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18 May 2018

[redacted]

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RE Official Information Act (the "Act") request CDHB 9833

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

- **The most recent Detailed Engineering Evaluation (DEE) of the Riverside, Parkside buildings at Christchurch Hospital AND for all buildings at Burwood Hospital except the new building opened in 2016.**

As requested we are providing a partial response to your Official Information Act request as follows:

Parkside
Please find attached as **Appendix 1** the Detailed Seismic Assessment Update for the Parkside Building completed by Holmes Consulting Group and dated 30 March 2016. Please note that this assessment was obtained under the previous assessment regime and the building has not yet been re-assessed in accordance with the current assessment regime (the "*The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments, July 2017 Version 1* ("NZSEE, 2017")).

Riverside
Please find attached as **Appendix 2** the Detailed Seismic Assessment Update for the Riverside Central Building completed by Holmes Consulting Group and dated 20 December 2017. As noted in the Executive Summary "Background" section on page v, this report is a reassessment in accordance with NZSEE, 2017.

We are actively working on processing the Burwood Hospital reports for you as requested and will have them to you as soon as is reasonably practicable as CDHB 9833 Part Two Response. These reports include:

- Orthopaedic Outpatients;
- Administration Building;
- Boiler House; Chapel;
- Engineering Services Building
- Nurse Hostel West;
- Orthopaedic Rehabilitation Unit;
- Physical Medicine;
- Spinal Unit;
- Surgical Services Unit and Surgical Operating Suites;
- Surgical Block;
- Maori Health;
- Birthing Unit and Minor Procedure Unit
- Milner Lodge and
- 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

DETAILED SEISMIC ASSESSMENT REPORT

STRUCTURAL AND CIVIL ENGINEERS

CHRISTCHURCH HOSPITAL

REPORT 3 – REVISION 5

PARKSIDE A, B, C & D

PREPARED FOR

CANTERBURY DISTRICT HEALTH BOARD

106186.20

30 MARCH 2016

CHRISTCHURCH HOSPITAL – DETAILED SEISMIC ASSESSMENT REPORT

REPORT 3 – PARKSIDE A, B, C & D

Prepared For:
CANTERBURY DISTRICT HEALTH BOARD

Date: 7 June 2016
Project No: 106186.20
Revision No: 5

Prepared By:	Reviewed By:
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Holmes Consulting Group LP
Christchurch Office

REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
28/10/11	1	Interim Report for Review
07/02/13	2	Report update to include results from NLTHA for Blocks A and D including Emergency Department Extension. Update to Block B and C capacities based on results of NLTHA from A and D. Update to include results of material testing, levels survey and further damage observations.
30/05/14	3	Report update to include updated results from Block A NLTHA. Update to include results from revision of material strengths, degrading column model implementation and updated scale factors.
9/10/15	4	Report update to include updated strengthening recommendations.
07/06/16	5	Report updated to include Basement Condition Survey Review and repairs completed to date.

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Holmes Consulting Group has been engaged by Canterbury District Health Board to complete a detailed structural review of the Christchurch Hospital Campus following the Lyttelton Earthquake. A series of reports have been compiled as part of this. 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, should be read in conjunction with the base report and refer to the repair specification.

This report covers the structural damage sustained by the Parkside Blocks A, B, C and D at Christchurch Hospital, 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 Lyttelton Earthquake in particular has subjected the building to strong ground motions which were up to 70 to 80 % of the full Design Basis Earthquake load for these buildings; however, it was relatively short in duration when compared to current code.

Building Form

Parkside consists of four blocks: A, B, C and D. The buildings were all designed and constructed in the mid 1980s and are separated from one another by 100mm seismic gaps. The buildings are of similar construction and the floor levels align allowing the four buildings to be used as one. Several modifications have been made since the buildings were constructed; the most significant of these was the addition of the Emergency Department Extension to the north side of Block A in 2007. For the purposes of this assessment the Parkside Blocks are deemed to be Importance Level 4 buildings, as they contain medical emergency and surgical facilities.

The Blocks are 5 and 6 storey reinforced concrete structures with a basement services space and a lightweight steel roof over. The suspended concrete floors are typically formed as flat slabs with thicker drop panels at column and wall locations. The lateral loads are resisted by internal shear wall cores. The walls and columns are generally supported on a 1.5m deep foundation raft slab. Link bridges cantilever horizontally off the face of Blocks A and B to the north linking Parkside with the Clinical Services Building. The Emergency Department Extension (EDE) is a two level steel framed structure connected to the north side of Block A. The EDE relies on Block A to resist lateral loads.

The Parkside Blocks were designed to behave in a ductile manner. Although there have been several revisions of the New Zealand Concrete Structures Standard since the Parkside Buildings were designed, they appear to have been well detailed and generally meet all detailing requirements of the current standard.

The glazed canopies to the east of Block A and the south of Blocks C and D are reported separately.

Observed Damage

Preliminary and detailed observations have been made of the damage sustained as a result of the Lyttelton earthquake.

The damage observed, typical for all 4 Blocks, included:

- Settlement damage including rotation of the buildings towards the south, horizontal offset of Block A relative to Block B (approximately 30mm to the north at the Third Floor) can be observed by offset handrail and concrete columns. Residual deformations also visible in the Lower Ground Floor of the Block B Link Bridge.
- Minor to moderate cracking of the shear walls.
- Cracking and some heaving of the basement slab.
- Cracking to slabs on grade and suspended slabs.
- Cracking to the stair flights and anecdotally the stairs have become more sensitive to vibrations.
- Minor spalling and cracking of the upper level beams and columns.
- Damage to floors and floor coverings at the seismic gap.
- Cracks to the basement tunnel walls, crawl space slab and the walls at the junction of the Blocks.
- In one location, spalling of a Lower Ground Floor column exposing rusted reinforcing.
- Damage to internal fit-out including some ceiling tiles falling out or showing signs of movement and deformation or partition walls out-of-plane.

Concrete strengths were tested in four locations on the core walls by Holmes Solutions LP. The probable concrete strength used is 40MPa below Lower Ground Floor Level and 25MPa above Lower Ground Floor Level.

Testing has also been completed by Holmes Solutions LP on the steel reinforcement crossing cracks in the Lower Ground Floor slab and shear core walls to determine the extent of strain hardening and therefore loss of potential future strain capacity. In both test locations the steel reinforcement has been shown to have a measurable loss in strain capacity, which means the ability for these elements to undergo future movement has been reduced. Some bars tested, although had a strain capacity reduction, would still meet the current code requirements for steel reinforcement bars.

Verticality and levels surveys of the buildings have been completed by Fox and Associates. The levels survey indicated that all of the Blocks have undergone differential settlement; with falls recorded from north to south in all Blocks. The levels survey indicates an overall differential settlement of 102 mm between the points measured, with a maximum slope measured of 1:180. Typical slopes across the building were in the order of 1:400. Steeper slopes were measured in the south side of Blocks A, B, C and D than the north which suggests the foundation rafts on the south side of the buildings have rotated more than the foundation rafts on the north. A differential settlement of 53 mm was recorded in the Emergency Department Extension with the east side measured lower than the west.

A geotechnical investigation was carried out by Tonkin & Taylor Ltd and it was concluded that the observed damage is likely to have been caused by residual displacements due to the dynamic loads that were applied to the ground below the building foundation during the earthquakes rather than liquefaction. The geotechnical report also concluded that:

- It is likely that the capacity of the existing foundations to resist static and seismic loads is the same as it was prior to the 22 February 2011 earthquake.

- The differential settlements and building movements may have resulted in a minor redistribution of foundation loads, but this is not believed to be significant.
- In future SLS2 level or greater earthquakes, ground damage of a similar nature to that observed from the 22 February 2011 earthquake may occur, i.e. further settlements of the foundations and rotation of the buildings.

A survey was carried out on the exterior of the building by Goleman Exterior Building Care. The earthquake damage observed included: torn sealant between precast concrete cladding panels, damage to flashings between the buildings, a couple of areas of spalled concrete on the precast cladding panels and columns, minor cracks in the panels and columns, two offset panels, settlement and damage to paths and paving and damaged soffit linings adjacent to the seismic joints.

Pre-Earthquake Building Capacity

Non-Linear Time History Analyses (NLTHA) have been completed to determine the structural capacities of Blocks A and D (including the Emergency Department Extension). The capacities for Blocks B and C were derived from an assessment of individual element capacities and a comparison with the performance of the Block A and Block D NLTHAs. The model for Block A has been updated in Revision 3 to include updated concrete strengths and degrading columns and the results were used to derive capacities for the other Blocks.

Assessments have also been made of the performance of the stairs, the precast concrete cladding panels, the potential of pounding of adjacent buildings, the performance of the link bridges and the performance of the ceilings and partition walls. In addition, the performance of the retaining walls around the building have also been assessed.

Table ES- 1 presents the capacities of the primary structural systems as well as secondary elements for the four main Parkside buildings and the EDE. It can be seen that the primary structure of the four main buildings have Ultimate Limit State (ULS) capacities of 78 % Design Basis Earthquake (DBE) loading and Collapse Limit State (CLS) capacities of 89 % DBE loading.

Table ES- 1: Summary of results for the Parkside Buildings as a percent of Design Basis Earthquake (%DBE) loading (IL4)

	Block A	Block B	Block C	Block D	(EDE)
Onset of significant damage (equivalent to the Ultimate Limit State (ULS) performance of a new building).	78 % DBE	78 % DBE	78 % DBE	78 % DBE	30 % DBE
Onset of damage that has a high probability of leading to partial or total collapse (the Collapse Limit State (CLS))	89 % DBE Margin of 1.14 over ULS	89 % DBE Margin of 1.14 over ULS	89 % DBE Margin of 1.14 over ULS	89 % DBE Margin of 1.14 over ULS	55 % DBE Margin of 1.8 over ULS
Equivalent performance relative to a new building: Onset of CLS divided by factor of safety against collapse of 1.8 i.e. CLS/1.8 (CLS/1.5)	49 % (59 %)	49 % (59 %)	49 % (59 %)	49 % (59 %)	30 % (37 %)
Precast panel connections ULS	<34 % DBE	<34% DBE	<34 % DBE	<34 % DBE	N.A.
Pounding	40 % DBE with Block B	40 % DBE with Block A	50 % DBE with School of Medicine	67 % DBE with Block C	30 % DBE with link bridge
Link Bridges ULS	33 % DBE	33 % DBE	N.A.	N.A.	N.A.
Interstorey drift > 0.5%	33% DBE	67% DBE	50% DBE	42% DBE	<30 % DBE

The onset of ULS in the primary structure of the main buildings at 78 % DBE is due to earthquake induced flexural rotations at the base of the shear core walls exceeding allowable limits.

The onset of CLS is also due to earthquake induced flexural rotations at the shear core wall bases, but is also due to some walls exceeding shear stress limits. New buildings typically have a margin between ULS and CLS of between 1.5 and 1.8; however, there is only a margin of 1.14 for these buildings. This means that the performance of the Parkside buildings, in a significant seismic event, is unlikely to be equivalent to a new building.

The ULS capacity of the Emergency Department Extension (EDE) is only 30 % DBE loading. The capacity of the EDE is limited by struts used in the roof bracing; however, there are several other elements in this extension that also have capacities less than 67 % DBE loading.

The Link Bridges that connect Parkside Blocks A and B to the Clinical Services building also have a low capacity to resist approximately 33 % DBE. It should also be noted that the assessment indicated the gravity capacity of the Block B link bridge was only 85 % of full code loading demand.

The capacity of the precast panels is limited by a combination of insufficient deformation capacity of their connections in Blocks A and C and due to the out-of-plane capacity of the connections in Blocks B and D. The precast panels, and onset of pounding, typically have a lower ULS capacity in the south side of Block A, all of Block B and the south side of Block D. This is because the interstorey deflections are estimated as larger in these areas, especially above Level 2. The deflections are larger because of the reduction in structure above Level 2 and the concrete Plant Room Level present on the south side of Block A, all of Block B and the south side of Block D.

The capacity of the stairs is not included in the above table; however, prior to the earthquakes these were rigidly connected floor to floor (remedial measures that separate the floors are underway). In their pre-earthquake condition, the inter-storey deflections in a DBE event would have caused significant damage with progressive degradation of the stairs during the earthquake shaking. The stairs were damaged in the recent earthquakes. As the stairs are required for safe egress of the building following an earthquake, in their pre-earthquake condition they were deemed to be a Critical Structural Weakness. Work is current and on-going to remediate this Critical Structural Weakness.

The capacity of the retaining walls is not included in the table above. Analysis of the retaining walls around the site show that the Parkside retaining walls have a capacity of 60% DBE IL2 level loads, while the crib wall to the emergency department carpark has a capacity of 45% DBE IL2 level loads. The lower height Hagley Ave retaining walls have a capacity of 70% DBE IL2 level loads.

The ceiling and internal partitions have been addressed separately in the Holmes Consulting Group report dated 23 May 2011 [9].

Importance Level 4 structures are also expected to meet two levels of serviceability requirements. The first of these, Serviceability Limit State 1 (SLS1), is met by the Main Block buildings, but not by the Emergency Department Extension. The second serviceability requirement, SLS2, is only required for IL4 structures and requires them to be operational after a 1 in 500 year event (equivalent to 55 % DBE loading for an IL4 structure). The Parkside buildings do not meet this criterion due to the onset of damage at lower load levels and high interstorey drifts. To limit damage to typical internal fit-outs and services to a level that would allow continued function interstorey drifts need to be less than 0.5 %. It can be seen in Table ES- 1 that interstorey drifts exceed this for Blocks A, C and D, and the EDE at load levels less than 55 % DBE loading.

If the buildings were assessed as an Importance Level 3 structure, the primary structural systems in the main buildings would be assessed as having an ULS capacity of 108 % DBE which is full capacity required for this Importance Level. However, as noted above, the margin between ULS and CLS is only 1.14 therefore, in a significant seismic event it is unlikely that the building will perform in an equivalent manner to a new IL3 building.

Post-Earthquake Building Capacity

Based on our observations of building damage and assessment of building capacity to the date of this report, we do not consider the Parkside buildings to have any significant reduction in gravity load resistance.

The lateral load resistance has been reduced relative to the pre Darfield (September 2010) capacity. This is due to the cracking to the shear core walls which has reduced the stiffness of the buildings and the loss of some strain capacity of the steel reinforcement indicated by the strain hardness testing.

Recommended Repairs

Recommendations regarding repair work required are provided in the report. Typical repair includes epoxy injection of cracks in concrete elements and repair of spalling. Discussion is provided on how to repair elements that have a reduced capacity to sustain future cyclic loading due to the potential strain hardening of steel reinforcement in these elements.

Strain hardening of steel reinforcement measured indicates a reduction in the future capacity of the buildings to sustain cyclic loading, therefore possible repair of these elements is also discussed.

Following the repairs recommended the lateral load resisting performance of all of the Blocks should be restored to close to pre-Darfield (September 2010) earthquake capacity.

Recommended Strengthening

As the stairs were identified as Critical Structural Weaknesses, a strengthening scheme has been developed which separates the stair flights between floors by introducing support at the mid-height landing. This work has been issued separately and is currently underway.

A concept strengthening scheme to improve the capacity of all parts of the building to above 67 % DBE is presented. This includes:

- Strengthening the upper levels (above level 2) to increase the stiffness and reduce deflections at these upper levels. The precast panels and stairs, even when strengthened, are not capable of resisting the current levels of interstorey drifts at the upper levels.
- Strengthening the Emergency Department Extension by adding more steel cross bracing to the roof and north wall.
- Strengthening the Link bridges by removing the precast cladding and adding a external diagonal brace back to the main buildings, as well as increasing the seismic separation between these and the Clinical Services building.
- Increasing the deformation capacity of the precast cladding panels by replacing their existing connections with ones with larger oversized holes.
- Increasing the seismic gap between the upper levels of Block B and C and the School of Medicine building.
- Increasing the margin between ULS and CLS of the shear walls by installing new confinement and additional shear capacity.
- Ensuring movement capability between the ground floor cantilevering slab and surrounding hard surface finishes at all entranceways into the Parkside buildings. Potential strengthening to the separate crib wall to the east of the building.

Discussion is also included on strengthening to higher levels and improving the Serviceability Limit State performance of the buildings.

Our damage observations have generally been visual only and limited to representative samples. Limited testing of material properties has been completed. 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.

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 detailed structural review of the Christchurch 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 Christchurch 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. The repair specification has been prepared to include repair details for typical damage observed in buildings on the Christchurch Hospital Campus and is referred to as required in the specific building reports.

1.1 SCOPE OF WORK

This report is on the Parkside Buildings – Blocks A, B, C and D at Christchurch Hospital, Riccarton Ave, 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 (CSWs) 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 June 2011 and the December Earthquake that struck at 3.18pm on the 23rd December 2011. The Lyttelton Earthquake, in particular, subjected the building to strong ground motions which approached the current code loading demand for buildings of this nature, however was relatively short in duration when compared to current code.

The capacity of each of the Parkside Buildings has been assessed relative to current code loading in both the buildings pre-earthquake undamaged states and in their post-earthquake damaged states. The post-earthquake assessments summarise the effects of the damage identified to both the gravity and lateral load resisting elements. Repair options to restore the capacity of the buildings to pre-earthquake levels for strength, durability and stiffness have been included. Where required, strengthening options have also been provided.

The glazed canopies to the east of Block A and the south of Blocks C and D are reported separately.

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 damage observations have generally been visual only and limited to representative samples, as described in our record of observations. Limited testing of material properties has been completed by Holmes Solutions LP. 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 Parkside buildings prior to the Darfield Earthquake.

The information for review included the original structural drawings [3,4,7], some of the original architectural drawings [5], the structural specification for the Emergency Department Extension [6]. Geotechnical information from the Post Earthquake geotechnical assessments has also been used to assess the pre-earthquake building condition [24,25] as have material strengths from recent testing of the concrete [18].

2.1 BUILDING FORM

Parkside consists of four blocks: A, B, C and D. The buildings were all designed and constructed in the mid 1980s and are separated from one another by 100mm seismic gaps. The buildings are of similar construction and the floor levels align allowing the four buildings to be used as one. Several modifications have been made since the buildings were constructed, the most significant of these was the addition of the Emergency Department Extension (EDE) to the north side of Block A. Figure 2-1 is an aerial view of the Parkside Buildings indicating their position relative to one another and the adjacent buildings.

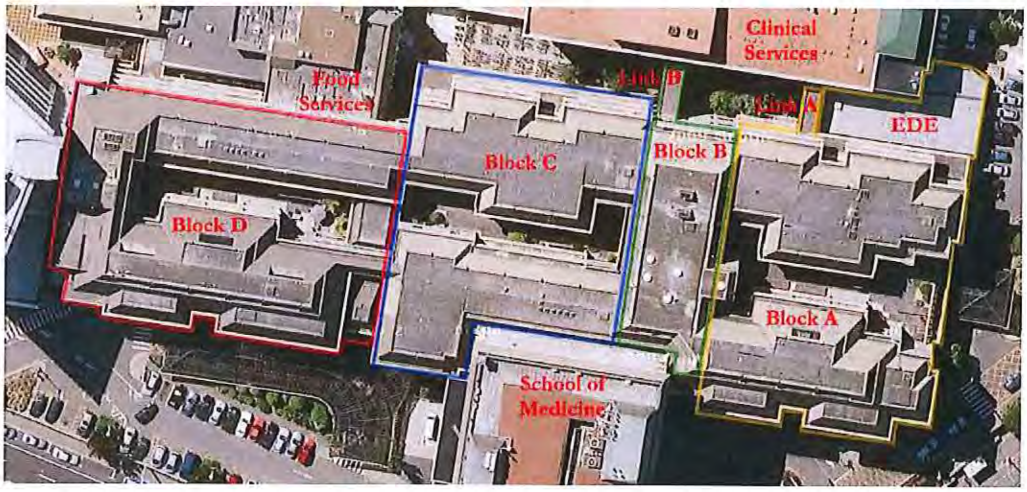


Figure 2-1: Parkside Plan indicating Building Names used in Report

For the purposes of this assessment the Parkside Blocks are deemed to be Importance Level 4 (IL4) buildings, as they contain medical emergency and surgical facilities, .i.e. they are an essential post disaster facility.

2.1.1 Block A

Block A is a 6 storey reinforced concrete structure with a basement services space and a light weight steel framed roof. The original building is approximately rectangular in plan, 41 x 43m, with a system of internal walls and columns and a perimeter frame. Above Level 2 the building is split into two wings with a void between.

On the south side of the building the ground is at Ground Floor level; however, on the north side of the building, the ground is lower and is at Lower Ground Floor level. It can be seen in Figure 2-2 that a retaining wall supports the ground on the south side of the building, there is also a retaining wall along the east side of the building. The retaining walls are not physically connected to the building at the Ground Floor level, i.e. there is a sliding detail between the Ground Floor of the building and the retaining wall such that they move independently.

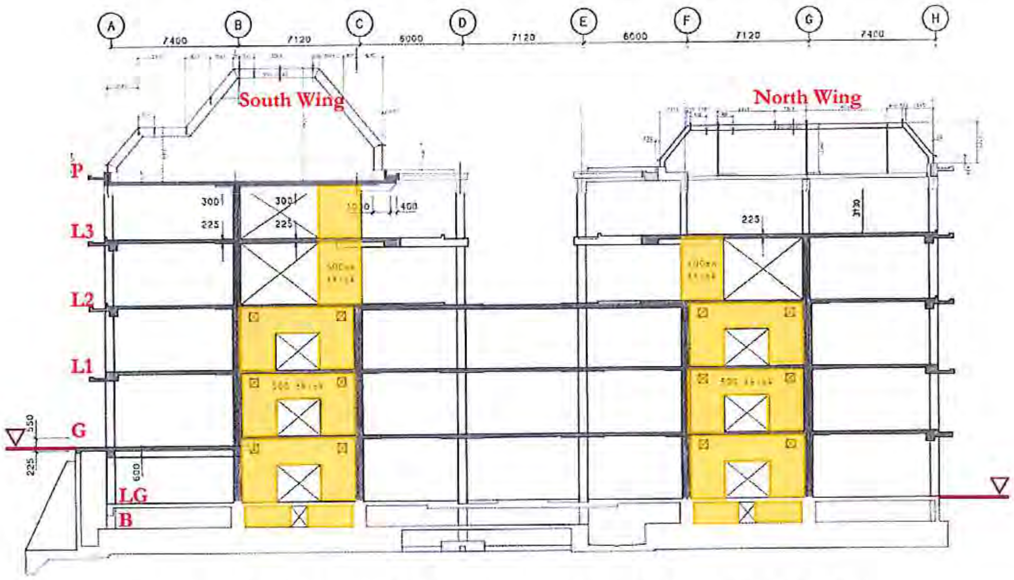


Figure 2-2: Section looking west through Parkside Block A

A link bridge corridor extends from the northern face of the building at the Ground Floor level and Level 1 to the Clinical Services building. The link bridge is separated from the Clinical Services building by a 100mm seismic gap.

