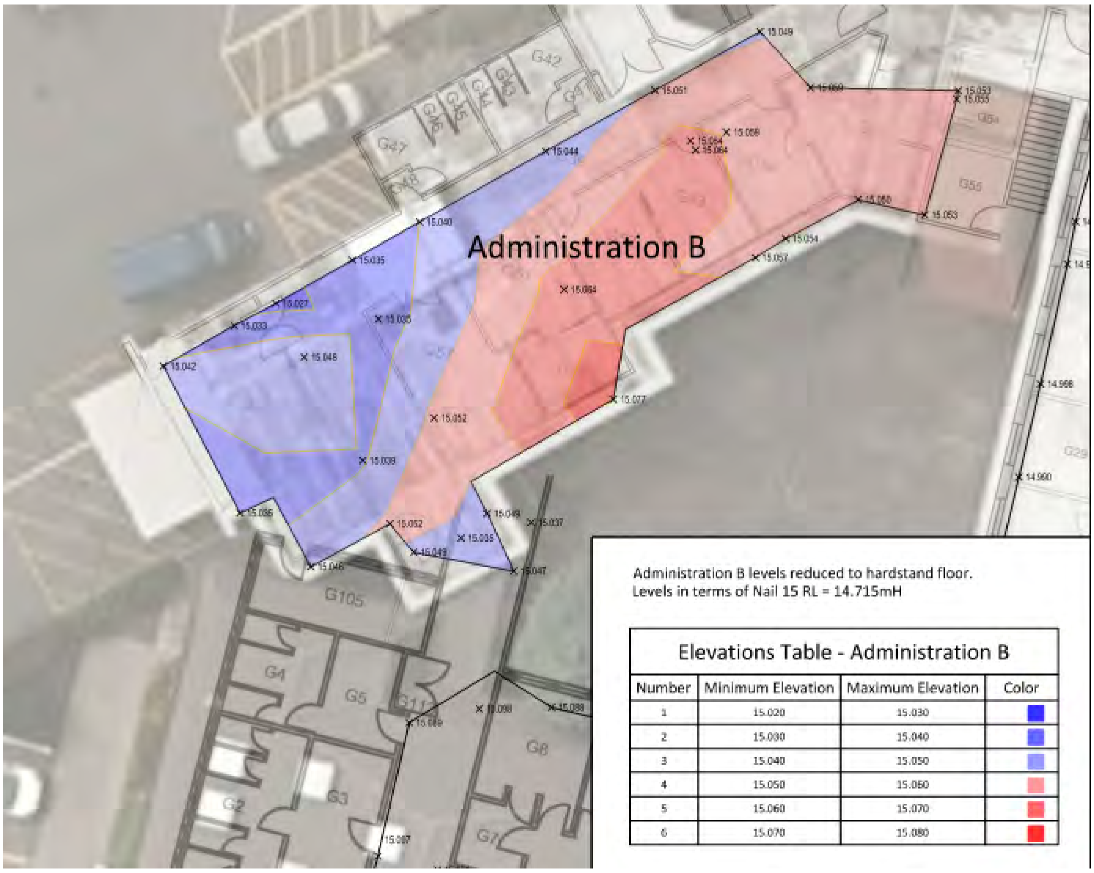


Figure 3-1: Maori Health Unit – Level Survey

3.6 SUMMARY OF BUILDING DAMAGE

The following is a summary of the observations made for the building, and our conclusions as to their condition and seismic load resisting capacity. These observations do not specifically distinguish between damage caused by the Darfield Earthquake, the Lyttelton Earthquake, or any significant aftershocks thereafter, such as those that occurred on 13th June 2011, 23rd December 2011 and 2nd January 2012.

The Maori Health Unit appears to have performed relatively well for a building of this type and age. The majority of the structural damage is cracking of the concrete footings at areas of reduced section (vents) and cracking of the linings on the timber framed walls and ceilings. A summary of the typical structural damage observed is as follows:

- **Differential Ground Settlement –** A worst case differential ground settlement of approximately 40 mm has occurred across the length of the building, resulting in permanent slopes in the ground floor framing.
- **Minor Cracking to Sub-floor Walls -** Settlement induced cracking of concrete sub-floor walls, particularly at areas of reduced section (vents)
- **Cracking to Wall Linings** – Cracking and general distress has been noted to internal and external wall linings, primarily at corners, openings and along wall board joints.
- **Cracking to non-structural elements** – Damage to door jambs and ceiling finishes.
- **Water Damage** – In eastern room (next to lift shaft) water damage to the ceiling linings has been noted.
- **Lift Shaft** – No damage was noted to the external cladding of the lift shaft and only very minor damage noted to the internal wall linings. Based on this it is very unlikely that any damage has been sustained by the steel structure so further, intrusive investigations are not required.

Section 4, Table 4-1 provides a photographic summary of the typical damage observed. A full record of our detailed observations and repairs required can be found in Appendix A.

3.7 FURTHER INVESTIGATIONS REQUIRED

3.7.1 Investigations Required For Further Assessment

Destructive exploration was undertaken in a number of locations in order to verify the assumptions made in the first revision. The areas investigated were as follows:

- Verify size and spacing of fixings between stud wall framing and sub-floor walls below.
    - *2-M12 anchor bolts at 1500 mm spacing were observed*
- Validate fixings between existing ceiling framing and top of bracing walls.
    - *90 x 3.55 mm nails (load path is complete)*

3.7.2 Investigations to be completed during Building Repairs

- Re-inspection of building will be required upon completion of any re-levelling works to determine if any additional damage has occurred.

- Investigation into ceiling water damage in eastern most room to establish cause of water presence and whether or not this is earthquake related. The condition of the water proof membrane should be checked.

3.8 POST-EARTHQUAKE BUILDING CAPACITY

Based upon our observations to date, we do not consider the Maori Health Unit to have any significant reduction in gravity load resistance. The damage observed to the interior and exterior wall sheeting will have resulted in some reduction in lateral load capacity, although it is difficult to quantify the actual percentage reduction in strength. While there has been some reduction in strength, according to the Department of Building and Housing, *Revised Guidance on Repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence* [16], the primary result of the damage noted will be a reduction in the overall stiffness of the building. The reduction in stiffness may cause ongoing concerns in regards to the buildings performance, primarily to contents and non-structural elements.

The differential settlement noted will also have resulted in some reduction in capacity, but again this is difficult to quantify. The primary concern will be reduced ability to absorb additional differential settlement prior to the onset of more severe damage to the foundations of the building and super structure above.

The damage observed will require repair to restore the strength, stiffness, durability and performance of lateral bracing system. The repair work required is outlined in Section 4. Following the recommended repairs to the structural damage noted, the lateral load capacity of the existing structure will be restored to close to pre-earthquake levels, which are summarised in Section 2.5

4. OBSERVED DAMAGE AND REQUIRED REPAIRS

4.1 PRIMARY DAMAGE OBSERVED AND REPAIRS REQUIRED

This section covers the damage noted during our detailed assessment of the building. Note that our observations have been restricted to structural aspects of the building 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.

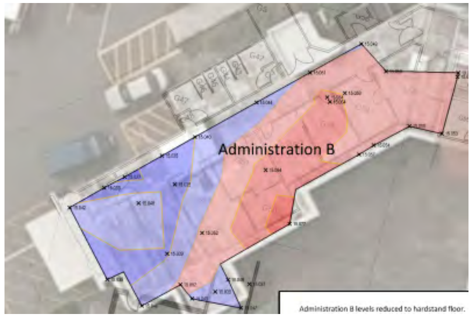
Table 4-1 provides a photographic summary of the observed damage and typical repairs required. The table should be read in conjunction with Appendix A – Record of Observations and Appendix B – Location Reference Plans.

In general, the aim of the repair work indicated is to restore the structure to its pre-earthquake state as close as practicable, while maintaining the utility of the building. The repairs presented attempt to address the loss of strength, stiffness and durability of the structural elements due to the damage noted.

It should be noted that more damage may be identified during the repair works and (if required) additional repair details will be specified accordingly.

Please note that if building is to be re-levelled, all repair works are to be completed after the building has been re-levelled to a satisfactory condition as further damage to the wall and ceiling linings can be expected during the re-levelling process.

Table 4-1: Photographic Summary of Primary Damage Observed and Repairs Required

Damaged Item	Location	Recommended Repair	Example (Photo refs as Appendix A)
1. Foundations			
1.1. Localised areas of differential settlement have resulted in slope of ground floor timber framing of up to 0.4% (1:240)	Refer: Appendix C - Survey of Levels	If the CDHB deems the floor slopes as represented in *Appendix C – Survey of Levels* to be unacceptable; remediation of floor levels would be required through localised lifting of the structure. See section 4.2 for additional information.	
1.2. Minor cracking to sub-floor concrete walls, primarily at areas of reduced section (i.e. vents)	Mostly on south side of building	Epoxy inject all cracks in concrete walls >0.2mm as per the HCG Repair Specification [2] For cracks greater than 0.5-0.6mm, HCG confirm the integrity of the reinforcement at top and bottom of wall. If reinforcement is damaged, an engineered repair will be required. Refer to HCG specification.	 DSCF1350

Damaged Item	Location	Recommended Repair	Example (Photo refs as Appendix A)
2. Walls			
2.1. Damage to vertical wallboards where relative movement with pipe has occurred	West side of building in alcove	Repair/replace damaged exterior linings. *Repair specification by others*	 DSCF1347
2.2. Typical cracking at edges of plasterboard sheets, particularly at wall intersections and off the corners of door and window openings	Along corridor and in corners of office rooms	Replace damaged ceiling boards with new gypsum board sheets. All wall boards to remain are to be re-fixed. See discussion in Section 4.3 for additional information.	 P1080515

Damaged Item	Location	Recommended Repair	Example (Photo refs as Appendix A)
3. Ceilings			
3.1. Cracking at ceiling panel edges	Throughout corridor	*Aesthetic repair – specification by others*	 DSCF1351
3.2. Cracking through plasterboard ceiling panels	Ambulance bay	*Aesthetic repair – specification by others*	 DSCF1353

Damaged Item	Location	Recommended Repair	Example (Photo refs as Appendix A)
3.3. Cracking through plasterboard lining surrounding roof beam	Eastern most room	*Aesthetic repair – specification by others*	 DSCF1341
3.4. Water damage to ceiling lining	Eastern most room	Further investigation required (Section 3.7). *Repair by others*	 DSCF1347

Damaged Item	Location	Recommended Repair	Example (Photo refs as Appendix A)
4. Lift Shaft			
4.1. Minor distress of wall linings at corners. As the damage observed to linings is very minor, it is very unlikely the steel frame has sustained any damage so further intrusive investigations are not required.	Edge of plasterboard panels in corners of shaft	*Repair would be for aesthetic reasons only if required. This area is not in public view.*	 P1040009

4.2 DISCUSSION ON BUILDING RE-LEVELLING AND REPAIR OF FOOTINGS

The level survey, completed by Fox & Associates [6] has indicated a total differential ground settlement across the building of approximately 40mm (see Appendix C for complete level survey). While the differential settlement has been noted throughout, the worst case permanent slope in the elevated ground floor framing, based upon the level survey, is approximately 1:250 (or 0.4 %). The recorded slopes are within the typical acceptable range for standard occupancy buildings of timber framed construction; however, given the nature of the patient group occupying the building, CDHB may wish to pursue re-levelling of the building.

If CDHB deems the slopes unacceptable, the Maori Health Unit can be re-levelled through the use of mechanical jacking. This could be achieved by disconnecting the ground floor framing from the existing foundation system, jacking the floor up to level and then reconnecting the floor to the concrete sub-floor walls and footings below. Alternatively, the jacking plane could be lowered below the perimeter footings. For either option additional adjustment of the interior piles will likely be required.

It should be noted that the re-levelling discussed above is not expected to increase the seismic performance of the building or reduce the potential for future differential settlements. Instead this option is intended to re-level the building without making the future performance of the building any worse than it was prior to the earthquakes.

It should also be noted that during the re-levelling process there is the risk that additional damage could occur to the building linings and exterior veneers. Appropriate contingencies would need to be provided and it would be sensible to complete other repair work required on the buildings only after the re-levelling process has been completed.

4.3 REPAIR OF WALL BRACING

The wall linings of the interior bracing walls have been damaged in numerous places and require repair. Based on the movement it is also believed the wall lining fixings have been damaged throughout the building. This has resulted in a reduction to the ongoing strength and stiffness of all the bracing walls. In order to reinstate the pre-earthquake strength and stiffness to the bracing walls, the repair recommendation is to remove all cracked or damaged sections of wall linings and replace them with new gypsum wallboard sheathing. The new gypsum board is to be fixed in accordance with GIB 'ezybrace' GS2-N specifications [17] (or equivalent). All existing internal wall linings to remain are to be re-fixed to existing studs in a similar manner. A new finish is then to be applied to all interior walls.

5. STRENGTHENING REQUIRED

As stated in Section 2.5, all components of the Maori Health Unit are above 33 % DBE so the building is not considered to be Earthquake Prone. However, due to the minimal connection between the wall framing and the foundation wall, the overall system of the building is reduced to 45 %. As this is below 67 %, the building could be considered to be an Earthquake Risk.

A simple strengthening scheme has been devised to improve the overall capacity of the building to over 67 % DBE should the CDHB wish to improve the building performance. Strengthening should be carried out by installing extra bolts to improve the connection between the bottom-plate and the foundation wall. Currently the M12 bolts are spaced at 1500 mm centres so extra M12 bolts should be installed at half-way between the existing bolts resulting in bolts at 750 mm spacing. This will increase the capacity of this connection to close to 100 % if this is carried out around the perimeter of the building.

6. REFERENCES

1 *CHDB – Burwood Campus – Detailed Seismic Assessment Report – Base Report*, Holmes Consulting Group, November 2011.

2 *CHDB – Burwood Campus – Detailed Seismic Assessment Report – Repair Specification*, Holmes Consulting Group, November 2011.

3 *Administration Building Burwood Hospital*; Architectural Drawings, Manson Seward & Stanton Registered Architects & Civil Engineers, Christchurch

4 *Burwood Hospital Survey of Existing Buildings,* North Canterbury Hospital Board, Cutter Pickmere Douglas Architects, 1976.

5 *Burwood Hospital Post Earthquake Geotechnical Assessment*, Tonkin and Taylor Ltd. June 2011.

6 *CDHB – Burwood Field Survey*, Fox & Associates, June 2011

7 *NZS 95 – 1955*, New Zealand Standard Code of Building By-Laws – Part IX Light Timber Construction.

8 *Structural Design Actions Part 0: General Principles, AS / NZS 1170.0:2002*, Standards New Zealand, 2002.

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

10 Department of Building and Housing, *Compliance Document for New Zealand Building Code - Clause B1 – Structure, Amendment 10 (Canterbury)*, Department of Building and Housing, Wellington, 19 May 2011.

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

12 *Seismic Rehabilitation of Existing Buildings*, ASCE 41-06, 2007

13 *CDHB – Burwood Hospital Campus – Rapid Visual Inspection: 106186.03SR1,* Holmes Consulting Group, 24 February 2011

14 *CDHB – Burwood Hospital – Rapid Seismic Assessment Post June 13th Earthquakes: 106186.03SR4* Holmes Consulting Group, 14 June 2011.

15 *CDHB – Burwood Campus – Post Earthquake Rapid Visual Assessment Post 2 Jan 2012 5.5M EQ*, Holmes Consulting Group, 9 Jan 2012

16 *Department of Building and Housing, Revised guidance on repairing and rebuilding houses affected by the Canterbury earthquake sequence*, Department of Building and Housing, Wellington, November 2011.

17 *GIB EzyBrace Systems*, June 2011

18 *Structural Design Actions Part 5: Earthquake Actions – New Zealand, NZS 1170.5:2004*, Standards New Zealand, 2004.

APPENDIX A

Record of Observations

APPENDIX A – RECORD OF OBSERVATIONS & REPAIRS

Inspection date: 7 May 2012 through to 18 May 2012

KEY	
N	No repair required
Y	Repair required
F	Further investigation required
C	Repair complete

Level
E - exterior
SF - subfloor
G - ground

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
E	G12	Outside west side of building	Timber wall	Cracking in timber board where relative movement to pipe has occurred	**Y**	All cracked and damaged wall boards are to be removed and relined with equivalent board sheeting. All existing wall boards to remain are to be re-fixed to the timber stud walls	DSCF1347
SF	G5-7	South side of building	Perimeter foundation wall	Concrete cracking from corner of grates mostly <0.4 but some up to 1 mm	**Y**	For all cracks greater than 0.2mm and less than 1mm, epoxy inject cracks in accordance with HCG specification. (Cracks greater than 1mm require investigation to confirm the integrity of the steel reinforcement. Refer to HCG Specification	DSCF1350
G	Corridor	Typical	Ceiling	Minor cracking at panel edges	**Y**	All cracked and damaged ceiling boards are to be removed and relined with gypsum board sheeting. All existing ceiling boards to remain are to be re-fixed to the timber studs	DSCF1351

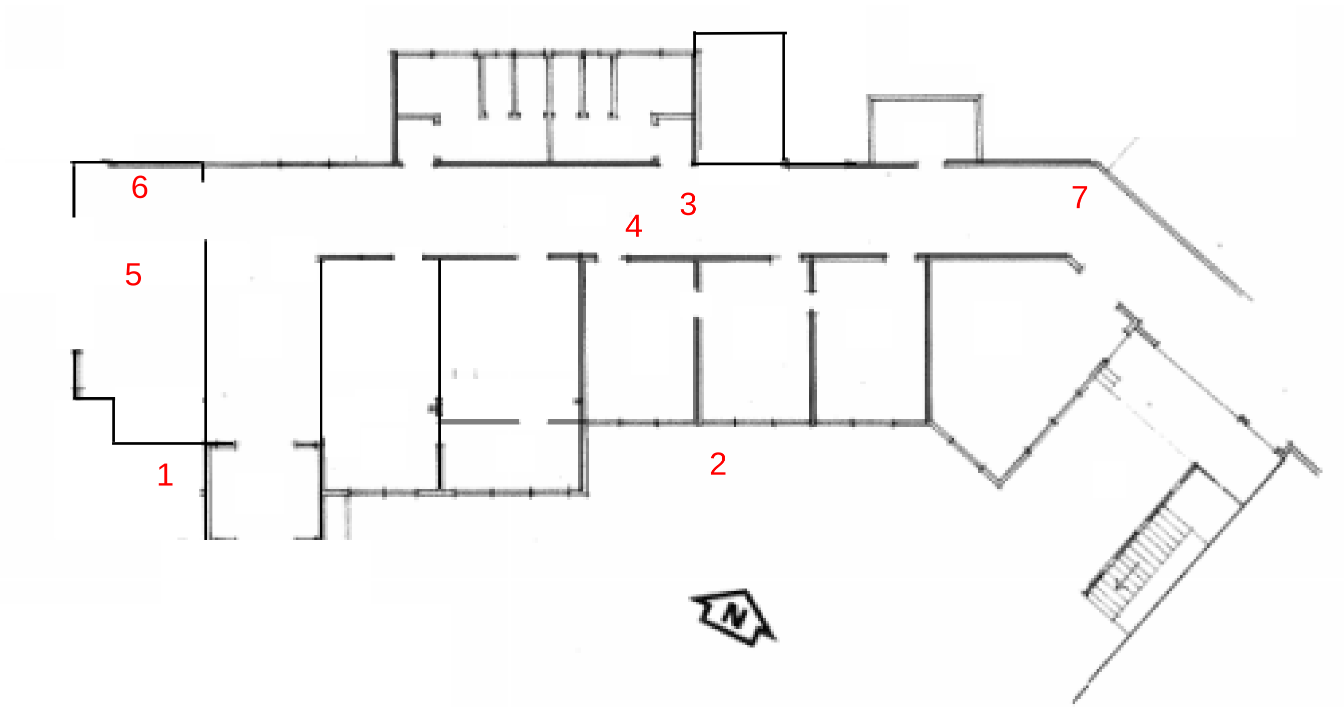
Refer to Table 3.1 and HCG Specification for repair details

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	Ambulance bay	Towards noth side of room	Ceiling	Cracking in ceiling gib at edge of panel	**Y**	All cracked and damaged ceiling boards are to be removed and relined with gypsum board sheeting. All existing ceiling boards to remain are to be re-fixed to the timber studs	DSCF1353
G	Ambulance bay	North side	Ceiling service duct	Cracking around edge	**Y**	Cosmetic repair to be specified by others	DSCF1354
G	G06	S-W corner of room	Ceiling-wall interface	Minor cracking around ceiling edge and at wall interace	**Y**	Cosmetic repair to be specified by others	DSCF1359
G	Typical	Typical	Wall	Cracking in gib sheets between panels and in corners of rooms between gib edges	**Y**	All cracked and damaged wall boards are to be removed and relined with gypsum board sheeting. All existing wall boards to remain are to be re-fixed to the timber stud walls	DSFC1361 P1080515
G	East room	Ceiling beam	Beam	Cracking to lining	**Y**	All cracked and damaged wall boards are to be removed and relined with gypsum board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs	DSCF1341
G	East room	Ceiling	Ceiling lining	Water damage to ceiling lining which has peeled away	**F**	Further investigation required to assess cause of damage	DSCF1347

Refer to Table 3.1 and HCG Specification for repair details

APPENDIX B

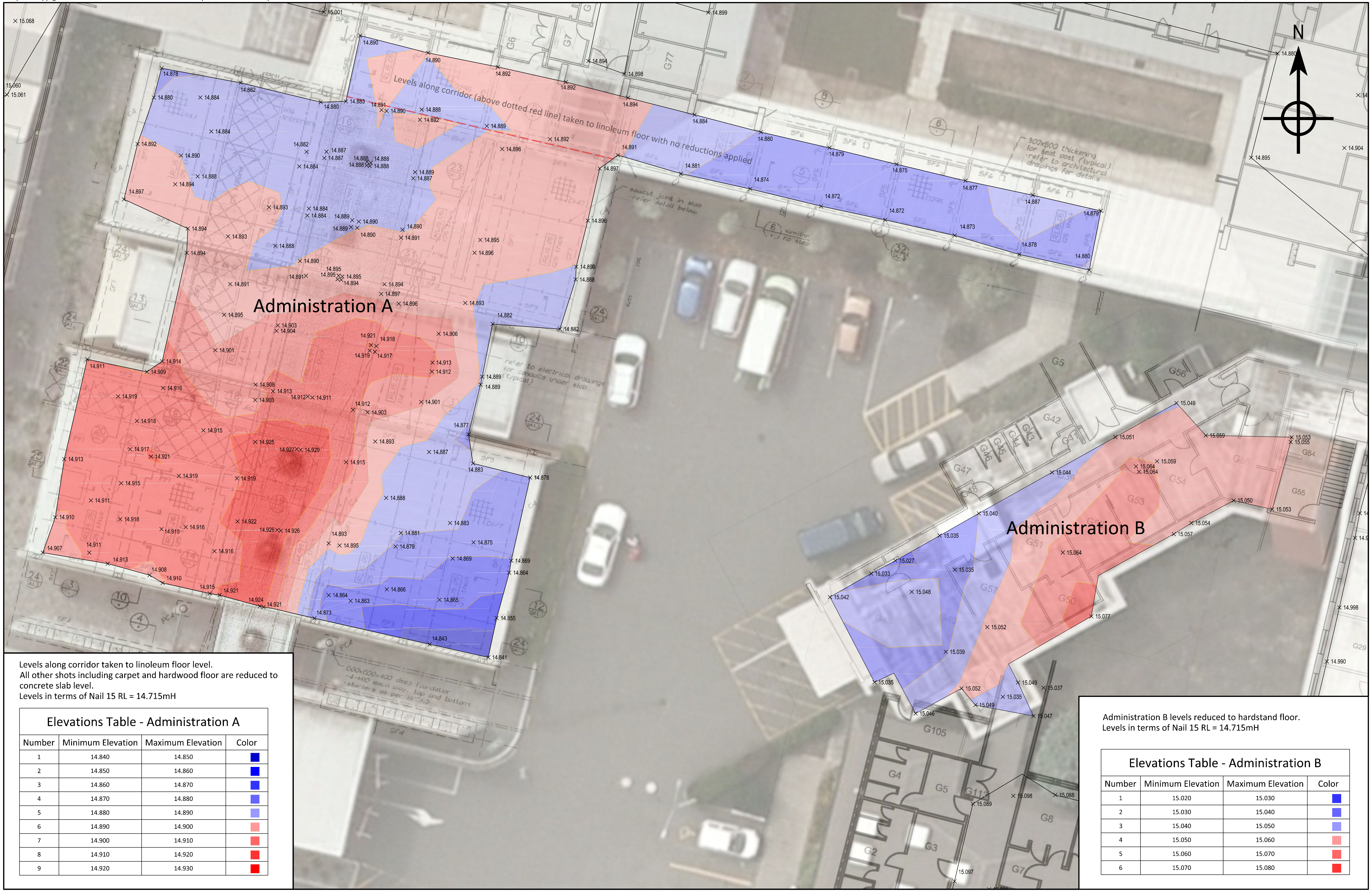
Photograph reference plan


6
3
4
7
5
1
2
N

APPENDIX C

Level survey location plan

K:\2462C CDHB Burwood\dwg\2462C 20111020 Elevations Survey.dwg : 18 Jun 2012 1:21 p.m. : 4 Administration



Levels along corridor taken to linoleum floor level.
All other shots including carpet and hardwood floor are reduced to concrete slab level.
Levels in terms of Nail 15 RL = 14.715mH

Elevations Table - Administration A

Number	Minimum Elevation	Maximum Elevation	Color
1	14.840	14.850	
2	14.850	14.860	
3	14.860	14.870	
4	14.870	14.880	
5	14.880	14.890	
6	14.890	14.900	
7	14.900	14.910	
8	14.910	14.920	
9	14.920	14.930	

Administration B levels reduced to hardstand floor.
Levels in terms of Nail 15 RL = 14.715mH

Elevations Table - Administration B

Number	Minimum Elevation	Maximum Elevation	Color
1	15.020	15.030	
2	15.030	15.040	
3	15.040	15.050	
4	15.050	15.060	
5	15.060	15.070	
6	15.070	15.080	

No.	Revision	Approved	Date
G	Med Records, Pain Mgmt, Admin B, and Surgical infill added	MJM	18/06/2012
F	Beacon House, BSU Hostel, Chapel, Tapper and Milner added	MJM	18/04/2012
E	Audit observations post Dec 23 2011 (not included in this Revision)	MJM	01/02/2012
D	Wards 7 & 8, Surgical Block, Melrose Chairs & Corridor added	MJM	02/12/2011
C	Phys Med, Parafed, Food Services, BIRS, Boilerhouse, EDC added	MJM	24/11/2011
B	Nurses Hostel & Champion Centre, SOU elevations added	MJM	18/11/2011
A	Administration and Allan Bean Centre elevations added	MJM	31/10/2011

Scale	1:100
Reduced A3	1:200
Designed	----
Drawn	DHP
Checked	MJM
Date	18/06/2012

Burwood Elevations Survey

Administration

SURVEYORS · URBAN DESIGNERS · CONSULTANTS · LAND DEVELOPMENT
Fox & Associates

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P.O.Box 13-390
CHRISTCHURCH

Job No. 2462C
Sheet No. 4 of 20
Rev. G

A1 Original Size

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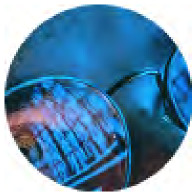
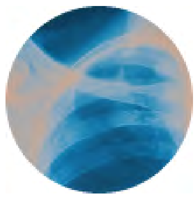


DETAILED SEISMIC ASSESSMENT REPORT

STRUCTURAL AND CIVIL ENGINEERS



BURWOOD HOSPITAL CAMPUS

REPORT 3 – BIRTHING UNIT & MINOR PROCEDURE UNIT

PREPARED FOR

CANTERBURY DISTRICT HEALTH BOARD

106186.59

INTERIM REPORT REV 2 – 24 MARCH 2014

BURWOOD HOSPITAL CAMPUS – DETAILED SEISMIC ASSESSMENT REPORT

REPORT 3 - BIRTHING UNIT AND MINOR PROCEDURE UNIT

Prepared For:
CANTERBURY DISTRICT HEALTH BOARD

Date: 24 March 2014
Project No: 106186.59
Revision No: 2

Prepared By:

Lachlan Beecher
PROJECT ENGINEER

Reviewed By:

Eric McDonnell
SENIOR PROJECT ENGINEER

Updated By:

Peter Grange
STRUCTURAL ENGINEER

Reviewed By:

Jenny Fisher
PROJECT DIRECTOR

Holmes Consulting Group LP
Christchurch Office

REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
30/04/12	1	Interim results of quantitative assessment (Phase 3) for discussion
24/03/14	2	Added Section 3.7, Further Investigations Required and IL3 values included

CONTENTS

APPENDICES

TABLES Page

FIGURES Page

The Birthing Unit and Minor Procedures Unit building (formerly known as wards 7 & 8) is a two storey timber framed structure that was designed and constructed in 1944. The building is comprised of three ward wings; a Main Wing running in the north-south direction, and two side Connecting Wings running in the east-west direction located at the southern end of the Main Wing. A small two storey Toilet Block Extension was added to the west side of the Main Wing shortly after the buildings initial construction date.