On the north side of the Ground Floor there is a 30 x 16 m extension. This was designed in 2007 [7]. This addition will be referred to as the Emergency Department Extension (EDE) and is a steel framed structure. The EDE has a composite steel and concrete Ground Floor slab and light-weight steel roof. The south side of the floor slab is supported by the original Block A Ground Floor structure, and the north side by steel columns that sit on screw pile foundations.

The original building is clad with precast concrete panels that are bolted to the perimeter frame structure, except for the west side which is adjacent to Block B. The concrete panel connections typically have a slotted or oversized hole for construction tollerances. The east façade of Block A is shown in Figure 2-3.

Figure 2-3: Photo – East elevation of Parkside Block A

The suspended conventionally reinforced concrete floors are typically formed as flat slabs with thicker drop panels at column and wall locations. A beam element runs around the perimeter of the slab and around the edge of the internal voids at the upper levels. Load bearing walls and columns are generally supported on 1.5m deep foundation raft slabs. The south wing has one more level of suspended concrete floor slab than the north wing.

The steel roof system transfers lateral loads via portal frame action to the upper concrete slabs. The concrete floor slabs act as structural diaphragms to distribute the lateral loads to the shear walls.

The lateral loads are resisted by four internal shear cores. At Level 2 and below, the shear cores each consist of four coupled L-shaped walls linked to form a 7 m x 7 m square, and are founded on the 1.5m deep raft slab. Above Level 2 the walls are partially discontinued. One leg of each "L" is eliminated and the remaining leg acts as a cantilevered shear wall, creating a vertical discontinuity in building stiffness and strength. The core walls are shaded yellow in Figure 2-2.

Under lateral loading the link bridge structure cantilevers off the face of the main building using a combination of the concrete floor slab and the steel floor beams that are bolted to the perimeter frame.

The EDE relies on the main building to resist lateral loads. The Ground Floor cantilevers out from the Original Block A Ground Floor slab. There is also some steel cross bracing in the roof and in the north wall to transfer lateral loads from the north side of the roof down to the Ground Floor concrete slab.

2.1.2 Block B

Block B is a 6 storey reinforced concrete structure with a basement services space and a light weight steel framed roof. There is a small 7th level near the centre of the building which forms the Lift Machine Room. The building is approximately rectangular in plan, 17 x 44m, with a system of internal walls and columns and a perimeter frame. A section through Block B is shown in Figure 2-4.

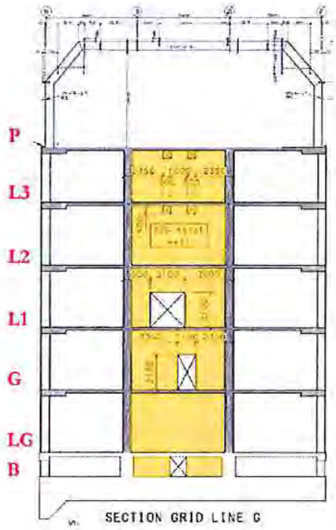


Figure 2-4: Section looking north through Parkside Block B

On the south side of the building the ground is at Ground Floor level; however, on the north side of the building, the ground is lower and is at Lower Ground Floor level.

A link bridge extends from the northern face of the building at Lower Ground, Ground Floor and Level 1 to the Clinical Services Block. The link bridge is separated from the Clinical Services Block by a 100mm seismic gap.

Block B is clad on the north face with precast concrete panels that are bolted to the perimeter frame structure. Block B is adjacent to Blocks A and C on the east and west and partially adjacent the University of Otago Christchurch School of Medicine building to the south. The floor levels align with those of the School of Medicine and access between these buildings is provided at several levels.

The suspended conventionally reinforced concrete floors are typically formed as flat slabs with thicker drop panels at column and wall locations. A beam element runs around the perimeter of the slab and around the edge of the internal voids at the upper levels. Load bearing walls and columns are generally supported on 1.5m deep foundation raft slabs.

Block B is smaller than the other Parkside buildings and therefore has only two internal shear cores (rather than four). The lateral loads are resisted by the shear cores which extend up to the underside of the Plant Room floor. The shear cores each consist of four walls linked to form a 7 m x 5 m square, and are founded on the 1.5m deep raft slabs. The shear core wall in Figure 2-4 is shaded yellow, in the east-west direction the shear walls cantilever up from the raft slabs, in the north-south direction a coupled wall system is present.

The steel roof system transfers lateral loads via portal frame action to the Plant Room Level concrete slab. The concrete floor slabs act as structural diaphragms to distribute the lateral loads to the shear walls.

Under lateral loading the link bridge structure cantilevers off the face of the main building using a combination of the concrete floor slab and the steel floor beams that are bolted to the perimeter frame.

2.1.3 Block C

Block C is a 5 storey reinforced concrete structure with a basement services space and a light weight steel framed mezzanine floor and roof. The building is approximately rectangular in plan, 45 x 42m, with a system of internal walls and columns and a perimeter frame. Above Level 2 the building is split into two wings with a void between. A section through Block C is shown in Figure 2-5.

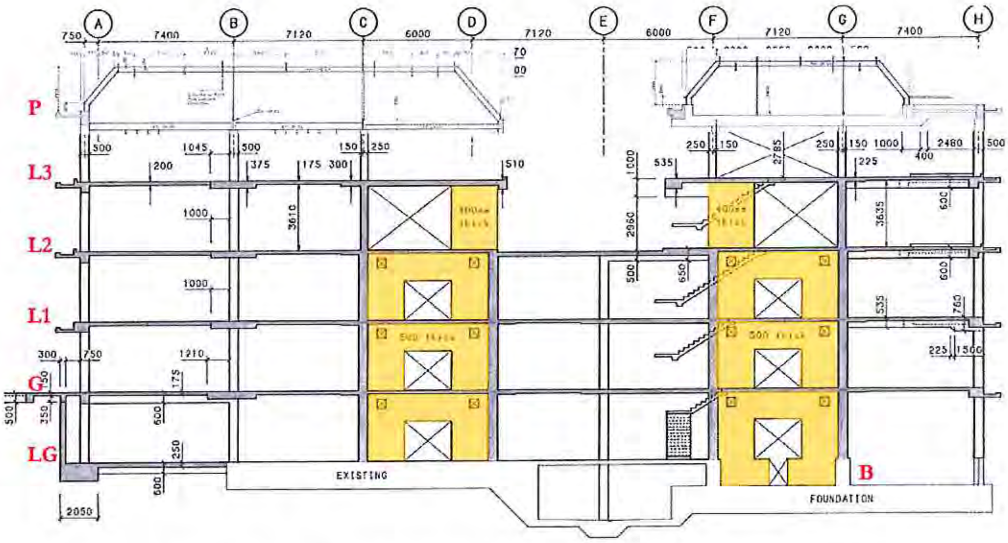


Figure 2-5: Section looking west through Parkside Block C

On the south side of the building the ground is at Ground Floor level; however, on the north side of the building, the ground is lower and is at Lower Ground Floor level.

A section of floor was infilled in Block C as part of the wards extension project in 2012. This was done with lightweight materials.

The structure for Block C is similar to that of Block A with the exception that the Plant Room Level is a lightweight mezzanine supported on steel beams that form part of the portal frame roof structure, rather than a concrete floor slab. The lateral loads from the mezzanine and roof are transferred by portal action to the Level 3 concrete slab and then into the shear wall cores (which are shaded yellow in Figure 2-5).

Block C is clad on the north and partially on south face with precast concrete panels that are bolted to the perimeter frame structure. The concrete panel connections typically have a slotted or oversized hole for construction tollerances. Block C is adjacent to Blocks B and D on the east and west and partially to the University of Otago Christchurch School of Medicine building to the south. The floor levels align with those of the School of Medicine and access between these buildings is provided at several levels.

2.1.4 Block D

Block D is a 6 storey reinforced concrete structure with a basement services space and a light weight steel framed roof, there is also a small 7th level at the west end that forms the Lift Machine Room. Block D is the largest of the Parkside buildings; it is approximately rectangular in plan, 62 x 41m, with a system of internal walls and columns and a perimeter frame which is partially in-filled with concrete blockwork on the north side. Above Level 2 the building becomes "C" shaped with a void opening from the east side. On the south side of the building the ground is at Ground Floor level; however, on the north side of the building, the ground is lower and is at Lower Ground Floor level. A section through Block C is shown in Figure 2-6.

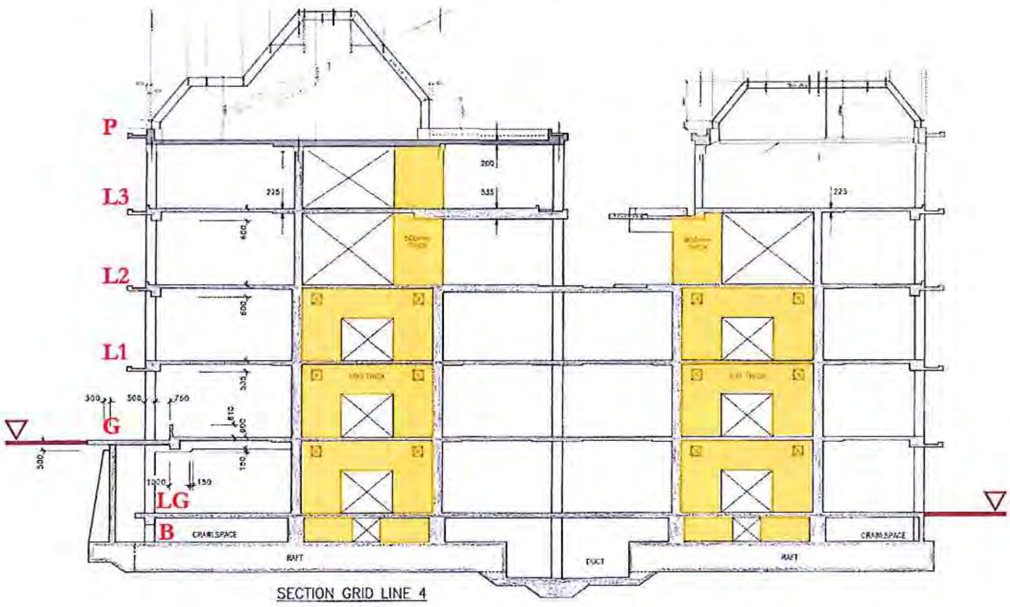


Figure 2-6: Section looking west through Parkside Block D

A retaining wall supports the ground on the south side of the building; there is also a retaining wall along the west side of the building. The retaining walls are not physically connected to the building at the Ground Floor level.

The structure for Block D is similar to that of Block A, with the exception that the lateral loads being resisted by a combination of the four internal shear cores and a coupled shear wall on the west elevation. Similar to Block A the north wing does not have a full concrete floor diaphragm at Plant Level.

The Lift Machine Room Level (which is above the main plant level) also has a concrete floor diaphragm. This level is supported by the coupled shear wall on the west side of the building and two internal columns.

Block D is adjacent to Block C on the east and has links to the Food Services Building to the north and Christchurch Women's Hospital to the west. There is a 100 mm separation between Parkside D and the Food Services building and over a 500 mm separation between Block D and the Christchurch Women's Hospital.

2.2 PRE-EARTHQUAKE BUILDING CAPACITY

This section discusses the capacity of the Parkside buildings prior to the Darfield Earthquake.

The discussion is divided into several parts. Section 2.2.1 defines the terminology used in this section. The loads that Parkside Blocks would have been originally designed to resist are compared to those required by the current codes in Section 2.2.2. A comparison of structural detailing used in the buildings compared to what would likely be implemented in an equivalent new building are also presented in this section.

Non-Linear Time History Analyses (NLTHA) have been completed to determine the structural capacities of Blocks A and D.

Section 2.2.3 provides a description of the NLTHA process and is broken down into the earthquake load level (Section 2.2.3.1), the material properties used (Section 2.2.3.2) and the modelling assumptions made (Section 2.2.3.3).

Each block is then discussed separately. The results calculated from the NLTHA for Block A are presented in Section 2.2.4 and those for Block D in Section 2.2.6. The capacities for Blocks B and C are presented in Sections 2.2.7 and 2.2.8 respectively; these were derived from an assessment of individual element capacities and a comparison with the performance of the Block A and Block D NLTHAs. The model for Block A has been updated in this latest revision and the results used to derive capacities for the other Blocks.

Assessments have also been made of the performance of the stairs (Section 2.2.9), the precast concrete cladding panels (Section 2.2.10), the potential of pounding of adjacent buildings (Section 2.2.11), the performance of the link bridges (Section 2.2.12) and the performance of the ceilings and partition walls (Section 2.2.13). These sections cover the performance of these items for all blocks.

An overall summary of performance of all Blocks is presented in Section 2.2.16.

2.2.1 Terminology used in this section

2.2.1.1 Design Basis Earthquake

The capacity of the buildings under earthquake actions discussed in this section is compared to the loads that a similar building would be designed to today. A new building would be designed to resist an earthquake known as the Design Basis Earthquake (DBE). The DBE is based on Ultimate Limit State loads calculated with reference to the buildings physical location, local soil conditions, building type, fundamental period and importance level. The DBE is calculated in accordance with the Structural Design Actions Standard, Part 5: Earthquake Actions – New Zealand, NZS1170.5:2004 [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 Base Report; however, for this type of building they essentially increase the design loads by 36 %.

In this report 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.

Unless noted otherwise, 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 [10]. The guidelines allow some relaxation of requirements for existing buildings compared to new, and therefore existing buildings shown to achieve 100 % of DBE loading may not achieve the same level of seismic performance as a new building designed to achieve minimum compliance with the building code.

2.2.1.2 Critical Structural Weaknesses

This assessment also considered 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. CSWs are described in more detail in the Christchurch 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. In Non-Linear Time History Analyses, the Collapse Limit State (CLS) is reached when a CSW is identified.

When CSWs are identified, the Engineering Advisory Group Draft Guidelines recommend a margin over collapse to be used to provide an acceptable risk of collapse [11]. This is to ensure existing buildings are assessed against a similar level of performance to that of new buildings. The Guidelines recommend a factor of 2 for qualitative assessments. As discussed in the Base Report, for detailed assessments a factor of 1.8 is used, as it is generally accepted that for well detailed new buildings there is typically a margin of at least 1.5 to 1.8 over the ultimate limit state capacity.

In the following sections, for the estimated CLS and the capacity of elements with brittle failure mechanisms that are considered to be CSWs, both the un-factored capacity as a percent of Design Basis Earthquake (DBE) is presented, along with this value divided by 1.8 and 1.5 to allow comparison with new buildings.

2.2.1.3 Pounding

Pounding is a term used to describe when buildings move laterally relative to one another during earthquake shaking and collide together. Pounding can result in increased damage to structures especially if the adjacent structures are of different heights and/or the floor levels do not align. The Parkside buildings are typically separated from each other and adjacent buildings by 100 mm seismic gaps above the Lower Ground Floor level. The buildings will move independently and pounding between the buildings is possible. The earthquake load levels this is expected and the likely consequence is discussed in Section 2.2.11.

2.2.2 Code Comparison

Parkside Blocks A, B, C and D were designed in around 1986 to a predecessor of the current NZ Building Code, most likely comprising NZS4203:1984 [12] for loadings and NZS3101:1982 [13] for concrete design.

A comparison of the load levels for the Parkside Buildings represented by NZS4203:1984 and the current loadings standard, NZS1170.5:2004 [8], incorporating the recent increase in seismicity for Christchurch, is plotted in Figure 2-7.

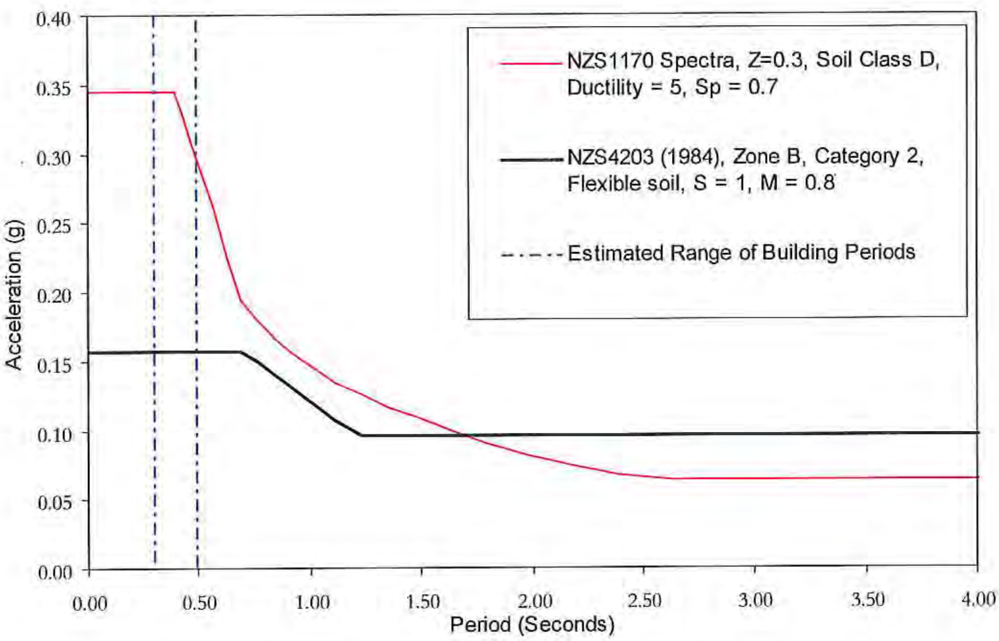


Figure 2-7: Comparison of Design Codes, IL4 Building

The comparison assumes the buildings were detailed for a high level of ductility. The buildings have fundamental periods between 0.3 and 0.5 seconds, therefore, the comparison of design codes indicates that the Parkside buildings should have the capacity to resist between 45 and 55% of the current Design Basis Earthquake (DBE) loads.

As well as an increase in load levels between the loading standard the building was designed to and the current loadings standard, there has also been a change in the classification of buildings and the serviceability performance requirements. In particular, the current standard requires that Importance Level 4 buildings remain operational after a 1:500 year event; this is referred to as the Serviceability Limit State 2 (SLS2), and is the equivalent to a full Design Basis Earthquake (DBE) for a "normal" IL2 building. The Parkside buildings would not have specifically been designed to meet SLS2 requirements, as this requirement did not apply at the time of design in the 1980's.

Although there have been several revisions of the New Zealand Concrete Structures Standard since the Parkside Buildings were designed, they appear to have been well detailed and generally meet all requirements of the current concrete standard, NZS3101 [14]. This means that the full ductility they were designed to sustain should be able to be achieved.

However, it should be noted that lessons have been learnt from the Canterbury earthquakes that will likely lead to revisions of the current code. The Structural Engineering Society New Zealand (SESOC), has published a practice note on recommended changes to how conventional structural systems are designed following the Canterbury Earthquakes [15]. These provisions are also covered in the draft amendment to the Concrete Structures Standard NZS3101. This guidance recommends an increase in the minimum quantity of reinforcement in concrete walls, which the Parkside buildings do not meet, it also recommends additional confinement steel for ductile walls such as those used in the Parkside cores. As such, the performance of these buildings is not expected to be as good as new wall buildings going forward.

The Parkside Blocks were designed to behave in a ductile manner. A ductile structure is one that is detailed such that it can dissipate energy in an earthquake and sustain, without significant loss of strength, repeated reversing displacements. The higher the ductility, the lower the loads for which the building is designed to resist due to the greater yielding of the structure. Consequently there will be greater damage to the structure in an earthquake. An elastic structure, ductility (μ) of 1, resists the DBE loads without yielding (or ductile behaviour) and will have minimal or no damage. The maximum permitted ductility for a reinforced concrete structure is a function of the structural type and geometry with a maximum of 5 for these types of structures. When the Parkside Buildings reach their Ultimate Limit State they are likely to have sustained significant damage and may not be economic to repair.

2.2.3 Blocks A and D – NLTHA Models

Non-Linear Time History Analyses (NLTHAs) have been completed to determine the building capacities of Blocks A and D. 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 assessment philosophy forms the basis of what is termed performance based design. Parameters used in the NLTHA model are discussed in the following sections.

The NLTHA models for both buildings initially showed similar behaviour and results. As such, when further analysis has been done on the Block A NLTHA model the results have been interpolated for Block D to save analysis costs and time.

2.2.3.1 Earthquake Load Level

Seismic loads used in the NLTHA model were based on the requirements of NZS1170.5 [8]. For time history analysis, the code specifies a minimum of three time histories scaled such that the records envelope the code response spectrum.

The appropriate scale factors were determined using the following parameters:

Design Life:	50 years
Zone factor, Z:	0.30 (Christchurch revised)
Subsoil Class:	D (Deep or soft soil)
Importance Level, I:	4
Risk Factor, R:	1.8
Structural Period, T:	$\leq$0.4s
Structural Performance Factor, S_p:	0.7

Table 2-1 lists the three earthquake records used, together with the scaling factors calculated for the buildings.

Table 2-1 Earthquake records used in NLTHA with scaling factors

	Scaling Factor for R=1.0			
Earthquake	**Block A (X)**	**Block A (Z)**	**Block D (X)**	**Block D (Z)**
El Centro Array #9 (Imperial Valley, USA) 19 May 1940	1.10	1.14	1.18	1.13
Kalamata (Greece) Earthquake, 13 Sep 1986, Nomapxia	1.01	1.01	0.84	0.81
Llay Llay (Chile) Earthquake, 3 March 1985	0.76	0.80	1.01	1.02

2.2.3.2 Material Properties

In-situ material testing was undertaken by Holmes Solutions and the results were interpreted by Holmes Consulting Group. Interpreted results were then implemented in the computer model. In other instances, NZSEE recommendations were used to develop probable strengths from specified minimum strengths on the drawings.

- Concrete strength was based on results from material testing undertaken by Holmes Solutions. A concrete strength (f'_c) of 40MPa was used for elements below the lower ground floor level and 25MPa for elements at or above the lower ground floor level.
- Flexural reinforcing for the cantilevered walls and all of the columns was specified as high strength (characteristic strength of 410MPa and probable of 460MPa). The remainder of the reinforcing for the walls and beams are specified, generally, as mild steel reinforcing (characteristic of 300MPa and probable of 320MPa).
- The properties of the steel in the EDE wing included 300MPa for the structural I-sections and hollow steel columns. The vertical cross-bracing properties were based on standard plate capacities (250MPa). Strengths provided by the manufacturer were used for the Reidbar horizontal bracing at roof level (500MPa). A factor of 1.1 was to account for probable strengths as per the NZSEE guidelines.

2.2.3.3 Model Assumptions

The following assumptions were made in the modelling of Blocks A and D:

- Parkside Blocks A and D have regular concrete floors comprising typically of 175mm in-situ two-way concrete slabs. Column capitals and thickened slab sections (slab beams) are provided to resist shear from gravity and seismic loading. The concrete floors are modelled as rigid diaphragms.

- The roofs over the north and south towers consist of light weight metal mansard roofs over structural steel framing. The framing was evaluated in the initial modal analysis of Block A and found to perform to 100% DBE loading. The roof framing in Block D is similar to that in Block A. The roofs were therefore not modelled in the non-linear time history analysis models for Blocks A and D except as mass contributing to the weight of the upper storeys.

- The structures are supported on a 1.5m reinforced concrete mat foundation. Mat foundations allow the building to rock about its base in the event of an earthquake, and this has been reflected in the model. Rocking permits energy dissipation but may contribute to inter-storey drifts.

- Fixity to adjacent buildings was not modelled. All buildings are seismically separated from each other except at Lower Ground Floor and Basement Level. Parkside Block A is adjacent to Block B on its west side and Parkside Block D is adjacent to Block C to the east, however, at the building interfaces the mat foundation is 300mm thick rather than 1.5m. It is assumed that the connection between the two structures is not rigid enough to significantly affect their behaviour.

- Soil springs were not included in the model; the soil is assumed to act rigidly. This is conservative for shear wall buildings because the fixity at the base of the walls results in greater axial and flexural loads. The introduction of soil springs would also likely lead to period lengthening and a reduced demand on the structures.

- Gap elements were used at the base and at each storey level of the shear walls to permit flexural yielding of the walls if it occurs.

- The lift machine room penthouse in Block D is supported by coupled shear walls on the west side of the building and two columns. Additional braces were added to the model to remove instability in the east-west direction to allow the analysis to be completed. These braces are not part of the existing building but additional assessment has shown they are not required.

- Investigations have indicated that at the Ground Floor level the retaining walls present on the south and east sides of Block A, and south and west sides of Block D, are not physically attached to the Ground Floor slabs. Therefore, these walls have not been included in the models and lateral loads are taken out of the model at the Lower Ground floor level.

2.2.4 Non-linear Time History Analysis (NLTHA) Limit States

The NLTHA provides more accurate information on when the elements in the building are likely to reach a capacity that result in significant structural damage and would be considered as the building reaching its Ultimate Limit State (ULS) capacity and also when individual elements experience sufficient damage that might lead to the onset of partial collapse of the building, its Collapse Limit State (CLS).

At their Ultimate Limit State (ULS), buildings are expected to suffer significant structural damage with the potential of being rendered uneconomical to repair and unable to be re-entered. At the Collapse Limit State (CLS), some elements may be on the point of collapse. The CLS is not typically checked in new building design; however, adhering to current standards should ensure there is a margin of 1.5 to 1.8 between the ULS and CLS. The NLTH analyses completed for this assessment permitted the identification of this limit state and therefore permitting the comparison of the performance of the Parkside Buildings to new buildings built in accordance with the current building standards.

The NLTHA can also be used to assess the Serviceability Limit States required by the Loading Standard [16]. All buildings are required to meet the SLS1 (Serviceability Limit State 1) requirements, for these to be met no structural or non-structural components should require repair in a 1 in 25 year event. Importance Level 4 (IL4) structures, such as the Parkside buildings, are also required to meet the SLS2 (Serviceability Limit State 2) requirements. This limit state requires IL4 buildings to remain operational after an event which is equivalent to the ULS Design Basis Earthquake (DBE) of a "normal" building (an Importance Level 2 structure), a 1 in 500 year event. It should be noted that the SLS2 requirements were not in the loadings code when the Parkside Buildings were designed.

2.2.5 Block A - Non-linear Time History Analysis (NLTHA) Results

A summary of the capacities predicted by the NLTHA model for Block A is given in Table 2-2. The table includes an estimate of the onset of the buildings ULS, an estimate of the onset of the CLS and the CLS divided by a factor of safety of 1.8 and 1.5 to allow comparison with the equivalent performance of a new building.

All capacities are presented as a percent Design Basis Earthquake (%DBE) loading assuming the building is Importance Level 4 (IL4).

The Emergency Department Extension (EDE) relies on the Original Block A building for both gravity and lateral support and therefore this was also included in the NLTHA model. Due to the different structural systems used in the EDE, the results are presented separately.

Table 2-2: Summary of results for Block A as a percent of Design Basis Earthquake (%DBE) loading (IL4)

	Original Building	**EDE**
Onset of significant damage (equivalent to the Ultimate Limit State (ULS) performance of a new building).	78% DBE	30 % DBE
Onset of damage that has a high probability of leading to partial or total collapse (the Collapse Limit State (CLS))	89 % DBE Margin of 1.14 over ULS	55 % DBE Margin of 1.8 over ULS
Equivalent performance relative to a new building: Onset of CLS divided by factor of safety against collapse of 1.8 i.e. CLS/1.8 (CLS/1.5)	**49 % (59 %)**	**30 % (37 %)**

The results indicate that the reliable ULS capacity of the main Block A building is approximately 78 % DBE loading, but with only a margin of 1.14 between ULS and collapse.

The capacity of the Emergency Department Extension is lower, with the onset of ULS at 30 % DBE and CLS at 55 % DBE.

2.2.5.1 Onset of Ultimate Limit State (ULS)

The Ultimate Limit State (ULS) of the main Block A building is reached at around 78 % DBE loads for an Importance Level 4 building. A screen shot of the NLTHA model indicating elements that have exceeded ULS limits is shown in Figure 2-8. One wall panel undergoes flexural rotations beyond their ULS limits (this wall panel is indicated by the purple/blue colour and circled in red for clarity). ULS limits for wall panel rotations are set to provide a margin of safety over the longitudinal bars in the wall from buckling and/or fracturing which would lead to reduced wall capacity.

The other elements shaded orange and pale blue are ones that are likely to sustain some damage, but are still in a condition where they continue to contribute to the stability of the structure. The beams and floors are not shown in the figure for clarity. The NLTHA indicates that the building will perform as the original designer intended with plastic hinges forming at the bases of the core walls.

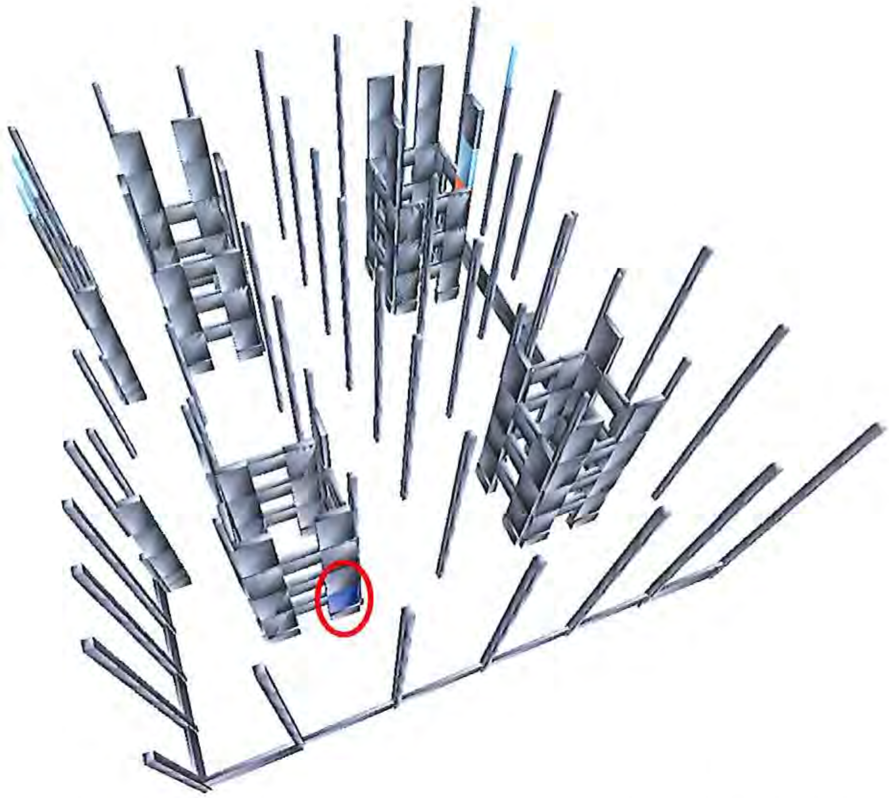


Figure 2-8: Elements that exceed Ultimate Limit State capacity at 78 % DBE (IL4) - View from north-east Corner

At 78 % DBE (the ULS capacity) the maximum interstorey drifts predicted by the NLTHA for the main Block A building are 2.4% in the X (east-west) and 2.6% in the Z (north-south) direction. This magnitude of drift is experienced between Level 3 and the Plant Room Level in the south wing of the building. The maximum interstorey drift limit at ULS for a new building is 2.5%. However, a NLTHA looks at the impact of drifts on specific building elements and as such the 2.5% 'limit' is only really used as an indicator of likely damage. In addition, drift critical elements such as stairs and seismic gaps are addressed specifically.

In the Emergency Department Extension (EDE), the first element to fail is a 150PFC brace that forms part of the roof cross-bracing system. This element has to transfer the roof loads from the roof bracing to the wall bracing. The 150PFC is unrestrained between its supports and when in compression it is estimated to buckle at around 25 % DBE loads. Some load will be shared by the adjacent DHS roof purlin, which increases the reliable capacity to 30 % DBE. This is considered the EDE's ULS capacity.

The roof brace is not the only element in the EDE with an estimated capacity of less than 67 % DBE loading for an IL4 structure. The other elements that become critical are:

- Wall cross-brace welded connections (35 % DBE)
- Post fixed stud anchors which connect EDE roof back into Block A (40 % DBE)
- Roof cross bracing – 12 mm diameter Reid Brace (40 % DBE)
- Wall bracing (50 % DBE)

- Cantilevered floor beams that pick up wall bracing (50 % DBE)

The drifts of the EDE Ground Floor are similar to the drifts of the main Block A building as the floor slabs of these two structures are connected. However, the inter-storey drifts between the Ground Floor slab and roof of the EDE are up to 3.0 % at 55 % DBE loading, this exceeds the limit allowed for new buildings (2.5 %). The high drift is due to the flexibility of the steel structure that ties the EDE roof back down to the Ground floor slab and Block A. Drifts of this magnitude would likely lead to significant damage of fit-out and cladding.

The Emergency Department Extension was designed in 2007. It was designed as an Importance Level 3 building. We are currently assessing the Parkside buildings as Importance Level 4 structures. This load increase, combined with the recent increase in seismic zone factor results in a 1.9 times increase in loading demand. This increase appears to be the primary reasons for the low capacity of these elements in the current assessment.

2.2.5.2 Onset of Collapse Limit State (CLS)

The Collapse Limit State is reached at around 89% DBE loads for an Importance Level 4 building. A screen shot of the NLTHA model indicating elements that have exceeded CLS limits is shown in Figure 2-9 (critical elements circled). There are two elements that exceed their CLS criteria,, one of these is a wall at Lower Ground Floor that exceeds flexural rotation limits (shaded pink and circled). The other element is a wall at level 1 that exceeds shear capacity in combination with having high axial loads (shaded black and circled). The elements shaded orange and pale blue are ones that are likely to sustain some damage, but are not considered critical to the stability of the structure. It can be seen that several other core walls have elements shaded purple/dark blue between the Lower Ground and Ground Floor levels; this means they have exceeded their ULS limits but not yet their CLS limit.

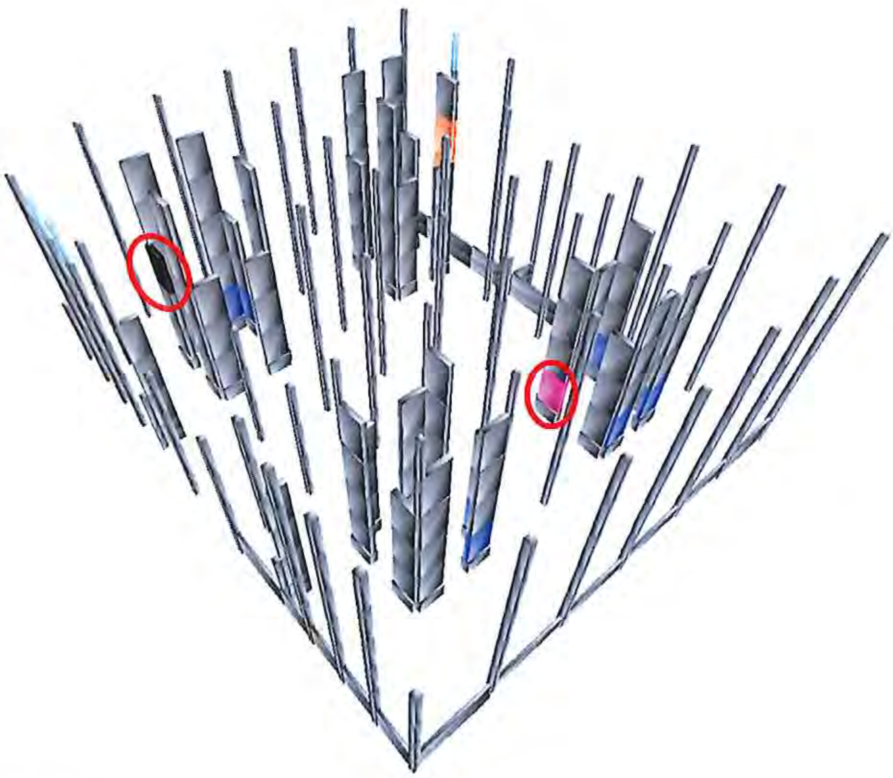


Figure 2-9: Elements that exceed Collapse Limit State capacity at 89 % DBE (IL4) - View from North-east Corner (beams and floors not shown for clarity)

In the Emergency Department Extension, the elements that first exceed critical collapse limits are two steel beams that support the Ground Floor. These beams cantilever over a primary steel floor beam and pick up the wall bracing above, the two beams can be seen circled in Figure 2-10. The NLTHA indicates these beams yield and reach their ULS capacity at around 50 % DBE loads and CLS capacity at around 55 % DBE.

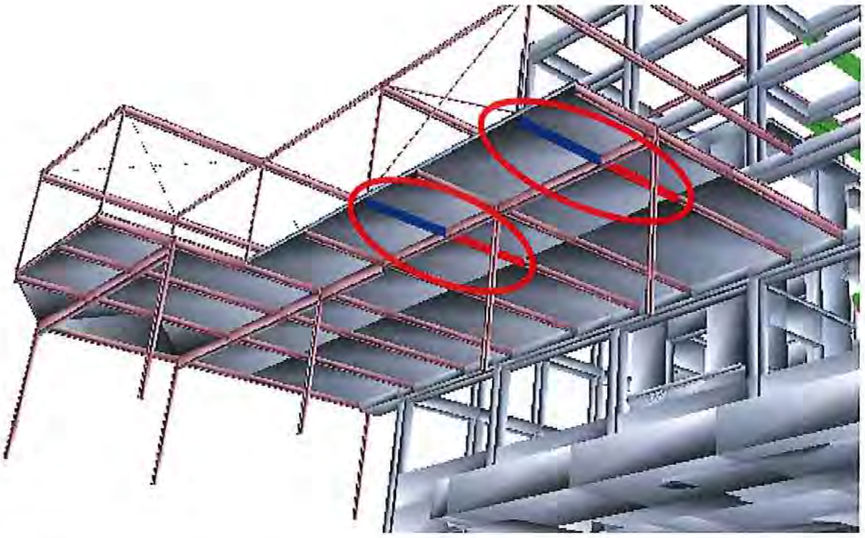


Figure 2-10: Underside of Emergency Department Extension (EDE) with critical beams circled

The Emergency Department Extension was designed as a nominally elastic structure and as such the code did not require it to incorporate principals of capacity design. Therefore, when loads higher than it was designed for are applied an undesirable hierarchy of failure develops. The cantilever beams (which are part of the gravity structure) yield and could result in this part of the floor failing. As such, they are considered a Critical Structural Weakness; however due to the low capacity of other elements, they do not currently govern the structure's performance.

2.2.5.3 Serviceability Limit States

At the SLS1 load level, the NLTHA indicates the majority of the main Parkside Block A structure remains undamaged. The inter-storey drifts are less than 0.4 % and therefore unlikely to cause significant damage to non-structural components.

At the SLS1 load level no structural elements in the Emergency Department Extension (EDE) yield and therefore remain undamaged. However, the interstorey drifts between the Ground Floor and Roof are up to 1 %. At this drift it is likely the internal fit-out will be damaged.

At the SLS2 load level (approximately 55 % DBE) numerous elements are indicated as becoming damaged in the main Block A building therefore the SLS2 criteria is not met. All of the elements are beam elements, generally at the upper levels (above Level 2) in the southern half of the building (refer to Figure 2-11 where damaged elements are shaded orange). The reason for this is that above Level 2 the building splits into two (with a courtyard between). Both halves have a concrete floor at Level 3, but only the southern half has a concrete floor at the Plant Room Level. This additional concrete level results in additional load on the southern half of the building; however, there is no increase in structure to account for this.

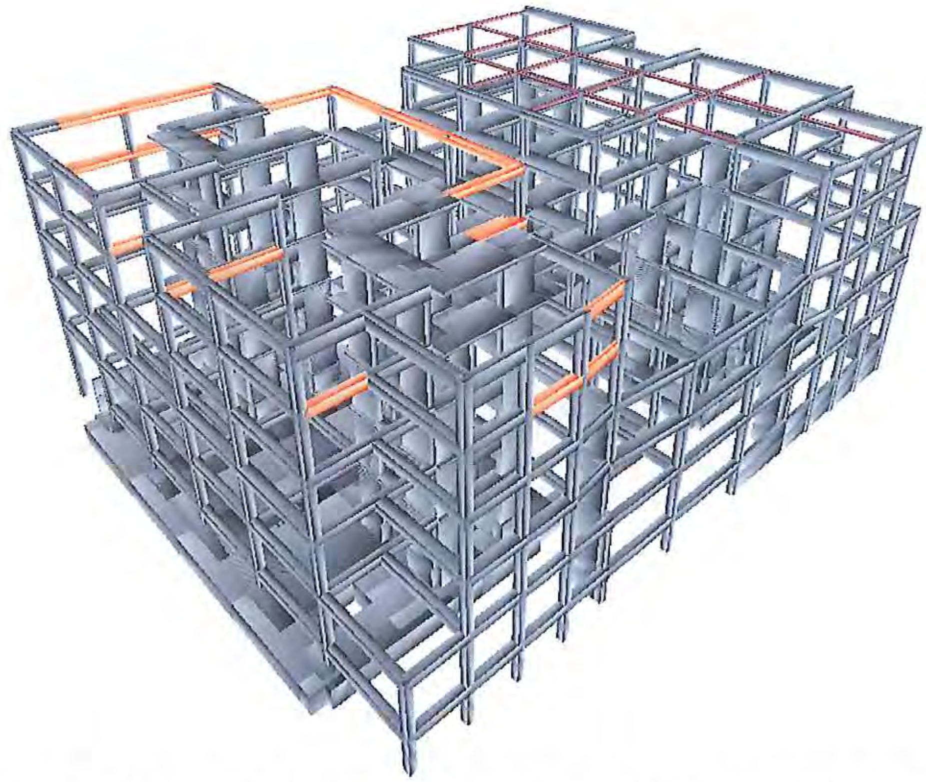


Figure 2-11: Elements that exceed SLS2 limits (55 % DBE) - View from South-east Corner

The interstorey drifts of the main Block A building at the SLS2 load level is indicated in Figure 2-12. These are the inter-storey floor drifts taken at the centre of mass. Drifts at the outer corners of the buildings are higher than those recorded at the centre of mass, due to torsional effects and accidental eccentricity and are approximately 2.5% at 67% IL4 maximum. It can be seen that below Level 2 the drifts are typically 0.5 to 0.6 %, this level of drift would be expected to cause some damage to internal fit-out, but building function could likely be maintained (or resumed in within a short time frame). Above Level 2 the inter-storey drifts are up to 1.75 % in both directions. This would cause significant damage to fit-out and may damage plant and services in these levels. These areas would not remain operational after a SLS2 event. The lower levels likely rely on services and other functions from the upper levels; therefore the Parkside Block A buildings is not considered to meet the SLS2 requirements.

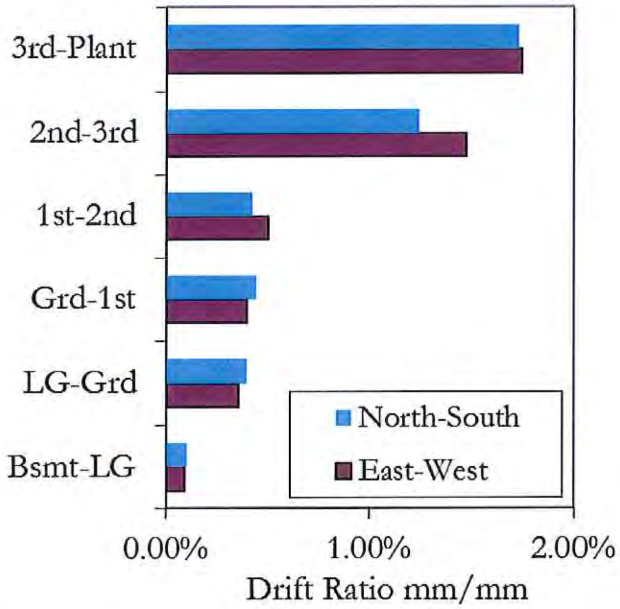


Figure 2-12: Inter-storey drifts up the building at the SLS2 load level

The EDE extension does not meet SLS2 criteria as at this load level ULS and CLS are exceeded for this part of the structure.

2.2.5.4 Building Drift

As noted above, and can be seen in Figure 2-12, the building drifts are significantly higher above Level 2. This is because there is a change in the structural system above this level. Between the Basement and Level 2 the lateral loads are primarily resisted by core walls connected by beams, above Level 2 the number of walls halve and there are not linking beams between them. A cross-section through Block A indicating this discontinuity in structure up the building and the differences between the north and south wings can be seen in Figure 2-13.