The roof assembly of the building consists of corrugated asbestos cement roof sheathing over diagonal timber sheathing. The first floor and ground floor diaphragms consist of straight tongue and groove board sheathing over timber floor joists. The gravity and lateral support of the building is provided by exterior timber stud walls clad in stucco and interior stud walls clad in gypsum wall board. The building is supported by exterior and interior sub-floor concrete walls and isolated internal concrete piers.

The information available for the review included: a 1976 campus wide survey of existing buildings [3], a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [4], along with a level survey of the building completed by Fox & Associates [5].

The Birthing Unit and Minor Procedures Unit building appears to have performed as would be expected for a building of this type and age. The bulk of structural earthquake related damage is typified by cracking of the concrete footings at areas of reduced section (vents) and cracking of the linings on the timber framed walls and ceilings.

Significant earthquake induced differential ground settlement has also been noted between the toilet block extension off the Main Wing, resulting in a drop of 40mm over a 1m length of the elevated ground floor (1:25 or 4% slope). The most severe cracking to the internal wall and ceiling linings corresponds to this area of greatest differential settlement.

It is believed that the significant damage observed occurred during the 22nd February event. Further observations of the earthquake damage observed have been included in the body of this report.

For the purposes of this assessment the CDHB Birthing Unit and Minor Procedures Unit building has been considered to be an Importance Level 2 building (IL2). The Importance Level 3 (IL3) values are provided in brackets and generally show a drop of approximately 33% DBE.

Based on our analysis, the primary lateral force resisting elements of the Main Wing have been assessed to have a pre-earthquake capacity to resist approximately 55% (40% IL3) of the demand required by the current loading code Design Basis Earthquake (DBE) in the east-west (transverse) direction and approximately 40% DBE (30% Il3) in the north-south (longitudinal) direction. The East and West Connecting Wings have been assessed to have a pre-earthquake capacity to resist approximately 55% DBE (40% IL3) in the east-west (longitudinal) direction and 40% DBE (30% IL3) in the north-south (transverse) direction. The Toilet Block Extension has been assessed at approximately 45% DBE (35% IL3) in the east-west direction and 100%

DBE (80% IL3) in the north-south direction. The capacity of the three wings and the Toilet Block Extension is governed by the ground floor timber bracing walls of the building.

The actual percentage reduction in lateral bracing capacity of the internal and external timber framed walls as a result of the damage observed is hard to quantify. Although there is some reduction in strength due to the damage noted, the primarily affect is to the ongoing stiffness of the building. The reduced stiffness will result in larger future displacements during seismic events and additional damage to interior linings and building contents.

There has also been some reduction in the capacity of the building as a result of the differential settlements noted. As a result of this, it is expected that there will be a reduction in the future differential settlement the building could absorb before severe distress to the structure occurs. In addition, while the resulting slopes in the ground floor outside of the Toilet Block are within the typical acceptable range, CDHB may wish to pursue re-levelling of the building due to the nature of the buildings occupancy, and ongoing serviceability concerns.

The minimum repairs required to reinstate the building to its pre-earthquake undamaged condition have been included in Section 4. This includes replacement or re-levelling of the Toilet Block extension and the repair and re-fixing of the wall and ceiling linings throughout.

In addition to the repairs, recommended strengthening concepts to increase the seismic performance of the Birthing Unit and Minor Procedures Unit, and bring the assessed capacity above 67% DBE, have been included in section 5. Proposed strengthening includes additional wall bracing to the exterior ground floor walls.

1. INTRODUCTION

Holmes Consulting Group has been engaged by Canterbury District Health Board (CDHB) to complete a full structural review of the Burwood Hospital Campus following the Lyttelton Earthquake. A series of reports have been compiled as part of this process. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The specific building reports, like this one on the Birthing Unit and Minor Procedures Unit, should be read in conjunction with the base report, and refer to the repair specification.

The Burwood Hospital campus base report covers the purpose and scope of the structural review. The current statutory requirements relevant to earthquake damaged buildings are outlined and the level of shaking experienced at the site estimated. Earthquake induced ground settlement damage across the campus is also discussed. The repair specification has been prepared to include repair details for typical damage observed in buildings on the Burwood Hospital Campus and is referred to as required in the specific building reports.

1.1 SCOPE OF WORK

This report is on the Birthing Unit and Minor Procedures Unit located at 245 Mairehau Road, Burwood, Christchurch. The report identifies the general form of the structure, along with the gravity and lateral load resisting systems. Each component of the structural system was reviewed based upon the information available and any potential Critical Structural Weaknesses (CSW's) were noted.

The report also identifies the structural damage observed to date as a result of the series of Earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake, in particular, subjected the building to strong ground motions which significantly exceeded the current code loading demand for buildings of this nature.

The capacity of the Birthing Unit and Minor Procedures Unit has been assessed relative to current code loading in the buildings pre-earthquake undamaged state and in its post-earthquake damaged state. The post-earthquake assessment summarizes the effects of the damage identified on both the gravity and lateral load resisting elements. Repair options to restore the buildings capacity to pre-earthquake levels for strength, durability and stiffness have been included. The repair options aim to maintain the buildings utility. Where required, strengthening options have also been provided, along with a discussion on the building's likely performance under Maximum Considered Earthquake (MCE).

1.2 LIMITATIONS

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. Secondary elements, such as windows and fittings, have not generally been reviewed.

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 and/or strengthening of the building have been completed.

2. PRE-EARTHQUAKE BUILDING CONDITION

This section discusses the form and capacity of the building prior to the Darfield Earthquake

2.1 BUILDING FORM

The Birthing Unit and Minor Procedures Unit (formerly known as wards 7 & 8) was designed in 1944 and constructed in the period thereafter.

The Birthing Unit and Minor Procedures Unit building originally comprised of two main ward wings running in the north-south direction which were connected via an east-west link at the southern end. However, the western side main wing (formally known as wards 5 & 6) was demolished between 2001 and 2005 to allow construction of a new Orthopaedic Quad.

Presently, the building consists of three two storey wings; a Main Wing running in the north-south direction and two side Connecting Wings running in the east-west direction off the southern end of the Main Wing. A small two storey Toilet Block Extension was added to the west side of the Main Wing shortly after the buildings initial construction date.

Figure 2-1: Birthing Unit and Minor Procedure Unit (formerly Wards 7 & 8)

The information available for the review included: a 1976 campus wide survey of existing buildings [3], a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [4], along with a level survey of the building completed by Fox & Associates [5]. As no original architectural or structural drawings were available, we have relied heavily on the 1976 survey and our site observation of the building.

All three wings and the Toilet Block Extension of the Birthing Unit and Minor Procedures Unit have corrugated asbestos cement sheet roofing supported by diagonal timber sheathing on battens over lightweight timber roof framing. The roof framing spans to load bearing timber stud walls at the perimeter of the buildings and along either side of the central corridors. The roof and ceiling framing are connected to interior and exterior walls via timber top plates.

The first floor and ground floor of the building are constructed of straight tongue and groove sheathing boards over timber floor joists.

At ground floor, the floor framing comprises of timber joists supported by bearers at approximately 1500 centres. The bearers are supported around the perimeter by concrete piers (that appear to be cast integral with the perimeter concrete subfloor walls), interior sub-floor walls and interior isolated concrete piers. The ground floor wall framing bottom plate is assumed to be connected directly to the concrete sub-floor walls with 3/8 inch (9.5mm) diameter bolts cast into concrete piers at regular centres (assumed to be 1800mm). The ground floor framing is bolted to the internal concrete piers with a single 9.5mm cast in bolt.

The exterior load-bearing walls are constructed of timber framing and are lined on the exterior with asbestos cement sheeting and a stucco finish. The interior walls are timber framed stud walls lined with a combination of gypsum wallboard and fibrous wall sheeting with a plaster finish. Timber stud walls are aligned above and below level 1 floor, and are spaced between 2.4 and 6m centres to divide the space.

A portion of the ground floor at the west side of the central corridor is constructed of suspended reinforced in situ concrete slabs supported by reinforced concrete sub-floor walls. The area below the suspended concrete slab serves as the service tunnel beneath the building (Refer to Figure 2-2).

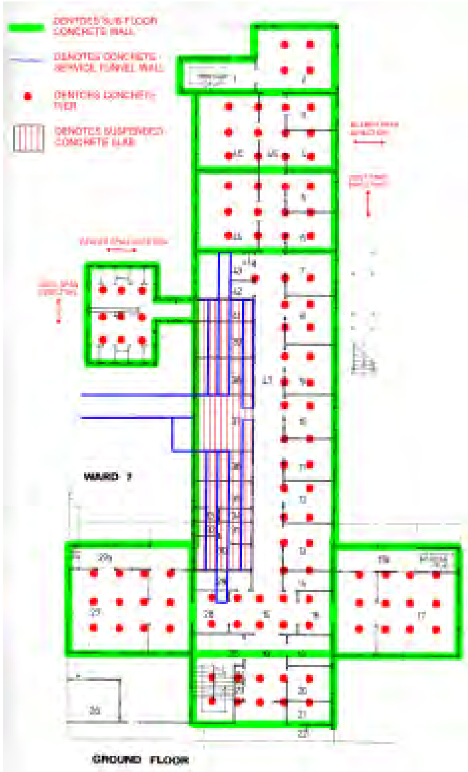


Figure 2-2: 1976 Survey – Foundation plan

Figure 2-3: 1976 Survey – Ground and First Floor Plan

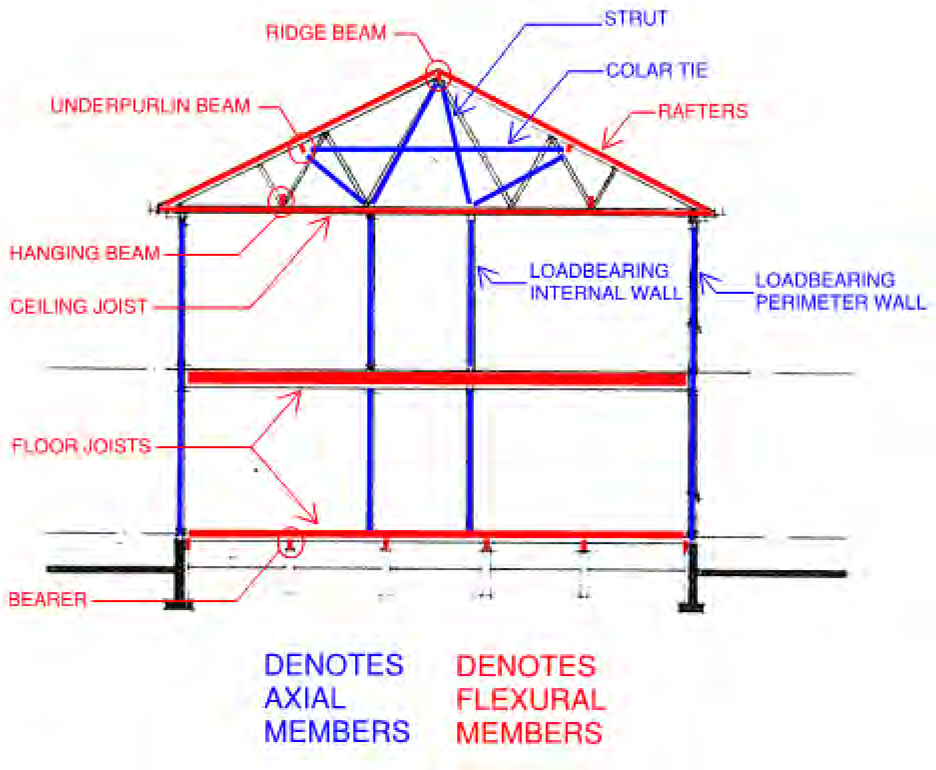


Figure 2-4: Typical Building Section

Figure 2-5: Roof Space

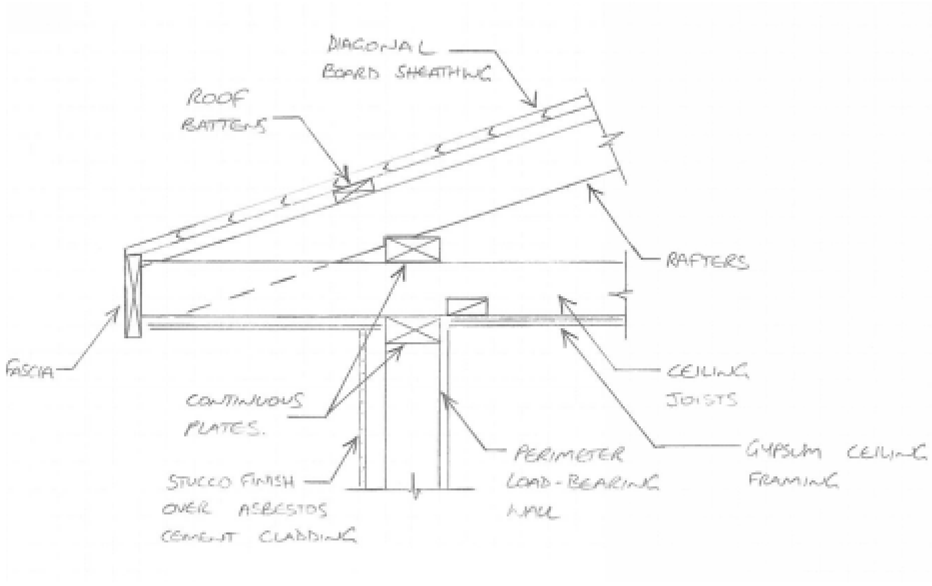


Figure 2-6: Typical Roof Support Detail

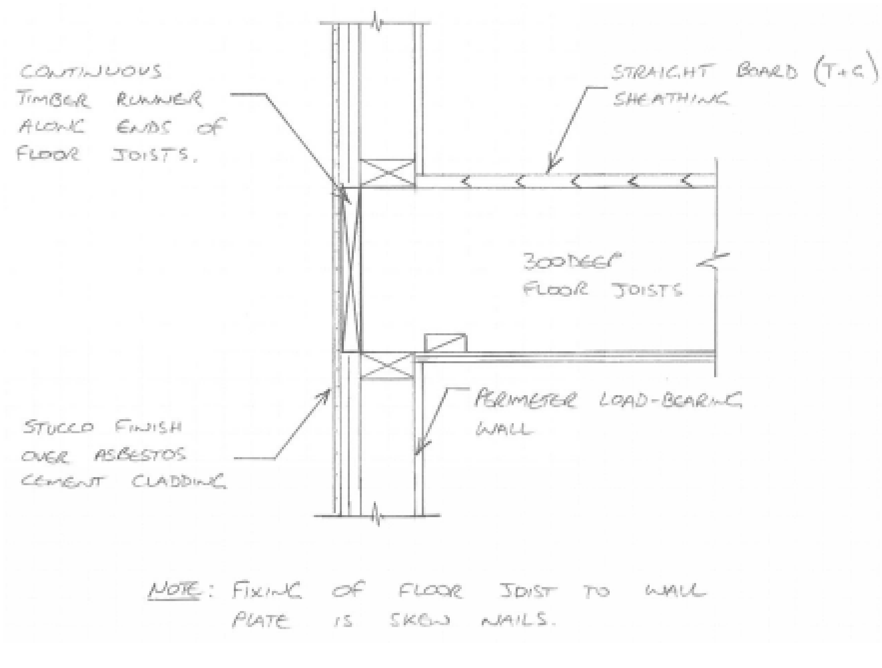


Figure 2-7: Typical First Floor Support Detail

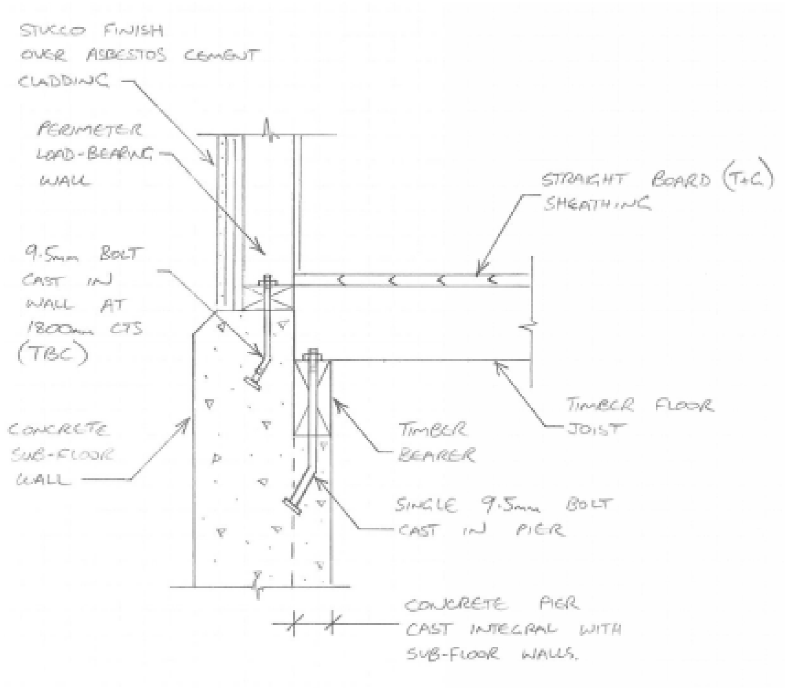


Figure 2-8: Typical Ground Floor Support Detail

Figure 2-9: Sub-floor Framing and Connection to Walls

Figure 2-10: Internal Pier (Vertical Bolt Through)

Figure 2-11: Water Tanks in Roof Space (Braced in Both Directions)

2.2 LATERAL LOAD RESISTING SYSTEMS

The primary lateral force resisting system of the Birthing Unit and Minor Procedures Unit consists of timber framed roof, floor and ceiling diaphragms, which transfer lateral loads to sheet clad timber bracing walls.

The roof diaphragm consists of diagonal board sheathing over timber roof framing. The first and ground floor diaphragms consist of straight tongue and groove board sheathing over timber floor joists.

The exterior bracing walls consist of a stucco finish over asbestos cement wall cladding. The interior walls consist of fibrous wall board sheathing and gypsum wall boards.

The sub-floor bracing walls consist of continuous concrete exterior concrete walls and intermittent interior concrete walls.

The roof diaphragm and supporting timber framing transfer seismic loads from the roof into the perimeter external walls and internal transverse walls. In the longitudinal direction there is no bracing present from the roof diaphragm to the internal longitudinal walls, so the gypsum board ceiling diaphragm has been assumed to distribute seismic roof loads to the interior longitudinal walls. It is also assumed that floor diaphragms are adequately fixed to all walls above and below to transfer the required seismic loads.

2.3 PRE-EARTHQUAKE BUILDING CAPACITY – DIRECT CODE COMPARISON

The building capacity under earthquake actions discussed in this section is compared to the capacity that a similar building would be designed to today. A new building of the form of the Birthing and Minor Procedures Unit would be designed to either the *Structural Design Actions Standard, Part 5: Earthquake Actions – New Zealand*, NZS 1170.5:2004[9] or the *New Zealand Standard Timber Framed Buildings*, NZS 3604:2011[12], incorporating the amendments made to these standards as a result of the Lyttelton Earthquake. These changes are outlined in the Amendment 10 of the Building Code [8]. The implications of the recent amendments are discussed more fully in the Burwood Hospital Campus Base Report. For a building of this type the amendments essentially result in an increase to the design loads of 36% when compared to pre-earthquake NZS 1170.5:2004[9] design levels and 67% when compared to pre-earthquake NZS 3604:2011[12] design levels.

When the building was originally designed in 1944, the loading standard at the time was likely the *New Zealand Standard Model Building By-Law* NZSS95:1939 [10] and/or the NZSS95:1944 *New Zealand Standard Model Building By-Law for Light Timber Framed Construction* [11]. When these By-Laws were written, neither the seismology of the different areas within New Zealand, or the impact this could have on buildings was as well understood as it is today. Along with an increase in the seismic demands required by the change in the loading code over this period, the seismic detailing requirements have also progressed significantly resulting in more ductile and better performing buildings. In particular, the bracing requirements of a similar building design and constructed to current code requirements would be several times larger.

The current code requires a new building to be designed for an earthquake, known as the Design Basis Earthquake (DBE), which is based upon the buildings physical location, local soil conditions, building type, fundamental period and importance level.

As the Birthing Unit and Minor Procedures Unit is timber framed and has been assigned a standard importance level (IL2) it has been assessed to *New Zealand Standard Timber Framed Buildings*, NZS 3604:2011[12]. The requirements of NZS 3604:2011[12] incorporate the DBE earthquake for the specific site conditions. The bracing output is roughly equivalent to an Importance Level 2 building (with an associated DBE return period of 500 years), a risk factor, R = 1.0, and a wall bracing ductility factor, μ=3.5. The sub soil for the site has been taken as Soil Type D, which is consistent with the findings of a post-earthquake geotechnical investigation [4].

2.4 EQUIVALENT STATIC ANALYSIS TO NZS3604 (2011)

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 estimated 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.

Following the Lyttelton earthquake, a geotechnical report was conducted by Tonkin & Taylor, titled "Burwood Hospital Post Earthquake Geotechnical Assessment", dated June 2011 [4]. This report has been used to aid in the evaluation of the site conditions and the likely effect of the ground on the buildings past and future performance. The soil parameters described in the geotechnical report complete by Tonkin and Taylor have been used for the evaluation of the buildings existing foundation system.

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. These are described in more detail in the Burwood Hospital Base report [1] and include strength governed elements such as short columns and deflection governed elements such as floor and stair elements with inadequate support seating.

The limiting factor for the capacity of the building is the ability of the ceiling framing to re-distribute lateral loads to the bracing walls in the east-west direction. Because there is no contiguous ceiling diaphragm, lateral loads are required to be distributed through a combination of the sections of gypsum board clad ceilings, and through direct bending of the wall top plates and ceiling framing.

For the purpose of this evaluation several assumptions also had to be made in regards to the existing building properties. Specifically, the existing diaphragm properties of the diagonal and straight timber board roof and floor sheathing, along with the bracing capacity of interior and exterior walls were of primary concern. The expected strength values for these elements were taken from NZSEE 2006, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes* [14] and ASCE 41-06, *Seismic Rehabilitation of Existing Buildings* [14]. The diaphragm expected strength values presented in NZSEE2006 have been divided by 1.5 to account for built in overstrength. This value is based upon the data from the NEHRP ABK Program for which the data in NZSEE2006 is based. These values could be further refined through destructive investigations of the existing materials. The assumed diaphragm and shear wall factored expected strength values are as follows:

- Exterior Walls: Unblocked timber framed walls with stucco exterior finish over asbestos cement cladding and interior fibrous wall board with plaster finish. Expected strength = 5.1 kN/m (102 BU/m) $\mu = 2.2$
- Interior Walls: Unblocked timber framed stud walls with gypsum wallboard or fibrous wall board sheathing and plaster finish on two sides. Expected strength = 3.0 kN/m (60 BU/m) $\mu = 3.3$

- Roof Diaphragm: 1 inch x 4 inch diagonal straight timber board sheathing. Expected strength = 8.8 kN/m (176 BU/m) μ = 1.3

- First Floor Diaphragms: 1 inch x 4 inch straight timber board sheathing. Expected strength = 2.8 kN/m (56 BU/M) μ = 3.5

- Ground Floor Diaphragms: 1 inch x 4 inch straight timber board sheathing. Expected strength = 2.8 kN/m (56 BU/M) μ = 3.5

- Reinforced Concrete Subfloor Walls: Expected strength = 11.6 kN/m (233 BU/m)

The bracing requirements in NZS 3604:2011[12] assume a ductility factor, μ = 3.5 for the bracing walls and diaphragms. To account for the less ductile existing walls outlined above, the wall bracing demands from NZS 3604:2011[12] have been factored up proportionally as required in our analysis. Values for the bracing supplied by the reinforced concrete sub-floors walls have been taken from NZS 3604:2011[12].

The foundations have been assessed at an ultimate bearing capacity of 150kPa, for factored loads, as per recommendations provided by Tonkin and Taylor.

A summary of the capacity of each primary lateral element as a percentage of the demand imposed by the Design Basis Earthquake (DBE) for each wing has been noted in the Tables 2-1 to 2-3, and are expressed as a %DBE.

Building Element	**%DBE (IL2)**	**%DBE (IL3)**	**Comments**
First Floor Walls – N-S E-W	100% 100%	85% 85%	
Ground Floor Walls - N-S E-W	40% 55%	30% 40%	
Roof Diaphragm – N-S E-W	100% 100%	90% 100%	
Ceiling Diaphragm – N-S E-W	100% 100%	90% 90%	
First Floor Diaphragm - N-S E-W	100% 70%	100% 55%	East-west direction governed by capacity of transfer diaphragm where walls below have been removed.
Ground Floor Diaphragm - N-S E-W	100% 100%	90% 90%	
Sub-Floor Walls - N-S E-W	100% 85%	100% 67%	Limited in east-west direction by shorter cross walls in service tunnel area (i.e. do not extend across full width)

Table 2-1: Main Wing – Seismic Assessment %DBE

Building Element	%DBE (IL2)	%DBE (IL3)	Comments
First Floor Walls – N-S E-W	75% 100%	60% 85%	
Ground Floor Walls - N-S E-W	40% 55%	30% 40%	
Roof Diaphragm – N-S E-W	100% 100%	90% 100%	
Ceiling Diaphragm – N-S E-W	100% 100%	90% 95%	
First Floor Diaphragm - N-S E-W	70% 100%	55% 90%	
Ground Floor Diaphragm – N-S: E-W	80% 90%	60% 70%	
Sub-Floor Walls - N-S E-W	100% 100%	100% 100%	

Table 2-2: East & West Connecting Wings - Seismic Assessment %DBE

Building Element	%DBE (IL2)	%DBE (IL3)	Comments
First Floor Walls – N-S E-W	100% 85%	100% 67%	E-W direction restricted by toilet block walls. Can achieve 100% if link walkway framing is securely tied in to main North-South Wing.
Ground Floor Walls - N-S E-W	100% 45%	80% 35%	E-W direction restricted by toilet block walls. Can achieve 100% if link walkway framing is securely tied in to main North-South Wing.
Ceiling Diaphragm – N-S E-W	100% 100%	90% 95%	
First Floor Diaphragm - N-S E-W	100% 100%	100% 100%	
Ground Floor Diaphragm – N-S: E-W	100% 100%	100% 100%	
Sub-Floor Walls - N-S E-W	100% 100%	100% 100%	

Table 2-3: Toilet Block Extension - Seismic Assessment %DBE

A review of the drawings available also noted no Critical Structural Weaknesses (CSW's). There are three locations on the ground floor where internal transverse walls have been removed from below walls above (refer figure 2-3), but in general the interior and exterior walls are well distributed and stacking. The ground floor framing is bolted to the concrete piers integral with the exterior sub-floor walls and additional fixings are assumed between the ground floor framing and the top of the concrete sub-floor walls. The ground floor framing is bolted to the internal concrete piers with a single 9.5mm cast in bolt.

Methodology to improve the seismic performance of the buildings and provide strengthening to achieve 67% DBE (IL2) have been included in Section 5.

3. POST EARTHQUAKE BUILDING CONDITION

This section covers the structural damage sustained by the Birthing Unit and Minor Procedures Unit, and its effect on the buildings capacity to resist seismic loads, as a result of the series of earthquakes which includes the Darfield Earthquake that struck at 4:36am on 4th September, 2010, the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011, the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake subjected the building to strong ground motions which significantly exceed the full design earthquake load for buildings of this nature and appears to have caused the majority of the earthquake damage observed, after the Darfield event.

3.1 THE LYTTELTON EARTHQUAKE

The Fundamental Period of the buildings is estimated to be between 0.2 and 0.4 seconds. 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 strong motion data presented in the Base Report, it appears the Lyttelton Earthquake produced ground shaking intensities of approximately 60-120% of the current Ultimate Limit State design spectra for an Importance Level 2 building of nominal ductility.

It should be noted that the Lyttelton Earthquake was very short in terms of strong shaking produced with the strong motion only lasting for duration of 5-7 seconds. Rupture of an alpine fault is expected to contain 50 to 60 seconds of strong motion.