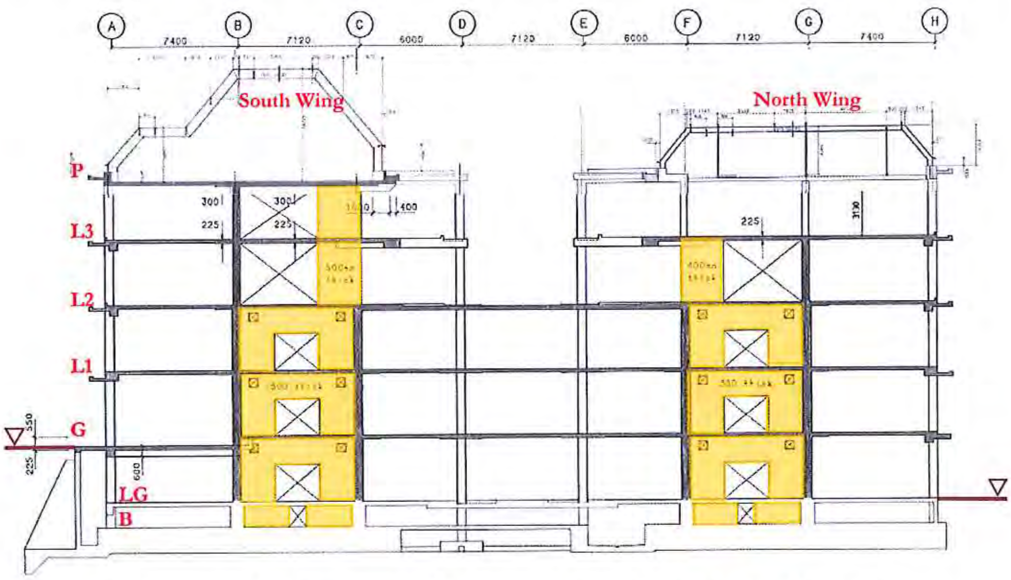


Figure 2-13: Section looking west through Parkside Block A

The NLTHA also indicates that the drifts at Level 3 of the south wing are higher than the north wing. This is because the Plant Level of the south wing is concrete, and heavier than the plant level of the north wing which is light-weight. This additional concrete level results in additional load on the southern half of the building; however, there is no increase in structure to account for this.

Discussion on the effect building drift has on the stairs, precast cladding panels and on pounding between adjacent buildings is provided in Sections 2.2.9, 2.2.10 and 2.2.11 respectively.

2.2.6 Block D - Non-linear Time History Analysis (NLTHA) Results

A summary of the capacities predicted by the NLTHA model for Block D is given in Table 2-3. The table includes an estimate of the onset of the buildings ULS, an estimate of the onset of the CLS and the CLS divided by a factor of safety of 1.8 and 1.5 to allow comparison with the equivalent performance of a new building.

All capacities are presented as a percent Design Basis Earthquake (%DBE) loading assuming the building is Importance Level 4 (IL4).

As outlined in section 2.2.3 above, the results for Block D have been interpolated from the Block A analysis results. This is because the initial models for the two blocks showed similar results and mechanisms and as such it was deemed not necessary to develop the Block D model further.

Table 2-3: Summary of results for Block D as a percent of Design Basis Earthquake (%DBE) loading (IL4)

	Block D
Onset of significant damage (equivalent to the Ultimate Limit State (ULS) performance of a new building).	78 % DBE
Onset of damage that has a high probability of leading to partial or total collapse (the Collapse Limit State (CLS))	89 % DBE Margin of 1.14 over ULS
Equivalent performance relative to a new building: Onset of CLS divided by factor of safety against collapse of 1.8 i.e. CLS/1.8 (CLS/1.5)	**49 % (59 %)**

The results indicate that the reliable ULS capacity of the main Block D building is approximately 78 % DBE, but with only a margin of 1.14 between ULS and collapse.

2.2.6.1 Onset of Ultimate Limit State (ULS)

The Ultimate Limit State (ULS) of the Block D building is reached at around 78 % DBE loads for an IL4 building. The elements which are expected to exceed ULS limits first are sections of the Lower Ground to Ground floor walls which are expected to exceed flexural rotation limits for the reinforcing. As with Block A, several beam elements also expected to exceed their ULS limits and sustain some damage, but these elements are not deemed critical and do not contribute to the stability of the structure as a whole.

At 78 % DBE (the ULS capacity) the maximum interstorey drifts predicted by the NLTHA for the Block D building are in the order of 2.5%, similar to Block A. This level of drift is likely to result in pounding with the adjacent buildings.

Discussion on the effect building drift has on the stairs, precast cladding panels and on pounding between adjacent buildings is provided in Sections 2.2.9, 2.2.10 and 2.2.11 respectively.

2.2.6.2 Onset of Collapse Limit State (CLS)

The Collapse Limit State (CLS) is reached at around 89 % DBE loads for an IL4 building. As with Block A, the expected failure mechanism is at the CLS limit state are walls at the Lower

Ground Floor to Ground Floor level reaching their CLS deformation limits for flexural rotations. At this load level the flexural demand on these walls might cause the vertical bars to buckle and fracture which reduces the wall capacity and could lead to some loss in gravity load carrying capacity. Several other wall elements will also likely sustain damage and exceed their ULS limit state at this time.

2.2.6.3 Serviceability Limit States

At the SLS1 (Serviceability Limit State 1) load level, the NLTHA indicates that the Parkside Block D structure remains undamaged (no yielding of structural elements modelled is indicated). The inter-storey drifts are less than 0.3 % and therefore unlikely to cause damage to non-structural components or fit-out.

At the SLS2 (Serviceability Limit State 2) load level, which is equivalent to 55 % DBE loading for an IL4 building, the building performs in a similar manner to Block A. It is expected that some beam elements in the upper floors will yield which will lead to damage that may require repairs.

The interstory drifts at the lower levels are expected to be around 0.5% in both directions. This level of drift would be expected to cause some damage to a standard internal fit-out, but building function could likely be maintained (or resumed in within a short time frame). Where the fit-out has been specifically designed to accommodate these drifts the building function should be resumed more quickly.

Above Level 2 the building form alters and the drifts are expected to increase to above 1% in both directions. This level of displacement would cause damage to fit-out and may damage plant and services in this level. It is likely that these levels would not remain operational after a SLS2 event. The lower levels likely rely on services and other functions from the upper levels; therefore the Parkside Block D building is not considered to meet the SLS2 requirements.

2.2.7 Block B (IL4) ETABS Assessment

Block B has a different primary structural system to the other three blocks. It only has two shear cores and these are cantilevered shear walls rather than the coupled walls used in the other Blocks. The full shear cores extend up the height of the building to the underside of the plant level.

An assessment has been completed on the capacity of the shear cores. As the wall detailing meets the requirements of the current concrete structures standard [14] this has been used to determine the wall performance. The assessment indicates a similar performance to Block A, and as such the capacity for Block A has been extrapolated to Block B. This gives an Ultimate Limit State (ULS) capacity of around 78 % DBE. The limiting factor is the ductility allowed in this type of wall (a ductility of 4.5). If a Non-Linear Time History Analysis (NLTHA) was completed for Block B it is expected that Block B would have a similar margin over collapse as Blocks A (a margin of 1.14) and therefore the Collapse Limit State (CLS) is expected to occur at around 89 % DBE. It is also expected that the walls would reach their CLS by exceeding CLS rotation limits rather than failing in shear.

Block B appears to meet the SLS1 (Serviceability Limit State 1) requirements. At this load level, the Parkside Block B structure has been assessed as remaining elastic (no yielding of structural elements is indicated) and therefore it is likely to remain undamaged. The inter-storey drifts are less than 0.3 % and therefore unlikely to cause damage to non-structural components or fit-out.

At the SLS2 (Serviceability Limit State 2) load level, which is equivalent to 55 % DBE loading for an IL4 building, there is likely to be some damage at the base of the shear cores (cracking between the Lower Ground and Ground Floors) which exceeds the amount allowed by the code. To meet SLS2 requirements the structural ductility must be limited to 2. The interstorey drift at the SLS2 load level is expected be around 0.3 % and therefore should not cause significant damage to the building fit-out or impact the continued occupancy of the building.

Because the structural system in Block B is different to the other Blocks, the drifts up the building are also expected to be different. There is no reduction in the structural system up the building and therefore the jump in interstorey drift magnitude noted in the other Blocks above Level 2 is not expected in Block B. Therefore the overall drift is expected to be smaller, especially in the east-west direction.

Discussion on the effect building drift has on the stairs, precast cladding panels and on pounding between adjacent buildings is provided in Sections 2.2.9, 2.2.10 and 2.2.11 respectively.

2.2.8 Block C (IL4) ETABS Assessment

The structure of Block C is similar to that of Block A, except that there is no concrete floor at the plant level (similar to the north side of Block A). Block C has four cores constructed with coupled shear walls.

An assessment has been completed on the capacity of the shear cores. As the coupled wall detailing meets the requirements of the current concrete structures standard [14] this has been used to determine the wall performance. The assessment indicates that the building performs in a similar manner to Block A and as such has an Ultimate Limit State (ULS) capacity of around 78 % DBE. The limiting factor is the ductility allowed in this type of wall (a ductility of 5).

The capacity is similar to the ULS capacity determined for Block A, which would be expected because of the building similarities. As such, if a Non-Linear Time History Analysis (NLTHA) was complete for Block C it is expected that it would have a similar margin over collapse as Block A (a margin of 1.14) and therefore the Collapse Limit State (CLS) is expected to occur at around 89 % DBE. The building drifts are likely to be similar to the north wing of Block A as this wing also has no concrete plant level.

Block C appears to meet the SLS1 (Serviceability Limit State 1) requirements. At this load level, the Parkside Block C structure has been assessed as remaining elastic (no yielding of structural elements is indicated) and therefore it is likely to remain undamaged. The inter-storey drifts will be similar to Block A (less than 0.4 %) and therefore unlikely to cause significant damage to non-structural components.

At the SLS2 (Serviceability Limit State 2) load level, which is equivalent to 55 % DBE loading for an IL4 building, there is likely to be some damage at the base of the shear cores (cracking between the Lower Ground and Ground Floors) which exceeds the amount allowed by the code. To meet SLS2 requirements the structural ductility must be limited to 2. The interstorey drift at the SLS2 load level is expected be between 0.5 % and 0.6 % and therefore will likely cause some damage to the building fit-out.

The inter-storey drifts for Block C are assumed to be the same as the south side of Block A. Discussion on the effect building drift has on the stairs, precast cladding panels and on pounding between adjacent buildings is provided in Sections 2.2.9, 2.2.10 and 2.2.11 respectively.

2.2.9 Stairs

There are seven flights of stairs in the four blocks. A typical section through a section of stair is shown in Figure 2-14.

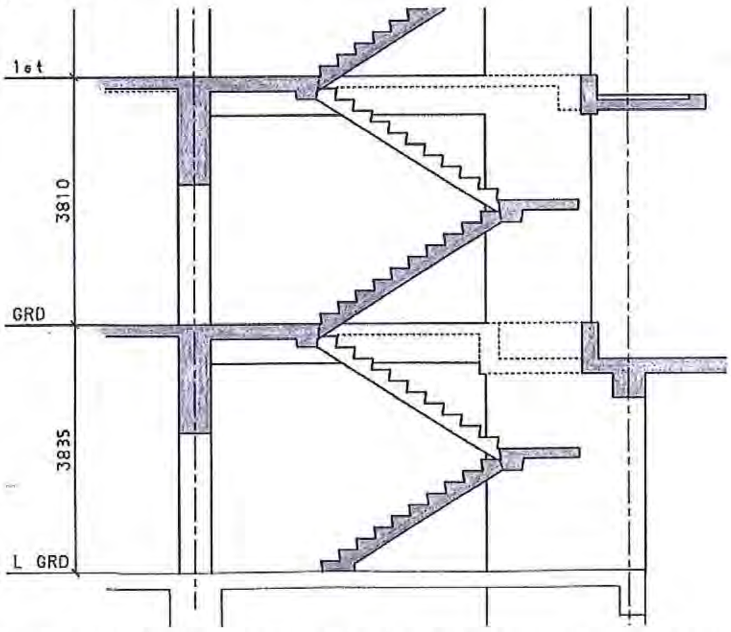


Figure 2-14: Section through typical reinforced concrete stair

The stairs are constructed of reinforced concrete; they zigzag between each floor level and, prior to the Canterbury Earthquakes, they did not have a mid height landing support. The stairs flights acted as tension/compression struts to support themselves under gravity loads. The stairs were rigidly connected floor to floor and because of this they were damaged as the building moved in the recent earthquakes.

In their pre-earthquake condition, the inter-storey deflections in a Design Basis Earthquake event would have caused significant damage to the stairs with progressive degradation of the stairs during the earthquake shaking as evidence in the post earthquake damage observations. As the stairs are required for safe egress of the building following an earthquake, in their pre-earthquake condition the stairs were deemed to be a Critical Structural Weakness.

Strengthening of the stairs is underway and is described in 5.1.1.

2.2.10 Precast Cladding Panels and Concrete Block Infill

The perimeter of the Parkside buildings, and the interior courtyards, are clad with precast concrete panels. There are numerous different cladding panel types and numerous ways the panels are connected to the main structure. Many of the panels span between floor levels and therefore subject to earthquake induced interstorey drifts.

The majority of these panels are connected by 24 mm diameter bolts cast into the concrete panel at all four corners. The bolts connect to steel angle brackets through 60 diameter oversized holes (both top and bottom of the panel), which if correctly installed provide movement tolerance to accommodate interstorey drift. The angle brackets are connected back into the concrete floor slab or beam soffit. One such angle bracket is shown in Figure 2-15.

Figure 2-15: Bottom Precast Panel Connection - Block D Plant Room Level

A review of these typical panel connections indicates that where the panel connections have been constructed according to the drawings, these panels should be able to withstand 1.0 % interstorey drifts. This drift is accommodated partially by the panel rotating and partially by the top bolts moving across the oversized holes. The 1.0 % drift capacity assumes that the bolts are positioned centrally within the oversized hole and are free to move relative to the bracket. This appears to have been the intention of the original design as it appears to allow construction tolerance to be accommodated at the fixing into the building and double lock nuts are used to ensure the washers are not hard up against the angle bracket. Inspection of a number of panel connections has indicated that in many instances, the movement allowed for by oversized bolt holes has been largely taken up by construction tolerance. Consequently the ability of the bolts to slide across the oversized holes has been reduced and the interstorey drift capacity of these connections is assessed as being less than 1.0%.

An assessment of the expected interstorey drift under increasing seismic loads indicates that the drift demand on precast panels on the Blocks that have no additional plant room area is less than 1 % until a 78 % Design Basis Earthquake (DBE) level event. However, the south wing of Block A, all of Block B and the south wing of Block D all experience higher drift demands above Level 2 than the other Blocks due to the additional concrete plant levels and therefore these areas experience drifts over 1 % at around 40 % DBE. As an example the Block A panels that are likely to be damaged at around 40% DBE loading, assuming bolts are placed centrally within the oversized holes, are shaded in the building elevations in Figure 2-16 and Figure 2-17.

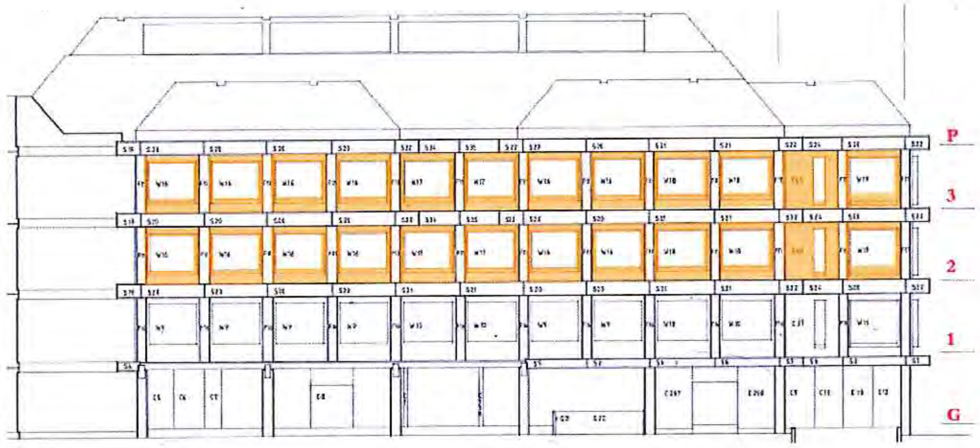


Figure 2-16: South Elevation of Block A indicating precast panels likely to be damaged at around 40 % DBE loading, assuming panel connection bolts placed centrally within oversized holes

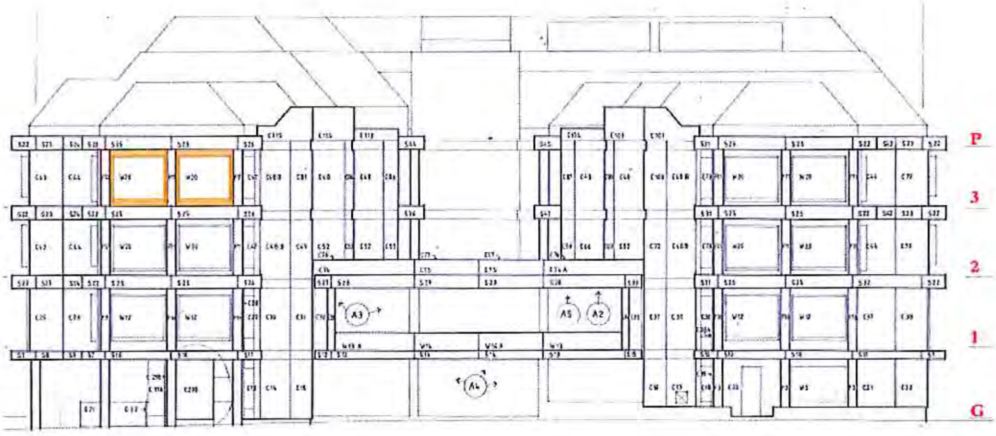


Figure 2-17: East Elevation of Block A indicating precast panels likely to be damaged at around 40 % DBE loading, assuming panel connection bolts placed centrally within oversized holes.

The interstorey drift capacity of the panel connections is determined by the extent to which the oversized bolt hole tolerance has been taken up by construction tolerance. Where the oversized bolt hole tolerance is taken up by construction tolerance, it is likely that the panels will be damaged at an earthquake loading of less than 34% DBE. It is likely that a large number of the panels around the buildings were constructed with insufficient drift tolerance and will be damaged at this loading level. These precast panels are assessed as one of the elements which govern the performance of the Parkside buildings. . At this load level, the connections back to the building are likely to become damaged. This is because, once the movement allowance has been exceeded, the capacity appears to be governed by the bolts cast into the precast panels cracking the concrete that encases them. There is potential that the panels will detach and fall into or away from the building at this load level, so it is recommended that the panel connections be inspected to confirm their interstorey drift capacity and remediated if required.

In a Maximum Considered Event (MCE), precast panels should be prevented from detaching from the building around entry and exit areas. This requirement is currently not met for an IL4 level MCE due to insufficient interstorey drift allowance in the existing connections. The remainder of the panels in Block A, as well as those in Blocks B, C and D, should have the capacity to be subjected to drifts larger than 1 % without detaching from the building.

Therefore they should be able to be subjected to an IL4 MCE level earthquake and not detach from the building. It is recommended that panel connections be inspected to confirm their interstorey drift capacity and remediated if required.

To date, only minor damage has been observed to these precast panels; however, the Block A panel connections are enclosed in the internal wall linings and therefore have not been inspected. Inspection of a selection of these connections is recommended in Section 3.9.1. Some inspection work has been undertaken in Block C, Level 2, where wall linings were removed as part of a room upgrade. The Panel fixings were observed at this time and the details found to be consistent with the assumptions made in the capacity calculations to date. No damage was observed in these connections.

When subjected to out-of-plane loading the majority of the precast cladding panel connections are estimated to have a capacity of over 67 % DBE loading. However, above Level 2, solid panels wider than 2 m (and those with only small windows), may have a capacity less than this (estimated at around 50 % DBE loading). It is estimated there are around 40 such panels, around 30 of which are on the west side of Block D. The remainder are on Blocks A, B and other elevations of Block D. Further investigations are required to confirm the panel connection type and layout for these panels.

Along the north wall of Block D some of the exterior concrete frame (below Level 2) is filled with concrete block. A 20 mm gap is provided between infill and adjacent concrete structure. This gap is sufficient to sustain an in-plane building drift of 0.62 % before the infill is damaged. The NLTHA indicates that below Level 2 the building drifts along the north side remain below this magnitude until a 78 % DBE. This is above the current estimated ULS capacity of the main concrete core walls and therefore the Concrete Block Infill walls are not likely to govern the building performance. The capacity of the Concrete Block Infill to out-of-plane loading is around 95 % DBE.

The Emergency Department Extension (EDE) is clad in a light-weight composite panel (Alucobond) on its north and east sides. The east elevation should remain undamaged until around 55 % DBE loading as the displacement along this side is determined by the Block A drifts. However, the cladding on the north side of the EDE will likely be damaged at low loads (30 % DBE) because of the low capacity and large interstorey deflections estimated in this direction.

2.2.11 Pounding between Adjacent Buildings

The Parkside Blocks A, B, C and D are separated by seismic 100 mm gaps above the Lower Ground Floor. The four Blocks also all have seismic gaps at interfaces with other adjacent buildings. The New Zealand Society for Earthquake Engineering (NZSEE) recommendations for the assessment of existing buildings provides a method to calculate the distance buildings must be separated by before pounding must be considered [10]. The NZSEE method has been used to determine the likelihood of the Parkside Buildings pounding with each other and with adjacent structures. The method takes into account the likely deflection of the buildings each side of the seismic gap and the likelihood of maximum displacements occurring simultaneously.

Similar to when assessing the capacity of existing buildings, the NZSEE criteria for pounding between existing buildings is less stringent than required for new buildings, and therefore existing buildings shown to have a sufficient seismic gap at 100 % of Design Basis Earthquake (DBE) loading may not achieve the same level of seismic performance as a new building designed to achieve minimum compliance with the Building Code.

The seismic gap between each of the Parkside Blocks is 100 mm wide. This is not sufficient to prevent pounding between the Blocks in a DBE event. The Non-Linear Time History Analyses completed for Block A and Block D provide good estimates of building drift. These analyses indicate that the Plant Room Level floors, on the south side of Block A and Block B are the first likely to pound at a load level around 40 % DBE. The interface between these two buildings was damaged in the recent earthquakes. The next location likely to pound is the south side of the Level 3 floors of Block A and Block B, which are likely to pound at around 55 % DBE loading. Floors at Level 2 and below are not likely to pound until an event over 67 % DBE loading.

All the floor levels in the Parkside Blocks align; therefore the impact of pounding is not as significant as if the floors did not align. The pounding will cause local damage at the junctions of the Blocks. Block C is one storey shorter than the adjacent Blocks B and D and therefore pounding of these Blocks is likely to cause increased forces in the top level of Blocks B and D and thus an increased amount of cracking to the shear walls and structure at this level. Pounding transmits short duration, high amplitude forces that will likely increase damage to building contents. However, this pounding of the Blocks is unlikely to lead to collapse or partial collapse of the building.

The Emergency Department Extension (EDE) is rigidly connected to Parkside Block A, but has a 100 mm gap to the Clinical Services Building. As only the Ground Floor of the EDE is concrete, this gap has been calculated to be sufficient for the IL4 DBE drifts. The buildings will likely pound under the MCE event, but as the floor levels align the damage is unlikely to lead to collapse or partial collapse of the EDE building.

The Link Bridges from Blocks A and B to the Clinical Services building are connected to the Parkside Blocks and separated from Clinical Services by a 100 mm seismic gap. This gap appears to be sufficient for drift demands up to those resulting from around 72 % DBE loading. Above this load level it is likely the Link Bridge roof will pound with the 2nd floor of the Clinical Services building. Although the floor heights of the two buildings align, in this instance the consequence of pounding is higher as it might compromise the lateral resisting system of the Link Bridges due to the length of the cantilever from the Parkside buildings. The capacity at the onset of pounding, divided by a margin of safety of 1.8 results in a capacity of only 40 % DBE (or 48 % DBE if divided by 1.5). The Block A Link Bridge will likely also pound with the Emergency Department Extension (EDE) along its west side; however, the consequence of this is not thought to be significant as the EDE will have a similar deformation to the Block A which the link bridge is also connected to.

On the south side of Blocks B and C, the School of Medicine Building is also separated by a 100 mm gap. Our analyses suggest that the Plant Level of Block B will pound with the School of Medicine building at around 50 % DBE loading. Level 3 of Blocks B and C will pound soon after this; however, Level 2 and below are not expected to pound until loads over 67 % DBE.

The School of Medicine Building is an IL2 facility and therefore 50 % DBE loading for an IL4 building is the ULS loading demand for this building. The floor levels in the School of Medicine Building and the Parkside Buildings align, but the School of Medicine is taller with four additional concrete levels. The effect of pounding is likely to be more significant to the School of Medicine building than to the Parkside Buildings; however, as it only occurs at the equivalent ULS loading for the School of Medicine Building this building is likely to meet its own performance requirements. However, damage to the School of Medicine Building at load levels greater than it ULS could affect the function of the Parkside Buildings which are expected to achieve higher performance levels.

It is recommended that the performance of the School of Medicine Building is considered in the assessment of the Parkside Buildings. Ways to mitigate the expected pounding are discussed in Section 5.2.5.

Pounding of Block D with the Christchurch Women's Hospital is unlikely. The seismic separation between these two building is over 500 mm (the Christchurch Women's Hospital is Base Isolated and is expected to have drifts in the order of 450 mm).

2.2.12 The Link Bridges to Clinical Services (IL4)

The link bridges to Clinical Services extend out from Blocks A and B. Lateral loads are resisted using a combination of the concrete floor slab with drilled and epoxied starters and bolted steel beams on the east and west faces of the link bridges. Figure 2-18 shows the link bridge that extends out from Block B.

Figure 2-18: Photo of east side of Link Bridge out from Block B (Clinical Services on right)

The capacities of the link bridges to cantilever horizontally from the Parkside Blocks are given in Table 2-4. The higher levels have lower capacities due to the increase in demand up the height of the building. The capacity of Level 2 governs both bridges and makes them considered Earthquake Prone.

Table 2-4: Capacity of Link Bridges

	%DBE (IL4)	
	Link bridge from Block A	**Link bridge from Block B**
Level 2	33%	33%
Level 1	40%	45%
Ground Level	55%	60%

The Lower Ground level of the Block B link sits on top of a service tunnel (see Figure 2-19). Because of this, several of the link columns are not supported by concrete pads (which are typical), but instead are supported by the tunnel, and the reinforced concrete up-stand beams that span over the tunnel. Review of these beams indicates that the east beam has insufficient capacity to resist the maximum design gravity load case. The capacity has been estimated as 85 % of current design loads using probable material strengths.

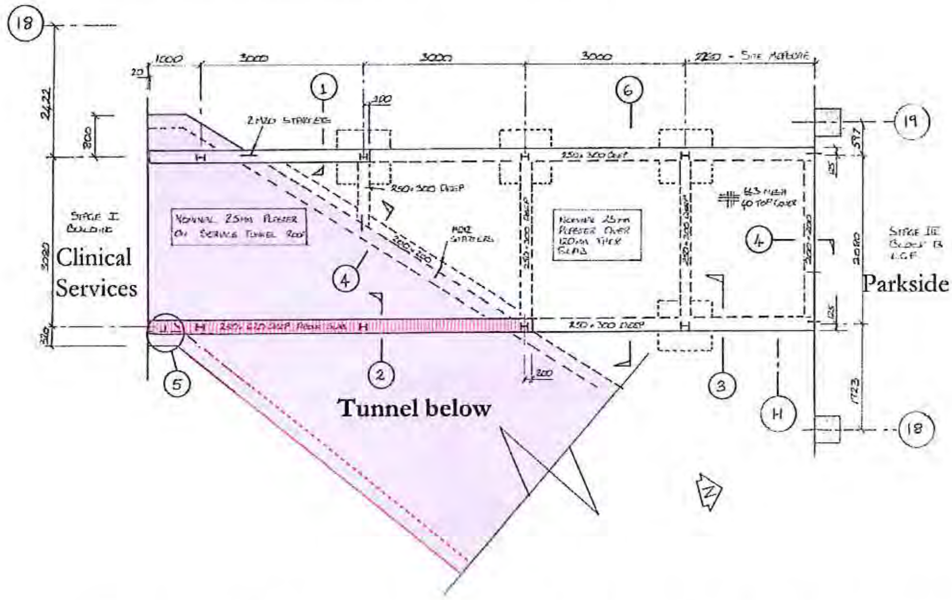


Figure 2-19: Lower Ground Floor Plan of Link B indicating location of tunnel below

As discussed in Section 2.2.11, pounding of the link bridges with the adjacent Clinical Services Building is likely to occur at 72 % DBE loading. This may compromise the lateral resisting system of the Link Bridges and as such the capacity at the onset of pounding is divided by a margin of safety of 1.8. This results in a capacity of only 40 % DBE (or 48 % DBE if divided by 1.5).

2.2.13 Ceilings and Internal Partitions for all Blocks

The ceiling system is a suspended acoustic plaster tile system. The majority of the internal timber partition walls are as high as the ceilings on either side of them and in the original configuration the ceiling panels were placed tight throughout the buildings. It is understood from the original installers that the ceiling tile system was used to restrain the top of the walls.

The ceiling and internal partitions have been addressed separately and the results are reported in the Holmes Consulting Group report dated 23 May 2011. [17]

2.2.14 Services

The mechanical services, plant and their fixings have not specifically been part of our structural review. It is recommended these are assessed for both damage and future capacity. We are happy to provide advice on likely earthquake induced interstorey and inter-building movements as well as design accelerations.

Services crossing seismic gaps between buildings are especially vulnerable and to meet the Serviceability Limit State 2 requirements of Importance Level 4 structures need to remain operational in this level event.

2.2.15 Retaining Walls

All of the Parkside buildings were built with a counterfort retaining wall. This means that the retaining wall that retains the soil at basement level is separate from the building itself. See Figure 2-20 below, which shows the general arrangement. The retaining walls do not support any gravity loads from the building itself.

Details from the original drawings indicate that the ground floor concrete slab cantilevers out from the building and rests on top of the retaining wall. There is no positive connection between the slab and retaining wall and the retaining wall has been detailed to cantilever. As such the two elements will move independently in a seismic event. Evidence of this movement has been observed around the south east side of Block A where there is an approximate 10-20mm gap in places between the top of the wall and the slab.

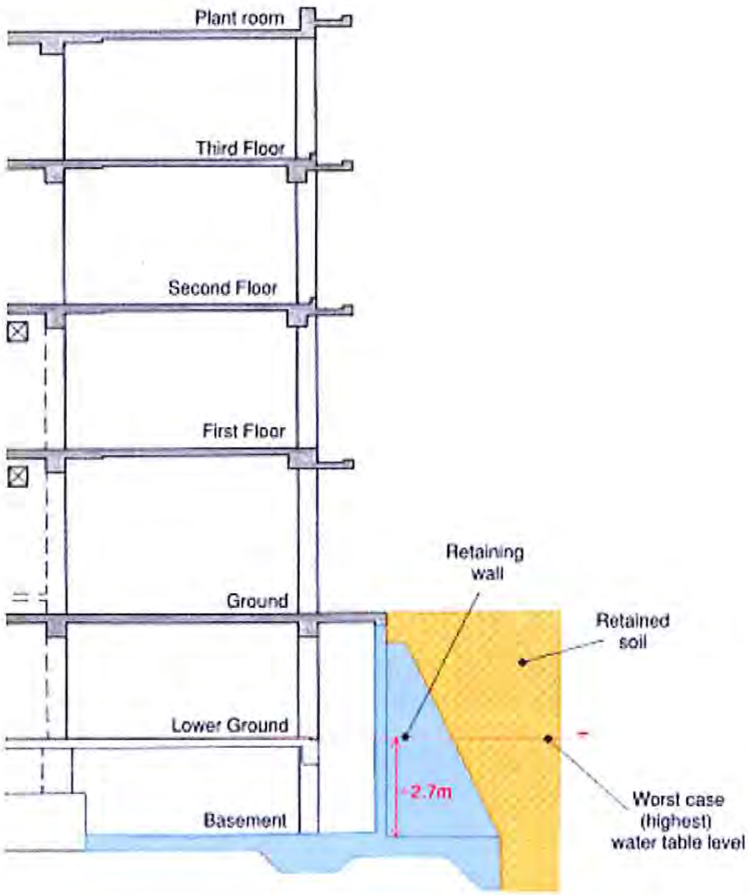


Figure 2-20: Parkside Block A Partial Elevation showing Retaining Walls

A separate analysis of the capacity of the retaining walls has been undertaken and this work shows that the Parkside retaining walls have the capacity to resist 60% of the Design Basis Earthquake (DBE) under Importance Level 2 (IL2) loads before the allowable soil bearing pressure is exceeded.

There are several other retaining walls around the site, that are not specifically connected to the Parkside building nor support the building itself. However, these walls do affect the functionality of the area surrounding the Parkside buildings. As such, they have been included in this assessment, and are shown in Figure 2-21 below.

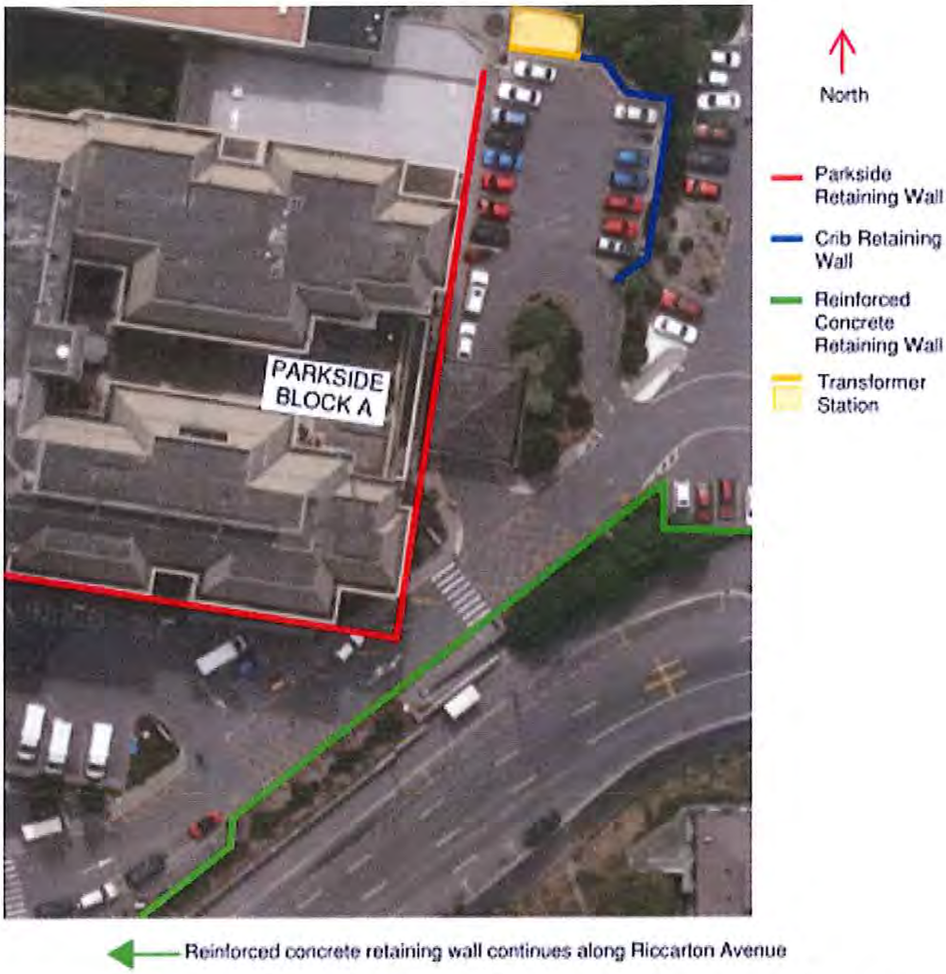


Figure 2-21: Retaining Walls around Christchurch Hospital Campus

The analysis of all the retaining walls showed that the concrete Crib retaining wall to the east of Block A (shown in Blue in Figure 2-21), retaining the Emergency Department car park, has the capacity to resist approximately 45% DBE of IL2 level loads. The driveway retaining wall (shown in Green in Figure 2-21) is of lower height than the crib retaining wall and is constructed of reinforced concrete. These walls are doubly reinforced concrete cantilever walls of similar construction to the Parkside retaining walls as they were built about the same time. Analysis shows that these walls have the capacity to resist 70% DBE loads at IL2 level.

A small transformer station has been constructed to the north-west of the crib retaining walls (shown in yellow in Figure 2-21). The existing crib retaining wall was removed in this location to construct the transformer station. HCG have acquired copies of the original drawings for the transformer room and are currently assessing its lateral capacity and the capacity of the retaining walls. A separate report will be issued on the capacity of the transformer.

2.2.16 Parkside Pre-earthquake Capacity Summary

Non-Linear Time History Analyses (NLTHA) have been completed to determine the structural capacities of Blocks A and D (including the Emergency Department Extension (EDE)). Additional analysis on Block A were interpolated for Block D due to their similar performances. The capacities for Blocks B and C were derived from an assessment of individual element capacities and a comparison with the performance of the Block A NLTHA. The Parkside Buildings have been assessed as an Importance Level 4 (IL4) facility.

Assessments have also been made of the performance of the stairs, the precast concrete cladding panels, the potential of pounding with adjacent buildings, the performance of the link bridges and the performance of the ceilings and partition walls.

Table 2-5 presents the capacities of the primary structural systems as well as secondary elements for the four main Parkside buildings and the EDE. It can be seen that the primary structure of the four main buildings have Ultimate Limit State (ULS) capacities of 67 % Design Basis Earthquake (DBE) loading and Collapse Limit State (CLS) capacities of 78 % DBE loading.

Table 2-5: Summary of results for the Parkside Buildings as a percent of Design Basis Earthquake (%DBE) loading (IL4)

	Block A	Block B	Block C	Block D	(EDE)
Onset of significant damage (equivalent to the Ultimate Limit State (ULS) performance of a new building).	78 % DBE	78 % DBE	78 % DBE	78 % DBE	30 % DBE
Onset of damage that has a high probability of leading to partial or total collapse (the Collapse Limit State (CLS))	89 % DBE Margin of 1.14 over ULS	89 % DBE Margin of 1.14 over ULS	89 % DBE Margin of 1.14 over ULS	89 % DBE Margin of 1.14 over ULS	55 % DBE Margin of 1.8 over ULS
Equivalent performance relative to a new building: Onset of CLS divided by factor of safety against collapse of 1.8 i.e. CLS/1.8 (CLS/1.5)	49 % (59 %)	49 % (59 %)	49 % (59 %)	49 % (59 %)	30 % (37 %)
Precast panel connections ULS	<34 % DBE	<34 % DBE	<34% DBE	<34 % DBE	N.A.
Pounding	40 % DBE with Block B	40 % DBE with Block A	55 % DBE with School of Medicine	67 % DBE with Block C	30 % DBE with link bridge
Link Bridges ULS	33 % DBE	33 % DBE	N.A.	N.A.	N.A.
Percent DBE when interstorey drift > 0.5 %	33 % DBE	67 % DBE	50 % DBE	42 % DBE	< 30 % DBE

The onset of ULS in the primary structure of the main buildings at 78 % DBE is due to earthquake induced flexural rotations at the base of the shear core walls exceeding allowable limits.

The onset of CLS is also due to earthquake induced flexural rotations at the shear core wall bases, but is also due to some walls exceeding shear stress limits. New buildings typically have a margin between ULS and CLS of between 1.5 and 1.8; however, there is only a margin of 1.14 for these buildings. This means that the performance of the Parkside buildings in a significant seismic event is unlikely to be equivalent to a new building.

The ULS capacity of the Emergency Department Extension (EDE) is only 30 % DBE loading. The capacity of the EDE is limited by struts used in the roof bracing; however, there are several other elements in this extension that also have capacities less than 67 % DBE loading.

The Link Bridges that connect Parkside Blocks A and B to the Clinical Services building also have a low capacity to resist approximately 33 % DBE. It should also be noted that the assessment indicated the gravity capacity of the Block B link bridge was only 85 % of full code loading demand.

The capacity of the precast panels is limited by a combination of insufficient deformation capacity of their connections in Blocks A and C and due to the out-of-plane capacity of the connections in Blocks B and D. The precast panels, and onset of pounding, typically have a lower ULS capacity in Block A. This is because the interstorey deflections are estimated as larger in Block A, especially above Level 2 in the south wing. The deflections are larger because of the reduction in structure above Level 2 and the concrete Plant Room Level present on the south side of Block A.

The capacity of the stairs is not included in the above table; however, prior to the earthquakes these were rigidly connected floor to floor (remedial measures that separate the floors are underway). In their pre-earthquake condition, the inter-storey deflections in a DBE event would have caused significant damage with progressive degradation of the stairs during the earthquake shaking. The stairs were damaged in the recent earthquakes. As the stairs are required for safe egress of the building following an earthquake, in their pre-earthquake condition they were deemed to be a Critical Structural Weakness.

The ceiling and internal partitions have been addressed separately in the Holmes Consulting Group report dated 23 May 2011 [9].

The ULS capacities estimated in the current assessment are slightly better than estimated by the code comparison completed in Section 2.2.2 (78 % DBE compared to 45-55% DBE). As the structural detailing meets the current code requirements the low performance of the building is predominantly due to increases in seismic demand for this type of facility and for Canterbury.

Importance Level 4 structures are also expected to meet two levels of serviceability requirements. The first of these, Serviceability Limit State 1 (SLS1), appears to be met by the Main Block buildings, but not by the Emergency Department Extension. The second serviceability requirement, SLS2, is only required for IL4 structures and requires them to be operational after a 1 in 500 year event (equivalent to 55 % DBE loading for an IL4 structure). The Parkside buildings do not meet this criteria due to the onset of damage at lower load levels and high interstorey drifts. To limit damage to typical internal fit-outs and services to a level that would allow continued function interstorey drifts need to be less than 0.5 %. It can be seen in Table 2-5 that interstorey drifts exceed this for Blocks A, C and D, and the EDE at load levels less than 55 % DBE loading.

As noted in Section 2.2.2, the Parkside Buildings were designed utilising a high ductility, therefore, when the Parkside Buildings reach their Ultimate Limit State they are likely to have sustained significant damage and may not be economic to repair.

For comparison, if the buildings were assessed as an Importance Level 3 structure the capacities would be as presented in Table 2-6. It can be seen that the ULS for the primary structural systems in the main buildings is 108 % which greater than the full capacity required for this Importance Level. However, as noted above, the margin between ULS and CLS is only 1.14 therefore, in a significant seismic event it is unlikely that the building will perform in an equivalent manner to a new IL3 building.

SLS2 requirements are not mandatory for IL3 buildings.

Table 2-6: Summary of results for the Parkside Buildings as a percent of Design Basis Earthquake (%DBE) loading (IL3)

	Block A	Block B	Block C	Block D	(EDE)
Onset of significant damage (equivalent to the Ultimate Limit State (ULS) performance of a new building).	108 % DBE	108 % DBE	108 % DBE	108 % DBE	42 % DBE
Onset of damage that has a high probability of leading to partial or total collapse (the Collapse Limit State (CLS))	123 % DBE Margin of 1.14 over ULS	123 % DBE Margin of 1.14 over ULS	123 % DBE Margin of 1.14 over ULS	123 % DBE Margin of 1.14 over ULS	76 % DBE Margin of 1.8 over ULS
Equivalent performance relative to a new building: Onset of CLS divided by factor of safety against collapse of 1.8 i.e. CLS/1.8 (CLS/1.5)	68 % (82 %)	68 % (82 %)	68 % (82 %)	68 % (82 %)	42 % (51 %)
Precast panel connections ULS	<34 % DBE	<34 % DBE	<34 % DBE	<34 % DBE	N.A.
Pounding	55 % DBE with Block B	55 % DBE with Block A	76 % DBE with School of Medicine	93 % DBE with Block C	42 % DBE with link bridge
Link Bridges ULS	46 % DBE	46 % DBE	N.A.	N.A.	N.A.
Percent DBE when interstorey drift exceeds 0.5 %	46 % DBE	93 % DBE	69 % DBE	58 % DBE	< 42 % DBE

3. POST EARTHQUAKE BUILDING CONDITIONS

This section covers the structural damage sustained by the Parkside Blocks 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 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 earthquake shaking experienced at the hospital site is outlined in the Base Report for the Christchurch Hospital Campus [1].

The fundamental periods of the Parkside Blocks have been estimated at between 0.3 seconds and 0.5 seconds. Based on the strong motion data downloaded, it appears that the earthquake produced shaking intensities between 70 and 80% of the new Design Basis Earthquake design spectra for an Importance Level 4 building.

It should be noted that the Lyttelton Earthquake was very short in terms of the strong shaking produced, with the strong motion only lasting for a duration of approximately 7-10 seconds. As a comparison, the DBE is based on the rupture of the Alpine Fault and is expected to contain up to 50 to 60 seconds of strong motion.

3.2 PRELIMINARY INVESTIGATIONS

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

- typical damage expected for buildings of this form
- a review of the original drawings [3,4,7]
- damage observed after the Darfield Earthquake (September 2010)
- damage observed during an initial walk around after the Lyttelton Earthquake (February 2012)

In conjunction with a review of the structural drawings and preliminary observations which were carried out on 25 and 26 February 2011, the following areas were identified as areas of primary damage;

- Flexure and shear cracking of the shear walls.