3.2 PRELIMINARY INVESTIGATIONS

Preliminary investigations were carried out to identify 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
- Review of available survey documentation
- Damage observed during 'Rapid' Structural Assessments following the Lyttelton Earthquake [18], June 13th, 2011 aftershocks [19, 20], December 23rd, 2011 [21] and January 2nd, Earthquakes [22].
- Review of previous Holmes Consulting Group assessments on the building [6, 7]

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

- Connections of timber framing to foundation supports
- Damage to roof framing at connections to timber wall framing
- Cracking to linings of timber framed walls and ceilings
- Distress to timber framed floor diaphragms
- Cracking in continuous concrete footings due to liquefaction induced differential settlement
- Displacement of ground around perimeter of building

Rapid Level 2 Assessments were carried out on the 24th February 2011[18] and on the 14th June 2011 [19, 20]. An additional Visual Structural Assessment was completed on the 5th January, 2012 following the 23rd December 2011 and 2nd Jan 2012 events [22]. These structural observations involved a complete walk around the exterior and throughout the interior of the building. The following primary areas of damage were identified from the damage assessments:

- Cracking in external footings, primarily in areas of reduced section
- Differential ground settlement concentrated between the Main Wing and Toilet Block Extension and in the East and West Wing Connecting Wing
- Cracking to internal and external wall linings and ceiling finishes.

A review of the above information on the building type and preliminary observation highlighted this building as requiring a detailed inspection. The aim of the detailed inspection was to determine the cause and full extent of damage to the building, particularly the elements identified for potential damage above. These items were targeted to identify if damage had occurred and to what extent the damage had reduced the capacity of the buildings lateral load resisting system to withstand future seismic events.

3.3 DETAILED OBSERVATIONS

Further detailed inspections and structural explorations have been carried out following the initial assessments to ascertain the full extent of structural damage. The detailed structural observations were completed between 29 February and 8 March, 2012. A full record of these observations can be found in Appendix A, with reference plans describing the location labelling used found in Appendix B. A full photographic record of the observations is available electronically on request. The detailed structural observations identified the following additional damage to those items identified in the initial rapid assessments:

- Additional occurrences of cracking of exterior and internal linings

3.4 GEOTECHNICAL REVIEW

A review of the ground damage and conditions was carried out by Tonkin & Taylor for the Burwood Hospital Campus. A subsequent report titled *Burwood Hospital Post Earthquake Geotechnical Assessment* was issued in June 2011 [4]. The geotechnical review concluded that the settlement and damage to building foundations and slabs on the Burwood Hospital Campus was likely due to the liquefaction of underlying soil layers. It is believed that excessive pore water pressures have been dissipated and that further settlement is not expected to occur, unless another significant event was to occur.

It is estimated that the building foundations have settled a total of 110mm – 200mm overall with a differential settlement of approximately 50mm noted across the length of the East and West Connecting Wings. The differential settlement is particularly noticeable at the link between the Main Wing and the Toilet Block Extension, were a there is a drop in the ground floor of approximately 40mm over a 1m length (1:25 or 4% slope).

Based up the geotechnical report provided by Tonkin & Taylor [4] the potential for future total and differential settlements at the building site varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

3.5 LEVEL SURVEY

A detailed survey of the ground floor levels in the Birthing and Minor Procedures Unit was conducted by Fox & Associates and issued on 2nd December, 2011 [5]. The survey indicates a differential settlement of approximately 46mm over a 25m length across the length of the East and West Connecting Wing along with differential settlements at the north end of the main wing. The maximum overall drop in the ground floor framing of 57mm has been recorded, although no clear pattern of settlement is present. Typical worst case slopes in the ground floor framing are on the order of 1:250 to 1:300 (0.3-0.4%). The worst case differential settlement is at the junction between the Main Wing and the Toilet Block Extension, resulting in a permanent slope in the ground floor of approximately 40mm over a 1m length (1:25 or 4% slope).

The resultant slope between the Main Wing and the Toilet Block Extension is way beyond the typical acceptable range will require repair. Options include demolishing and reconstruction the Toilet block or localised mechanical jacking of the toilet block structure.

The slopes noted across the rest of the Birthing and Minor Procedures Unit are within the typical acceptable range, however given the nature of the buildings occupancy CDHB may wish to pursue re-levelling of the building, as the slopes noted will affect the ongoing serviceability of the building. Further discussion on re-levelling is included in Section 4.1.

A discussion on re-levelling on a campus wide basis is also included in the Burwood Hospital campus base report. This includes a study on the effect of re-levelling individual buildings on the serviceability of the hospital campus as a whole.

For the extent of the differential settlement noted to the building see the level survey included in Appendix C.

3.6 SUMMARY OF BUILDING DAMAGE

The following is a summary of the observations made for the building, and our conclusions as to their condition and seismic load resisting capacity. These observations do not specifically distinguish between damage caused by the Darfield Earthquake, the Lyttelton Earthquake, or any significant aftershocks thereafter, such as those that occurred on 13th June 2011, 23rd December 2011 and 2nd January 2012. Despite not being able to specifically distinguish when individual damage observed occurred, it is believed that the majority of the damaged can be linked to the February 22nd event.

The Birthing Unit and Minor Procedures Unit building appears to have performed relatively well considering the age and type of the building and the seismic actions experienced at the site. The bulk of structural damage is typified by cracking of the concrete footings at areas of reduced section (vents) and cracking of the linings on the timber framed walls and ceilings. The structural damage sustained by the building as a whole would be categorized as minor to moderate due to the reduction in lateral capacity of the building caused by the cracking of the linings (and loosening of fixings) to the timber framed walls and ceilings, which provide the primary gravity and lateral support to the building. The link between the main wing and the toilet block has also been compromised due to the significant differential settlements noted in this area. A summary of the typical structural damage observed is as follows:

- **Differential Ground Settlement** – As previously noted differential settlement of up to 57mm have been observed. The worst case differential settlement, and associated distress to the superstructure, has been noted at the junction between the Main Wing and the Toilet Block.

- **Distress to Timber Wall and Ceiling Framing –** Distress to the timber wall and ceiling framing and connections has occurred at the link between the Main Wing and the Toilet Block. The damage is a result of the large differential settlements which have occurred in this area.

- **Cracking of Wall Finishes** – Cracking, and general distress has been noted to internal and external wall linings, primarily at corners, openings and along wall board joints. This includes damage to the interior gypsum wall boards and the external stucco cladding. Based upon the movements observed it is believe the wall board fixings have been damaged as well.

- **Cracking of Ceiling Finishes** – Cracking, and general distress has been noted to ceiling linings, primarily at corners, openings and along wall board joints. Based upon the movements observed it is believe the wall board fixings have been damaged as well.

- **Damage to Non-structural Elements** - Cracking to non-structural elements such as window reveals, door jambs and ceiling finishes

In Section 4, Table 4-1 provides a photographic summary of the typical damage observed. A full record of our detailed observations and repairs required can be found in Appendix A.

3.7 FUTURE INVESTIGATIONS REQUIRED

3.7.1 Investigations Required for Damage Assessment

Several assumptions were made in the completion of the pre-earthquake (undamaged state) and post-damaged (damaged state) structural assessments. Destructive exploration is still required in a few locations in order to verify these assumptions. The areas requiring further investigation to finalise the assessments are as follows:

- Determine the existing fixings between the exterior timber stud wall and the concrete subfloor wall below. In particular, determine the fixing type, size and spacing of the timber bottom plate to the top of the concrete subfloor wall below.

The ground floor framing is bolted to the concrete piers integral with the exterior sub-floor walls with a single 9.5mm cast in bolt. The fixings between the exterior wall and the foundation wall were not able to be viewed.

- Investigate water tank fixings in the roof space to the timber framing below.

The water tanks have been directly connected into the purlins with proprietary steel brackets such that they have some restraint for lateral loads in both directions.

3.7.2 Investigations Required During Repairs

The following investigations will be required during repairs:

- The hold-down fixings of the timber framing in the bracing walls should be checked for damage and ability to transfer new bracing loads if recommended wall strengthening proceeds.

- Re-inspection of building will be required upon completion of any re-levelling works to determine if any additional damage has occurred.

3.8 POST-EARTHQUAKE BUILDING CAPACITY

Based upon our observations to date, we do not consider the Birthing Unit and Minor Procedures Unit building to have any significant reduction in gravity load resistance. The damage observed to the interior and exterior wall sheeting will have resulted in some reduction in lateral load capacity, although it is difficult to quantify the percentage reduction in strength. While there has been some reduction in strength, according to the Department of Building and Housings, *Revised Guidance on Repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence*, the primarily result of the damage noted will be a reduction in the stiffness of the wall bracing. The reduction in stiffness will cause ongoing concerns in regards to the buildings performance, primarily to building contents and non-structural elements. There will also be some addition reduction in capacity due to the differential ground settlement observed.

The damage observed will require repair to restore the strength, stiffness, durability and performance of lateral bracing system. The repair work required to reinstate the building to pre-earthquake levels is outlined in Section 4. Following the recommended repairs to the structural damage noted the lateral load capacity of the existing structure will be restored to close to the earthquake levels, which are summarised in Section 2.4.

Recommendations for strengthening to improve seismic performance and bring the building to above 67% DBE are included in Section 5.

4. OBSERVED DAMAGE AND REQUIRED REPAIRS

This section covers the damaged noted during our detailed assessment of the building. Note that our observations have been restricted to structural aspects of the building 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.

Table 4-1 provides a photographic summary of the observed damage and typical repairs required for the Birthing Unit & Minor Procedures Unit. Table 4-1 should be read in conjunction with Appendix A – Record of Observation and Appendix B – Reference Plans. The Repair Specification [2] referred to in Table 4-1 has been issued separately.

In general, the aim of the repair work indicated in this section is to restore the structure to its pre-earthquake state, as far as practicable, while maintaining the utility of the building. The repairs presented attempt to address the loss of strength, stiffness and durability of the structural elements due to the damage noted.

It should be noted that more damage may be identified during the repair works and (if required) additional repair details will be specified accordingly.

Please note that all repair works are to be completed after any re-levelling work to the building has been completed to a satisfactory condition, as further damage to the wall and ceiling linings can be expected during the re-levelling process.

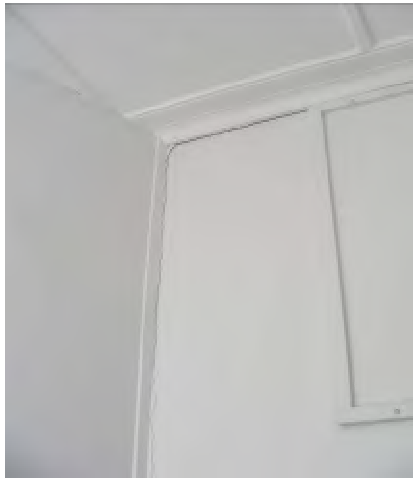
Further recommendations for improvement to the buildings seismic performance, and to achieve a minimum capacity of 67% DBE have been included in Section 4

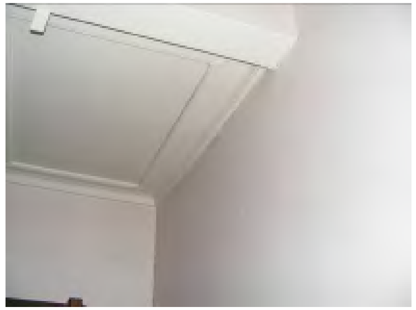
Table 4-1: Photographic Summary of Primary Damage Observed and Repairs Required

Damaged Item	Photo Ref: Location	Recommended Repair	Example
1. Foundations and Service Tunnel			
1.1. Localised areas of differential settlement have resulted in slope of ground floor timber framing of up 0.4% (1:250) across the East and West Connecting Wings and 4% (1:25) at the entry link to the Toilet Block Extension	Refer: Appendix C - Survey of Levels	Remediation of the floor levels will be required at the link to the Toilet Block Extension. If CDHB deems the remainder of the floor slopes across the Main, East and West Connecting Wing as represented in Appendix C – Survey of Levels, to be unacceptable, additional remediation of floor levels could be carried out through localised lifting of the structure. See section 4.1 for additional discussion on building re-levelling.	
1.2. Vertical and horizontal cracks to sub-floor external walls	Refer: Appendix A – Record of Observations	Epoxy inject cracks that are less than 1mm, in accordance with HCG specification. For cracks identified that are greater than 1mm, advise the engineer for inspection to confirm the integrity of the steel reinforcement. If damage has occurred to the steel reinforcing, an engineered repair will be required. Refer to HCG specification.	

Damaged Item	Photo Ref: Location	Recommended Repair	Example
1.3. Cracking and spalling to entrance ramp	Refer: Appendix A – Record of Observations	Remove balustrade and break out damaged concrete hob to inspect reinforcement. Re-concrete hob and reinstate balustrade *(may require additional dowel bars into existing wall pending inspection after breakout).*	
1.4. Cracking and spalling to sub-floor piers cast integral with sub-floor support walls	Refer: Appendix A – Record of Observations	Side fix bearer to sub-floor concrete walls with minimum two post installed bolts to re-instate vertical and lateral load-path from ground floor framing diaphragm into sub-floor wall.	
2. External Walls			
2.1. Cracking to external wall cladding between upper and lower level windows	Refer: Appendix A – Record of Observations	*Repair specification of external finishes to be completed by others.*	

Damaged Item	Photo Ref: Location	Recommended Repair	Example
2.2. Cracking to external wall cladding below lower level windows	Refer: Appendix A – Record of Observations	*Repair specification of external finishes to be completed by others.*	
2.3. Separation of external wall cladding and eave soffit in corners as a result of differential movement between toilet extension block and Main Wing	Refer: Appendix A – Record of Observations	Once re-levelling of the toilet block has occurred, all timber framing will need to be re-fixed and secured, with exterior finishes subsequently repaired. *Repair specification of external finishes to be completed by others.*	
3. Internal Walls			
3.1. Vertical and horizontal cracking to wall claddings	Refer: Appendix A – Record of Observations	Repair all cracked or damaged wall boards with new gypsum board sheets. All wall boards to remain are to be re-screwed as per sub-section 4.2	

Damaged Item	Photo Ref: Location	Recommended Repair	Example
3.2. Cracking to wall linings in corners	Refer: Appendix A – Record of Observations	Repair all cracked or damaged wall boards with new gypsum board sheets. All wall boards to remain are to be re-screwed as per sub-section 4.2	
3.3. Cracking of wall linings above openings	Refer: Appendix A – Record of Observations	Repair all cracked or damaged wall boards with new gypsum board sheets. All wall boards to remain are to be re-screwed as per sub-section 4.2	

Damaged Item	Photo Ref: Location	Recommended Repair	Example
3.4. Separation of cornice from wall linings	Refer: Appendix A – Record of Observations	Aesthetic repair to cornices. Repair specification by others	
4. Roof Framing and Ceilings			
4.1 Cracking in ceilings	Refer: Appendix A – Record of Observations	Repair all cracked or damaged ceiling lining with new gypsum board sheets. All ceiling sheets to remain are to be re-screwed as per sub-section 4.2	

4.1 DISCUSSION ON BUILDING RE-LEVELLING

The level survey, completed by Fox & Associates has indicated a total differential ground settlement across the East and West Connecting Wings of approximately 50mm. While differential settlement has been noted throughout (see Appendix C for complete level survey) the worst differential settlement noted has occurred at the link between the Toilet Block Extension and the Main Wing. The worst case permanent slope in the elevated ground floor framing in this region (based upon the level survey) is approximately 1:25 (or 4% slope).

Remediation of the floor levels is required to the Toilet Block Extension. Re-levelling could be achieved through demolition and reconstruction of the Toilet Block or potentially through mechanical jacking. If mechanical jacking is pursued it would involve jacking disconnecting the superstructure from the sub-structure, jacking the ground floor up to a level position, and then reconnecting the floor to the concrete sub-floor walls, internal pier footings.

The slopes noted across the rest of the Birthing and Minor Procedures Unit are within the typical acceptable range, however given the nature of the buildings occupancy CDHB may wish to pursue re-levelling of the building, as the slopes noted will affect the ongoing serviceability of the building. If CDHB chooses to pursue re-levelling of the Main, East and West Wings, they could be re-levelled through the use of mechanical jacking, using the same process as outlined for the Toilet Block Extension.

A discussion on re-levelling on a campus wide basis is also included in the Burwood Hospital campus base report. This includes a study on the effect of re-levelling individual buildings on the serviceability of the hospital campus as a whole. In particular this building is located directly adjacent to the Surgical Block and thus any re-levelling of these two buildings would need to be coordinated to ensure the final floor elevations align.

During the re-levelling process there is a risk that addition damage could occur to the buildings linings, exterior block veneer, etc. and appropriate contingencies should be provided.

For the extent of the work proposed see Figure 4-1 below.

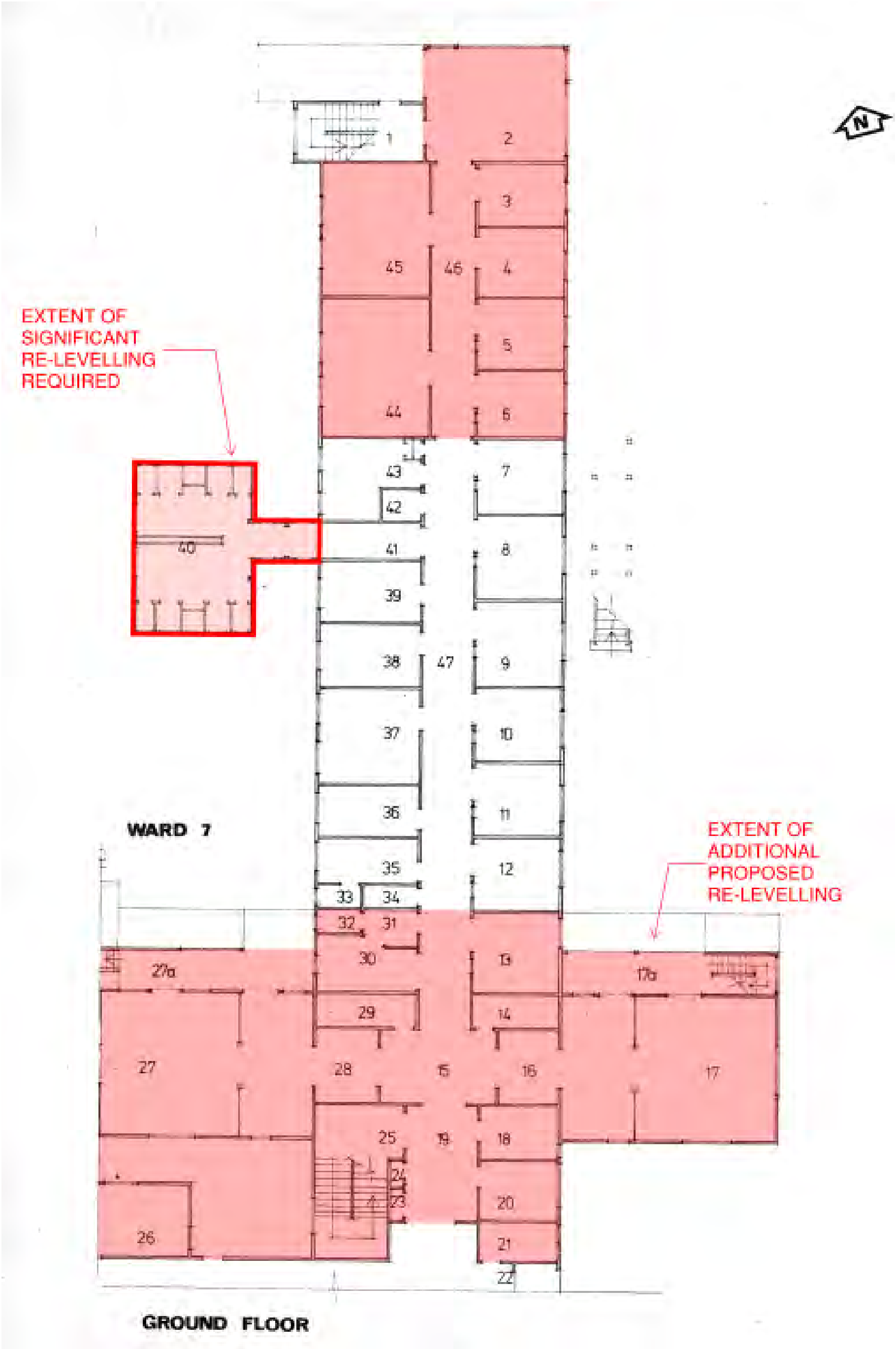


Figure 4-1: Foundation Plan – Damage Repairs

It should be noted that re-levelling the building through the use of mechanical jacking will not reduce the potential for future differential settlements. The ground conditions under the building will remain roughly as they were prior to the earthquakes. Based up the geotechnical report provided by Tonkin & Taylor [4] the potential for future total and differential settlements at the building site would remain between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

To improve the future performance of the building, and reduce the potential for future differential settlements, would likely require the entire footprint of the building to be either piled or the ground under all the sub-floor wall footings, service tunnels and the partial basement improved. *Further geotechnical investigations would be required into the type and depth of ground improvement required.*

4.2 REPAIR OF WALL BRACING

The wall linings to the interior and exterior bracing walls have been damaged in locations and require repair. Based upon the movement observed it is also believed the wall lining fixings have been damaged throughout. This will have resulted in a reduction to the ongoing strength and stiffness of the building. In order to reinstate the pre-earthquake strength and stiffness to the bracing walls, the repair recommendation is to remove all cracked or damaged sections of the wall linings and replace them with new gypsum board sheathing. The new gypsum board sheathing is to be fixed in accordance with GIB 'ezybrace' GS2-N specifications (or equivalent). All existing internal wall linings to remain are to be re-fixed to the existing studs in a similar manner. Upon completion of the repairs a new finish is to be applied to all interior walls.

All repairs to wall bracing are to be completed after any re-levelling to the building has been completed.

Note: The fixings of the walls to the sub-floor concrete walls below will need to be checked for damage and the ability to transfer the new bracing loads.

4.3 REPAIR OF CEILING DIAPHRAGMS

Similarly to the wall linings, the ceiling diaphragm and its fixings have been damaged and require repair. This is particularly important at the roof level where the ceiling linings distribute lateral loads to the corridor bracing walls. The repair recommendation is to remove any cracked or damaged sections of ceiling lining and replace them with new gypsum board sheathing, fixed in accordance with GIB ceiling diaphragm specifications. All existing ceiling linings that are undamaged are to be re-fixed to existing ceiling joists in a similar manner. Upon completion of the repairs a new finish is to be applied to all ceiling linings.

All repairs to wall bracing are to be completed after any re-levelling to the building has been completed.

Figure 4-2: Ground and First Floor Plan – Damage Repairs

5. STRENGTHENING RECOMMENDED

The primary lateral force resisting system of the Birthing Unit and Minor Procedures Unit consists of timber framed roof, floor and ceiling diaphragms, which transfer lateral loads to sheet clad timber bracing walls.

As noted in Section 2, Pre-Earthquake Building Condition, and Section 3, Post-Earthquake Building Condition, the lateral load resisting capacity of the Main Wing, East-West Connecting Wings and the Toilet Block Extension (as a percentage of the loads imposed by the Design Basis Earthquake) have been assessed at approximately 40% DBE (30% IL3), 40% DBE (30% IL3) and 45% DBE (35% DBE) respectively.

Provided the repairs in Section 4 are implemented the existing wall bracing capacity of the Main Wing and the East-West Connecting Wings will be above 67% DBE in the east-west direction, and at approximately 50% DBE in the north-south direction.

Strengthening recommendations to improve the seismic performance and bring the capacity of the entire building above 67% DBE have been included in sub-sections below.

5.1 STRENGTHENING WORKS TO ACHIEVE 67% DBE (IL2)

Additional Ground Floor Wall Bracing – Additional exterior wall bracing is recommended at the ground floor level in the north-south direction of the Main Wing and East-West Connecting Wings as well as in the east-west direction of the Toilet Block Extension in order to bring the assessed capacity of the building above 67% DBE.

The additional bracing could consist of new plywood sheathing or GIB 'Braceline' (or equivalent) applied to the inside face of the exterior wall. The strengthening would remain independent of the required repairs to the exterior stucco finish. Alternatively if the exterior finishes were to be removed a new layer of plywood could be installed to the exterior face of the walls.

In conjunction with the new plywood sheathing (or GIB 'Braceline') new holdowns to the concrete sub-floor walls below would likely be required at either side of each window or door opening.

The extent of the additional bracing proposed has been included in Figure 4-2.

Figure 5-1: Ground Floor and First Floor Plan – Recommended Strengthening

To increase the capacity in the east-west direction of the Toilet Block Extension, we propose tying the structure into the main North/South Wing rather than increasing the bracing capacity of the existing walls.

Anchor Bolts to Sub-floor Walls – In some cases it appears as though the anchor bolts connecting the ground floor framing to the concrete sub-floor walls below have corroded. Additional anchors between the ground floor framing and the concrete sub-floor walls at these areas are required to ensure vertical and lateral loads are transferred to the sub-floor concrete walls. It is likely that the installation of additional anchor bolts will also be required at the strengthened wall locations noted in Figure 5-1.

If the strengthening measures are to be implemented further investigation would be required to validate assumptions made. This includes the verification of existing material properties and all connections between bracing members (roof, walls and diaphragms). Additional fixings and isolated holdown anchors may be required.

6. REFERENCES

1. *Burwood Hospital – Detailed Seismic Assessment Report – Base Report*, Holmes Consulting Group, November 2011.

2. *Burwood Hospital – Detailed Seismic Assessment Report – Earthquake Repair Specification*, Holmes Consulting Group, July 2011.

3. *Burwood Hospital Christchurch – Survey of Existing Buildings – Wards 7 and 8*, Cutter Pickmere Douglas – Architects, 1976

4. *Burwood Hospital Post Earthquake Geotechnical Assessment*, Tonkin and Taylor Ltd., June 2011.

5 *Burwood Elevation Survey – Revision E*, Fox & Associates, January 2012

6 *Burwood Hospital Campus – Seismic Risk Assessment Report*, Holmes Consulting Group, April 2002

7 *Burwood Hospital Campus – 2007 Seismic Risk Assessment Update,* Holmes Consulting Group, June 2007

8 *Compliance Document for New Zealand Building Code - Clause B1 – Structure, Amendment 10 (Canterbury)*, Department of Building and Housing, Wellington, 19 May 2011.

9 *Structural Design Actions Part 5: Earthquake Actions – New Zealand, NZS 1170.5:2004*, Standards New Zealand, 2004.