- Cracking of and damage to the stairs
- Damage to the link bridge connections to the Parkside buildings

3.3 DETAILED STRUCTURAL OBSERVATIONS

The detailed structural observations were completed on the 5, 20 & 21 April, 2 & 3 May and 29 September 2011. A full record of these observations is attached in Appendix A, with reference drawings describing the location labelling attached in Appendix B. Since the initial detailed observations, further observation have been made as areas have been opened up for repairs and information from the capacity assessment has become available. These observations have been added to the record of observations tables in Appendix A and also have typically been recorded in Site Reports.

As well as our visual observations, material testing has been completed to determine the concrete strength of the core walls [18] and for the potential for strain hardening of the steel reinforcement [19]. A levels survey has been completed [21,22], as has a geotechnical investigation [24,25,26] and a survey of façade damage [27]. The results of these investigations are summarised below and have assisted in creating an overall picture of building performance.

3.4 SUMMARY OF BUILDING DAMAGE

The following is a summary of our observations of the buildings reviewed, and our conclusions as to their condition and seismic load resisting capacity.

Table 4-1 through Table 4-5 provide a photographic summary of the observed damage and the typical repair work required. A full record of the detailed observations can be found in Appendix A.

3.4.1 Block A

Settlement Damage

Rotation of Block A to the north was observed in the horizontal offset of the handrails and concrete columns in the corridor and in movement on the seismic gap at the junction of Blocks A and B at the upper levels. The horizontal offset of Block A relative to Block B is approximately 30mm to the north at the Third Floor. This relates to approximately a 1.8mm/m lean.

The geotechnical report states that this is due to differential settlement caused by compaction of the ground due to the large forces on the building in the February earthquake. The levels survey undertaken by Fox and Associates [22] indicates a measured differential settlement over the Block A building footprint of 82 mm generally from north to south, the survey results are discussed in more detail in Section 3.6.

Some earthquake induced settlement was observed in the fill around the services tunnel in the Ambulance Bay to the south of Block A and in the paving to the north.

Shear Cores

Moderate cracking was observed in the internal shear walls in the cores. Cracking was observed up the entire height of the walls. This damage is typical for Blocks A, B, C and D. The maximum diagonal crack width observed was 0.75mm in width.

Strain hardness testing has been completed on a sample of steel crossing cracks in the core walls at the Lower Ground Floor. Further discussion on this testing is provided in Section 3.5.3.

Basement Slab and Foundations

Cracking was observed in the crawl space slab and through the top of the raft foundation in the Basement. A significant area of the crawl space slab between the raft foundations has been damaged. While the crawl space slab is on grade, it appears to have been detailed to span between the raft and foundation beams, with reinforcing steel top and bottom, both ways. The damage could be due to increased water pressure below the slab induced by the earthquakes or as a response to earthquake induced movements and forces from the raft slabs and super-structure.

Floor Slabs

Cracking is observed in the slab on grade in the Lower Ground Floor in the corridor adjacent to the entry to the services tunnel. This slab on grade is reinforced with mesh and is in an area between the basement corridor wall and the western foundation beam for Block A. Seismic forces and movement of the retaining wall are likely to have caused these cracks.

Cracking to the suspended slabs has been noted throughout the building. Cracking of up to 1.6mm in width has been observed in the suspended slab of the Lower Ground Floor. Cracking is observed in the soffit of the Lower Ground Floor slab from the Basement Level. Some shrinkage cracking has also been observed at the Lower Ground and Level 1 suspended slabs. This is not a result the earthquakes, but some of it may have been enlarged due to the earthquake induced actions.

Stairs

Cracking was observed in the stair flights over the full height of the building. This cracking of the stairs occurred in Blocks A, B, C and D. From anecdotal feedback, some of the stair flights are noticeably more sensitive to vibration than prior to the Lyttelton Earthquake. The stairs are constructed with no mid height landing support and support themselves by tension and compression struts in the stair flights. The stairs are therefore connected floor to floor. The cracking in the stairs is due to the seismic forces generated in the stair due to the inter-storey drifts, i.e. due to the floors to which the stairs are connected moving by a different amount during the earthquake.

Concrete Beams and Columns

Significant spalling has been observed on the Lower Ground Floor to a single column near the entry to the services tunnel and the basement access. Some rusting of the reinforcing is visible on the reinforcing exposed. This rusting is likely to have weakened the concrete in this area leading it to be more susceptible to spalling during the earthquake.

Spalling and cracking of the concrete was observed in some locations in the beams and columns at the upper levels, however the scale and extent is not significant. This is typical for Blocks A, B, C and D.

Seismic Joints

There were signs that the seismic joints had experienced movement. In some locations, such as the corridors to Block B at the Lower Ground Floor there are signs of residual displacement.

Roof Steel

No damage was noted to the roof steel work; however, many of the connections back down to the up-stand concrete beams around the Plant Room Level floor slab showed signs of movement. Often grout under the base plates had connections, bolt nuts were loose and occasionally bolts were bent.

Basement Walls

Cracking was observed in the basement walls near the entry to the basement by the services tunnel. This is at the control joint between Block A and the access extension and is likely to be due to the relative movement between the two areas.

At the interface between the retaining walls around the south and east sides of the building and the Ground Floor suspended slab there are signs of movement and some spalled concrete. These elements are not physically connected and the damage indicates the Ground Floor has been moving relative to the retaining wall. There appears to be some residual vertical and horizontal deformation along the east side, but it is unclear if this is due to a permanent deformation of the retaining wall, the main structure, or a combination of both. Further investigation is required to determine the cause of this movement.

Link Bridge

A crack has been noted in the concrete slab at the interface between the Link Bridge and the Main Building (Level 1). There was also some damage to the internal linings around the seismic gap at the Clinical Services end. The Cracking and lining damage are likely due to the relative movement of these two structures.

Emergency Department Extension

Some cracking has been noted to the floor slab across the joint between the Emergency Department Extension and the main Block A structure. Spalling was observed in the floor slab over the joint between the Emergency Department Extension, the main Block A structure and the basement tunnel.

Fit-out

Typical damage to the ceilings and internal partition walls has been described in a separate report [17]. Damage included heavy ceiling tiles falling out, dropping slightly and showing signs of movement on their supports. Review of the ceiling support rails indicated some of these had bowed, sagged and/or twisted and some vertical metal straps that the ceiling system was suspended from were also damaged. Damage to the partition walls included deformation out-of-plane. This damage was typical for Blocks A, B, C and D.

Façade

Some damage was observed to the façade, this was typical for all buildings and is discussed in more detail in Section 3.8 which summaries finding from the façade survey completed by Golemans.

3.4.2 Block B

Settlement Damage

The levels survey undertaken by Fox and Associates [22] indicates a measured differential settlement over the Block B building footprint of 68 mm generally from north to south, the survey results are discussed in more detail in Section 3.6. Residual deformations can be seen in the Lower Ground level of the link bridge to the Clinical Services Building.

Shear Cores

Cracking of the shear cores observed was similar to that noted for Block A above.

Floor Slabs

Cracking is observed in the slab on grade in the Lower Ground Floor in a room adjacent to the entry to the southeast services tunnel. This slab on grade is reinforced with mesh.

Cracking of up to 1.0mm in width has been observed in the suspended slab of Ground Floor and Level 1.

Stairs

Cracking of the stairs observed was similar to that noted for Block A above.

Concrete Beams and Columns

Cracking of the upper levels beams and columns observed was similar to that noted for Block A above.

Seismic Joints

A vertical displacement was observed in the Lower Ground Floor in one room on the seismic joint between Blocks B and C. This vertical displacement was in the overlay slab which had delaminated and displaced vertically.

Roof Steel

As noted for Block A above.

Link Bridge

Similar to in Block A cracking has been observed in the concrete floor slabs at the interface between the Link Bridge and the Main Building. There was also some damage to the internal linings around the seismic gap adjacent to Clinical Services. The cracking and lining damage are likely due to the relative movement of these two structures.

Façade

Bending in a bolt in the connection of a precast panel to the building was observed in one location. There were no washers present in the connection where the damage occurred. All other connections observed in this area had washers. With no washers, the bolt is unlikely to have had the capacity or stiffness to resist the forces caused by the earthquake.

A horizontal crack was observed in one of the precast panels viewed. No cracks were recorded in the Goleman survey of the exterior of the building.

3.4.3 Block C

Settlement Damage

The levels survey undertaken by Fox and Associates [22] indicates a measured differential settlement over the Block C building footprint of 91 mm from north to south as well as a 44 mm differential from west to east, the survey results are discussed in more detail in Section 3.6.

Shear Cores

Cracking of the shear cores observed was similar to that noted for Block A above.

Floor Slabs

Extensive cracking, up to 1.5mm in width, has been observed throughout the suspended slabs of the Ground Floor, Level 2 and Level 3.

Precast Concrete Panels

Minor cracking to precast panels, up to 0.4mm in width, was observed at Level 2 and Level 3.

Stairs

Cracking of the stairs observed was similar to that noted for Block A above.

Concrete Beams and Columns

Cracking of the upper levels beams and columns observed was similar to that noted for Block A above.

Seismic Joints

A vertical displacement was observed in the Lower Ground Floor in one room on the seismic joint between Blocks B and C. This vertical displacement was in the overlay slab which had delaminated and displaced vertically.

Damage was observed in the floor coverings at the seismic gap between Blocks C and D at several levels. In the locations where this was observed the tiles and lino spanned across the seismic joints with no allowance for movement. The Blocks move independently and their relative movement during the earthquake would have caused the damage to the floor coverings.

Roof Steel

As noted for Block A above.

Basement walls

Vertical cracking at regular centres (between 0.8 and 1.2 m) have been noted along the southern wall of the central corridor through the Block C basement that runs east-west. The top of this wall is part of the southern Block C raft foundation and the lower portion is a retaining wall. The cracks are between 0.3 and 1 mm wide. There is also a horizontal crack up to 0.7 mm wide which appears to coincide with the construction joint between the raft and wall.

3.4.4 Block D

Settlement Damage

The levels survey undertaken by Fox and Associates [22] indicates a measured differential settlement over the Block D building footprint of 89 mm from north to south across the building, the survey results are discussed in more detail in Section 3.6.

Shear Cores

Cracking of the shear cores observed was similar to that noted for Block A above.

Floor Slabs

A continuous and large crack, greater than 2.0mm in width, has been observed in the Lower Ground suspended floor slab. Strain hardening testing has been undertaken on the steel reinforcement which runs across the crack, the results of this testing are discussed in Section 3.5.2. Cracking has been identified in the suspended slabs of the Lower Ground and Ground floors. Cracking has been identified in the slab on grade at the southwest corner of the Lower Ground Floor.

Blockwork Walls

Minor cracking has been observed in the blockwork walls at the northern elevation of the Lower Ground Floor.

Stairs

Cracking of the stairs observed was similar to that noted for Block A above.

Concrete Beams and Columns

Cracking of the upper levels beams and columns observed was similar to that noted for Block A above.

Seismic Joints

A vertical displacement was observed in the Lower Ground Floor in one room on the seismic joint between Blocks B and C. This vertical displacement was in the overlay slab which had delaminated and displaced vertically.

Roof Steel

As noted for Block A above.

Basement

Some cracking and damage was observed in the walls in the basement. The most significant cracking has occurred at the concrete wall that spans across the seismic gap between Blocks C and D. This cracking would have been caused by the relative movement between the blocks during the earthquake.

Lift Machine Room

Some minor spalling was observed at the landing in the Lift Machine Room.

3.5 MATERIALS TESTING

3.5.1 Concrete Strength

Testing of the concrete strength has been carried out by Holmes Solutions LP in the shear walls of the north shear core at Lower Ground in Block B. The testing was carried out in August 2011 using a Proceq Silverschimdt Rebound Hammer. Calibration was carried out using the Proceq 10th percentile curve. The use of the 10th percentile curve provides a conservative estimate of the concrete strength. The probable strength of the concrete could be 20 to 25% higher than the results achieved. The results could be calibrated against results from cylinder tests; however this has not been carried out to date.

A copy of the Holmes Solutions report is attached in Appendix D [18].

Concrete strengths were tested in four locations and the measured strengths had an average of 23 MPa, 25.5 MPa, 27.5 MPa and 28 MPa. Concrete strengths for the lower ground level and the upper levels are noted as different on the drawings. Adjustments to the concrete strengths in the model were made to account for the two different strengths used in construction.

3.5.2 Strain Hardness Testing Steel in Concrete Floor Slabs

Detailed investigations of the Parkside structures identified a series of cracks in the Lower Ground Floor slab of Block D. Although the cracks were not continuous, they appeared to follow a similar line across a number of rooms. In some places the cracks had a width of greater than 2.0 mm. Desktop calculations have shown that the steel reinforcement across a crack of this width may have undergone significant yielding. To determine if significant yielding had occurred (and as a consequence a loss in strain capacity of the reinforcement bars) two slab locations, both top and bottom, were chosen to test.

Holmes Solutions LP carried out the tests on the reinforcement to determine the level of strain hardening that had occurred in the bars due to the earthquakes. A copy of the Holmes Solutions report is attached in Appendix D [19].

The results indicate that all bars tested had undergone some yielding and experienced some loss in strain capacity. Strain capacity is the measure of the bars ability to elongate without strength degradation. A bar crossing a crack in the concrete must elongate for the crack to form, if the elongation yields the bars crossing the crack, some strain hardening occurs and results in a loss in strain capacity. Steel reinforcement has a finite amount of strain capacity before it rapidly looses strength and snaps.

A summary of the results is provided in Table 3-2 and a graphical representation of these results is presented in Figure 3-2. The average potential induced strains were between 3.61 to 5.75% (out of a total pre-earthquake strain capacity of around 20.5 %). The range of potential induced strains is indicated by the darker hatched area of Figure 3-2. The potential lost strain capacity was between 12 and 35 % of the peak strain, this is indicated by the larger shaded zone in Figure 3-2.

Table 3-1: Summary of Strain Hardness Testing of Concrete Slab

Test Reference	Parkside Block and room number	Location	Bar diameter (mm)	Average Potential Induced Strain (%)	Potential Lost Strain Capacity (%)	Does remaining strain capacity meet code requirements?*
109356-12A	D – L232	Upper side of the Lower Ground Floor Slab	16	5.75	22-35	No
109356-12B	D – L232	Upper side of the Lower Ground Floor Slab	16	3.63	12-27	No
109356-13	D – B09	Under side of the Lower Ground Floor Slab	16	3.61	15-21	Yes
109356-14	D – B09	Under side of the Lower Ground Floor Slab	16	4.22	16-27	No

* Remaining strain capacity meets current code requirements if potential lost strain capacity is less than 25 %.

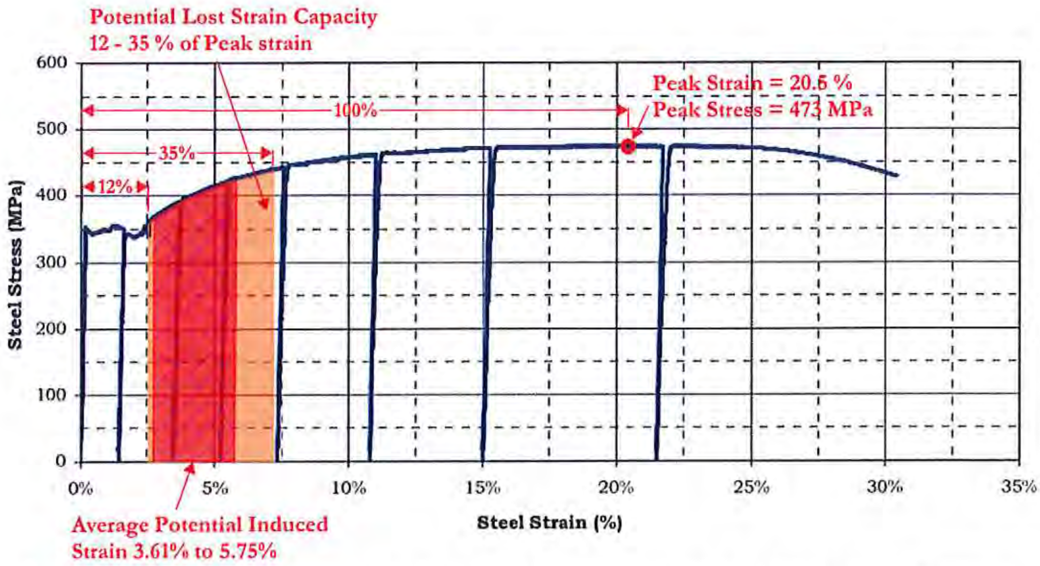


Figure 3-1: Stress-strain curve for steel reinforcement in Parkside buildings, shaded area indicates strain hardness testing results from Block D, Lower Ground Floor Slab

There has been a measurable loss in strain capacity in the steel reinforcement crossing these cracks. This means the amount the bars can elongate before they fail has been reduced, which in turn means the ability for the slab to undergo future movement has been reduced.

The potential loss of strain hardening capacity of the steel reinforcing bars across the cracks in the Lower Ground Floor slab is considered significant. However, the stress-strain curve determined from testing a piece of steel removed from the structure suggests it has a higher strain capacity than required by the current steel reinforcement code for ductile steel (15 % is required and 20.5 % was measured) [20]. Therefore, although some capacity has been lost, the slab still satisfies the requirements of a new building if less than 25 % strain capacity has been lost. The final column in Table 3-2 shows that one of the four bars tested has a potential lost strain capacity that is still within this limit. However, the other three possibly have lost more than 25 % strain capacity and therefore the remaining strain capacity is less than required by the current code.

Steel was only tested at four locations and these were all crossing the same concrete crack. However, this crack is the largest noted to date in observations of the suspended concrete floor slabs (2-3 mm wide). Therefore, it is reasonable to assume that steel crossing smaller cracks will not have experienced strain hardening that reduces its capacity less than current code requirements and this is a localised issue.

3.5.3 Strain Hardness Testing of Steel in Concrete Core Walls

The Non-Linear Time History Analyses (NLTHA) indicated that during the Lyttelton Earthquake the vertical steel reinforcement in the core walls between the Lower Ground Floor and the Ground Floor could have undergone significant yielding. Although the cracks in these walls were not observed to be larger than 0.5 mm, it is possible they were larger than this during shaking and have closed up under gravity once the shaking stopped. To determine if significant yielding had occurred (and as a consequence a loss in strain capacity of the vertical bars) several locations were chosen to test. In the four Parkside Blocks there are a total of 14 shear cores. Three different shear cores, in three different Blocks were chosen for testing of bars at the Lower Ground Floor level.

Holmes Solutions LP carried out the tests on the reinforcement to determine the level of strain hardening that had occurred in the bars due to the earthquakes. A copy of the Holmes Solutions report is attached in Appendix D [19].

The results indicate that all bars tested had undergone some yielding and experienced some loss in strain capacity. Strain capacity is the measure of the bars ability to elongate without strength degradation. A bar crossing a crack in the concrete must elongate for the crack to form, if the elongation yields the bars crossing the crack, some strain hardening occurs and results in a loss in strain capacity. Steel reinforcement has a finite amount of strain capacity before it rapidly looses strength and snaps.

A summary of the results is provided in Table 3-2 and a graphical representation of these results is presented in Figure 3-2. The average potential induced strains were between 2.73 and 6.41% (out of a total pre-earthquake strain capacity of around 20.5 %). The range of average potential induced strains is indicated by the hatched shaded area in Figure 3-2. The potential lost strain capacity was between 12 and 41 % of the peak strain, this is indicated by the larger shaded zone in Figure 3-2.

Table 3-2: Summary of Strain Hardness Testing of Concrete Core Walls

Test Reference	Parkside Block and room number	Location	Bar diameter (mm)	Average Potential Induced Strain (%)	Potential Lost Strain Capacity (%)	Does remaining strain capacity meet code requirements?*
109356-1	C – L152	NW core, corner of core wall	24	2.73	12-16	Yes
109356-2	C – L152	NW core, centre of core wall	20	3.75	14-24	Yes
109356-3	B – L113	N core, centre of core wall (near corner)	24	6.41	24-41	No
109356-4	D – L240	SW core, centre of core wall	20	5.56	18-41	No
109356-5	D – L240	SW core, adjacent door	24	5.03	20-30	No

* Remaining strain capacity meets current code requirements if potential lost strain capacity is less than 25 %.

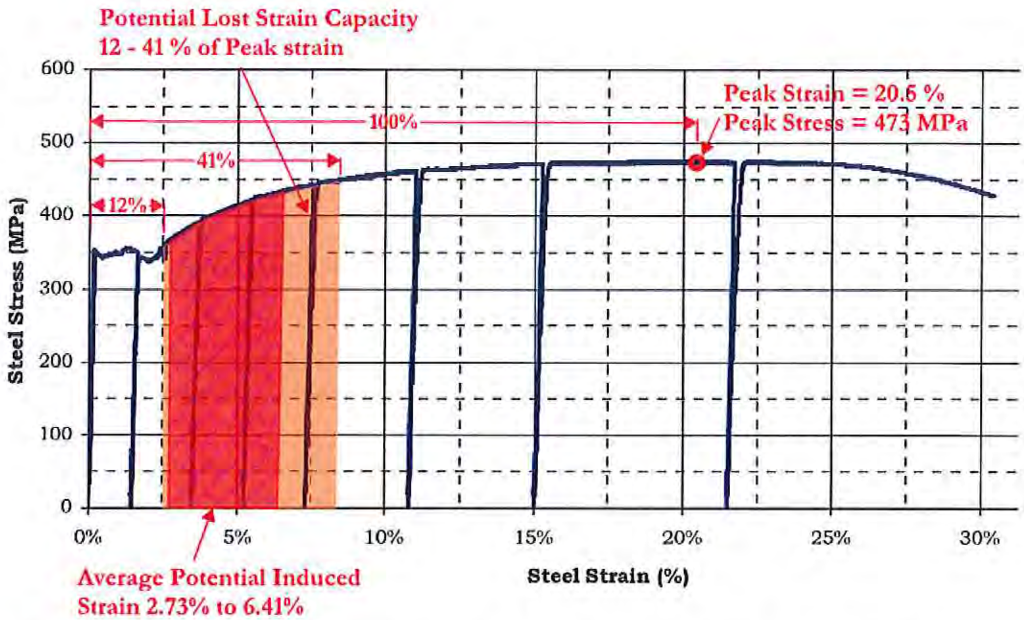


Figure 3-2: Stress-strain curve for steel reinforcement in Parkside buildings, shaded area indicates strain hardness testing results from core walls

There has been a measurable loss in strain capacity in the steel reinforcement crossing these cracks. This means the amount the bars can elongate before they fail has been reduced, which in turn means the ability for the slab to undergo future movement has been reduced.

The potential loss of strain hardening capacity of the vertical reinforcing bars in the shear cores is considered significant. However, the stress-strain curve determined from testing a piece of steel removed from the structure suggests it has a higher strain capacity than required by the current steel reinforcement code for ductile steel (15 % is required and 20.5 % was measured) [20]. Therefore, although some capacity has been lost, the walls still satisfy the requirements of a new building if less than 25 % strain capacity has been lost. The final column in Table 3-2 shows that two of the five bars tested have a potential lost strain capacity within this limit. However, the other three possibly have lost more than 25 % strain capacity and therefore the remaining strain capacity is less than required by the current code.

It should be noted that the steel tested was crossing crack widths between 0.2 and 0.5 mm and testing was carried out in only five locations in the Lower Ground Floor. There are cracks of similar widths up the height of the building in each shear core and therefore it is possible that strain hardening and loss of strain capacity is not restricted to bars in the Lower Ground Floor level walls only, but is likely more widespread.

3.6 LEVELS SURVEY

A verticality survey was carried out by Fox & Associates on 16 June 2011 and the results are summarised in their report dated 28 June 2011 [21].

The results of the verticality survey indicated tilts of between 1 mm/m and 4 mm/m. No clear pattern in the direction or magnitude of the verticality measurements was noted. The Parkside buildings are clad with precast concrete panels with a rope finish which are bolted to the perimeter frames. It is likely that the tilts measured have been impacted by construction tolerance and the finish to the panels.

A levels survey was carried out by Fox & Associates in May 2012 and the results are summarised in their drawings titled *CDHB – Parkside – Floor Levels* dated 24 May 2012 [22]. Levels were taken on the Lower Ground Floor in Blocks A, B, C and D and on the underside of the Ground Floor in Blocks A and B where differential movements of Blocks A and B have been observed on site. The levels taken on the underside of the Ground Floor confirmed the slopes measured on the Lower Ground Floor.

Fox and Associates extended the levels survey in November 2012 to include the Emergency Department Extension at Ground Floor level. The results are shown on their drawings titled CDHB – Christchurch Hospital – Emergency Department Elevations [23].

The levels survey indicates that all of the Blocks have undergone differential settlement with falls recorded from north to south in all Blocks. A summary of these results is shown in Table 3-3.

Table 3-3: Summary of Levels Survey Results

	Maximum Differential Measured (mm)	Differential in north-south Measured (mm)	Maximum Slope Measured	Typical Slope Measured
Block A	82	81	1:180	1:430
EDE	53	-	1:220	-
Block B	68	54	1:200	1:370
Block C	94	91	1:185	1:400
Block D	89	79	1:190	1:400
	Total = 102	-	**Max = 1:180**	**Ave = 1:400**

Steeper slopes were measured on the south side of the buildings than the north which suggests the foundation rafts on the south side of the buildings have rotated more than the foundation rafts on the north. Cracking has been observed in the Ground Floor slab that appears to be consistent with this change in slope. Typically the buildings have one more concrete suspended level on the south side than they do on the north.

The differential settlement of the four Blocks has not been even. As well as the slope down to the south, differential settlements have been recorded across Block C in the east-west direction at 44mm. Localised differential settlements in the east-west direction have been recorded at the east end of Blocks D and A. These differential settlements relate to changes in foundations from stiff rafts foundation beams to thinner slabs. The differential settlements recorded on the Lower Ground Floor are shown in Figure 3-3.

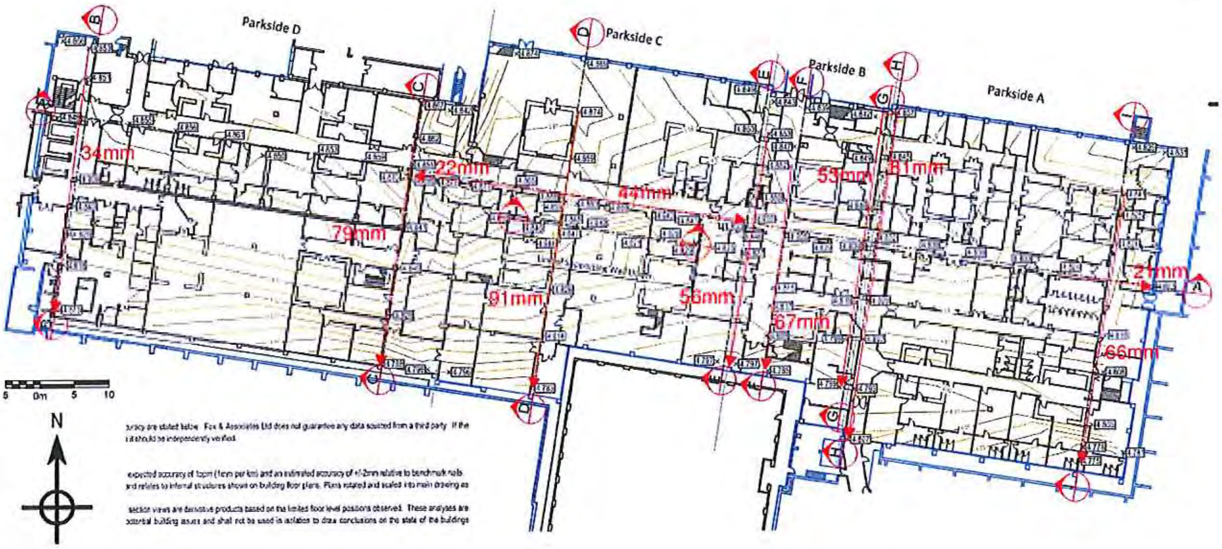


Figure 3-3: Differential Settlements Recorded in the Levels Survey.

The levels survey indicated a step at the interface between Blocks A and B (typically between 5 and 10 mm), this is consistent with observations on site of step at seismic joints up building and illustrates that the settlement of the four blocks has not been even. A horizontal offset of Block A relative to Block B can also be observed at the seismic joints and is shown in Figure 3.1 by the offset at the handrail.

Figure 3-4: Parkside A-B Junction at 1st Floor

Slopes of up to 1: 180 have been measured on the southeast corner of Blocks A and D and the northeast corner of Block D.

Differential settlement was also observed in the Emergency Department Extension survey of the Lower Ground Floor. The levels survey was taken on a suspended concrete floor and therefore measured some floor sag as well as differential settlement. The summary values presented in Table 3-3 were derived only using measurements taken over column locations and therefore do not include the influence of floor sag; the column locations have been circled in Figure 3-5. In Figure 3-5 the high areas are shaded red, while the low areas are shaded blue. It can be seen that the columns along the east side have settled more than the others.

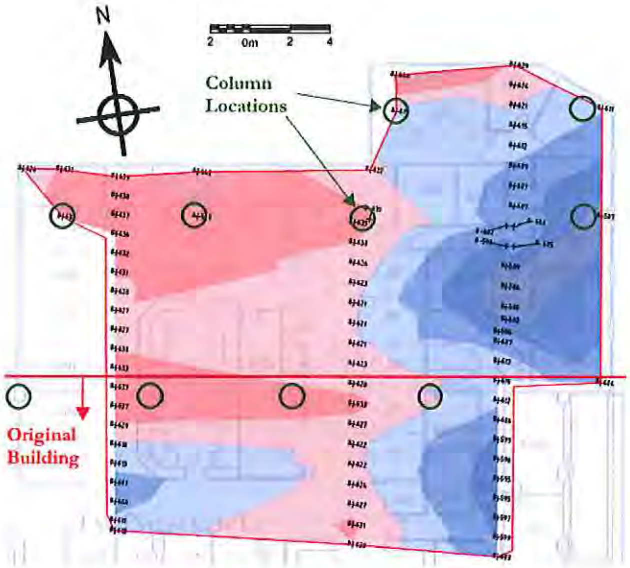


Figure 3-5: Levels Measured on the Ground Floor of the Emergency Department Extension

The measured differential levels are also consistent with the observations made in the Tonkin and Taylor post earthquake geotechnical assessment [24,25], this assessment is discussed further in Section 3.7.

3.7 GEOTECHNICAL INVESTIGATION

Geotechnical investigations have been carried out by Tonkin & Taylor Ltd and the results are summarised in their reports dated June 2011 [24] and September 2011 [25].

The investigation concluded that there is a non-liquefiable gravel layer present from basement level to 6.0m below basement level. There is a dense sand layer between 6.0 and 8.5m below basement level which liquefied during the 22 February 2011 earthquake.

The earthquake ground damage observed included settlement and rotation of Parkside Block A relative to Block B with Block A appearing to have moved to the north and east and settlement to the south of the Parkside Blocks adjacent to the hospital entry.

The geotechnical report concludes that the observed damage is unlikely to have been caused by liquefaction of the sand layer below the basement. The observed damage is more likely to have been caused by residual displacements due to the dynamic loads that were applied to the ground below the building foundation during the earthquakes.

The geotechnical report also concludes that:

- Further ground damage is not expected in a future SLS1 earthquake

- In future SLS2 level or greater earthquakes, ground damage of a similar nature to that observed from the 22 February 2011 earthquake may occur, i.e. settlements of the foundations and rotation of the building.

- It is likely that the capacity of the existing foundations to resist static and seismic loads is the same as it was prior to the 22 February 2011 earthquake.

- The differential settlements and building movements may have resulted in a minor redistribution of foundation loads, but this is not believed to be significant.

Tonkin & Taylor have also prepared a memo on likely cause of the observed differential settlement [26]. The memo states that the observed settlement is considered likely to have occurred as a results of the 22 February 2011 earthquake due to dynamic loads from the building exceeding the elastic range of the foundation soils, resulting in residual displacement of the building foundations.

The possibility of elastic settlements is discussed, total elastic settlements are expected to have been less than 25mm and geotechnical investigations do not indicate different ground conditions across the building footprint that would results in the settlement pattern observed in the levels survey. The memo also states that settlements due to consolidation or creep are unlikely to have been significant.

3.8 FAÇADE SURVEY

A survey was carried out on the exterior of the building by Goleman and the earthquake damage observed is outlined in their report dated 4 October 2011 [27].

The damage recorded included:

- Torn sealant between precast cladding panels
- Damage to flashings between the buildings
- Areas of spalled concrete on the panels and columns
- Cracks in the panels and columns
- Offset panels
- Settlement and damage to paths and paving
- Damaged soffit linings adjacent to the seismic joints.

The repair work for these items is detailed in Table 4-5.

3.9 FURTHER INVESTIGATIONS REQUIRED

3.9.1 Investigations Required During Repairs

The following investigations are required during repairs:

- Detailed crack mapping of concrete elements (walls, columns, beams and slabs), especially in areas where concrete is currently obscured by linings and furnishings. This work is currently being undertaken and is ongoing.

- Destructive investigations of cast-in fixing of one of the typical precast panels that has been removed from the building during recent retrofit works.

- Opening up to inspect a selection of precast panel connections in the South wing of Block A, Levels 2 and 3.

- Opening up to inspect a selection of precast panel connections on the west elevation of Block D, Levels 2, 3 and the Plant level.

- Opening up to inspect the precast concrete cladding panel connections above main entrances and exits.

Depending on the outcome of the further investigations additional further opening up and/or repair details maybe required.

3.10 POST EARTHQUAKE BUILDING CAPACITY

Based on our investigations to the date of this report, we do not consider the Parkside Buildings to have any significant reduction in gravity load resistance. However, it should be noted that the assessment did indicate that the gravity capacity of the Block B link bridge is only 85 % of the current code requirements (this is discussed in Section 2.2.12).

The cracking to the shear core walls observed to date in the four blocks has reduced the stiffness of the buildings. In addition, the strain hardness testing completed on a sample of steel reinforcement crossing these cracks indicates some strain capacity of the steel reinforcement has been lost. Although the lost strain capacity in some bars is small enough that the strain capacity still meets current codes, it still reduces the ability of the elements to sustain future strain and continue to have sufficient strain capacity.

Differential settlement has been measured, the current magnitude of this is unlikely to lead to global instability; however, the impact of these deformations on future building performance needs to be considered.

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 completion of the recommended repair of the structural damage, the lateral load resisting performance of the structure should be restored to close to what it was prior to the Darfield earthquake.

4. OBSERVED DAMAGE & REPAIRS RECOMMENDED

4.1 TYPICAL OBSERVED DAMAGE & REPAIRS RECOMMENDED

Table 4-1, 4-2, 4-3, 4-4 and 4-5 provide a photographic summary of the observed damage and typical repairs required for the Parkside Blocks A, B, C and D and the façades respectively. These tables should be read in conjunction with the Appendix A Record of Observations and Appendix B Reference Plans. The Repair Specification [2] referred to in the Tables has been issued separately.

It should be noted that some of these repairs are complete, or underway, this is not recorded in the following tables of typical damage, but is noted in the more detailed Record of Observations provided in Appendix A.

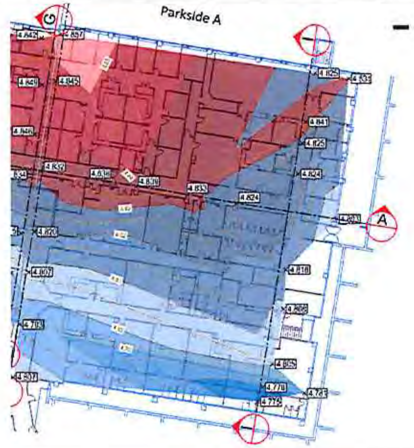
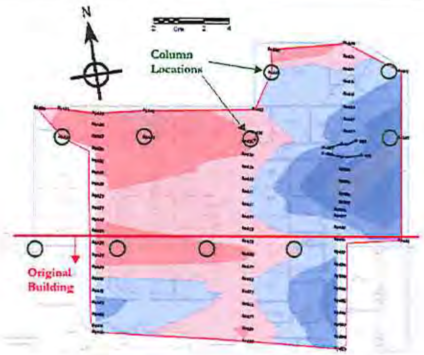
Generally the aim of the repair work indicated is to restore the structure to its pre-earthquake state as far as practicable. The repairs address strength, stiffness and durability of the structural elements. As building re-levelling is a complex issue this has been discussed in more detail in Section 4.2.

Recommended strengthening to improve the buildings lateral load capacity is outlined in Section 5.

In some areas of potential damage the structure was not inspected due to linings or claddings obstructing inspection. It should be noted that more damage may be identified during the repair works and (if required) additional repair details will be specified accordingly.

Note that our observations have been restricted to structural aspects only. Waterproofing elements, electrical and mechanical equipment, fire protection and safety systems, service connections, water supplies and sanitary fittings have not been inspected or reviewed, and secondary elements such as windows and fittings have not generally been reviewed.

Table 4-1: Block A – Photographic Summary of Damage Observed & Repairs Required

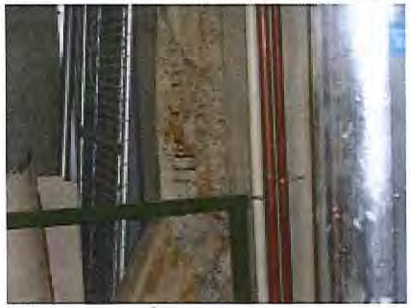
Damaged Item	Location	Recommendation	Example
1. Settlement			
1.1. Differential settlement of up to 81mm. Slope in the Lower Ground Floor slab of 1:180	Measured on the Lower Ground Floor and underside of the Ground Floor	For discussion on the remediation work required see Section 4-2.	
1.2. Differential settlement in Emergency Department Extension of up to 53 mm and slope between column supports of 1:220	Measured on the Ground Floor	For discussion on the remediation work required see Section 4-2.	

Damaged Item	Location	Recommendation	Example
1.3. Asphalt damage and settlement	Ambulance bay on the south side of Block A	Repair to asphalt is being addressed as part of the tunnel repair	
2. Shear Cores			
2.1. Flexural / shear cracks between 0.2mm and 0.5mm	Internal shear core walls.	Inspect the shear core walls at all levels and epoxy inject all cracks that are greater than 0.3mm in width in accordance with the HCG Specification.	

Damaged Item	Location	Recommendation	Example
3. Slabs			
3.1. Slab cracks up to 2.5mm wide in slab-on-grade.	Lower Ground Floor slab on grade in corridor L17	The drawings indicate that this is a slab on grade reinforced with 665 mesh located between the basement wall to the east and the columns and foundation beam on the west face of Block A. The crack and its width indicate that the mesh is likely to have fractured and the tie between the exterior foundation beam and the remainder of the building compromised. Break out the slab between Grids A and D and reinstate with a reinforced slab as per Sketch SK100 and SK100A Rev A in Appendix C.	 
3.2. Floor Cracks – up to 1.6mm in width in suspended slabs.	Lower Ground slab in Rm. L68, L72, L75, L89-L91, L121, soffit above L22, Rm. 1.77 Note: inspections of floor slabs is ongoing.	Repair recommended on case-by-case basis. General repair is to epoxy inject all cracks that are greater than 0.3mm in width in accordance with the HCG Specification.	

Damaged Item	Location	Recommendation	Example
3.3. Floor cracks – up to 1.0mm in width	Across joint between main Block A building and Emergency Department Extension Slab	Repair recommended on case-by-case basis. General repair is to epoxy inject all cracks that are greater than 0.3mm in width in accordance with the HCG Specification.	
3.4. Spalling to slab	Slab across joint between main Block A building, Emergency Department Extension and basement tunnel	Repair to be confirmed by HCG	

Damaged Item	Location	Recommendation	Example
4. Stair cracking			
4.1. Cracking to the stairs at all levels	Stair Nos. 1 and 2 – cracks in the soffits at all levels , crack in stair at change in direction at 1st Floor	Epoxy inject cracks that are 0.3mm or greater in width in accordance with the HCG specification. Strengthening details for the stairs to provide support for the stairs at the mid-height landing and separate the stairs so that they are not connected floor to floor have been issued as a separate package.	 
			

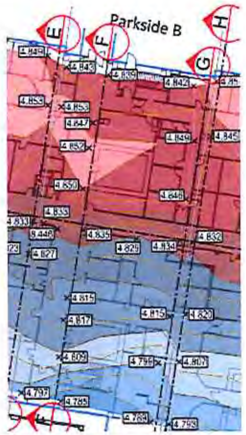
Damaged Item	Location	Recommendation	Example
5. Concrete Columns			
5.1. Cracking to concrete columns	Level 2 courtyard columns	Check all columns and epoxy inject cracks greater than 0.2mm in width as per HCG specification. Repair areas of spalling in accordance with the HCG specification.	
5.2. Spalling of cover concrete on column	Lower Ground Room L121	Break back to expose rusted reinforcing. Wire brush exposed reinforcing that has rusted and treat with Sika Monotop Primer. Repair areas of spalling in accordance with the HCG specification.	
6. Beams			
6.1. Spalling to beams	Level 2 courtyard perimeter beam	Repair areas of spalling in accordance with the HCG specification.	

Damaged Item	Location	Recommendation	Example
7. Seismic Joints			
7.1. Permanent displacement at seismic joint of 5mm at Lower Ground Floor	Lower Ground Floor in L117	Replace seismic gap cover with a cover of greater width.	
8. Roof Steel			
8.1. Damage to base plate connections (spalling of grout and loose/bent bolts)	Around perimeter of Plant Room Level floor	Replace grout. Tighten bolts where nuts loose. Replace bent bolts.	

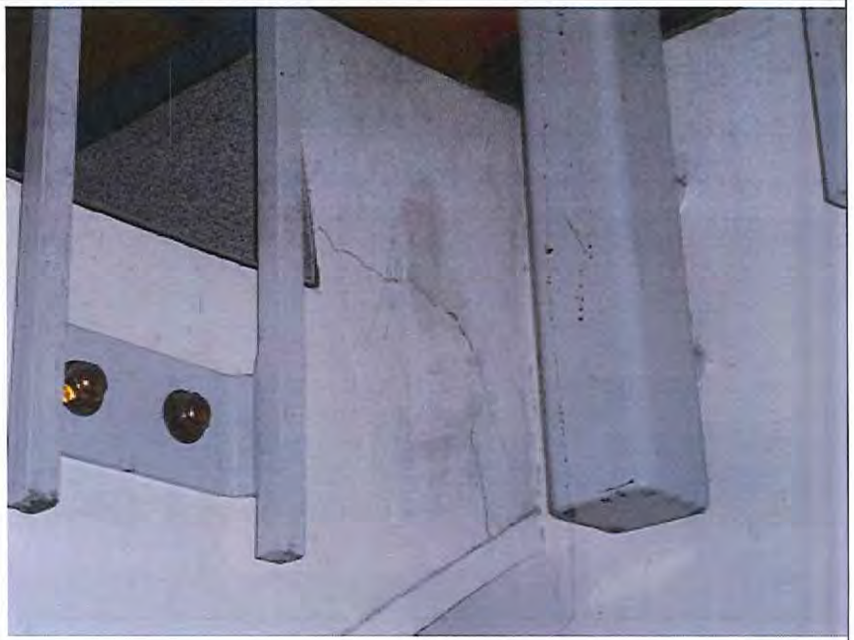
Damaged Item	Location	Recommendation	Example	
9. Basement				
9.1. Cracking to concrete wall	B03A /B03 west wall	Break out damaged concrete locally between structural walls so that construction of damaged wall and repair methodology can be determined.		
9.2. Horizontal crack in the tunnel wall – approximately 0.3mm in width	In tunnel B03	Check all tunnel walls and epoxy inject cracks greater than 0.2mm in width as per HCG specification. HCG have undertaken a condition survey of this area and the current recommendation is that wide spread epoxy injection of historic shrinkage cracks or cracks widened by the earthquakes is not considered necessary – refer HCG report dated April 2014. [33]		

Damaged Item	Location	Recommendation	Example	
9.3. Spalling of concrete at the top of the retaining walls and signs of residual movement (25 mm residual in east-west direction of east retaining wall)	Area B01 along south and east retaining walls at interface with Ground Floor slab	Where gap has developed need to reinstate water seal. Further investigation is required to determine cause of residual displacement. Repair areas of significant spalling in accordance with the HCG specification.		
9.4. Spalling of the concrete slab in the crawl space	Area B01	HCG has issued separate repair documentation. Repair requirements are supplied in Site Report 60 (SR#60) dated 23 September 2013.		
10. Link Bridge to Clinical Services				
10.1. Crack in the concrete slab at the junction of the link bridge and the building	Junction of link bridge and building in Room 1.104 at Level 1	Check all link bridge slabs at the junction of the building at all levels and epoxy inject cracks greater than 0.3mm in width as per HCG specification.		