10 *NZS 95 – 1939, New Zealand Standard Code of Building By-Laws.*

11 *NZS 95 – 1944, New Zealand Standard Code of Building By-Laws – Part IX Light Timber Construction.*

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

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

14 *Seismic Rehabilitation of Existing Buildings,* ASCE 41-06, American Society of Civil Engineers, 2007

15 *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury – Part 2 Evaluation Procedure*, Engineering Advisory Group, July 2011

16 *Revised Guidance on Repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence*, Department of Building and Housing, November 2011

17 *Practice Note – Design of Conventional Structural Systems Following the Canterbury Earthquakes*, SESOC, December 2011

18 *CDHB – Burwood Hospital Campus – Rapid Visual Inspection: 106186.03 Site Report 1,* Holmes Consulting Group, February 2011

19 *CDHB – Burwood Hospital – Rapid Seismic Assessment Post June 13th Earthquakes: 106186.03 Site Report 4,* Holmes Consulting Group, 14 June 2011

20 *CDHB – Burwood Hospital – Rapid Seismic Assessment Post June 13th Earthquakes: 106186.03 Site Report 5,* Holmes Consulting Group, 15 June 2011

21 *CDHB – Burwood Hospital – Post Earthquake Rapid Structural Assessment following 5.8 and 6.0 Magnitude Earthquakes, 106186.03 Site Report* 8, Holmes Consulting Group, 24 December 2011

22 *CDHB – Burwood Hospital – Post Earthquake Rapid Structural Assessment following 2nd January 5.5 Magnitude Earthquakes, 106186.03 Site Report 9*, Holmes Consulting Group, 9 January 2012

Appendix A

Record of Observations

APPENDIX A – RECORD OF OBSERVATIONS & REPAIRS

Inspection date: 29 February

KEY	
N	No repair required
Y	Repair required
F	Further investigation required
C	Repair complete

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	External	Building Exterior	General view of the North end of the building	**N**		001
GND	External	Building Exterior	General view of the North-East end of the building	**N**		002
GND	External	Building Exterior	General view of the North-West end of the building	**N**		003
GND	External	Sub-floor Concrete Wall	Horizontal tapered cracking at corner of wall located below floor level	**Y**	For all cracks greater than 0.2mm and less than 1mm, epoxy inject cracks in accordance with HCG specification. Cracks > 1mm require further investigation to confirm the integrity of the steel reinforcement. Refer to HCG specification.	004
GND	External	External Concrete Footpath Pavement	Continuous horizontal cracking across pavement	**Y**	Repair Specification by others	005

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	External	External Concrete Footpath Pavement	Concrete spalling and damage to concrete hob at balustrade fixing location	Y	For all cracks greater than 0.2mm and less than 1mm, epoxy inject cracks in accordance with HCG specification. Cracks > 1mm require further investigation to confirm the integrity of the steel reinforcement. Refer to HCG specification.	006
GND	External	External Wall	Vertical crack in external wall lining between upper level and lower windows	Y	Aesthetic repair to external wall cladding. Repair specification by others.	007
GND	External	External Wall	Numerous vertical cracks in external wall lining between upper level and lower windows	Y	Aesthetic repair to external wall cladding. Repair specification by others.	008
GND	External	External Wall	Tapered vertical crack in external wall lining off lower level window	Y	Aesthetic repair to external wall cladding. Repair specification by others.	009
GND	External	External Wall	Vertical crack in external wall lining between upper and lower level windows	Y	Aesthetic repair to external wall cladding. Repair specification by others.	010
GND	External	External Wall	Patched vertical crack in external wall lining between upper and lower level windows	Y	Aesthetic repair to external wall cladding. Repair specification by others.	011
GND	External	Sub-floor Concrete Wall	Vertical crack in sub-floor concrete wall at corner of stair well	Y	For all cracks greater than 0.2mm and less than 1mm, epoxy inject cracks in accordance with HCG specification. Cracks > 1mm require further investigation to confirm the integrity of the steel reinforcement. Refer to HCG specification.	012
GND	External	External Wall and Eave Soffit	Vertical and horizontal separation of toilet block from main North-South wing at high level	Y	Aesthetic repair to external wall cladding after re-levelling of toilet block. Repair specification by others.	013-014

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	External	Sub-floor Concrete Wall	Spalled concrete adjacent concrete service tunnel	Y	For all cracks greater than 0.2mm and less than 1mm, epoxy inject cracks in accordance with HCG specification. Cracks > 1mm require further investigation to confirm the integrity of the steel reinforcement. Refer to HCG specification.	015
GND	External	External Wall	Cracked external wall lining	Y	Aesthetic repair to external wall cladding. Repair specification by others.	016
GND	External	External Wall	General view of external wall lining material (assumed part of demolition works to construct new courtyard area)	N		017-018
GND	External	External Wall	General view of concrete service tunnel access hatch	N		019
GND	External	External Wall	General view of South-East corner of building	N		020
GND	1	Internal Wall	Horizontal and vertical cracks in wall lining inside stair access well	Y	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	021-022
GND	1	Internal Wall	Tapered vertical cracking off top of door into stair well	Y	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	023
GND	1	Ceiling lining	Separation of cornice from wall lining and cracking in ceiling above stairs	Y	All cracked and damaged ceiling linings are to be removed and re-lined with gypsum ceiling sheets. All existing ceiling sheets to remain are to be re-fixed to the ceiling joists.	024

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	Sub-floor area	Sub-floor Concrete Wall	Cracking and spalling to sub-floor piers cast integral with sub-floor support walls	Y	Side fix bearer to sub-floor concrete walls above pier with minimum two post installed bolts to re-instate vertical and lateral load-path from ground floor framing diaphragm into sub-floor wall.	025
1st	2	Internal Wall	Patched horizontal and vertical cracks re-opened off door and ceiling/wall junction	Y	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	037
1st	2	Internal Wall	Vertical crack below window	Y	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	038
1st	Corridor	Internal Wall	Tapered vertical crack above doorway at North end of corridor	Y	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	039-040
1st	Corridor	Internal Wall	Continuous vertical crack in wall lining located in corridor	Y	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	041
1st	5	Ceiling lining	Damage to ceiling lining at junction with cornice	Y	All cracked and damaged ceiling linings are to be removed and re-lined with gypsum ceiling sheets. All existing ceiling sheets to remain are to be re-fixed to the ceiling joists.	042

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
1st	43	Ceiling lining	Damage to ceiling lining at junction with cornice	Y	All cracked and damaged ceiling linings are to be removed and re-lined with gypsum ceiling sheets. All existing ceiling sheets to remain are to be re-fixed to the ceiling joists.	043
1st	Corridor	Internal Wall and Ceiling	Damage to ceiling and wall lining at bulkhead above double door	Y	All cracked and damaged wall boards and ceiling linings are to be removed and re-lined with gypsum wall boards and ceiling sheets. All existing sheets to remain are to be re-fixed to the timber studs walls and ceiling joists.	044
1st	Corridor	Internal Wall	Two vertical cracks above doors to rooms no. 42 and 43	Y	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	045
1st	42	Ceiling lining	Tapered crack in ceiling lining	Y	All cracked and damaged ceiling linings are to be removed and re-lined with gypsum ceiling sheets. All existing ceiling sheets to remain are to be re-fixed to the ceiling joists.	046
1st	40-41	Wall, Floor and Ceiling lining	Extensive cracking in floor, wall and ceiling lining in walkway to toilet block as a result of differential settlement between toilet block and main ward wing	Y	Replacement to wall, floor and ceiling linings to be carried our after re-levelling of toilet block extension.	047-053
1st	40	Internal Wall and Ceiling	Cracking and paint damage along wall and ceiling junction	Y	All cracked and damaged wall boards and ceiling linings are to be removed and re-lined with gypsum wall boards and ceiling sheets. All existing sheets to remain are to be re-fixed to the timber studs walls and ceiling joists.	054

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
1st	Corridor	Internal Wall	Vertical crack in corridor above door opening	**Y**	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	055
1st	Corridor	Internal Wall	Vertical crack in corridor above door opening	**Y**	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	056
1st	Corridor	Internal Wall and Ceiling	Vertical crack in corridor above door opening and damage between cornice and wall lining	**Y**	All cracked and damaged wall boards and ceiling linings are to be removed and re-lined with gypsum wall boards and ceiling sheets. All existing sheets to remain are to be re-fixed to the timber studs walls and ceiling joists.	057
1st	36	Ceiling lining	Tapered crack in ceiling	**Y**	All cracked and damaged ceiling linings are to be removed and re-lined with gypsum ceiling sheets. All existing ceiling sheets to remain are to be re-fixed to the ceiling joists.	058
1st	36	Internal Wall and Ceiling	Damage between cornice and wall lining	**Y**	All cracked and damaged wall boards and ceiling linings are to be removed and re-lined with gypsum wall boards and ceiling sheets. All existing sheets to remain are to be re-fixed to the timber studs walls and ceiling joists.	059
1st	35	Internal Wall	Tapered vertical crack above doorway (view of crack in photo 57 from inside room 35)	**Y**	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	060

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
1st	15	Internal Wall and Ceiling	Separation of cornice from wall lining above double door	**Y**	All cracked and damaged wall boards and ceiling linings are to be removed and re-lined with gypsum wall boards and ceiling sheets. All existing sheets to remain are to be re-fixed to the timber studs walls and ceiling joists.	061
1st	23	Internal Wall	Separation of wall lining at intersecting corner	**Y**	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	062
1st	Southern Stair well	Internal Wall and Ceiling	Vertical cracks of window and damage to wall lining at cornice inside stair well	**Y**	All cracked and damaged wall boards and ceiling linings are to be removed and re-lined with gypsum wall boards and ceiling sheets. All existing sheets to remain are to be re-fixed to the timber studs walls and ceiling joists.	063-065
1st	19	Internal Wall	Separation of wall lining at intersecting corner	**Y**	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	66
1st	11	Internal Wall and Ceiling	Damage between cornice and wall lining	**Y**	All cracked and damaged wall boards and ceiling linings are to be removed and re-lined with gypsum wall boards and ceiling sheets. All existing sheets to remain are to be re-fixed to the timber studs walls and ceiling joists.	067
GND	Southern Stair well	Internal Wall	Vertical crack in wall lining	**Y**	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	068

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	Southern Stair well	Internal Wall	Tapered vertical crack in wall lining off bottom of window	**Y**	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	069
GND	Entry corridor to minor precedures unit	Internal Wall	Vertical crack in North end of bulkhead	**Y**	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	070
GND	Entry corridor to minor precedures unit	Internal Wall and Ceiling	Cracking and damage to wall and ceiling linings around cornice and South end of bulkhead	**Y**	All cracked and damaged wall boards and ceiling linings are to be removed and re-lined with gypsum wall boards and ceiling sheets. All existing sheets to remain are to be re-fixed to the timber studs walls and ceiling joists.	072
GND	34	Internal Wall	Vertical cracking in wall linings located at corners above doorway	**Y**	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	073
GND	35	Internal Wall and Ceiling	Wall lining separation from bottom of cornice	**Y**	All cracked and damaged wall boards and ceiling linings are to be removed and re-lined with gypsum wall boards and ceiling sheets. All existing sheets to remain are to be re-fixed to the timber studs walls and ceiling joists.	074

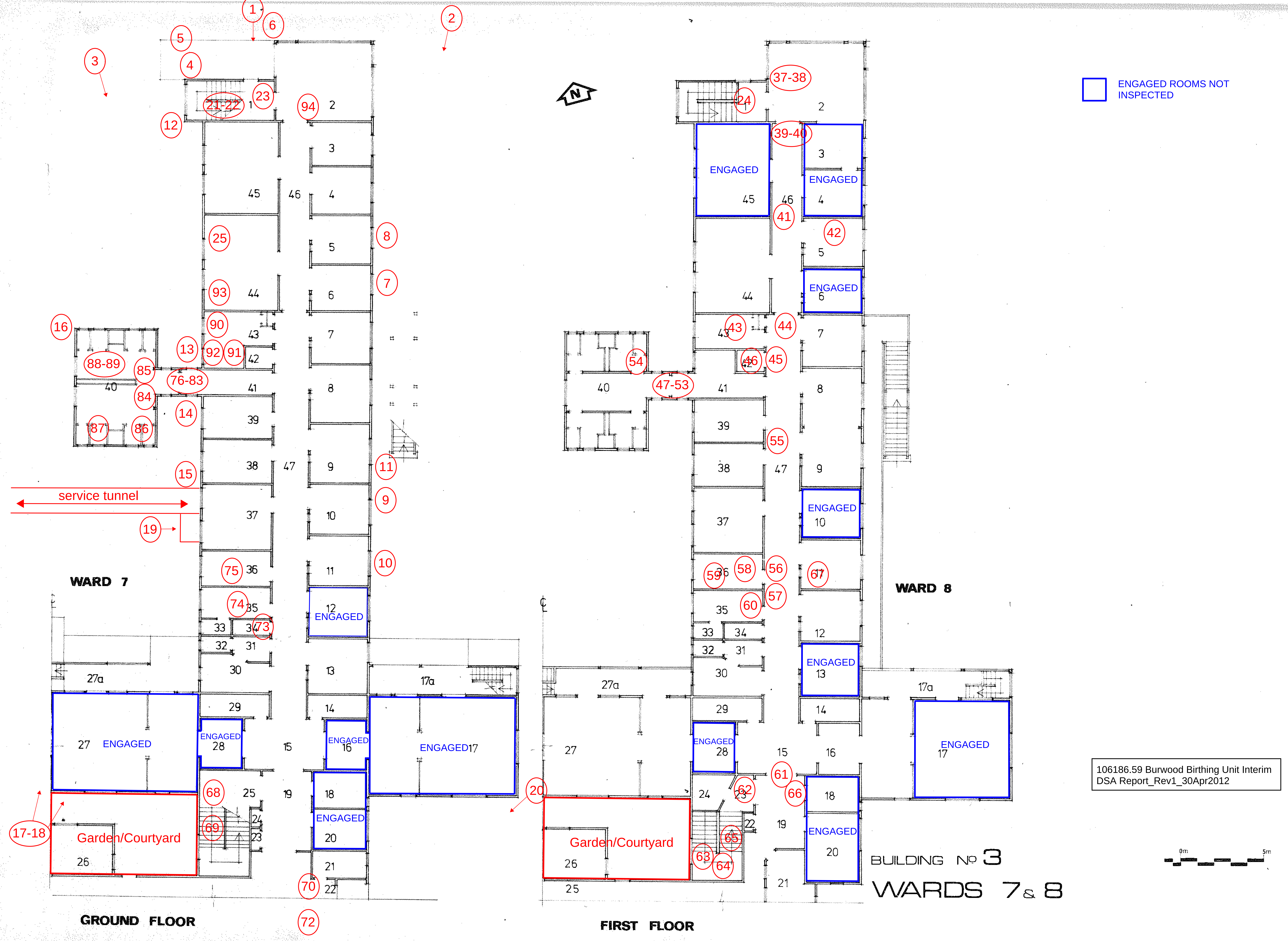
Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	36	Internal Wall and Ceiling	Wall lining separation and cracking from bottom of cornice	**Y**	All cracked and damaged wall boards and ceiling linings are to be removed and re-lined with gypsum wall boards and ceiling sheets. All existing sheets to remain are to be re-fixed to the timber studs walls and ceiling joists.	075
GND	41	Internal Wall and Ceiling	Extensive cracking in floor, wall and ceiling lining in walkway to toilet block as a result of differential settlement between toilet block and main ward wing	**Y**	Replacement to wall, floor and ceiling linings to be carried our after re-levelling of toilet block extension.	076-083
GND	40	Internal Wall and Ceiling	Continuous ceiling crack across entry to toilets. Horizontal cracking in wall lining off corner of window	**Y**	Repair to wall, floor and ceiling to be carried our after re-levelling of toilet block extension. Repair specification by others.	084
GND	40	Ceiling lining	Continous ceiling crack across entry to toilet entry area	**Y**	All cracked and damaged ceiling linings are to be removed and re-lined with gypsum ceiling sheets. All existing ceiling sheets to remain are to be re-fixed to the ceiling joists.	085
GND	40	Internal Wall	Horizontal cracking between top of windows in along Southern wall of toilet entry area	**Y**	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	086
GND	40	Internal Wall	Horizontal cracking between top of windows in along Southern wall of toilet area. Extends up through corner of wall at West end	**Y**	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	087

Level	Room Number	Building Element	Observations	Repair Required	Repair	Photo Reference
GND	40	Ceiling lining	Continous ceiling crack across toilet corridor	Y	All cracked and damaged ceiling linings are to be removed and re-lined with gypsum ceiling sheets. All existing ceiling sheets to remain are to be re-fixed to the ceiling joists.	088-089
GND	43	Internal Wall	Damage to wall lining in corner of room	Y	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	090
GND	43	Internal Wall	Damage to wall lining in corner of room	Y	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	091
GND	43	Internal Wall	Diagonal cracking in wall lining off corner of administation window (internal view of cracks shown in 077-079)	Y	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	092
GND	44	Internal Wall and Ceiling	Damage to wall lining and cornice-ceiling junction	Y	All cracked and damaged wall boards and ceiling linings are to be removed and re-lined with gypsum wall boards and ceiling sheets. All existing sheets to remain are to be re-fixed to the timber studs walls and ceiling joists.	093
GND	2	Internal Wall	Vertical crack of entry door to corridor	Y	All cracked and damaged wallboards are to be removed and re-lined with gypsum wall board sheeting. All existing wall boards to remain are to be re-fixed to the timber studs walls.	094

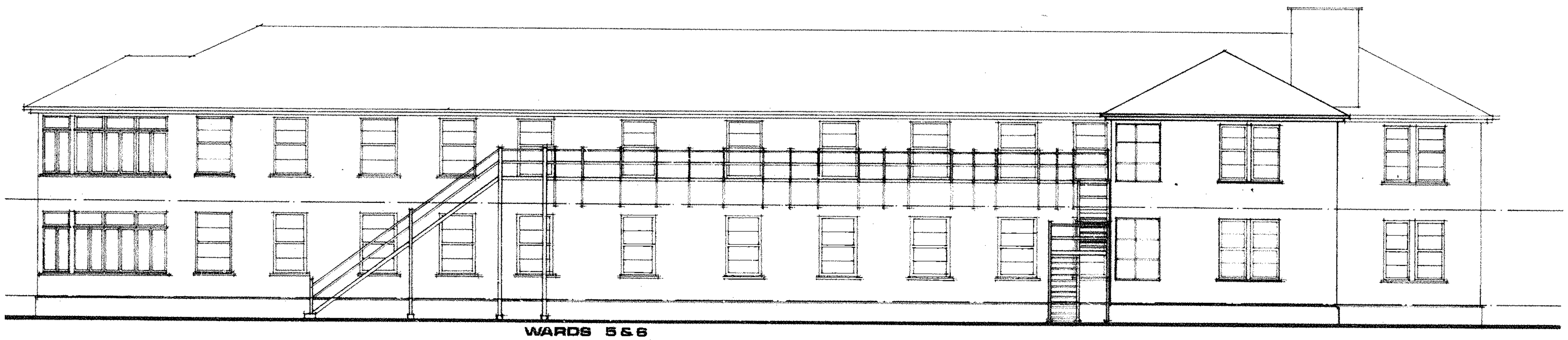
NOTE: Rooms noted as 'ENGAGED' on photo refernce plan were either locked or engaged at time of inspection and were not assessed

Appendix B

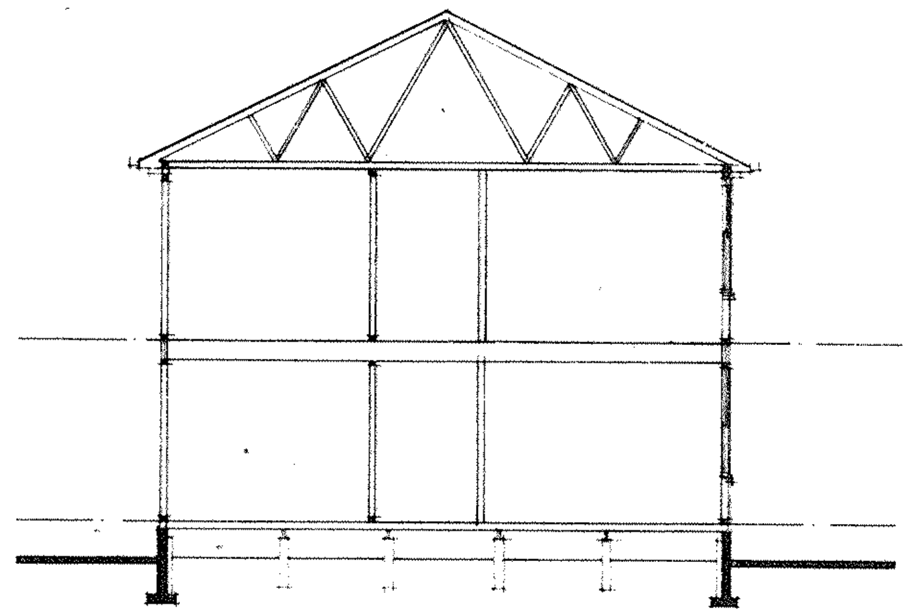
Reference Plans



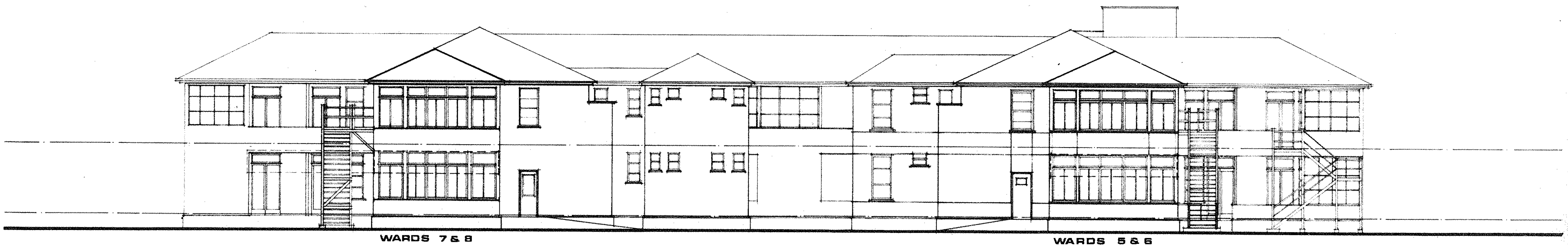
106186.59 Burwood Birthing Unit Interim DSA Report_Rev1_30Apr2012



WEST ELEVATION



SECTION XX



NORTH ELEVATION

BUILDING No 2&3

WARDS 5&6 , 7&8

106186.59 Burwood Birthing Unit Interim
DSA Report_Rev1_30Apr2012

Appendix C

Survey of Levels

All Wards 7 & 8 level observations reduced to hardwood floor level.
Benchmark = Nail 21

Elevations Table - Wards 7 & 8

Number	Minimum Elevation	Maximum Elevation	Color
1	14.960	14.970	
2	14.970	14.980	
3	14.980	14.990	
4	14.990	15.000	
5	15.000	15.010	
6	15.010	15.020	
7	15.020	15.030	

A1 Original Size

No.	Revision	Approved	Date
G			
F			
E			
D	Wards 7 & 8, Surgical Block, Melrose Chairs & Corridor added	MJM	02/12/2011
C	Phys Med, Parafed, Food Services, BIRS, Boilerhouse, EDC added	MJM	24/11/2011
B	Nurses Hostel & Champion Centre, SOU elevations added	MJM	18/11/2011
A	Administration and Allan Bean Centre elevations added	MJM	31/10/2011

032005

Scale	1:100
Reduced	A3 1:200
Designed	----
Drawn	DHP
Checked	MJM
Date	02/12/2011

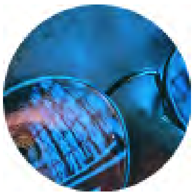
106186.59 Burwood Birthing Unit Interim DSA Report_Rev1_30Apr2012

Burwood Elevations Survey

Wards 7 & 8

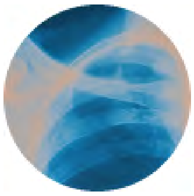
SURVEYORS
Fox & Associates
URBAN DESIGNERS
LAND DEVELOPMENT
CONSULTANTS
www.foxsurvey.co.nz
0800 FOX SURVEY
P.O.Box 13-390
CHRISTCHURCH

Job No. 2462C
Sheet No. 13 of 14
Rev. D



REPORT

STRUCTURAL AND CIVIL ENGINEERS



BURWOOD HOSPITL CAMPUS

REPORT 23 - MILNER LODGE

PREPARED FOR

CANTERBURY DISTRICT HEALTH BOARD

106186.76

INTERIM REPORT REV 3 – 13 JANUARY 2014

BURWOOD CAMPUS, MILNER LODGE – DETAILED SEISMIC ASSESSMENT REPORT

REPORT 23

Prepared For:
CANTERBURY DISTRICT HEALTH BOARD

Date: 13 January 2014
Project No: 106186.76
Revision No: 3

Prepared By:

Peter Grange
STRUCTURAL ENGINEER

Reviewed By:

Eric McDonnell
SENIOR PROJECT ENGINEER

Reviewed By:

Jenny Fisher
PROJECT DIRECTOR

Holmes Consulting Group LP
Christchurch Office

REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
26/7/12	1	Interim for Review
03/10/12	2	Revised recommendations based upon additional structural explorations
13/01/14	3	Updated to include signature of Project Director

CONTENTS

APPENDICES

TABLES Page

FIGURES Page

Holmes Consulting Group has been engaged by Canterbury District Health Board (CDHB) to complete a full structural review of the Burwood Campus following the Lyttelton Earthquake. A series of reports have been compiled as part of this process. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The specific building reports, like this one for the Milner Lodge, should be read in conjunction with the base report, and refer to the repair specification.

This report identifies the structural damage observed to date for the Milner Lodge Buildings as a result of the series of Earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The report summarises the effects of the damage on the lateral load capacity of the building, and provides structural repairs for the damage identified. The general form of the building, along with its capacity relative to current code levels, has been included for the buildings pre-earthquake undamaged state and post-earthquake state.

The Milner Lodge Buildings consists of two identical single storey buildings, with two units each, which were designed and constructed in 1983. The roofs are constructed of interlocking concrete roof tiles supported by timber roof purlins and prefabricated timber trusses. The trusses span to the external walls constructed primarily of timber framing with 15 mm profiled 'shadowlclad' plywood external claddings. The internal walls are lined with gypsum wallboard. A central 190 mm reinforced concrete masonry separation wall divides the two units in each building in the north-south direction. Short reinforced concrete masonry wing walls support the central wall out-of-plane and provide in-plane bracing for the front and back of the units. The buildings are supported on a reinforced concrete ground floor slab, with shallow reinforced concrete strip footings beneath the load bearing walls.

The information available included a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [7], along with a level survey completed by Fox and Associates [3].

The Milner Lodge Buildings appears to have performed fairly well considering the seismic actions and earthquake induced differential ground settlement experienced at the site. Although no significant damage has been observed to the superstructure, some damage has been noted to the wall linings. Separation has also been noted between the concrete slabs of the residential units and the adjacent carport structures. Earthquake induced differential ground settlement of the foundations has occurred, resulting in a worst case permanent slope of 50 mm over a 10 m length (1:200 or 0.5%).measured in the ground floor slab across the length of the building.

It is believed that the majority of the damage observed, including the onset of damage, occurred as a result of the 22nd February event. Further observations of the earthquake damage observed have been included in the body of this report.

Based upon a review of the information available, and the site investigations completed, the primary lateral force resisting elements of the Milner Lodge were assessed in their pre-earthquake undamaged state. For the purposes of this assessment the Milner Lodge has been considered to be Importance Level 2 building (IL2, R=1.0).

Based on this review the assessed capacity of the primary lateral load resisting elements of the building, relative to the demand imposed by the current loading code Design Basis Earthquake (DBE), have been assessed at approximately 70 % DBE in the north-south direction and approximately 100 % DBE in the east-west direction.

However, based on additional investigations completed the capacity of two connections have been found to be below 33% DBE. This includes the connection of the exterior east and west bracing walls to the foundation below and the connection of the timber collector elements to the end of the concrete block walls running in the east-west direction.

If the buildings were to be assessed for an increased importance factor, IL3, the seismic demand would increase by 30% (R=1.3) and as such the assessed capacities would be reduced proportionally.

The reduction in the lateral capacity of the building due to the earthquake damage observed is hard to quantify. Although the damage to the timber bracing wall linings will have resulted in some reduction in strength, the primary effect will be a reduced stiffness of the building along these bracing lines. This may result in larger lateral displacements at the east and west ends of the building, which could result in additional damage to interior linings and building contents in these areas.

The minimum repairs required to reinstate the building to its pre-earthquake undamaged condition have been included in Section 4. This includes repair of the damaged wall linings.

While the permanent slopes noted in the ground floor are right around the typical range for residential timber framed construction, they may be deemed unacceptable by CDHB based upon the building function. If the slopes are deemed unacceptable, re-levelling of the ground floor could be achieved through the use of mechanical jacking or underpinning grout.

As a result of the building being assessed at below 33% DBE, the Milner Lodge buildings are considered to be "Earthquake Prone" in terms of section 122 of the Building Act. Christchurch City Council current policy requires that buildings identified as "Earthquake Prone" be strengthened to 67% of current code requirements when seeking consent for repairs, which is the minimum strengthening we would recommend.

The limited capacity of the connections noted are also considered to be Critical Structural Weaknesses (CSW's) as they prevent the development of the capacity of the primary lateral load resisting elements.

A strengthening scheme for the connections identified as having a low capacity has been included in Section 5. This consists of the installation of additional anchor bolts and hold downs between the external bracing walls and the foundations below, along with strengthening the connection between the timber collector elements and the ends of the concrete block walls running in the east-west direction.

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 and/or strengthening of the building have been completed.

1. INTRODUCTION

Holmes Consulting Group has been engaged by Canterbury District Health Board to complete a full structural review of the Burwood Hospital Campus following the Lyttelton Earthquake. A series of reports have been completed as part of this. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The individual building reports, like this one, should be read in conjunction with the base report and refer to the repair specification.

The Burwood Hospital Campus base report covers the purpose and scope of the structural review. The current statutory requirements relevant to earthquake damaged buildings are outlined and the level of shaking experienced at the site estimated. Earthquake induced ground settlement damage across the campus is also discussed. The repair specification has been prepared to include repair details for typical damage observed in buildings on the Burwood Hospital Campus and is referred to as required in the specific building reports.

1.1 SCOPE OF WORK

This report is on the Milner Lodge buildings located at the Canterbury District Health Board (CDHB) Burwood Hospital Campus, approximately 7 km north-east of downtown Christchurch. The report identifies the general form of the structure, along with the gravity and lateral load resisting systems. Each component of the structural system was reviewed based upon the information available and any potential Critical Structural Weaknesses (CSW's) were noted.

The report also identifies the structural damage observed to date as a result of the series of Earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake, in particular, subjected the building to strong ground motions which significantly exceeded the current code loading demand for buildings of this nature.

The capacity of the Milner Lodge buildings has been assessed relative to current code loading in the buildings pre-earthquake undamaged state and in its post-earthquake damaged state. The post-earthquake assessment summarizes the effects of the damage identified on both the gravity and lateral load resisting elements. Repair options to restore the capacity of the building to pre-earthquake levels for strength, durability and stiffness have been included. The repair options aim to maintain the utility of the building. Where required, strengthening options have also been provided.

1.2 LIMITATIONS

Findings presented as a part of this project are for the sole use of the Canterbury District Health Board, 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. Because all of the structure has not been available for detailed inspection or evaluation, this report is limited to those elements available and engineering judgement as to the likely condition of unseen elements. 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. PRE-EARTHQUAKE BUILDING CONDITION

This section discusses the form and capacity of the building prior to the Darfield Earthquake, September 2010. The information available for this review included a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [7] along with a level survey of the building completed by Fox and Associates [3].

2.1 BUILDING FORM

The Milner Lodge buildings at the Burwood Hospital campus were designed and constructed in 1983. The two buildings are identical in plan, with two residential units each. The buildings are approximately 117 m^2 in plan area and spaced approximately 10 m apart, and as shown in Figure 2-1. A ground floor plan of one of the buildings is shown in Figure 2-2.

Figure 2-1: Milner Lodge – Plan View

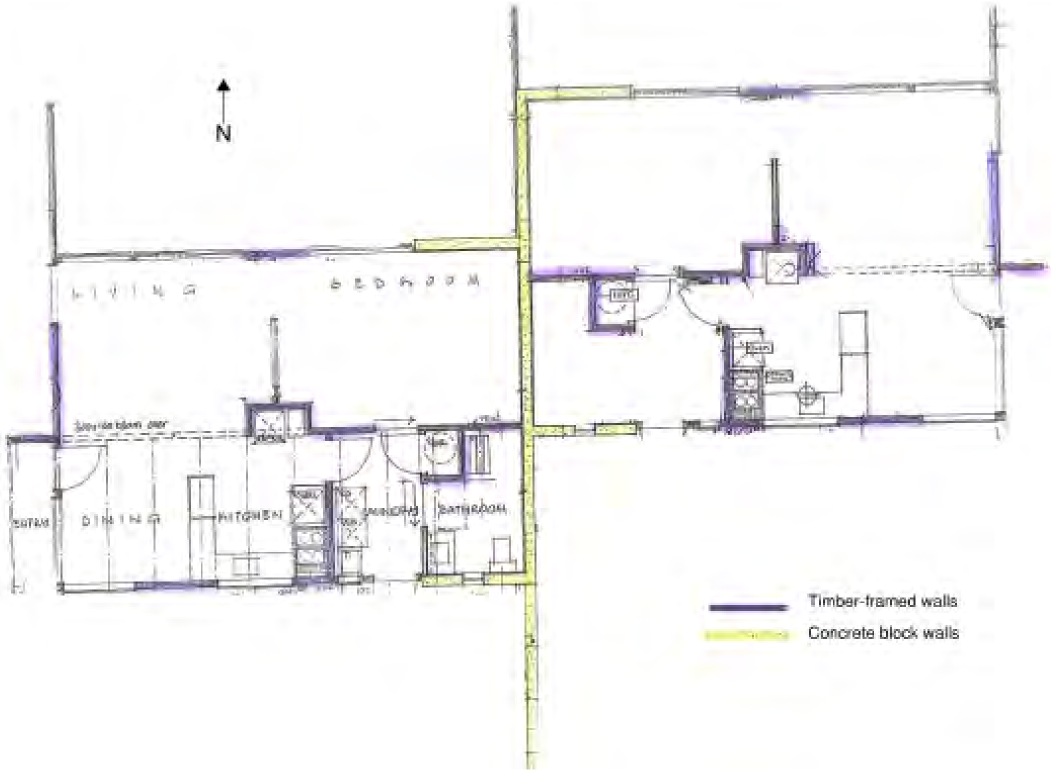


Figure 2-2: Milner Lodge – Ground Floor Plan

The information available included a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [7], along with a level survey completed by Fox and Associates [3].

The Milner Lodge buildings have interlocking concrete roof tiles supported by timber roof purlins and prefabricated timber trusses. The trusses span to the external walls which are constructed primarily of timber framing with 15 mm profiled external 'shadowclad' plywood claddings. The internal walls and ceilings are lined with gypsum board claddings. A central 190 mm reinforced concrete block wall divides the two units in each building in the north-south direction. Short reinforced concrete block wing walls extend off the central wall in the east-west direction at the front and rear of each unit (see Figure 2-2).

The buildings are founded on a reinforced concrete ground floor slab, with shallow reinforced concrete strip footings located beneath load bearing walls.

The north elevation of one of the units is shown in Figure 2-3.

Figure 2-3: Milner Lodge - North Elevation

Each of the units has a carport on the south side. The carport has a light weight roof over timber purlins which are supported by the central reinforced block wall on one side and timber posts that form a small storage unit on the opposite side. See Figure 2-4. The size and depth of the footings beneath these elements is unknown.

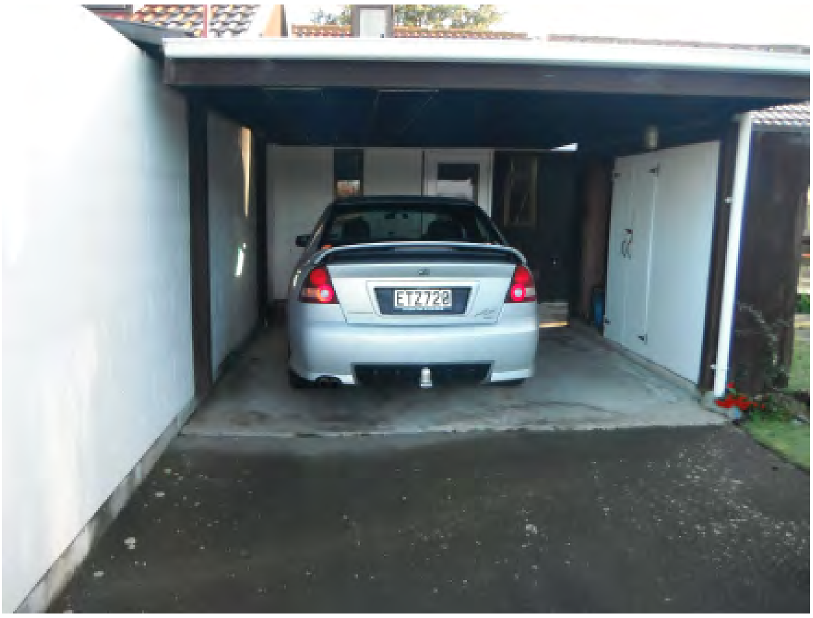


Figure 2-4: Milner Lodge - North Side (Carport)

2.2 LATERAL LOAD RESISTING SYSTEMS

In both north-south and east-west directions of the residential units, lateral loads are distributed to the bracing elements below through the gypsum board clad ceiling diaphragm. The bracing in the roof plane is nominal, but it has been assumed to be sufficient to transfer the seismic mass of the roof down to the ceiling plane.

In the north-south direction lateral loads are resisted by a combination of the reinforced concrete block walls, internal gypsum board lined timber bracing walls and the external plywood lined timber bracing walls on the east and west ends of the building. The contribution of the gypsum lined walls has been conservatively ignored as the block walls are much stiffer than these.

In the east-west direction lateral loads have been assumed to be resisted by the reinforced concrete block walls and the external plywood walls. The internal timber framed walls have not been considered due to stiffness incompatibility with the block walls. Because the block walls are concentrated at the centre of the building, and not well distributed the timber collector and the associated connection to the top of the block walls is critical.

For a summary of the assumed bracing wall elements see Figure 2-5 below. The non-bracing walls are shown in purple with the timber bracing walls shown in green and the block wall bracing sections shown in yellow.

Figure 2-5: Milner Lodge – Bracing Wall Locations

The lateral resistance of the carports is provided by the residential units in the north-south direction. In the east-west direction lateral resistance is provided by either the out-of-plane bending of the block walls or the storage units, or a combination there of. The ability of these elements to resist seismic loads will depend on the moment that can be developed by their respective foundations. As there is currently no information on these foundation elements footings an accurate seismic assessment of the carports cannot be completed at this time.

2.3 PRE-EARTHQUAKE BUILDING CAPACITY – DIRECT CODE COMPARISON

The building capacity under earthquake actions discussed in this section is compared to the capacity that a similar building would be designed to today. A new building would be designed to the *Structural Design Actions Standard, Part 5: Earthquake Actions – New Zealand*, NZS 1170.5:2004 [4] and incorporating the amendments made to this standard as a result of the Lyttelton Earthquake as outlined in the Amendment 10 of the Building Code [5]. The implications of the recent amendments are discussed more fully in the Burwood Hospital Campus Base Report however, for a building of this type the amendments essentially result in an increase to the design loads of 36 % when compared to pre-earthquake design levels.

When the building was originally designed in 1983, the loading standard at the time was the *New Zealand Loading Standard* – NZS4203:1976 [6]. When this standard was devised, neither the seismology of the different areas within New Zealand, or the impact this could have on buildings was as well understood as it is today. Along with an increase in the seismic demands required by the change in the loading code over this period, the seismic detailing requirements have also progressed significantly resulting in more ductile and better performing buildings.

The current seismic loading code, NZS 1170.5, requires a new building to be designed for an earthquake, known as the Design Basis Earthquake (DBE), which is based upon the buildings physical location, local soil conditions, building type, fundamental period and importance level.

As the original structural drawings, calculations and specifications for the building are not available, the exact design and loading assumptions originally made are unknown. For the purposes of this report, seismic loading assumptions have been made based on detailed physical observations of the building.

The Milner Lodge buildings are not regarded as essential hospital facilities by the CDHB and have therefore classified as an Importance Level 2 building in accordance with NZS 1170:2004 [4]. The associated return period of the DBE is 500 years, with a risk factor for design of R = 1.0. The sub soil for the site is taken as Soil Type D, which is consistent with the findings of a post-earthquake geotechnical investigation [7].

Based upon the period of construction, and the detailing of the lateral load resisting elements, the concrete block portion of the building has been assessed with ductility of $\mu = 2.0$, and the timber walls have been assessed with ductility of $\mu = 3.3$.

A comparison between the DBE of NZS4203:1976 [6] and NZS 1170:2004 [4] for the site is plotted in Figure 2-6. Based upon a fundamental building period below 0.40 seconds, the seismic demands on the structure required by the loading code have increased by approximately 50% since 1976.

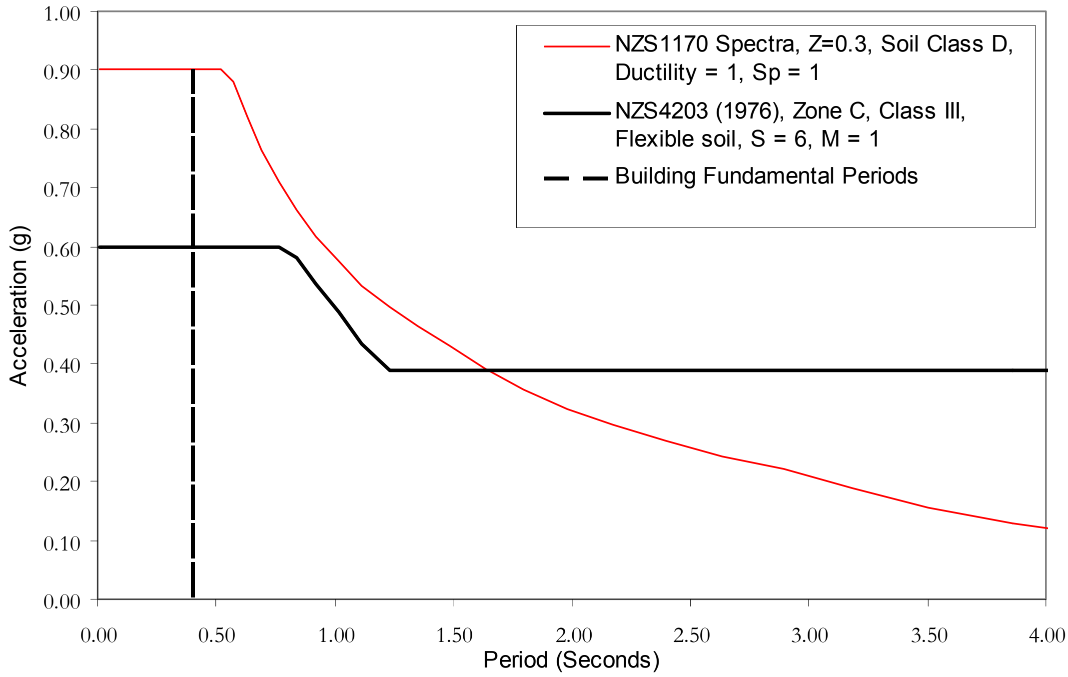


Figure 2-6: Comparison of Design Codes

2.4 EQUIVALENT STATIC ANALYSIS TO NZS1170.5 (2004)

In addition to the direct code comparison provided above, an equivalent static analysis to NZS1170.5: 2004 [4] has been carried out to gain a better understanding of the buildings estimated capacity when compared to current loading standards. The equivalent static analysis was carried out based upon floor plans, site measurements and as built observations.

Following the Lyttelton earthquake, a geotechnical report was conducted by Tonkin & Taylor, titled "Burwood Hospital Post Earthquake Geotechnical Assessment", dated June 2011 [7]. This report has been used to aid in the evaluation of the site conditions and the likely effect of the ground on the buildings past and future performance. The soil parameters described in the geotechnical report complete by Tonkin and Taylor has been used for the evaluation of the buildings existing foundation system. The foundations have been assessed at an ultimate bearing capacity of 150kPa, for factored loads, as per recommendations of this report.

The probable capacities of the structural elements 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 [8], Timber-framed buildings – NZS 3604:2011 [9], *Concrete Masonry Buildings Not Requiring Specific Engineering Design* – NZS 4229:1999 [10], *Design of Reinforced Masonry Structures* – NZS 1900:1964 [11] and *Historical Review of Masonry Standards in New Zealand* [12]. 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.

These are described in more detail in the Burwood Hospital Base report [1] and include strength governed elements such as short columns and deflection governed elements such as floor and stair elements with inadequate support seating.

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 the strength and deflection requirements.

For the purpose of this evaluation several assumptions also had to be made in regards to the existing building properties. The expected strength values for timber framed elements were taken from NZSEE 2006, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes* [8] and ASCE 41-06, *Seismic Rehabilitation of Existing Buildings* [13]. For the reinforced concrete block walls, the most conservative values (and allowable reinforcing) from NZS1900:1964 [11] were used. These values could be further refined through destructive investigations of the existing materials. The assumed diaphragm and shear wall expected strength values are as follows (assuming standard fixings and nail spacing):

- External Timber Framed Walls: Timber framed stud walls with external 15mm profiled plywood sheathing and internal gypsum board linings. Expected strength = 4.25 kN/m with a ductility of μ = 3.5.

- Ceiling diaphragm: Timber framed ceiling with gypsum board linings. Expected strength = 1.5 kN/m with ductility, μ = 3.5.

- Internal block walls: 190mm thick fully grouted and reinforced concrete block walls. Expected Strength = 15 kN/m with a ductility, μ = 2.0.

The foundations have been assessed at an assumed ultimate bearing capacity of 150kPa, for factored loads based upon the recommendations provided by Tonkin and Taylor [7].

The assessed capacity of the primary lateral load resisting elements of the building, relative to the demand imposed by the current loading code Design Basis Earthquake (DBE), have been assessed at approximately 70 % DBE in the north-south direction and approximately 100 % DBE in the east-west direction. In the north-south direction, the capacity of the building is limited by the exterior 15mm profiled (shadowclad) plywood lined bracing walls.

However, based on additional investigations completed, the capacity of two connections have been found to be below 33% DBE. This includes the connection of the timber collector elements to the end of the concrete block walls running in the east-west direction, and the connection of the exterior east and west bracing walls to the foundation below.

The assessed capacity of the block walls was initially completed assuming the minimum allowable reinforcement as per NZS1900:1964 [11]. Based upon this assumption the walls were assessed at 70% DBE under face loading. Scanning of the walls has since been completed showing that significantly more reinforcing was present than in this initial assumption so the capacity has been reassessed at 100 % DBE.

A summary of the capacity of each primary lateral element as a percentage of the demand imposed by the Design Basis Earthquake (DBE) have been noted in Table 2-1.

Table 2-1: Section 1 – Seismic Assessment %DBE

Building Element	%DBE	Comments
Wall bracing - E-W direction:	100 %	
Wall bracing - N-S direction:	70 %	Limited by capacity of small plywood bracing walls on the east and west ends of the building
Wall bracing connections	*20 %*	*CSW – insufficient connection between outer walls and foundations; this limits the lateral capacity of the wall.*
Block wall - Out-of-plane	100 %	
Ceiling diaphragm	100 %	
Collector connections	*20 %*	*CSW – insufficient connection between header and wall; load path disrupted (20% based upon capacity of remaining lateral load resisting system once the connection has failed).*
Foundations	100 %	
Carport	*70 %*	*Limited by number of nails connecting shed to foundation*

If the building were to be assessed for an increased importance factor, IL3, the seismic demand would increase by 30% (R=1.3) and as such the assessed capacities would be reduced proportionally.

A review of the drawings available identified the connection of the timber header beam / collector element to the top of the concrete block walls, running in the east-west direction, as critical elements to the overall performance of the building. Additional investigations completed found only a nominal connection of the timber header beam to the ends of the block walls (total 4 per building). The limited capacity of these elements does not enable the capacity of the block walls to be fully developed, and thus have been classified as Critical Structural Weaknesses (CSWs). If these connections were to fail, the secondary load path for the east and west ends of the building would be the short exterior timber bracing elements on the north and south faces of the buildings. If these walls are required to resist localized loads they would have an assessed capacity of approximately 20% DBE.

Likewise the connection between the base of the exterior sheet lined timber bracing walls, on the east and west faces of the building, to the foundation below has been found to consist of only two M6 anchor bolts for and 2.2 meter length of wall. This is limits the ability of the bracing load to transfer the required load to the foundation elements. As such these connections are considered to be Critical Structural Weaknesses (CSWs).

3. POST-EARTHQUAKE BUILDING CONDITION

This section covers the structural damage sustained by the Milner Lodge at Burwood Hospital Campus as a result of the series of earthquakes that includes the Darfield Earthquake that struck at 4:36am on 4th September, 2010 and the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011.

3.1 THE LYTTELTON EARTHQUAKE

The fundamental period of the building is estimated to be approximately 0.4 seconds. 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 strong motion data presented in the Base Report, it appears the Lyttelton Earthquake produced ground shaking intensities of approximately 60-120% of the current Ultimate Limit State design spectra for an Importance Level 2 building. The majority of the earthquake damage observed, or the onset of damage, appears to be as a result of this earthquake.

It should be noted that the Lyttelton Earthquake was very short in terms of strong shaking produced with the strong motion only lasting for duration of 5-7 seconds. Rupture of the Alpine Fault is expected to contain 50 to 60 seconds of strong motion.

3.2 PRELIMINARY INVESTIGATIONS

Preliminary investigations were carried out to identify 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
- damage observed during 'Rapid' Structural Assessments following the Lyttelton Earthquake, June 13th aftershocks and subsequent events

The following areas were identified for potential damage:

- movement or damage to structure associated with ground movement and/or settlement
- cracking to foundation slab
- cracking to concrete block walls
- distress at connection of timber roof framing to block walls

- distress and cracking of gypsum clad bracing walls
- distress and cracking of gypsum clad ceiling diaphragm
- damage to the heavy roof tile assembly

Rapid Level 2 assessments were carried out on 24th February 2011[14] and on 16th June [15] following the June 13th earthquakes. Additional Rapid Visual Structural Assessments were conducted on 5th January 2012 [16], following the 23rd December 2011 event. These structural observations involved a complete walk around the exterior of the building. The following primary areas of damage were identified from the damage assessments:

- signs of ground movement around perimeter (carport slab has displaced away from main building slab by up to 10 mm) and site asphalt has dropped up to 100 mm with a 20 mm crack in the concrete path
- there were no obvious signs of cracking to block work or damage to roof tile assembly

A review of the above information on the building type and preliminary observations highlighted this building as requiring a more detailed inspection. The aim of the detailed inspections was to determine the full extent of the damage caused to the building, particularly those elements identified for potential damage above. These areas were targeted to identify if damage had occurred, and to what extent the damage had reduced the capacity of the buildings lateral load resisting system to resist future seismic events.

3.3 DETAILED STRUCTUAL OBSERVATIONS

Further detailed inspections and structural explorations have been carried out following the initial assessments to ascertain the full extent of structural damage. The majority of the detailed structural observations were completed on 22nd June 2012.

A full record of these observations can be found in Appendix A. A full photographic record of the observations is available electronically on request. The detailed structural observation identified the following additional damage to those items noted in the initial rapid assessments:

- localized cracking of internal gypsum board linings
- further separation of the carport concrete slab from the perimeter of the house and cracking to concrete paths

3.4 GEOTECHNICAL REVIEW

A review of the ground damage and conditions was carried out by Tonkin & Taylor for the Burwood Hospital Campus. A subsequent report titled *Burwood Hospital Post Earthquake Geotechnical Assessment* was issued in June 2011 [7]. The geotechnical review concluded that the settlement and damage to building foundations and slabs on the Burwood Hospital Campus was likely due to the liquefaction of underlying soil layers. It is believed that excessive pore water pressures have been dissipated and that further settlement is not expected to occur, unless another significant event was to occur.

Based on the geotechnical report provided by Tonkin & Taylor [7] the potential for future total and differential settlements at the building site varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

3.5 LEVEL SURVEY

A detailed survey of the ground floor levels of the Milner Lodge buildings was conducted by Fox & Associates and issued on 18th April, 2012 [3]. The survey indicates a earthquake induced differential ground settlement has occurred on the site, resulting in a worst case permanent drop in the elevation of the ground floor slab of approximately 50mm over a 10 m length of the building (1:200 or 0.5% slope). This is on the edge of the typical acceptable range for standard occupancy buildings of timber framed construction, however given the nature of the patient group occupying the building, CDHB may wish to pursue re-levelling of the building. A discussion on how this could be achieved has been included in Section 4.2.

Figure 3-1: Milner Lodge – Level Survey

For the extent of the differential settlement noted to the building see the level survey included in Appendix B.

3.6 SUMMARY OF BUILDING DAMAGE

The following is a summary of the observations made for the building, and our conclusions as to their condition and seismic load resisting capacity. These observations do not specifically distinguish between damage caused by the Darfield Earthquake, the Lyttelton Earthquake or any significant aftershocks, such as those that occurred on the 13th June 2011, .the 23rd December 2011 or the 2nd January 2012.

The Milner Lodge buildings appear to have performed relatively well considering the seismic actions and differential ground settlement experienced at the site. Though there is significant ground movement around the buildings with separation of the carport concrete slabs, minimal damage has been noted to the building themselves. A summary of the typical damage observed is as follows:

- **Ground Movement** – Differential ground settlement and lateral spreading has been noted around Milner Lodge building sites. This includes a permanent drop in elevation of approximately 50mm in the ground floor slab and separation of up to 25 mm at the interface of the main building carport slabs.

- **Cracking to Wall Linings –** Few minor incidents of cracking and general distress has been noted to internal wall linings, primarily at corners, openings and along wall board joints.

While no specific damage was noted to the roof tile assembly, based upon the damage noted to other heavy roofs on the Burwood Campus site, we would recommended that a more detailed assessment be carried out by a qualified roofing contractor.

Table 4-1 provides a photographic summary of the typical type of damage observed. A full record of our detailed observations and repairs can be found in Appendix A.

3.7 FURTHER INVESTIGATIONS REQUIRED

3.7.1 Investigations Required For Further Assessment

Several assumptions were made in the completion of the pre-earthquake (undamaged state) and post-damaged (damaged state) structural assessments. Destructive exploration is required in a number of locations in order to verify these assumptions. The areas in which further investigation is required is as follows: Note: A report summarizing the investigations was completed by Naylor Love and dated 11th September 2012.

- At the interior and exterior lined timber bracing walls, verify the connection at the base of the walls is sufficient to develop the expected strength of the walls. This includes any holdown elements at the end of the exterior plywood bracing walls in addition to the size and spacing of typical fixings to foundation elements below.
    - The connection was made with only 2-M6 bolts. This connection is insufficient to get the full capacity out of the wall. Recommended strengthening has been included in Section 5.

- Validate the connection of the timber collector elements to the top of the concrete block walls running in the east-west direction.
    - Only a nominal connection was observed between the adjacent timber header beam / collector element and the top of the concrete block walls. There is a timber plate bolted to the top of the wall but it is not lapped with the header beam. The connection is not sufficient to drag the required loads back into the wall. Recommended strengthening has been included in Section 5.

- Validate fixings between existing ceiling diaphragm and the top of timber and block bracing walls.
    - No connection between the last rafter and the block wall was found. However, the roof diaphragm was found to be nailed to a timber plate which is bolted to the top of the concrete block wall.

- Validation of the reinforcing in the central concrete block wall. This investigation is likely to yield a higher reinforcement ratio than that assumed and thus a higher assessed capacity under face loading.
    - Significantly more reinforcing was present in the wall than initial assumed (approximately 16mm diameter bars at 600mm centres each way).
- A specialist assessment of the roof is recommended.
    - The specialist report has indicated that the roof is in generally good condition and that there is no repair or remedial work required.
- Verification of the footings beneath the block wall and the timber sheds in the carports areas is required.
    - The bottom plate of the timber shed walls were found to be connected to the foundation below with 'Hilti' powder activated pins at 800 mm centres. The footing width was found to be 600mm wide by 300 mm deep.

3.7.2 Investigations to be Completed During Building Repairs

- Re-inspection of the building will be required upon completion of any re-levelling works to determine if any additional damage has occurred.

3.8 POST-EARTHQUAKE BUILDING CAPACITY

Based upon our observations to date, we do not consider the Milner Lodge buildings to have any significant reduction in gravity load resistance. The damage observed to the interior wall linings will have resulted in some reduction in lateral load capacity, although it is difficult to quantify the actual percentage reduction in strength. The overall reduction in strength will be minimal, with the primary effect being a reduction in stiffness of the east and west ends of the buildings in the north-south direction. The reduction in stiffness may cause ongoing concerns in regards to the performance of the building, primarily to contents and non-structural elements at the east and west ends of the building.

The differential settlement noted will also have resulted in some reduction in capacity, but again this is difficult to quantify. The primary concern will be a reduced ability of the buildings to absorb future differential settlements prior to the onset of more severe damage to the foundations and superstructure of the buildings.

The damage observed will require repair to restore the strength, stiffness, durability and performance of the lateral bracing system. The repair work required is outlined in Section 4. Following the recommended repairs to the structural damage noted, the lateral load carrying capacity of the existing structure will be restored to close to pre-earthquake levels, which are summarised in Section 2.5.

As noted in Section 2, there are several critical connections which have been assessed below 33% DBE, and classified as Critical Structural Weaknesses (CSW's). As a result of the building being assessed at below 33% DBE, the Milner Lodge buildings are considered to be "Earthquake Prone" in terms of section 122 of the Building Act. Christchurch City Council current policy requires that buildings identified as "Earthquake Prone" be strengthened to 67% of current code requirements when seeking consent for repairs, which is the minimum strengthening we would recommend.

A strengthening scheme for the connections identified has been included in Section 5. This consists of the installation of additional anchor bolts and hold downs between the external bracing walls and the foundations below, along with strengthening of the connection between the timber collector elements and the ends of the concrete block walls running in the east-west direction.

4. DAMAGE OBSERVED & REPAIRS REQUIRED

4.1 PRIMARY OBSERVED DAMAGE AND REPAIRS REQUIRED

This section covers the damaged noted during our detailed assessment of the building. Note that our observations have been restricted to structural aspects of the building 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.

Table 4-1 provides a photographic summary of the observed damage and typical repairs required for the Milner Lodge buildings. Table 4-1 should be read in conjunction with Appendix A – Record of Observation.

In general, the aim of the repair work indicated in this section is to restore the structure to its pre-earthquake state, as far as practicable, while maintaining the utility of the building. The repairs presented attempt to address the loss of strength, stiffness and durability of the structural elements due to the damage noted.

It should be noted that more damage may be identified during the repair works and (if required) additional repair details will be specified accordingly.

Please note that if building is to be re-levelled, all repair works are to be completed after the building has been re-levelled to a satisfactory condition as further damage to the wall and ceiling linings can be expected during the re-levelling process.

Table 4-1: Photographs of Primary Observed Damage and Repairs Required

Damaged Item & Location	Damage	Recommendations	Example Photograph
1. Foundation			
1.1. Concrete slab on grade	At the interface of the main building and carport slabs, a gap of up to 25 mm has formed.	*Repair specification by others*	
1.2. Asphalt driveway	Some cracking has occurred in site asphalt adjacent to the carport slab.	*Repair specification by others*	

Damaged Item & Location	Damage	Recommendations	Example Photograph
2. Timber Framed Structure			
2.1. Wall Linings	There are a few cases of cracking to gypsum board wall linings, mostly off corners of door and window openings	Replace damaged wall boards with new gypsum board sheets. All wall boards to remain on timber walls assumed to provide lateral bracings are to be re-fixed. *All remaining lining repairs are to be specified by others.* For additional information see Section 4.3.	

4.2 DISCUSSION ON BUILDING RE-LEVELLING

The level survey, completed by Fox & Associates [3], has indicated differential ground settlement of approximately 50 mm over the length of the building (see Appendix B for complete level survey). The worst case permanent slope in the concrete slab on grade, based upon the level survey, is approximately 1:200 (or 0.5 %). This slope is on the edge of the typical acceptable range for standard occupancy buildings of timber framed construction. However, given the nature of the patient group occupying the building, CDHB may wish to pursue re-levelling of the building.

The two primary re-levelling options available for the Milner Lodge buildings include the use of either mechanical jacking or underpinning grout to raise the perimeter strip footings, along with the interior spread footings, to the elevation of the highest point noted for each individual building. The lowest point in both of the buildings is beneath the central block wall, and it is possible that all the footings of the building will require some degree of re-levelling treatment. For both options the extent of the re-levelling proposed would likely require the removal of the existing slab on grade.

There are advantages and disadvantages for each solution which extend beyond structural performance which will need to be considered by CDHB. These include continuity of operation, degree of re-levelling accuracy, risk of damage existing foundation system and/or superstructure, and the willingness of the re-levelling sub-contractor to provide a producer statement, amongst other items. From a structural standpoint, either option is acceptable provided the use of underpinning grout does not create any detrimental "hard points" under the building.

It should be noted that neither of the re-levelling options discussed above is expected to increase the seismic performance of the building or reduce the potential for future differential settlements. Instead the option presented are intended to re-level the building without making the future performance of the building any worse than it was prior to the earthquakes. To improve the future performance of the building, and reduce the potential for future differential settlements, would likely require the entire footprint of the building to be either piled or the ground under all the existing footings improved. Further geotechnical investigations would be required into the type and depth of ground improvement required.

Appropriate contingencies should be provided to account for the risks to the building foundations and superstructure during any re-levelling process.

4.3 REPAIR OF TIMBER WALL LININGS

The wall linings to the interior timber bracing walls have been damaged in some locations and require repair. The repair recommendation varies depending on whether or not the walls have been assumed to provide lateral bracing for the building.

At non-bracing walls, the repair to the wall linings will be aesthetic in nature only, and is to be specified by others.

At walls assumed to provide bracing, a structural repair will be required to reinstate the walls to their pre-earthquake strength and stiffness. This will include the replacement of any cracked or damaged sections of the wall linings with new gypsum board sheathing. The new gypsum board sheathing is to be fixed in accordance with GIB 'ezybrace' GS2-N specifications (or equivalent). All existing internal bracing wall linings to remain are to be re-fixed to the existing studs in a similar manner. For the locations of the assumed bracing walls, see Figure 2-5.

The primary lateral force resisting system of the Milner Lodge superstructure consists of a timber framed roof and a ceiling diaphragm which transfer lateral loads to sheet clad timber bracing walls and a central reinforced concrete block. As noted in Section 2, the assessed capacity of the primary lateral load resisting elements of the building, relative to the demand imposed by the current loading code Design Basis Earthquake (DBE), have been assessed at approximately 70 % DBE in the north-south direction and approximately 100 % DBE in the east-west direction.

However, several critical connections have been assessed below 33% DBE and identified as Critical Structural Weaknesses (CSW's). These are as follows:

- The connection of the external sheet lined timber bracing walls, on the east and west face of the building, to the foundation elements below. As there is only one bracing wall on these ends of the building the failure of these connections could throw the building into extreme torsion under loading in the north-south direction (two locations in total per building).

- The connection of timber header beam / collector element to the top of the concrete block walls running in the east-west direction. The reinforced concrete block walls (return walls) are the primary lateral load resisting elements in the east-west direction. As they are located in the centre of the building the load is required to be dragged back into the walls. The load path required for this to occur is through the timber header beams spanning the openings adjacent to the walls.

Strengthening schemes to address these critical connections are provided below. Following the completion of this strengthening, the capacity of the building would be approximately 70 % DBE.

5.1 BRACING WALL TO FOUNDATION CONNECTION

As the existing connection between the outer walls and the foundations is inadequate, the bracing capacity of these walls cannot be properly engaged. To improve this connection new holdowns should be installed at either end of each wall, along with intermediate anchor bolts in between. The location is shown on the plan in Figure 5-1.

As the wall cladding must be removed for this strengthening work, we would recommend new plywood panels be installed as bracing elements to increase the capacity of the walls. Holdowns should have the strength required to develop the capacity of the new plywood bracing walls. This should be in the form of 12.5 mm plywood on the interior side with 50 x 2.8 mm nails at 150 centres to perimeter and 300 mm to intermediate studs. Gib HandiBrac Hold Downs should be installed.

5.2 HEADER BEAM TO WALL CONNECTION

The connection between the header and the block wall is insufficient and as such the ceiling diaphragm is not properly engaged. The limited capacity of this connection means the load path to the block walls is incomplete. To strengthen the system, new steel straps should be installed on each side of the header beam to top of block wall connection. These would be bolted to the block wall and nailed into the timber header. This will provide the required tensile capacity to tie the two elements together and improve the performance of the structure. A strap of approximately 1.5 m is required in each location as shown in Figure 5-1.

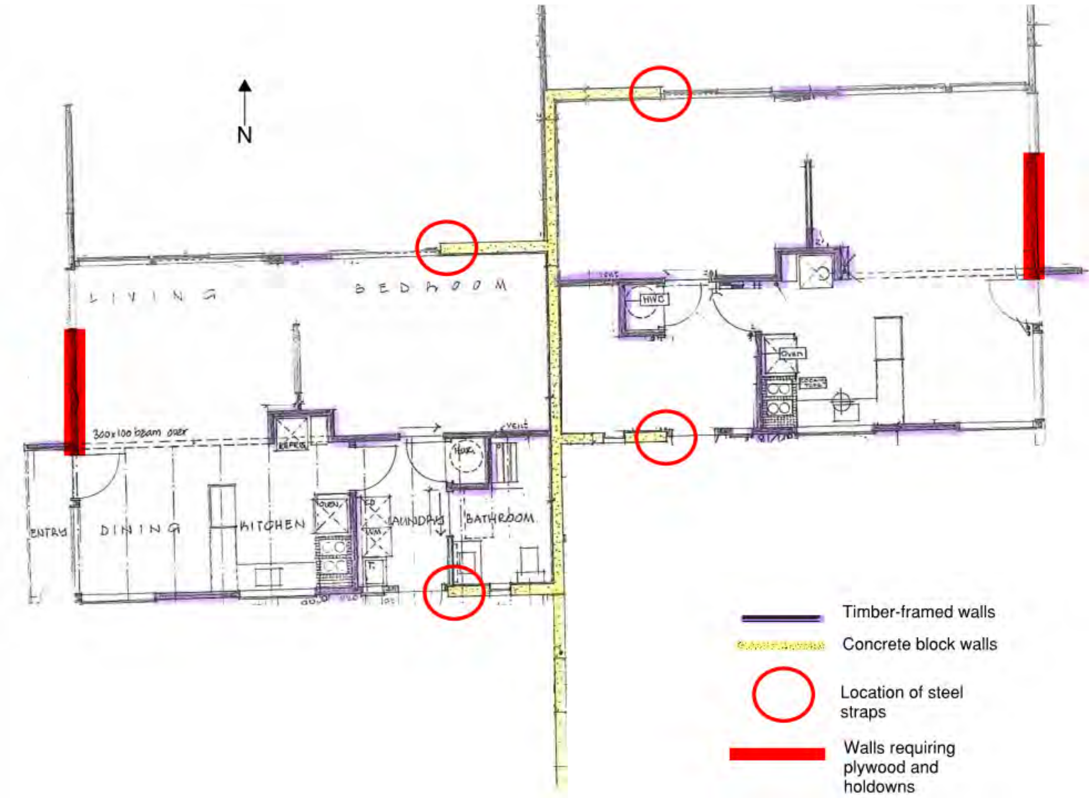


Figure 5-1: Location of steel straps between block wall and headers

6. REFERENCES

1 . *Burwood Hospital – Detailed Seismic Assessment Report – Base Report*, Holmes Consulting Group, November 2011.

2 *Burwood Hospital – Detailed Seismic Assessment Report – Earthquake Repair Specification,* Holmes Consulting Group, July 2011.

3 *CDHB – Burwood Field Survey*, Fox & Associates, June 2011

4 *Structural Design Actions Part 5: Earthquake Actions – New Zealand*, NZS 1170.5:2004, Standards New Zealand, 2004.

5 *Compliance Document for New Zealand Building Code - Clause B1 – Structure, Amendment 10 (Canterbury)*, Department of Building and Housing, Wellington, 19 May 2011.

6 *New Zealand Loading Standard NZS 4203: 1976*, Standards New Zealand

7 *Burwood Hospital Post Earthquake Geotechnical Assessment*, Tonkin and Taylor Ltd., June 2011.

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

9 *Timber-framed buildings, NZS 3604:2011*, New Zealand Standard

10 *Concrete Masonry Buildings Not Requiring Specific Engineering Design, NZS 4229:1999*, Standards New Zealand

11 *Masonry, NZSS 1900:1964, Chapter 6.2,* New Zealand Standard

12 Peter C Smith and Jonathan W Devine of Spencer Holmes Ltd, *Historical Review of Masonry Standards in New Zealand*, for the Royal Commission of Inquiry Building Failure Caused by the Canterbury Earthquakes, Aug 2011

13 *Seismic Rehabilitation of Existing Buildings*, ASCE 41-06, 2007

14 *CDHB – Burwood Hospital Campus – Rapid Visual Inspection: 106186.03 Site Report 1*, Holmes Consulting Group, February 2011

15 *CDHB – Burwood Hospital – Rapid Seismic Assessment Post June 13th Earthquakes: 106186.03 Site Report 6*, Holmes Consulting Group, 16 June 2011

16 *CDHB – Burwood Hospital – Rapid Seismic Assessment Post January 2nd Earthquakes: 106186.03 Site Report 9*, Holmes Consulting Group, 9 January 2012

APPENDIX A

Record of Observations

APPENDIX A – RECORD OF OBSERVATIONS & REPAIRS

Inspection date: 15 Feb 2012 through to 21 March 2012

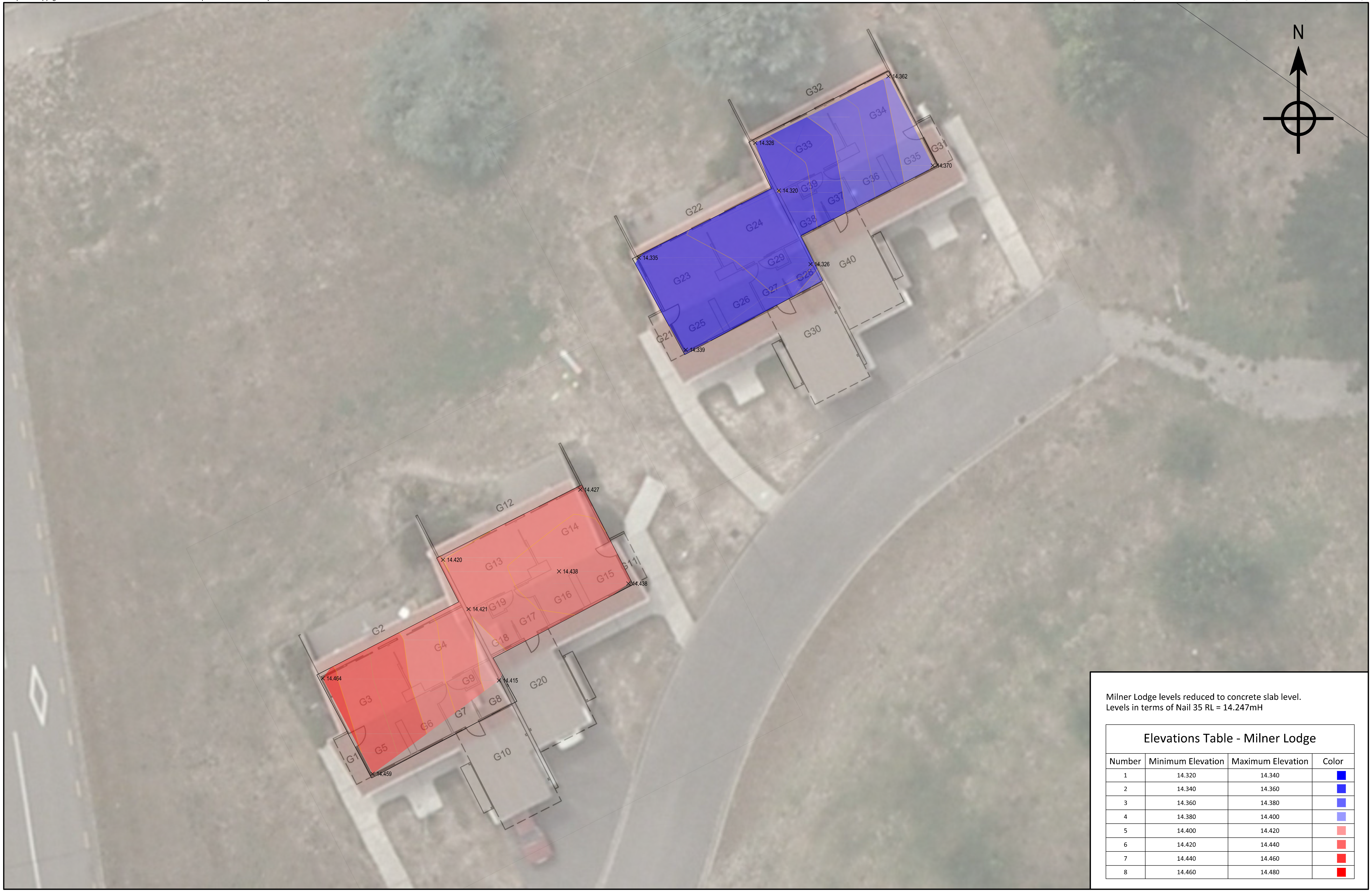
KEY	
N	No repair required
Y	Repair required
F	Further investigation required
C	Repair complete

Level
E - exterior
SF - subfloor
G - ground

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G		Typical	Wall linings	Minor cracking to gib in a few locations throughout the units	**Y**	Repair/replace damage linings	DSCF1082 - 3
G		Outside	Concrete ground slab	Movement between house slab and carport slab	**F**	Seal gap - specification by others	DSCF1085 - 6
G		Outside	Asphalt driveway	Cracking to asphalt driveway	**F**	Repair spcification by others	DSCF1084

APPENDIX B

Levels Survey



Milner Lodge levels reduced to concrete slab level.
Levels in terms of Nail 35 RL = 14.247mH

Elevations Table - Milner Lodge

Number	Minimum Elevation	Maximum Elevation	Color
1	14.320	14.340	
2	14.340	14.360	
3	14.360	14.380	
4	14.380	14.400	
5	14.400	14.420	
6	14.420	14.440	
7	14.440	14.460	
8	14.460	14.480	

No.	Revision	Approved	Date
G			
F	Beacon House, BSU Hostel, Chapel, Tapper and Milner added	MJM	18/04/2012
E	Audit observations post Dec 23 2011 (not included in this Revision)	MJM	01/02/2012
D	Wards 7 & 8, Surgical Block, Melrose Chairs & Corridor added	MJM	02/12/2011
C	Phys Med, Parafed, Food Services, BIRS, Boilerhouse, EDC added	MJM	24/11/2011
B	Nurses Hostel & Champion Centre, SOU elevations added	MJM	18/11/2011
A	Administration and Allan Bean Centre elevations added	MJM	31/10/2011

Scale		1:75
Reduced	A3	1:150
Designed		----
Drawn		DHP
Checked		MJM
Date		18/04/2012

Burwood Elevations Survey

Milner Lodge

SURVEYORS
Fox & Associates
URBAN DESIGNERS LAND DEVELOPMENT CONSULTANTS

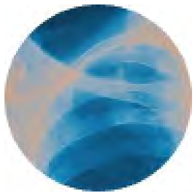
www.foxsurvey.co.nz
0800 FOX SURVEY
P.O.Box 13-390
CHRISTCHURCH

Job No. 2462C
Sheet No. 18 of 19
Rev. F

032005

DETAILED SEISMIC ASSESSMENT REPORT

STRUCTURAL AND CIVIL ENGINEERS



BURWOOD HSOPITAL CAMPUS

REPORT 25 – TAPPER UNITS

PREPARED FOR

CANTERBURY DISTRICT HEALTH BOARD

106186.79

INTERIM REPORT REV 2 – 4 MARCH 2014

BURWOOD HOSPITAL CAMPUS – DETAILED SEISMIC ASSESSMENT REPORT

REPORT 25 – TAPPER UNITS (FORMERLY SELF CARE UNITS)

Prepared For:
CANTERBURY DISTRICT HEALTH BOARD

Date: 4 March 2014
Project No: 106186.79
Revision No: 2

Prepared By:

Kelly Newton
PROJECT ENGINEER

Updated By:

Peter Grange
STRUCTURAL ENGINEER

Reviewed By:

Eric McDonnell
SENIOR PROJECT ENGINEER

Reviewed By:

Jenny Fisher
PROJECT DIRECTOR

Holmes Consulting Group LP
Christchurch Office

REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
19/09/12	1	Interim Report for Review
07/03/14	2	Further Investigations included

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Holmes Consulting Group has been engaged by Canterbury District Health Board (CDHB) to complete a full structural review of the Burwood Hospital Campus following the Lyttelton Earthquake. A series of reports have been compiled as part of this process. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The specific building reports, like this one on the Tapper Units, should be read in conjunction with the base report, and refer to the repair specification.

This report identifies the structural damage sustained by the Tapper Units as a result of the series of Earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The report summarises the effects of the damage on the lateral load capacity of the building, and provides structural repairs for the damage identified. The general form of the building, along with its capacity relative to current code levels, has been included for the buildings pre-earthquake undamaged state and post-earthquake state.

The Tapper Units (formerly the Self Care Units) was originally designed in 1978 and constructed in the period thereafter. It consists of a series of four residential units, located in the south-east corner of the campus. Each unit is set out with living and kitchen areas on the western half of the building, along with a bedroom and bathroom on the eastern half of the building.

The building has a lightweight standing seem metal roof over a layer of plywood sheathing and timber roof framing. The ground floor walls consist primarily of timber framed stud walls, with the exception of a concrete block separation wall between the units and a short concrete block return wall. The internal linings on the timber framed walls consist of gypsum plasterboard, while the exterior linings are a combination of horizontal weatherboard and a concrete block veneer. The buildings are supported on a reinforced concrete ground floor slab and what is assumed to be shallow reinforced concrete strip footings beneath.

The information available for the review included: a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [3], a 2009 Master Floor Plan provided from the CDHB's Maintenance and Engineering Department [4], and a level survey of the building completed by Fox & Associates [5].

The Tapper Units appear to have performed as would be expected for a building of this type and age considering the seismic actions and differential ground settlement experienced at the site. While the ground damage surrounding the building is relatively severe the observed damage to the building itself can be considered minor to moderate. The bulk of structural damage is typified by separation of the linings at the interface with block walls and cracking of the linings on the timber framed walls and ceilings.

Differential ground settlement has been noted around Tapper Units building including severe cracking and separation of exterior paving slabs around the building. A total drop of 83mm has also been noted in the ground floor slab across the length of the building resulting in a worst case measured slope in the slab of 1:180.

It is believed that the majority of the damage observed, including the onset of damage, occurred as a result of the 22nd February event. Further observations of the earthquake damage observed have been included in the body of this report.

Based upon a review of the information available, and the site investigations completed, the primary lateral force resisting elements of the Tapper Units were assessed in their pre-earthquake undamaged state. For the purposes of this assessment the Tapper Units building has been considered to be Importance Level 2 building (IL2, R=1.0).

The assessed capacity of the building, relative to the demand imposed by the current loading code Design Basis Earthquake (DBE), is approximately 100% DBE in the north-south direction and approximately 20% DBE in the east-west direction. In the east-west direction the %DBE is governed by the limited wall bracing provided, particularly on the north side of each unit.

If the buildings were to be assessed for an increased importance factor, IL3, the seismic demand would increase by 30% (R=1.3) and as such the assessed capacities would be reduced proportionally.

The reduction in the lateral capacity of the building due to the earthquake damage observed is hard to quantify. Although the damage to the timber bracing wall linings will have resulted in some reduction in strength, the primary effect will be a reduced stiffness of the building along these bracing lines. This may result in larger lateral displacements at the north and south ends of the building, which could result in additional damage to interior linings and building contents in these areas.

The building deformation due to the differential settlement will have resulted in some reduction in capacity, but again this is difficult to quantify. The primary concern from a structural standpoint will be a reduced ability of the building to absorb future differential settlements prior to the onset of more severe damage to the foundations and superstructure of the buildings.

As a result of the building being assessed at below 33% DBE, the Tapper Units are considered to be "Earthquake Prone" in terms of section 122 of the Building Act. Christchurch City Council current policy requires that buildings identified as "Earthquake Prone" be strengthened to 67% of current code requirements when seeking consent for repairs, which is the minimum strengthening we would recommend.

Despite the low % DBE, as a primarily light weight timber framed building, with natural built in redundancies, the building is unlikely to fail in a brittle manner.

The minimum repairs required to reinstate the building to its pre-earthquake undamaged condition have been included in Section 4. This includes repair of the damaged wall and ceiling linings, along with repairing the separation of ceiling and exterior eave linings at the interface withy the concrete block walls.

The overall differential settlement noted, and the associated slopes in the ground floor slab, are also outside the typical acceptable range for buildings of this construction type and will likely require re-levelling to restore the functionality of the building. Re-levelling options have been included in Section 4.2 and include the use of mechanical jacking or underpinning grout.

In addition to the repairs, recommended strengthening concepts to increase the seismic performance of the seismic performance of the building and bring its assessed capacity above 67% DBE have been included in Section 5.

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. Secondary elements, such as windows and fittings, have not generally been reviewed.

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 Canterbury District Health Board (CDHB) to complete a full structural review of the Burwood Hospital Campus following the Lyttelton Earthquake. A series of reports have been completed as part of this. These consist of a base report [1], a number of specific building reports and a repair specification [2]. The individual building reports, like this one on the Tapper Units, should be read in conjunction with the base report and refer to the repair specification.

The Burwood Campus base report covers the purpose and scope of the structural review. The current statutory requirements relevant to earthquake damaged buildings are outlined and the level of shaking experienced at the site estimated. Earthquake induced ground settlement damage across the campus is also discussed. The repair specification has been prepared to include repair details for typical damage observed in buildings on the campus and is referred to as required in the specific building reports.

1.1 SCOPE OF WORK

This report is on the Tapper Units (formerly the Self Care Units), Burwood Hospital, 255 Mairehau Road, Burwood, Christchurch. The report identifies the general form of the structure along with the gravity and lateral load resisting systems. Each component of the structural system was reviewed based upon the information available and any potential Critical Structural Weaknesses (CSW's) were noted.

The report also identifies the structural damage observed to date as a result of the series of Earthquakes, including: the Darfield Earthquake that struck at 4:36am on the 4th September, 2010; the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December, 2011. The Lyttelton Earthquake, in particular, subjected the building to strong ground motions which significantly exceeded the current code loading demand for buildings of this nature.

The capacity of the building has been assessed relative to current code loading in the buildings pre-earthquake undamaged state, and in its post-earthquake damaged state. The post-earthquake assessment summarizes the effects of the damage identified on both the gravity and lateral load resisting elements. Repair options to restore the buildings capacity to pre-earthquake levels for strength, durability and stiffness have been included. The repair options aim to maintain the buildings utility. Where required, strengthening options have also been provided.

1.2 LIMITATIONS

Findings presented as a part of this project are for the sole use of the Canterbury District Health Board (CDHB). 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 professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practising in this field at this time. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

Conclusions relate to the structural performance of the building under earthquake loads. We have not assessed the live load capacity of the floors, nor have we assessed the performance of non-structural components or building contents under earthquake loads.

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. Because all of the structure has not been available for detailed inspection or evaluation, this report is limited to those elements available and engineering judgement as to the likely condition of unseen elements. 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. PRE-EARTHQUAKE BUILDING CONDITION

This section discusses the form and capacity of the building prior to the Darfield Earthquake.

2.1 BUILDING FORM

The Tapper Units (formerly the Self Care Units) are located at the Canterbury District Health Board's (CDHB's) Burwood Hospital Campus, located 7km north-east of downtown Christchurch. The building was originally designed in 1978 and constructed in the period thereafter. It consists of a series of four residential units, located in the south-east corner of the campus. The single storey units are stepped in plan from south-west to north-east. Each unit is set out with living and kitchen areas on the western half of the building, along with a bedroom and bathroom on the eastern half of the building.

Figure 2-1: The Tapper Units - View from the North-East

The information available for the review included: a post-earthquake geotechnical assessment conducted for the campus by Tonkin & Taylor [3], a 2009 Master Floor Plan provided by the CDHB's Maintenance and Engineering Department [4], and a level survey of the building completed by Fox & Associates [5].

The Tapper Units have a lightweight standing seem metal roof over a layer of plywood sheathing. The plywood sheathing is supported by timber roof purlins which span between a series of timber roof beams and the internal load bearing walls below. Over the living and kitchen areas of each unit, the roof purlins and the timber beams are exposed (see Figure 2-2). Over the remainder of the unit, there is a dropped flat ceiling, framed with timber ceiling joists and clad with gypsum plasterboard linings.

Figure 2-2: Exposed Roof Framing over Living Area

The ground floor walls of the building consist of a mixture of timber framed stud walls and reinforced concrete block walls.

The perimeter walls are all timber framed, and clad on the south, east and west sides of the building, with a 100mm thick concrete block veneer. Based upon observations of similar veneer on site, it is believed the veneer is partially grouted with vertical reinforcement, with fixings back to the top of the timber framed walls. The remainder of the perimeter walls are clad externally with horizontal weatherboard.

The internal separation walls between the units, which run in the north-south direction, consist of 200mm thick concrete block walls, and are believed to be fully grouted and reinforced. On the north side of the building the separation walls extend past the face of the building to form the exterior 'wing' walls, which separate the exterior patio spaces provided for each unit. On the far east and west sides of the building, the 200mm thick wing walls interlock with the 100mm thick concrete block veneer along these lines.

At the centre of each unit there is a short 140mm thick concrete block return wall, which extends to the west off of the centre separation wall. These walls extend just above the ceiling line and are also assumed to be fully grouted and reinforced. The remainder of the internal walls are timber framed and lined with gypsum plasterboard.

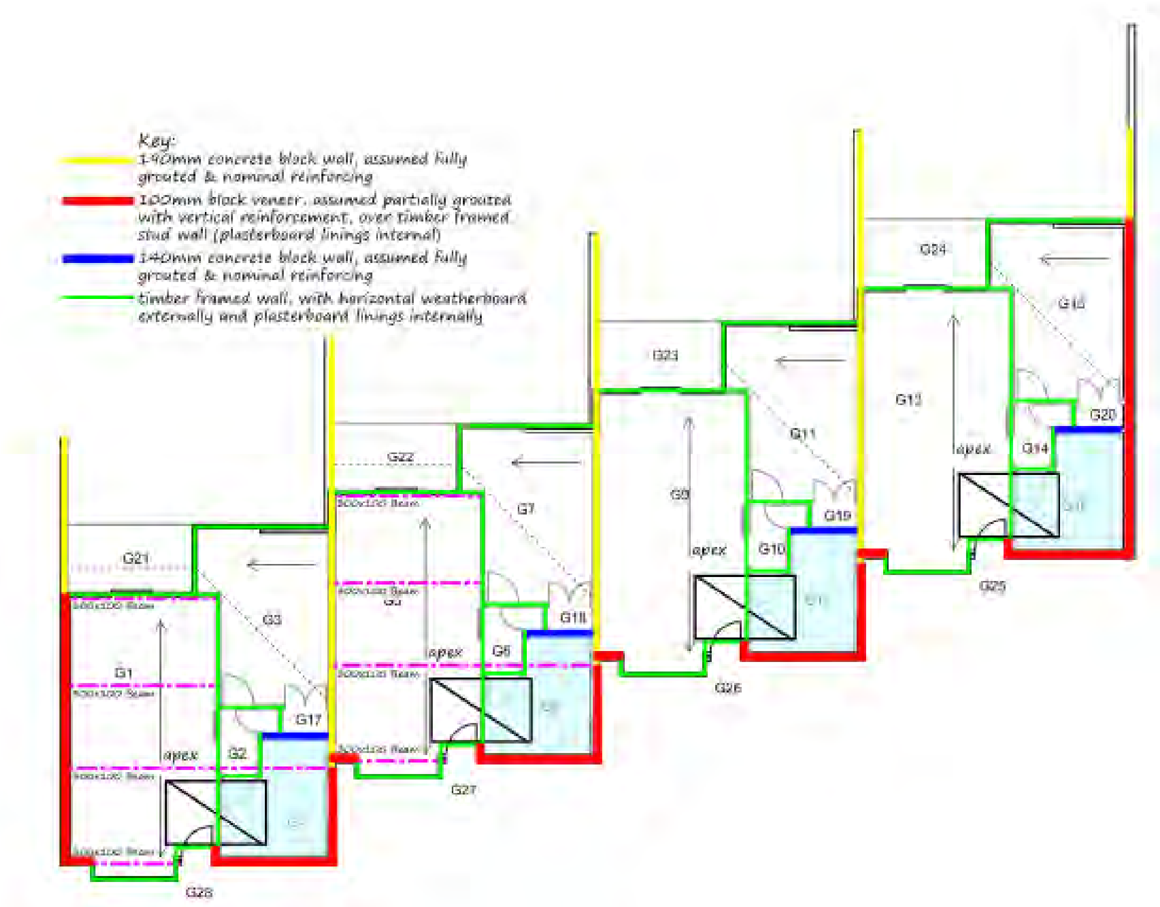


Figure 2-3: Ground Floor Plan – Wall Types

All load bearing walls within the building are assumed to be supported on reinforced concrete strip footings, or slab thickenings, while the ground floor comprises of an in-situ concrete slab on grade. The main slab of the building has not been tied in to the concrete paving at the front and rear of the building.

2.2 LATERAL LOAD RESISTING SYSTEMS

The lateral-force resisting system for the Tapper Units, in the north-south and east-west directions, consists of a combination of flexible roof diaphragms and wall bracing elements. The roof diaphragms are formed by the plywood roof sheathing, which in general, directly distributes lateral load to the wall bracing elements below.

The wall bracing elements consist of a combination of the reinforced concrete block walls and the gypsum lined timber framed stud walls. The external block veneer and weatherboard have not been considered to contribute to the lateral bracing capacity of the building. The bathroom walls have also not been considered to provide bracing due to the lining materials used on these walls.

As the internal 140mm thick concrete block return wall is not full height, lateral loads at this location are transferred to the wall through weak axis bending of the 200mm thick concrete block separation walls. The reinforcement connection these two elements is assumed to be sufficient to transfer the required load. See Figure 2-4 below.

Figure 2-4: Block Wall Intersection

All loads from the bracing walls are transferred directly to the continuous reinforced concrete footings below.

2.3 PRE-EARTHQUAKE BUILDING CAPACITY – DIRECT CODE COMPARISON

The building capacity under earthquake actions discussed in this section is compared to the capacity that a similar building would be designed to today. A new building would be designed to the *Structural Design Actions Standard, Part 5: Earthquake Actions – New Zealand*, NZS 1170.5:2004 [11] and incorporating the amendments made to this standard as a result of the Lyttelton Earthquake as outlined in the Amendment 10 of the Building Code [8]. The implications of the recent amendments are discussed more fully in the Burwood Hospital Campus Base Report, however, for a building of this type the amendments essentially result in an increase to the design loads of 36 % when compared to pre-earthquake design levels.

The original structural drawings, calculations and specifications were unavailable for review, so the exact design and loading assumptions originally made are unknown. For the purpose of this report seismic loading assumptions have been made based on physical observations of the building.

When the Tapper Units building was originally designed in 1978, the loading standard at the time was the *New Zealand Loading Standard* – NZS4203:1976 [12]. When these By-Laws were written, neither the seismology of the different areas within New Zealand, or the impact this could have on buildings was as well understood as it is today. Along with an increase in the seismic demands required by the change in the loading code over this period, the seismic detailing requirements have also progressed significantly resulting in more ductile and better performing buildings.

The current New Zealand loading code, NZS1170.5:2004 [11], requires a new building to be designed for an earthquake, known as the Design Basis Earthquake (DBE), which is based upon the buildings physical location, local soil conditions, building type, fundamental period and Importance Level.

The Tapper Units are not regarded as an essential hospital facility by the CDHB and are therefore classified as an Importance Level 2 building, in accordance with NZS 1170:2004 [11]. The associated return period of the DBE is 500 years, with a risk factor for design of R = 1.0 (no post-disaster or special function). The sub soil class for the site is taken as Soil Type D, which is consistent with the findings of the post-earthquake geotechnical investigation [3].

Based upon the period of construction the concrete masonry walls have been assumed to have nominal ductility, and as such have been assigned a ductility factor of μ =1.25. The timber framed bracing walls have been assigned a ductility factor of μ=3.3 based on the existing properties of the wall linings and the fixings to the foundation elements below.

A comparison between the Design Basis Earthquake of NZS4203:1976 [12] and NZS 1170.5:2004 [11] for the site is plotted below in Figure 2-5. Based upon a fundamental building period below 0.40 seconds, the seismic demands on the structure have increased by approximately 10% since 1978.

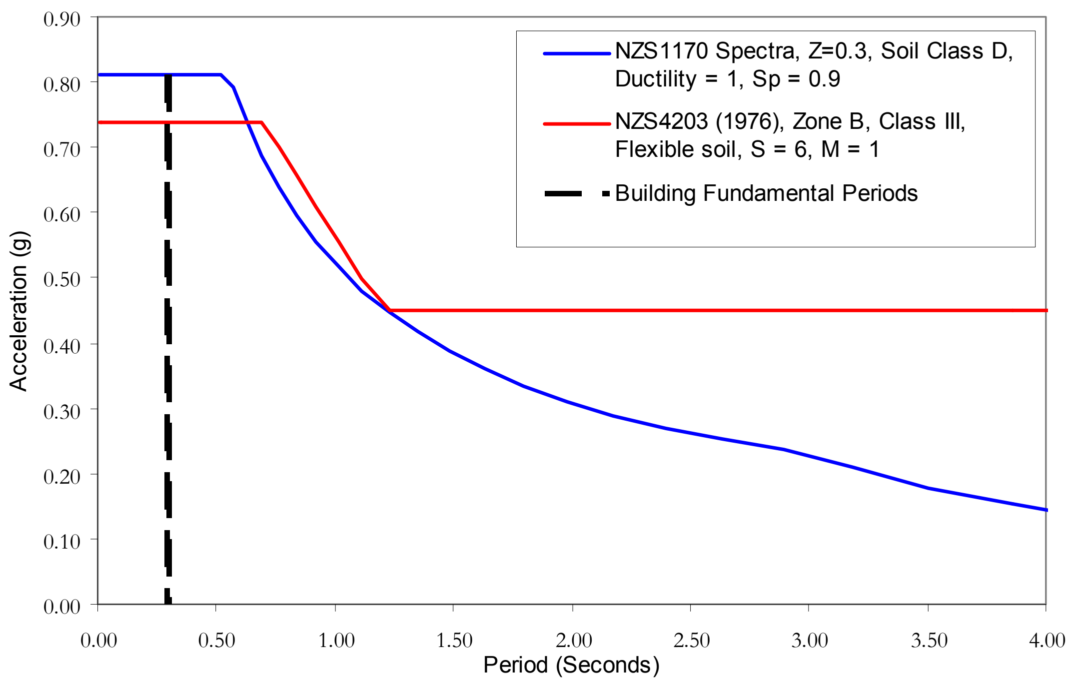


Figure 2-5: Comparison of Design Codes

2.4 EQUIVALENT STATIC ANALYSIS TO NZS1170.5 (2004)

In addition to the direct code comparison provided above, an equivalent static analysis to NZS1170.5: 2004 [11] has been carried out to gain a better understanding of the buildings estimated capacity when compared to current loading standards. The equivalent static analysis was carried out based upon floor plans, site measurements and as built observations.

Following the Lyttelton earthquake a geotechnical report was conducted by Tonkin & Taylor titled "Burwood Hospital Post Earthquake Geotechnical Assessment", dated June 2011 [3]. This reports has been used to aid in the evaluation of the site conditions and the likely effect of the ground on the buildings past and future performance. The soil parameters described in the report have also been used for the evaluation of the buildings existing foundation system.

The probable capacities of the structural elements 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 [1], Timber-framed buildings – NZS 3604:2011 [13], *Concrete Masonry Buildings Not Requiring Specific Engineering Design* – NZS 4229:1999 [14], *Design of Reinforced Masonry Structures* – NZS 1900:1964 [15] and *Historical Review of Masonry Standards in New Zealand* [17]. 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. These are described in more detail in the Burwood Hospital Base report [1] and include strength governed elements such as short columns and deflection governed elements such as floor and stair elements with inadequate support seating.

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 (DBE), and have been expressed as a %DBE. This includes checks for both the strength and deflection requirements.

For the purpose of this evaluation several assumptions had to be made in regards to the material properties of the existing building elements. The expected strength and ductility values for these elements were taken from NZSEE 2006, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes* [16] and ASCE 41-06, *Seismic Rehabilitation of Existing Buildings* [18], along with some degree of engineering judgement. These values could be further refined through destructive investigations of the existing building elements. The assumed expected strength and ductility for the various building elements is as follows:

- Concrete Block Walls: The 140mm and 190mm thick concrete block walls have been assumed to be fully grouted and reinforced to the minimum requirements at the time of construction. Effective compressive strength, $f'm$ = 4 MPa with ductility, $\mu = 1.25$.

- Interior Timber Framed Walls: Unblocked stud walls with gypsum plasterboard linings on two sides. Expected strength = 3.0 kN/m with ductility, $\mu = 3.3$.

- Exterior Timber Framed Walls: Unblocked stud walls with gypsum plasterboard linings on interior face. Exterior veneer/weatherboard does not contribute to assumed bracing capacity. Expected strength = 1.5 kN/m with ductility, $\mu = 3.3$.

- Roof Diaphragms: Plywood sheathing over unblocked roof purlins. Expected strength = 6.0 kN/m with ductility, $\mu = 3.5$.

The assessment of the buildings indicated that in the north-south direction the assessed lateral capacity of both the roof diaphragm and the ground floor wall bracing to be approximately 100% DBE. This is attributed to the regular spacing of both the reinforced masonry walls and the timber framed walls in the north-south direction.

In the east-west direction the roof diaphragm has been assessed at approximately 80% DBE. The ground floor bracing walls have been assessed at approximately 20% DBE due to the limited number of bracings walls, particularly on the north side of the units.

Both the 190mm concrete masonry walls and the 140mm concrete masonry walls have been assessed at 100% DBE under face loading.

As no foundation information is available these elements have not been included in this assessment.

A summary of the capacity of each primary lateral element as a percentage of the demand imposed by the Design Basis Earthquake (DBE) have been noted in Table 2-1.

Building Element	**%DBE (IL2)**	**Comments**
Roof Diaphragm – N-S Direction	100%	
Roof Diaphragm – E-W Direction	80%	Limited diaphragm capacity and by spacing of bracing walls in the east-west direction
Ground Floor Bracing Walls N-S Direction	100%	Including out-of-plane capacity of the 140mm Block Wall – Assuming walls are fully grouted and minimum required reinforcement
Ground Floor Bracing Walls E-W Direction	20%	Limited by the number of adequate bracing walls in the east-west direction
Inter-tenancy Walls – Out of Plane Capacity	100%	Assuming walls are fully grouted and minimum required reinforcement

Table 2-1: Seismic Assessment %DBE

If the building were to be assessed for an increased importance factor, IL3, the seismic demand would increase by 30% (R=1.3) and as such the assessed capacities would be reduced proportionally.

A review of the drawings available and site observations revealed no obvious critical structural weaknesses (CSW's) that could lead to premature collapse of the building.

3. POST-EARTHQUAKE BUILDING CONDITION

This section covers the structural damage sustained by the Tapper Units, along with the associated reduction in lateral load resisting capacity, as a result of the series of earthquakes that includes the Darfield Earthquake that struck at 4:36am on 4th September, 2010 and the Lyttelton Earthquake that struck at 12:51pm on the 22nd February, 2011; the June Earthquake that struck at 2.20pm on the 13th of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011.

3.1 THE LYTTELTON EARTHQUAKE

The fundamental period of the building is estimated to be between 0.2 and 0.4 seconds. 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 strong motion data presented in the Base Report [1], it appears the Lyttelton Earthquake produced ground shaking intensities of approximately 60-120% of the current Ultimate Limit State design spectra for an Importance Level 2 building of nominal ductility. The majority of the earthquake damage observed, or at least the onset of damage, appears to be as a result of this earthquake.

It should be noted that the Lyttelton Earthquake was very short in terms of strong shaking produced, with the strong motion only lasting for a duration of 5-7 seconds. Rupture of an alpine fault is expected to contain 50 to 60 seconds of strong motion.

3.2 PRELIMINARY INVESTIGATIONS

Preliminary investigations were carried out to identify 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
- review of available documentation (only a room numbering plan available)
- review of previous Holmes Consulting Group assessments on the building [6,7]
- damage observed during 'Rapid' Structural Assessments following the Lyttelton Earthquake, June 13th aftershocks and subsequent events

Following the review of the drawings, and previous work associated with this building, the following areas were identified for potential damage:

- movement or damage to structure associated with ground movement and/or settlement

- cracking to foundation slab and continuous concrete footings due to earthquake induced differential settlement

- cracking to concrete block walls

- distress at connection of timber roof framing to block walls

- distress and cracking of gypsum clad bracing walls

- distress and cracking of gypsum clad ceiling diaphragm

- damage to fixings at base of timber framed bracing walls

- cracking to linings of timber framed walls and ceilings

- damage to exterior block veneer and associated fixings to perimeter walls

- displacement of ground around perimeter of building.

Rapid Level 2 assessments were carried out on 24th February 2011[18] and on 15th June [19] following the June 13th earthquakes. The structural observations involved a complete walk around the exterior and throughout the interior of the building. The following primary areas of damage were identified from the damage assessments:

- cracking in exterior block veneer above entry to western most unit

- severe cracking, separation and differential settlement of exterior paving slabs around the building

A review of the above information on the building type and preliminary observations highlighted this building as requiring a detailed inspection. The aim of the detailed inspection was to determine the cause and full extent of damage to the building, particularly the elements identified for potential damage above. These items were targeted to identify if damage had occurred and to what extent the damage had reduced the capacity of the buildings lateral load resisting system to withstand future seismic events.

3.3 DETAILED OBSERVATIONS

Further detailed inspections and structural explorations have been carried out following the initial assessments to ascertain the full extent of structural damage. A detailed structural observation was completed on the 3rd May, 2012.

A full record of these observations can be found in Appendix A. Reference plans describing the location labelling can be found in Appendix B. Full photographic records of the observations are available electronically on request. The detailed structural observation identified the following additional damage to those items noted in the initial rapid assessments:

- cracking of internal linings both in the ceiling and walls, particularly in and around wall/ceiling joints

- separation between gypsum walls, ceilings and the concrete block separation walls

- separation between external walls and eave linings

It should be noted that during these observations any damage sustained by the concrete floor could not be examined due to the existing floor coverings. The differential settlement that the building experienced has likely caused cracking and damage to the existing slab. In order to record damage noted to these areas an intrusive exploration is required in which the floor linings will be required to be removed.

3.4 GEOTECHNICAL REVIEW

A review of the ground damage and conditions was carried out by Tonkin & Taylor for the Burwood Hospital Campus. A subsequent report titled Burwood Hospital Post Earthquake Geotechnical Assessment was issued in June 2011 [3]. The geotechnical review concluded that the settlement and damage to building foundations on the Burwood Hospital Campus was likely due to the liquefaction of underlying soil layers. It is believed that excessive pore water pressures have dissipated and that further settlement is not expected to occur, unless another significant event was to occur.

Based on the geotechnical report provided by Tonkin & Taylor the potential for future total and differential settlements at the building site varies between 0 to 20mm for a SLS event, and between 160 to 250mm for an ULS event.

3.5 LEVEL SURVEY

A detailed survey of the ground floor levels in the Tapper Units was conducted by Fox & Associates and issued on 18th April, 2012 [5]. The survey indicates a permanent slope in the ground floor from west to east along the length of the building. The total measured drop in the ground floor is approximately 83mm. The worst case localized drop in the elevation of the ground floor slab is approximately 19mm over a 3.8 m length, resulting in 1:180 slope in the ground floor slab (0.55%).

Based upon observations on site, including the extensive ground damage in the direct vicinity of the building, it is believed the slopes in the ground floor slab are primarily earthquake induced.

Figure 3-1: Tapper Units – Level Survey

The total differential settlement noted across the footprint of the building, and the associated slope in the slab on grade, exceeds the minimum typical acceptable value for standard occupancy buildings. As a result remediate of the floor levels is likely required to restore the functionality of the building. A discussion on the options available to re-level the building have been included in Section 4.2.

For the full extent of differential settlement noted to the building see Appendix C: Survey of Levels.

3.6 SUMMARY OF BUILDING DAMAGE (DARFIELD EARTHQUAKE)

The following is a summary of the observations made for the building, and our conclusions as to their condition and seismic load resisting capacity. These observations do not specifically distinguish between damage caused by the Darfield Earthquake, the Lyttelton Earthquake or any significant aftershocks, such as those that occurred on the 13th June 2011, .the 23rd December 2011 or the 2nd January 2012. Despite not being able to specifically distinguish when individual building damage observed occurred, it is believed that the majority of the damaged, or at least the onset of damage, can be linked to the February 22nd event.

The Tapper Units appear to have performed as would be expected for a building of this type and age considering the seismic actions and differential ground settlement experienced at the site. While the ground damage surrounding the building is relatively severe the observed damage to the building itself can be considered minor to moderate. The bulk of structural damage is typified by separation of the linings at the interface with block walls and cracking of the linings on the timber framed walls and ceilings.

A summary of the typical damage observed is as follows:

- **Ground Movement** – Differential ground settlement has been noted around Tapper Units building including severe cracking and separation of exterior paving slabs around the building. A total drop of 83mm has also been noted in the building slab on grade resulting in a worst case slope in the slab of 1:180. *The surface of the slab on grade nor the foundations were visible for observation.*

- **Cracking to Wall & Ceiling Linings -** Cracking to internal wall and ceiling linings, primarily at wall/ceiling board joints, wall/ceiling intersections and off corners of openings.

- **Separation at Concrete Block Wall** – Separation of inframing timber walls and ceiling diaphragms has been noted at the interface with the internal concrete block separation walls. Likewise separation has been noted between the eave soffit and the exterior walls.

- **Non-structural -** Cracking to non-structural elements such as door jambs and paint finishing.

Table 4-1 provides a photographic summary of the typical damage observed.

3.7 ADDITIONAL INVESTIGATIONS REQUIRED

Several assumptions were made in the completion of the Pre-earthquake (undamaged state) and Post-earthquake (damaged state) Structural Assessments. Destructive exploration is required in a number of locations in order to verify these assumptions.

3.7.1 Investigations Required For Further Assessment

The areas requiring further investigation to finalize the assessments are as follows:

- Scan block walls to confirm assumed reinforcing. This includes the interior concrete block walls, exterior concrete block 'wing' walls and the exterior block veneer. All items should be checked for typical reinforcing size and spacing in the horizontal and vertical directions.

  The block walls were scanned in six locations around the buildings and 'sounded' to check that the blocks are filled. Generally the bars were found to be at 400mm centres both horizontally and vertically. The diameter of the bars appeared to vary from 8mm to 20mm, though the majority were 16mm. Five out of six of the tested walls were found to be fully filled, though one small section in the middle of a wall was found to be unfilled. As the reinforcing size is greater and spacing smaller than initially assumed, there is no change to the reported capacities.

- Investigate fixings between timber bracing walls and concrete block separation walls.

  The fixings between the walls are made with 75mm Ramset gun nails at 600mm centres. This means there is at least 25mm penetration into the concrete block. This is sufficient connection to validate initial assumptions so there is no change to the calculated capacities.

- Check existing connection of timber roof beams to masonry wall for damage. These connections are either concealed or too high to easily be viewed.

  No damage to the investigated connection was found.

- Validate roof diaphragm and roof framing fixings to top of internal and external timber bracing walls and to the top of internal concrete block separation walls.

  The roof diaphragm consists of both 12mm plywood and 10mm gib board in separate areas. The plywood, over the bedrooms and bathrooms is fixed directly to the top of the rafters. The gib board, over the kitchen area and living areas, is also fixed to the top of the rafters. The fixings of these linings cannot be confirmed without lifting the roof.

- At the base of internal and external timber framed stud walls, determine the typical type, size and spacing of fixings to concrete slab/foundation elements below.

  The bottom plate of the internal and external walls are fixed to the concrete slab with 75mm Ramset gun nails at approximately 600mm centres. This confirms the initial assumptions so there is no change to the calculated capacities.

- A specialist assessment of the exterior block veneer is recommended to be completed by a qualified mason to review for damage and provide repair recommendations as required. This includes a review of the fixings of the block veneer to the exterior walls.

  The report by Simon Thelning [21] shows the block veneer to be in very good condition with minor repair work to be undertaken to mortar joints. The fixings of the block veneer back to the exterior walls are at 400mm centres vertically and 800mm horizontally.

3.7.2 Investigations to be Completed During Building Repairs/Strengthening

- Remove floor coverings to review damage to concrete slab on grade.

- Validate all existing timber stud wall framing and fixings to concrete slabs below where new wall linings are to be installed.

- Validate collector connection at top of wall at re-entrant corner on the north side of each unit.

- Re-inspection of building will be required upon completion of any re-levelling works, to determine if any additional damage has occurred.

3.8 POST-EARTHQUAKE BUILDING CAPACITY

Based upon our observations to date, we do not consider the Tapper Units to have any significant reduction in gravity load resistance. The damage observed to the interior wall linings on the timber bracing walls will have resulted in some reduction in lateral load capacity, although it is difficult to quantify the percentage reduction in strength. While there has been some reduction in strength, according to the Department of Building and Housings, *Revised Guidance on Repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence*, the primarily result of the damage noted will be a reduction in the stiffness of the wall bracing. The reduction in stiffness will be ongoing concerns in the east-west direction in regards to the buildings performance, primarily to building contents and non-structural elements.

The movement noted in the slab on grade is not believed to have significantly affected the existing capacity of the building. The building deformation due to the differential settlement will have resulted in some reduction in capacity, but again this is difficult to quantify. The primary concern will be a reduced ability of the building to absorb future differential settlements prior to the onset of more severe damage to the foundations and superstructure of the buildings.

The damage observed will require repair to restore the strength, stiffness, durability and performance of lateral bracing system. The differential settlement noted will also require re-levelling to restore the serviceability of the building. The repair work is outlined in Section 4. Following the recommended repairs the lateral load resisting performance of the existing structure will be restored close to the pre-earthquake levels, which are summarised in Section 2.4.

In its pre-earthquake and post-earthquake condition, the Tapper Units have been assessed at a capacity below 33% of the load imposed by the current loading standards DBE. As a result of being below 33% DBE the Tapper Units are considered to be "Earthquake Prone" in terms of section 122 of the Building Act. Christchurch City Council current policy requires that buildings identified as "Earthquake Prone" be strengthened to 67% of current code requirements when seeking consent for repairs, which is the minimum strengthening we would recommend.

Despite the low % DBE, as a primarily light weight timber framed building, with natural built in redundancies, the building is unlikely to fail in a brittle manner.

Recommendations for strengthening to improve seismic performance and bring the building to above 67% DBE are included in Section 5.

4. DAMAGE OBSERVED & REPAIRS REQUIRED

4.1 PRIMARY DAMAGE OBSERVED AND REPAIRS REQUIRED

This section covers the damage noted during our detailed assessment of the building. Note that our observations have been restricted to structural aspects of the building 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. At the time of inspection the ground floor slab in all units were covered with floor linings. As such any damage to the ground floor slabs of the building will be recorded after works have begun and the floor finishes are removed.

Table 4-1 provides a photographic summary of the observed damage and typical repairs required. The table should be read in conjunction with Appendix A – Record of Observations and Appendix B – Location Reference Plans. The Repair Specification [2] referred to in Table 4-1 has been issued separately.

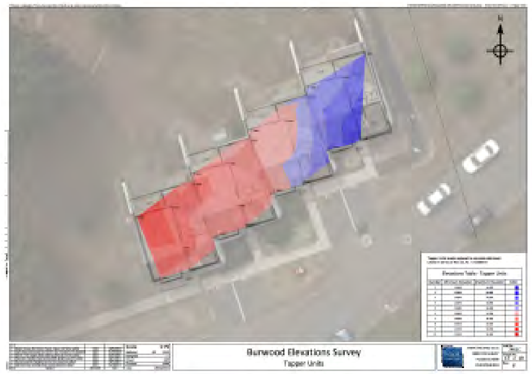
In general, the aim of the repair work indicated in this section is to restore the structure to its pre-earthquake state, as far as practicable, while maintaining the utility of the building. The repairs presented attempt to address the loss of strength, stiffness and durability of the structural elements due to the damage noted.

Based upon the low % DBE of the building in its pre-earthquake, undamaged state, we would recommend that any repairs be combined with a strengthening scheme to improve the performance of the building. Further recommendations for improvement to the buildings seismic performance, and to achieve a minimum capacity of 67% DBE have been included in Section 5.

It should be noted that more damage may be identified during the repair works and (if required) additional repair details will be specified accordingly.

Please note that if building is to be re-levelled, all repair works are to be completed after the building has been re-levelled to a satisfactory condition as further damage to the wall and ceiling linings can be expected during the re-levelling process.

Table 4-1: Tapper Units - Photographic Summary of Primary Damage Observed & Repairs Required

Damaged Item & Location	Damage	Recommended Repair	Example Photograph
1. Foundations			
1.1. Differential Ground Settlement	Earthquake induced differential settlement has resulted in a permanent slope in the concrete slab on grade of up to 1:180.	Remediation of floor levels requires re-levelling of the structure. See Section 4.2 for additional information.	
2. Ground Floor Walls			
2.1. Timber Framed Bracing Walls	Cracking at existing wall board joints	Replace damaged wall boards with new gypsum plasterboard linings. All wall boards to remain which are assumed to provide lateral bracings are to be re-fixed. *All remaining lining repairs are to be specified by others*. For additional information see Section 4.3.	

Damaged Item & Location	Damage	Recommended Repair	Example Photograph
2.2. Timber Wall / Block Wall Interface	Separation at interface of timber stud wall with internal concrete block separation wall.	*Further investigation required of fixings between timber framed stud wall and concrete block separation wall.*	

Damaged Item & Location	Damage	Recommended Repair	Example Photograph
3. Eave and Ceiling Linings			
3.1. Gypsum board ceiling linings	Cracking of ceiling linings, particularly off corners of openings. Separation of ceiling linings at interface with interior bracing walls.	Ceiling linings have not been assumed to provide bracing. *Repair specification to by others.*	  

Damaged Item & Location	Damage	Recommended Repair	Example Photograph
3.2 Exterior Eave Soffits	Separation of eave soffit lining and external walls	Any damaged soffit linings are to be replaced and re-fixed. Prior to repair of the soffit check the existing connection of adjacent timber roof beams to masonry wall for damage.	 

Damaged Item & Location	Damage	Recommended Repair	Example Photograph
4. Ground Damage			
4.1 Exterior Paving	Differential settlement and separation between external concrete paving and main building slab. Located at entrance and rear patio area, typical for all units	Repair specification by others	  

Damaged Item & Location	Damage	Recommended Repair	Example Photograph
4.2 Exterior Walkways	Severe cracking in exterior walkways and parking area.	Repair specification by others.	 

4.2 DISCUSSION ON BUILDING RE-LEVELLING

The level survey, completed by Fox & Associates [5], has indicated several areas of the building which contain a permanent slope in the ground floor slab. In general the slab on grade slopes downward from west to east along the length of the building, with a worst case measured slope in the ground floor slab of approximately 1:180 (0.55%). The total drop in the slab on grade over the length of the building is approximately 83mm.

The total differential settlement noted across the footprint of the building, and the associated slope in the slab on grade, exceeds the minimum typical acceptable value for standard occupancy buildings. As a result remediate of the floor levels is likely required to restore the functionality of the building.

Besides the slopes noted in the ground floor framing, the differential settlements observed will have resulted in some reduction in the capacity of the building, along with a reduction in the buildings ability to undergo future differential settlements before the onset of more severe damage.

The permanent slopes in the ground floor slab can be remediated through re-levelling of the building. This would entail lifted the entire structure up to the highest point located in the north-west corner of the building. For the extent of the proposed re-levelling see Figure 4-1 below.

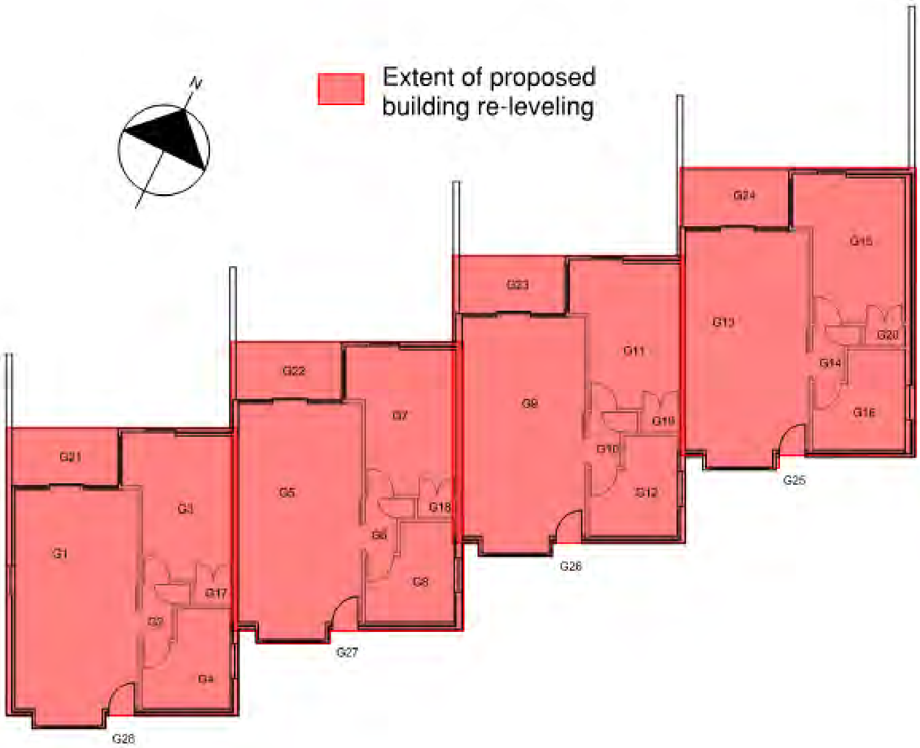


Figure 4-1: Foundation Plan – Re-Levelling Repairs

The two primary re-levelling options available include the use of either mechanical jacking or underpinning grout. There are advantages and disadvantages of each solution which extend beyond structural performance, and need to be considered by CDHB. These include continuity of operation, degree of re-levelling accuracy, risk of damage to existing foundation system and/or superstructure, and the willingness of the re-levelling sub-contractor to provide a producer statement, amongst other items.

From a structural standpoint, either option is acceptable provided the use of underpinning grout does not create any "hard points" under the building. If "hard points" are created during the re-levelling process the potential for future differential settlements can be increased. If this were to occur it would reduce the capacity of the building going forward.

Based upon the preliminary information provided by Tonkin & Taylor the soil profile throughout the Burwood Hospital (medium dense sand overlying dense sand) lends itself to localized lifting through underpinning grout techniques and should not create any undesirable "hard points" as described above. This concern should be further remediated as the re-levelling process will likely require the installation of underpinning grout under the entire footprint of the building.

The building could also be re-levelled through the use of mechanical jacking under the existing foundations. In this scenario the existing foundations would be jacked up to level, with the void created under the footings filled with cementicious grout.

With either option it is likely the existing slab on grade will be required to be partially demolished and replaced in conjunction with the re-levelling of the existing foundations. This is due to access requirements to the underside of the foundations. It is also likely that parts of the interior fit out will be required to be demolished and replaced as well.

It should be noted that neither option is expected to increase the seismic performance of the building or reduce the potential for future differential settlements. Instead the options presented are designed to re-level the building without making the future performance of the building any worse than it was prior to the earthquakes. To improve the future performance of the building, and reduce the potential for future differential settlements, would likely require the entire footprint of the building to be either piled or the ground under all the existing footings improved to the appropriate depth. Further geotechnical investigations would be required into the type and depth of ground improvement required.

During the re-levelling process there is also the risk that additional damage could occur to the building linings, exterior block veneer, etc. Appropriate contingencies should be provided.

The suitability of re-levelling the building through the use of either mechanical jacking or underpinning grout will need to be verified by a qualified sub-contractor in conjunction with the geotechnical consultant.

4.3 REPAIR OF WALL LININGS

The interior wall linings to the timber framed bracing walls have been damaged in locations and require repair. Based upon the movement observed it is believed the wall lining fixings have been damaged throughout. This has resulted in a reduction to the ongoing strength and stiffness of all the bracing walls.

In order to reinstate the pre-earthquake strength and stiffness to the bracing walls, the repair recommendation at these walls is to remove all cracked or damaged sections of the wall linings and replace them with new gypsum board sheathing. The new gypsum board sheathing is to be fixed in accordance with GIB 'ezybrace' GS2-N specifications (or equivalent). A new finish is then to be applied to all interior walls.

The repair recommendation for non-bracing walls is to be specified by others.

All repairs to wall bracing are to be completed after any re-levelling to the building has been completed.

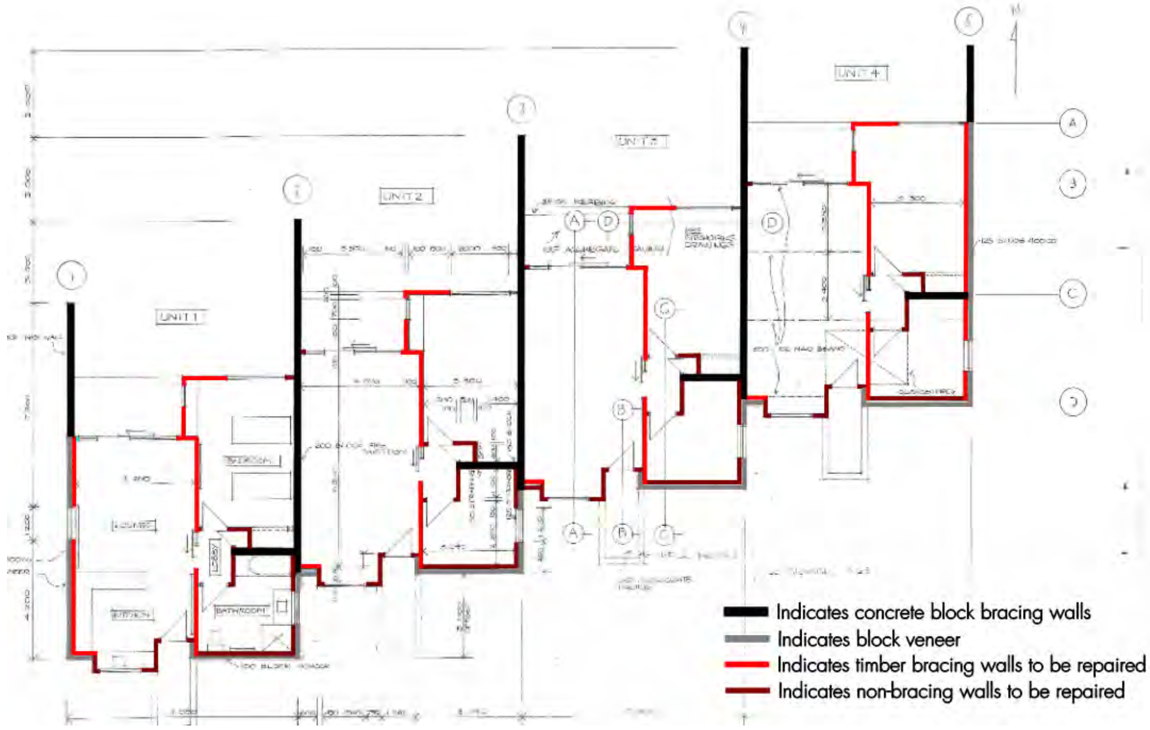


Figure 4-2: Extent of Timber Frame Wall Repairs

5. STRENGTHENING REQUIRED

The primary lateral load resisting system for the Tapper Units is consists of a plywood roof diaphragm which transfers lateral loads to the bracing walls below. The ground floor bracing walls are a combination of plasterboard clad timber stud walls and reinforced concrete block walls.

As noted in Section 2, the lateral load resisting capacity of the building in its Pre-Earthquake condition has been evaluated as a percentage of the loads imposed by the Design Basis Earthquake. Based upon the results of the evaluation, the Tapper Units building have been assessed at approximately 20% DBE in the east-west direction and approximately 100% in the north-south direction. The limiting factor on the assessed capacity of the building is the amount of bracing walls provided in east-west direction, particularly on the north side of the building.

Provided the repairs in Section 4 are implemented the buildings capacity will be restored to approximately pre-earthquake levels.

Upon completion of the repairs the assessed capacity of the building will still be below 33% DBE, and therefore still considered 'Earthquake Prone'. The Christchurch City Council Earthquake Prone Building Policy requires that applications for Building Consents, for repairs, ensure structural strength. The policy also requires that earthquake prone buildings be strengthened to resist a target of 67% of the new code loads.

Irrespective of the council requirements, we recommend that if the building is repaired, strengthening is also undertaken and 67% of the current code loads should be the minimum level considered. Strengthening recommendations to achieve 67% DBE, and improve the seismic performance of the building have been included in sub-sections below.

5.1 STRENGTHENING WORKS TO ACHIEVE 33% & 67% DBE

Based on the limited number and location of the lateral load resisting elements in the east-west direction, we recommend that the existing timber bracing walls on the north and south ends of each unit be strengthened. In addition we would recommend that a more direct load path be provided to the top of the internal 140mm concrete block return walls.

The difference of the additional wall bracing required to achieve 33% and 67% DBE is minimal, therefore we recommend that any strengthening implemented target 67% DBE.

In order to achieve the 67% DBE target the following strengthening measures are recommended:

Additional Ground-Floor Wall Bracing

The proposed timber bracing wall locations to be strengthened have been included in Figure 5-2 below. The new linings proposed are to be plywood based upon the lateral load resisting capacity required and are to be fixed as per 'Ecoply' recommendations. These linings will likely be required to be applied on the exterior face of the building due to the presence of plumbing on the interior face of the southern wall recommended to be strengthened. On the north side of the building the internal wall linings are to be repaired as per the recommendations in Section 4.3. Additional fixings down to the foundations will be required in conjunction with the new wall linings. This includes the addition of hold-downs at either end of the bracing walls to be strengthened.

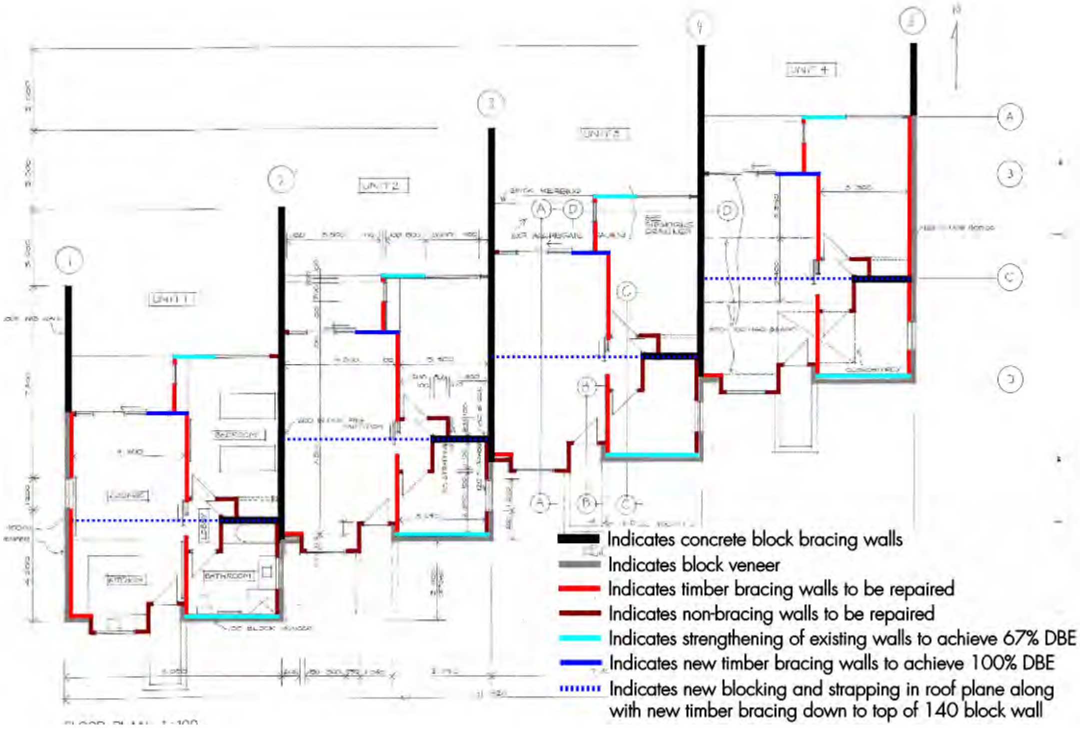


Figure 5-1: Recommended Strengthening

Additional strengthening to achieve 100% DBE could be achieved by extending the existing timber framed wall on the north side of the building, noted in Figure 5-1. This would require the removal of a window adjacent to the existing sliding glass door.

New Load Path to 140 Block Wall

Currently lateral loads are distributed to the 140mm concrete block return wall through weak axis bending of the 190mm concrete block separation wall. It is believed a more direct load path to this wall would increase the seismic performance of the building. This could be achieved by providing timber bracing up to the roof plane in conjunction with new timber blocking and strapping in the roof plane across the width of each unit (see Figure 5-1). The blocking would be installed between the existing roof rafters and the steel strap would be installed on the top side of the existing plywood roof sheathing.

6. REFERENCES

1. *CHDB – Burwood Campus – Detailed Seismic Assessment Report – Base Report*, Holmes Consulting Group, November 2011

2. *CHDB – Burwood Campus – Detailed Seismic Assessment Report – Repair Specification*, Holmes Consulting Group, November 2011

3. *Burwood Hospital Post Earthquake Geotechnical Assessment*, Tonkin and Taylor Ltd. June 2011

4. *CDHB – Tapper Units Master Floor Plan,* Maintenance and Engineering Department – Christchurch Hospital, 2009

5. *CDHB – Tapper Units Level Survey*, Fox & Associates, April 2012

6. *Burwood Hospital Campus – Seismic Risk Assessment Report*, Holmes Consulting Group, April 2002

7. *Burwood Hospital Campus – 2007 Seismic Risk Assessment Update,* Holmes Consulting Group, June 2007

8. *Compliance Document for New Zealand Building Code - Clause B1 – Structure, Amendment 10 (Canterbury)*, Department of Building and Housing, Wellington, 19 May 2011

9. Department of Building and Housing, *Revised guidance on repairing and rebuilding houses affected by the Canterbury earthquake sequence,* Wellington, November 2011

10. *Practice Note – Design of Conventional Structural Systems Following the Canterbury Earthquakes*, SESOC, December 2011

11. *Structural Design Actions Part 5: Earthquake Actions – New Zealand*, NZS 1170.5:2004, Standards New Zealand, 2004

12. *New Zealand Loading Standard NZS 4203: 1976*, Standards New Zealand

13. *New Zealand Standard Code for Timber-framed buildings, NZS 3604:2011*, New Zealand Standard

14. *Concrete Masonry Buildings Not Requiring Specific Engineering Design, NZS 4229:1999*, Standards New Zealand

15. *Masonry, NZSS 1900:1964, Chapter 6.2,* New Zealand Standard

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

17. Peter C Smith and Jonathan W Devine of Spencer Holmes Ltd, *Historical Review of Masonry Standards in New Zealand*, for the Royal Commission of Inquiry Building Failure Caused by the Canterbury Earthquakes, Aug 2011

18. *Seismic Rehabilitation of Existing Buildings,* ASCE 41-06, American Society of Civil Engineers, 2007

19. *CDHB – Burwood Hospital Campus – Rapid Seismic Assessment Post June 13th Earthquakes:* 106186.03SR01, Holmes Consulting Group, 24 February 2011

20. *CDHB – Burwood Hospital Campus – Rapid Seismic Assessment Post June 13th Earthquakes:* 106186.03SR05, Holmes Consulting Group, 15 June 2011

21. *S A Thelning Brick & Blocklayer, Burwood Hospital Earthquake Damage Report, 19 Oct 2012*

Appendix A

Record of Observations

APPENDIX A – RECORD OF OBSERVATIONS - Burwood Hospital - Tapper Units

Inspection date: 3/05/2012

KEY	
N	No repair required
Y	Repair required
F	Further investigation required
C	Repair complete

Note: At the time of inspection some repair work has previously been performed in Unit 4, post the series of Earthquakes. Also the ground floor slab was covered in floor finish and could not be examined.

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G26	Exterior	Footpath	Crack and separation between concrete footpath and building footings, at entrance. Typical for all units.	Y	Repair to be specified by others.	2
G	G23	Exterior	Footpath	Crack in patio concrete slab.	Y	Repair to be specified by others.	5
G	G23	Exterior	Footpath	Crack, separation and differential settlement between patio concrete slab and building footings, typical for all units.	Y	Repair to be specified by others.	7,8
G	G23	Exterior	Eaves Soffit Lining	separation between eaves soffit lining and timber door framing.	Y	Any damaged soffit linings are to be replaced and re-fixed.	9
G	G23	Exterior	Window framing.	separation between masonry separation wall and timber window framing/ceiling.	Y	Repair to be specified by others.	10
G	G26	Exterior	Eaves Soffit Lining	separation between eaves soffit lining and masonry veneer wall.	Y	Any damaged soffit linings are to be replaced and re-fixed.	11

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G26	Exterior	Footpath	Separation between footpath and concrete footing of building.	**Y**	Repair to be specified by others.	12
G	G10	Interior	Wall and ceiling.	Horizontal crack between wall and ceiling lining.	**Y**	At bracing walls replace all cracked or damaged wallboards with new gypsum plasterboard linings. Wall linings to remain are to be re-fixed as per the recommendations in Section 4.3. *Ceiling and non-bracing wall lining repairs are to be specified by others.*	13
G	G10	Interior	Wall and ceiling.	Horizontal crack between wall and ceiling lining.	**Y**	At bracing walls replace all cracked or damaged wallboards with new gypsum plasterboard linings. Wall linings to remain are to be re-fixed as per the recommendations in Section 4.3. *Ceiling and non-bracing wall lining repairs are to be specified by others.*	14
G	G9	Interior	Wall	Diagonal crack in wall lining around timber roof joist connection.	**Y**	At bracing walls replace all cracked or damaged wallboards with new gypsum plasterboard linings. Wall linings to remain are to be re-fixed as per the recommendations in Section 4.3. N*on-bracing wall lining repairs are to be specified by others.*	15
G	G9	Interior	Wall	Vertical crack and separation between walls corner.	**Y**	At bracing walls replace all cracked or damaged wallboards with new gypsum plasterboard linings. Wall linings to remain are to be re-fixed as per the recommendations in Section 4.3. N*on-bracing wall lining repairs, including repair at ceiling/wall seperation, are to be specified by others.*	16

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G9	Interior	Wall	separation between masonry separation wall and wall lining.	**Y**	At bracing walls replace all cracked or damaged wallboards with new gypsum plasterboard linings. Wall linings to remain are to be re-fixed as per the recommendations in Section 4.3. N*on-bracing wall lining repairs are to be specified by others.*	17
G	G22	Exterior	Wall	separation between masonry separation wall and timber window framing/ceiling.	**Y**	*Repair to be specified by others.*	18
G	G7	Interior	Wall and ceiling.	separation between ceiling panel and masonry separation wall, as well as separation and cracking between wall panels in the corner of the room.	**Y**	At bracing walls replace all cracked or damaged wallboards with new gypsum plasterboard linings. Wall linings to remain are to be re-fixed as per the recommendations in Section 4.3. N*on-bracing wall lining repairs, including repair at ceiling/wall seperation, are to be specified by others.*	19,20
G	G7	Interior	Wall	Vertical crack in wall lining above corner of window.	**Y**	At bracing walls replace all cracked or damaged wallboards with new gypsum plasterboard linings. Wall linings to remain are to be re-fixed as per the recommendations in Section 4.3. N*on-bracing wall lining repairs are to be specified by others.*	21
G	G8	Interior	Ceiling	Diagonal crack out from corner of ceiling penetration.	**Y**	*Repair to be specified by others.*	22
G	G8	Interior	Ceiling and wall.	Crack and separation between panels in ceiling penetration.	**Y**	At bracing walls replace all cracked or damaged wallboards with new gypsum plasterboard linings. Wall linings to remain are to be re-fixed as per the recommendations in Section 4.3. Ceiling and n*on-bracing wall lining repairs, including repair at ceiling/wall seperation, are to be specified by others.*	23

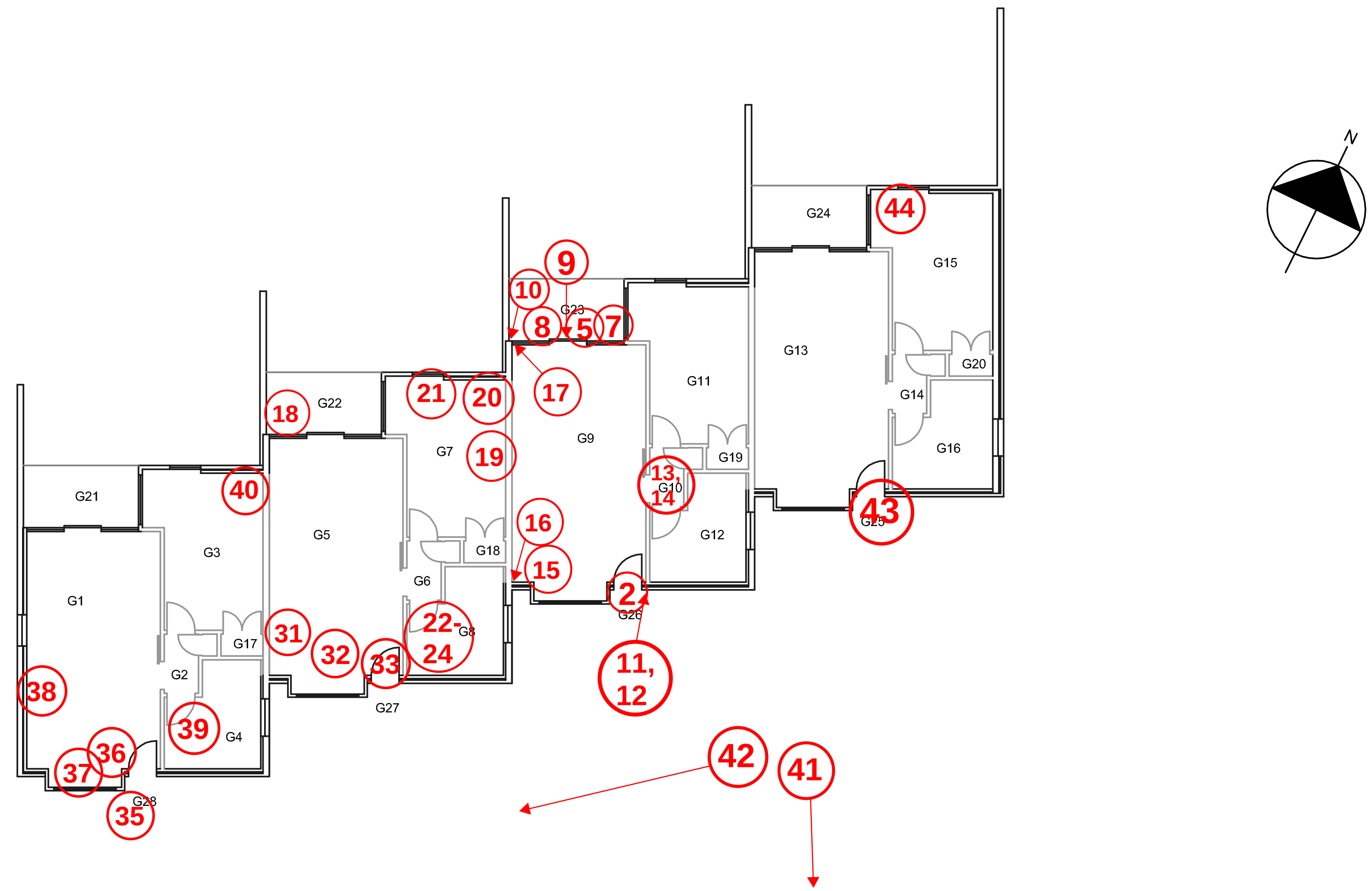
Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G8	Interior	Wall	Cracking in wall lining and framing of window in ceiling penetration.	**Y**	At bracing walls replace all cracked or damaged wallboards with new gypsum plasterboard linings. Wall linings to remain are to be re-fixed as per the recommendations in Section 4.3. N*on-bracing wall lining repairs are to be specified by others.*	24
G	G5	Interior	Wall	Vertical crack and minor separation between wall lining and masonry separation wall.	**Y**	At bracing walls replace all cracked or damaged wallboards with new gypsum plasterboard linings. Wall linings to remain are to be re-fixed as per the recommendations in Section 4.3. N*on-bracing wall lining repairs are to be specified by others.*	31
G	G5	Interior	Wall	separation between wall lining and cupboard timbers.	**Y**	At bracing walls replace all cracked or damaged wallboards with new gypsum plasterboard linings. Wall linings to remain are to be re-fixed as per the recommendations in Section 4.3. N*on-bracing wall lining repairs, including repair at ceiling/wall seperation, are to be specified by others.*	32
G	G5	Interior	Wall	Cracking and separation as well as lining peeling of wall around timber window framing, above doorway. Typical for all apartments.	**Y**	At bracing walls replace all cracked or damaged wallboards with new gypsum plasterboard linings. Wall linings to remain are to be re-fixed as per the recommendations in Section 4.3. N*on-bracing wall lining repairs are to be specified by others.*	33
G	G28	Exterior	Wall	Cracking and displacement of external weatherboard cladding.	**Y**	*Damaged weatherboard cladding is to be removed and replaced to make new.*	35

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G1	Interior	Wall	Minor separation between wall lining and timber roof joist.	**Y**	At bracing walls replace all cracked or damaged wallboards with new gypsum plasterboard linings. Wall linings to remain are to be re-fixed as per the recommendations in Section 4.3. N*on-bracing wall lining repairs, including repair at ceiling/wall seperation, are to be specified by others.*	36
G	G1	Interior	Wall and ceiling	Minor separation between wall and ceiling lining.	**Y**	At bracing walls replace all cracked or damaged wallboards with new gypsum plasterboard linings. Wall linings to remain are to be re-fixed as per the recommendations in Section 4.3. Ceiling and n*on-bracing wall lining repairs, including repair at ceiling/wall seperation, are to be specified by others.*	37
G	G1	Interior	Wall	Possible area of vertical crack in wall which has been repaired.	**N**		38
G	G4	Interior	Ceiling	Diagonal crack out from corner of ceiling penetration.	**Y**	*Repair to be specified by others.*	39
G	G3	Interior	Wall and ceiling.	Minor separation between ceiling lining and masonry separation wall.	**Y**	At bracing walls replace all cracked or damaged wallboards with new gypsum plasterboard linings. Wall linings to remain are to be re-fixed as per the recommendations in Section 4.3. Ceiling and n*on-bracing wall lining repairs, including repair at ceiling/wall seperation, are to be specified by others.*	40
G		Exterior	Footpath and parking area	Severe cracking and differential settlement of footpath and parking area.	**Y**	*Repair to be specified by others.*	41,42
G	G25	Exterior	Eaves Soffit Lining	separation between eaves soffit lining and masonry veneer.	**Y**	Any damaged soffit linings are to be replaced and re-fixed.	43

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G15	Interior	Wall	Vertical crack in wall lining above corner of window.	**Y**	At bracing walls replace all cracked or damaged wallboards with new gypsum plasterboard linings. Wall linings to remain are to be re-fixed as per the recommendations in Section 4.3. N*on-bracing wall lining repairs are to be specified by others.*	44

Appendix B

Reference Plans



AMENDMENT	DRAWN	CHECKED	DATE
MASTER DRAWING CREATED AND CDHB BORDER ADDED	BJT		17-06-09

BURWOOD HOSPITAL
TAPPER UNITS
MASTER FLOOR PLAN

Canterbury
District Health Board
Te Poari Hauora o Waitaha

DESIGNED:
DRAWN: BJT
CHECKED:
DATE: 17-06-09
XREFED DRAWING/S:

DWG No. 04142020
SCALE 1:100 AT A1
REVISED 17-06-09

Maintenance and Engineering Department - Christchurch Hospital

Appendix C

Survey of Levels

Tapper Units levels reduced to concrete slab level.
Levels in terms of Nail 22, RL = 14.698mH

Elevations Table - Tapper Units

Number	Minimum Elevation	Maximum Elevation	Color
1	14.840	14.850	
2	14.850	14.860	
3	14.860	14.870	
4	14.870	14.880	
5	14.880	14.890	
6	14.890	14.900	
7	14.900	14.910	
8	14.910	14.920	
9	14.920	14.930	
10	14.930	14.940	

A1 Original Size

No.	Revision	Approved	Date
G			
F	Beacon House, BSU Hostel, Chapel, Tapper and Milner added	MJM	18/04/2012
E	Audit observations post Dec 23 2011 (not included in this Revision)	MJM	01/02/2012
D	Wards 7 & 8, Surgical Block, Melrose Chairs & Corridor added	MJM	02/12/2011
C	Phys Med, Parafed, Food Services, BIRS, Boilerhouse, EDC added	MJM	24/11/2011
B	Nurses Hostel & Champion Centre, SOU elevations added	MJM	18/11/2011
A	Administration and Allan Bean Centre elevations added	MJM	31/10/2011

032005

Scale		1:75
Reduced	A3	1:150
Designed		----
Drawn		DHP
Checked		MJM
Date		18/04/2012

Burwood Elevations Survey

Tapper Units

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