Table 4-2: Block B - Photographic Summary of Damage Observed & Repairs Required

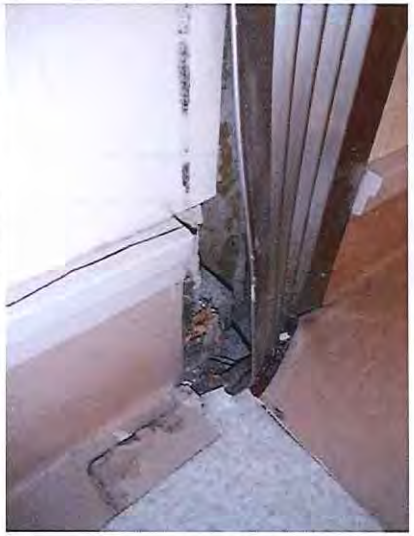
Damaged Item	Location	Recommendation	Example
1. Settlement			
1.1. Differential settlement of up to 54mm.	Measured on the Lower Ground Floor and underside of the Ground Floor	For discussion on the remediation work required see Section 4-2.	
1.2. Evidence of residual movement	Damage to tiled area north of Block B where service tunnel below.	For discussion on the remediation work required see Section 4-2.	
2. Shear Cores			

Damaged Item	Location	Recommendation	Example
2.1. Flexural / shear cracks between 0.2mm and 0.5mm	Internal shear core walls.	Inspect the shear core walls at all levels and epoxy inject all cracks that are greater than 0.3mm in width in accordance with the HCG Specification.	
3. Slabs			
3.1. Slab on grade cracks up to 0.4mm wide	Lower Ground Floor slab on grade in room L120	The drawings indicate that this is a slab on grade reinforced with mesh. Epoxy inject all cracks that are greater than 0.2mm in width in accordance with the HCG Specification.	

Damaged Item	Location	Recommendation	Example
3.2. Suspended slab cracks up to ~0.7mm	Ground Floor slab. Rm. G162, Rm. 3.176-179	Repair recommended on case-by-case basis. General repair is to epoxy inject all cracks that are greater than 0.3mm in width in accordance with the HCG Specification.	 
4. Stair cracking			
4.1. Cracking to the stairs at all levels	Stair Nos. 3 and 4 – cracks in the soffits at all levels and at the junction between the stairs and the support slab.	Epoxy inject cracks that are 0.3mm or greater in width in accordance with the HCG specification. Strengthening details for the stairs to provide support for the stairs at the mid-height landing and separate the stairs so that they are not connected floor to floor have been issued as a separate package.	

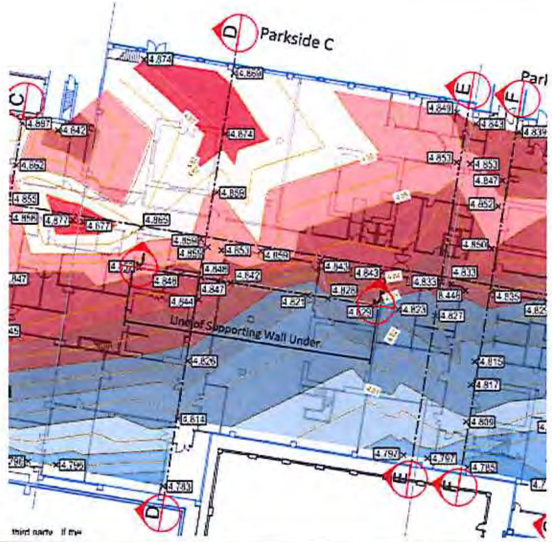
Damaged Item	Location	Recommendation	Example
5. Concrete Columns			
5.1. Cracking to concrete columns and some spalling	Level 3 columns	Check all columns and epoxy inject cracks greater than 0.2mm in width where external and 0.3mm where internal as per HCG specification. Repair areas of spalling in accordance with the HCG specification.	 
6. Precast Panels			
6.1. Cracks in precast panels up to 0.4mm in width	Panels in rooms G129 at Ground Floor level, 1.136 at First Floor	Check all panels and epoxy inject cracks greater than 0.2mm in width as per HCG specification.	

Damaged Item	Location	Recommendation	Example
6.2. Panel connection bolt has yielded	Panel in room 3.122.	Replace fixing. HCG to provide detail	
7. Beams			
7.1. Cracking in beams around courtyard	Level 3 courtyard surround	Check all beams/panels and epoxy inject cracks greater than 0.2mm in width as per HCG specification.	
8. Roof Steel			
8.1. Damage to base plate connections (spalling of grout and loose/bent bolts)	Around perimeter of Plant Room Level floor	Replace grout. Tighten bolts where nuts loose. Replace bent bolts.	No photo

Damaged Item	Location	Recommendation	Example
9. Link Bridge to Clinical Services			
9.1. Crack in the concrete slab near the junction of the link bridge and the building	Junction of link bridge and building in Room 1.117 at Level 1	Check all link bridge slab at the junction of the building at all levels and epoxy inject cracks greater than 0.3mm in width as per HCG specification.	
9.2. Damage to seismic joint	Between link bridge and Clinical Services Building	Non-structural repair. However, when repair is undertaken it should be ensured nothing crosses the seismic gap that will inhibit differential movement. Some existing timber framing may need to be trimmed back.	 

Damaged Item	Location	Recommendation	Example	
9.3. Damage due to differential settlement of service tunnel and surrounding ground.	In Lower Ground Floor level of link tunnel can see service tunnel location where it passes diagonally under link floor and also where extends outside under tiled area.	For discussion on remediation work requires see Section 4.2 on building re-levelling.		

Table 4-3: Block C - Photographic Summary of Primary Observed & Repairs Required

Damaged Item	Location	Recommendation	Example
1. Settlement			
1.1. Differential settlement of up to 91mm.	Measured on the Lower Ground Floor	For discussion on the remediation work required see Section 4-2.	
2. Shear Cores			
2.1. Flexural / shear cracks between 0.2mm and 0.4mm	Internal shear core walls.	Inspect the shear core walls at all levels and epoxy inject all cracks that are greater than 0.3mm in width in accordance with the HCG Specification.	

Damaged Item	Location	Recommendation	Example
3. Slabs			
3.1. Slab cracks up to 1.5mm wide	Cracks observed in soffit of slab over room L161 and underside of Lower Ground slab over B04. Ground Floor slab Rm. G.161-163, G167-172, G198. Cracks observed throughout south of Level 2 & 3 slab (observed during new wards project fit-out).	Repair recommended on case-by-case basis. General repair is to epoxy inject all cracks that are greater than 0.3mm in width in accordance with the HCG Specification.	
3.2. 15mm displacement across the seismic joint between Blocks B and C at Lower Ground	Room L127	Grind back overlay slab until slabs are level across the seismic gap. Reinstate the seismic gap cover.	

Damaged Item	Location	Recommendation	Example
4. Stair cracking			
4.1. Cracking to the stairs at all levels	Stair Nos. 5 – cracks in the soffits.	Epoxy inject cracks that are 0.3mm or greater in width in accordance with the HCG specification. Strengthening details for the stairs to provide support for the stairs at the mid-height landing and separate the stairs so that they are not connected floor to floor have been issued as a separate package.	No photo
5. Concrete Columns			
5.1. Cracking to concrete columns and some spalling	Level 3 columns	Check all columns and epoxy inject cracks greater than 0.2mm in width where external and 0.3mm where internal as per HCG specification. Repair areas of spalling in accordance with the HCG specification.	

Damaged Item	Location	Recommendation	Example
6. Concrete Beams			
6.1. Cracking in the concrete beams up to 0.3mm	Level 4	Check all beams and epoxy inject cracks greater than 0.3mm in width as per HCG specification.	
7. Seismic Joints			
7.1. Damage to floor coverings at the seismic gap between Blocks C and D	Ground Floor and Lower Ground Floor at entry to stair and Electricians room	Provide seismic gap cover to allow movement on the seismic joint without damages to floor coverings.	 
7.2. Damage to seismic joint covers	Level 4	Replace or repair covers where damaged	

Damaged Item	Location	Recommendation	Example
8. Roof Steel			
8.1. Damage to base plate connections (spalling of grout and loose/bent bolts)	Around perimeter of Plant Room Level floor	Replace grout. Tighten bolts where nuts loose. Replace bent bolts.	
9. Basement Walls			
9.1. Cracking to retaining wall. Cracks at 0.8 to 1.2m centres along the length of the wall (0.3mm - 1mm). Horizontal crack 1.6 m above basement slab	South wall of central east-west running corridor	Epoxy inject cracks greater than 0.2mm in width as per HCG specification. HCG have undertaken a condition survey of this area and the current recommendation is that widespread epoxy injection of historic shrinkage cracks or cracks widened by the earthquakes is not considered necessary – refer HCG report dated April 2014. [33]	 

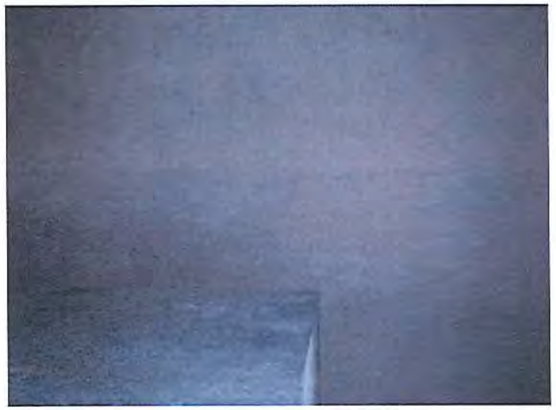
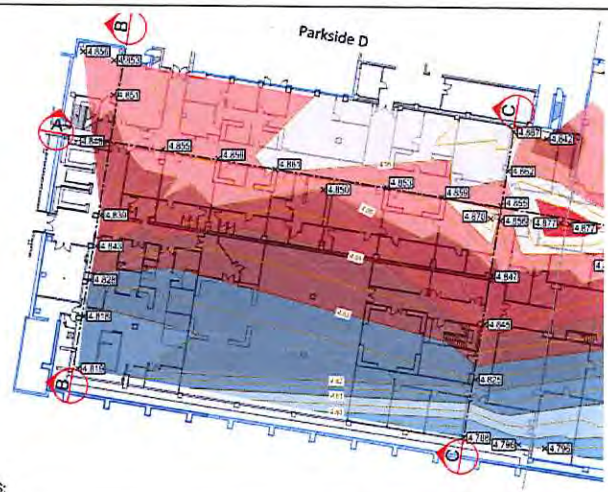
Damaged Item	Location	Recommendation	Example
10. Precast Concrete Panels			
10.1. Vertical cracking (~0.4mm max) at mid-span of precast panels	Along southern elevation of courtyard	Panels have been removed as part of the New Wards project	
10.2. Horizontal cracking (~0.4mm max) at mid-height of precast panels	Along south-western elevation of Block C	Panels have been removed as part of the New Wards project	

Table 4-4: Block D - Photographic Summary of Damage Observed & Repairs Required

Damaged Item	Location	Recommendation	Example
1. Settlement			
1.1. Differential settlement of up to 81mm. Slope in the Lower Ground Floor slab of approximately 0.6% (1:160)	Measured on the Lower Ground Floor	For discussion on the remediation work required see Section 4-2.	
2. Slabs			
2.1. Large (>2.0mm), continuous crack to suspended slab at Lower Ground	Crack has been observed running through rooms Rm. L228, 229A, 231, 232, 232A, 232E and 235. Investigations are ongoing.	Repair recommended on case-by-case basis. General repair is to epoxy inject all cracks that are greater than 0.3mm in width in accordance with the HCG Specification.	

Damaged Item	Location	Recommendation	Example
2.2. Cracking to slab on grade	Southwest corner of Block D, Ground Level. Rm. G374 and 375	Epoxy inject cracks that are 0.3mm or greater in width in accordance with the HCG specification. Note: Work has been completed.	
3. Shear Cores			
3.1. Flexural / shear cracks between 0.2mm and 0.4mm	Internal shear core walls.	Inspect the shear core walls at all levels and epoxy inject all cracks that are greater than 0.3mm in width in accordance with the HCG Specification.	No photo
4. Stair cracking			
4.1. Cracking to the stairs at all levels	Stair Nos. 6 & 7 – cracks in the soffits.	Epoxy inject cracks that are 0.3mm or greater in width in accordance with the HCG specification. Strengthening details for the stairs to provide support for the stairs at the mid-height landing and separate the stairs so that they are not connected floor to floor have been issued as a separate package.	

Damaged Item	Location	Recommendation	Example
5. Concrete Beams			
5.1. Cracking in the concrete beams up to 0.4mm	Level 4	Check all beams and epoxy inject cracks greater than 0.3mm in width as per HCG specification.	
6. Seismic Joints			
6.1. Damage to seismic joint and ramp at entry to Christchurch Women's	Level 3	Repair spalled concrete in accordance with HCG specification. Lift and relay or replace covers to seismic gap. Architectural detail provided by others.	
7. Roof Steel			
7.1. Damage to seismic joint and ramp at entry to Christchurch Women's	Around perimeter of Plant Room Level floor	Replace grout. Tighten bolts where nuts loose. Replace bent bolts.	No photo

Damaged Item	Location	Recommendation	Example
8. Basement			
8.1. Cracking to concrete slab in crawl space	Area BM.13	Inspect the crawl space slabs and epoxy inject the slab where cracks are 0.2mm or greater in width in accordance with the HCG specification. HCG have undertaken a condition survey of this area and the current recommendation is that widespread epoxy injection of historic shrinkage cracks or cracks widened by the earthquakes is not considered necessary – refer HCG report dated April 2014. [33]	No photo
8.2. Lift shaft walls – cracking up to 0.4mm in width	West lift shafts	Check all walls and epoxy inject cracks greater than 0.2mm in width as per HCG specification. HCG have undertaken a condition survey of this area and the current recommendation is that widespread epoxy injection of historic shrinkage cracks or cracks widened by the earthquakes is not considered necessary – refer HCG report dated April 2014. [33]	

Damaged Item	Location	Recommendation	Example
8.3. Damage to concrete infill wall at junction of Blocks C & D	Junction between Blocks D & C	Break out damaged concrete locally between structural walls so that construction of damaged wall and repair methodology can be determined.	 
8.4. Cracks to walls	Wing wall on north elevation	Check all walls and epoxy inject cracks greater than 0.2mm in width as per HCG specification. HCG have undertaken a condition survey of this area and the current recommendation is that widespread epoxy injection of historic shrinkage cracks or cracks widened by the earthquakes is not considered necessary – refer HCG report dated April 2014. [33]	

Damaged Item	Location	Recommendation	Example
8.5. Cracks to slab	South side crawl space, trench slabs and upper raft slab	Epoxy inject cracks greater than 0.2mm in width as per HCG specification. HCG have undertaken a condition survey of this area and the current recommendation is that widespread epoxy injection of historic shrinkage cracks or cracks widened by the earthquakes is not considered necessary – refer HCG report dated April 2014. [33]	No photo
8.6. Retaining wall cracks in walkways up to 0.4mm	Widespread throughout basement	Check all walls and epoxy inject cracks greater than 0.2mm in width as per HCG specification. HCG have undertaken a condition survey of this area and the current recommendation is that widespread epoxy injection of historic shrinkage cracks or cracks widened by the earthquakes is not considered necessary – refer HCG report dated April 2014. [33]	

Damaged Item	Location	Recommendation	Example
9. Lift Machine Room			
9.1. Spalled stair landing	At concrete landing at top of stair	Repair spalled concrete in accordance with the HCG specification.	

Table 4-5: Façade Damage – From Goleman's Survey

Damaged Item	Location	Recommendation	Example
1. Spalled Concrete			
1.1. Panels and Columns	Refer to Goleman Report.	Repair areas of spalling in accordance with the HCG specification.	 8. Sealant and Concrete Damage
2. Cracked Concrete			
2.1. Cracking to panels and columns	Refer to Goleman Report.	Epoxy inject cracks that are 0.2mm or greater in width in accordance with the HCG specification.	 3b  6

2.2.	Parapet beams cracks around bolt fixings	Refer to Goleman Report.	Where cracking around filled recess is minor, break out the mortar, check the bolt alignment and fixing and repair as spalled area in accordance with the HCG specification. Where the cracking is significant and spalling has occurred (refer photo 2b), break out and repair the spalled concrete in accordance with the specification and provide a new parapet beam fixing in accordance with Sk 102 Rev A. Where parapet beam has moved out of alignment, provide a new fixing as above without realigning the parapet beam. All fixings to this parapet are to be checked.	 6b

Damaged Item	Location	Recommendation	Example
3. Exterior paving			
3.1. Settlement and damage to paving	Refer to Goleman Report.	Specification by others	 2d  6d
4. Sealant			
4.1. Splitting of sealant and bond with concrete broken	Refer to Goleman Report.	Specification by others	 13. Torn Sealant

Damaged Item	Location	Recommendation	Example	
5. Flashings				
5.1. Lift shaft walls – cracking up to 0.4mm in width	Refer to Goleman Report.	Specification by others	 12b	 3c
6. Soffit Linings				
6.1. Broken soffit linings and damged fixings adjacent to seismic joints	Refer to Goleman Report.	Specification by others	 4d	 6a Broken Hardy board

Damaged Item	Location	Recommendation	Example
7. Panels			
7.1. Panels appear to be displaced	Refer to Goleman Report.	There are no signs of damage or movement in the sealant around the panel or the panel fixings. The internal bottom fixings are to be inspected.	 8. Displaced Parapet. Tipped back

4.2 BUILDING RE-LEVELLING

Re-levelling buildings the size of the Parkside Blocks is complicated and would require specific advice from both the Geotechnical Engineers and specialist re-levelling contractors to determine whether it is feasible. A scheme to look into this has been prepared by Uretek.

Re-levelling of the Parkside Blocks is complicated by the fact that foundations of the Blocks are separated by control joints and the upper levels by seismic separations. There will be the requirement to ensure that the re-levelling does not generate any steps between the Blocks that will affect the functionality of the buildings.

Each of the main buildings is supported by two main raft slabs with varying thickness infill slabs at varying levels in between which will also complicate the re-levelling process.

Another factor that must be considered when considering a re-levelling scheme is the connection and interface the Parkside buildings have with the adjacent structures. The Parkside Buildings are connected to the Clinical Services building by the Link Bridges, there are several services tunnels that enter the building as well as a pedestrian tunnel that links the Parkside Buildings with the St Asaph Street campus.

The School of Medicine Building, Food Services and Oncology are all linked to the Parkside and have the same floor levels. There is also a connection with the Women's Hospital; however, the floor levels of this building do not typically align with the Parkside floors.

4.3 REINSTATING CAPACITY TO PRE-EARTHQUAKE CONDITION

As described when discussing the post-earthquake building capacity in Section 2.2.16, it is likely the capacity of the building to sustain future earthquakes has been reduced. This is primarily due to the steel reinforcement in the concrete elements experiencing strain hardening during the recent earthquakes which has lead to a reduction in these elements ability to sustain further cyclic loading.

Typically, where concrete elements are cracked, repair has been achieved using epoxy injection. However, epoxy injection of the cracks will not reinstate degradation of the steel reinforcement as a result of strain hardening.

As there is no repair that will reinstate the capacity these elements have lost due to strain hardening, to repair these elements and reinstate the pre-earthquake utility of these areas would involve breaking out and replacing these individual bars or whole elements. New steel reinforcement would need to meet current code detailing requirements and be lapped with the existing bars.

Replacing individual bars crossing cracks might be possible in the floor slabs; and likely only required in cracks with a similar width to that tested (2-3 mm). This is likely to require a localised repair in a couple of locations. However, due to the quantity of transverse reinforcement in the core walls and the quantity of steel that as potentially strain hardened, replacing individual bars is not feasible and replacement of these walls might be required to reinstate the pre-earthquake strain capacity of the steel reinforcement.

5. STRENGTHENING RECOMMENDED

The Emergency Department Extension limits the capacity of the Parkside buildings and has a capacity of only 30 % DBE. The primary structural systems of the main Parkside Blocks (A, B, C and D) have the capacity to resist 78% of an Importance Level 4 (IL4) Design Basis Earthquake (DBE) loading. However, the onset of Collapse Limit State damage is estimated to occur at load levels only 1.14 times this. Other building elements, such as the link bridge structures to the Clinical Services Blocks and precast cladding panels have capacities less than 78 % DBE. The stairs were also identified as a Critical Structural Weakness due to insufficient deformation capacity and it has been calculated that the building performance does not meet the serviceability (SLS2) requirements of an IL4 structure.

The likely strengthening required to remove Critical Structural Weaknesses and the likely strengthening required to increase the capacity of the building to higher percentages of the current code are addressed separately herein. We have also included some discussion on improving the buildings performance in an SLS2 magnitude event.

5.1 STRENGTHENING TO REMOVE CRITICAL STRUCTURAL WEAKNESSES

5.1.1 Stairs

The stairs are constructed of reinforced concrete; they zigzag between each floor level. In their pre-earthquake configuration the stairs did not have a mid height landing support and were rigidly connected floor to floor.

The stairs cracked and anecdotally became more sensitive to vibration during the Lyttelton Earthquake and would be significantly damaged in a Design Basis Earthquake (DBE) and a Maximum Considered Earthquake (MCE) of longer duration. As the stairs are required for safe egress of the building following an earthquake, the stairs in their original condition are deemed to be a Critical Structural Weakness.

To remove the Critical Structural Weakness and ensure that the stairs will perform adequately, the stairs must be separated such that they are not connected to adjacent floors. This work is underway and progressively being completed stairwell by stairwell. HCG Stair Remediation and Seismic Gap Memo – Stair Strengthening and Separations Memo Revision 2 [30] indicates the work that has been completed to date as well as the work going forward required to meet the design requirements.

The strengthening is achieved by the addition of vertical support to the stair flights at the mid-height landing and separating the top flight from the bottom flight of the stairs by cutting them apart at the mid-height landing level. The stairs flights cantilever laterally out from each floor and are allowed to slide over the vertical support at the mid-height landing; this permits the adjacent floors to move relative to one another without damaging the stairs. The stair strengthening has been designed to for 67% IL4 level loads.

The strengthened stairs, upon construction completion, should be able to sustain the expected DBE drifts without significant damage and without the stair flights pounding together at the mid-height landing. The stairs will therefore function for building occupant egress.

At the Maximum Considered Event (MCE) load case (approximately 1.5-1.8 times the DBE loads), the strengthened stairs flights may pound together at the mid-height landing due to movement perpendicular to the stair span. In that case, the stairs will sustain some damage; however, they are unlikely to collapse.

5.2 STRENGTHENING TO ACHIEVE 67 % DBE (IL4) NEW BUILDING PERFORMANCE

Strengthening the Parkside buildings to have an ULS capacity above 67 % DBE will require strengthening of the Emergency Department Extension, the Link Bridges, some of the precast panels and requires mitigation to prevent the buildings pounding at loads less than this.

If the performance is to be improved to be equivalent to a new structure then the margin between the ULS and CLS of the primary structural systems will also need to be increased from its current 1.14 to between 1.5 and 1.8.

The following sections provide strengthening concepts to increase the building capacity to 67 % DBE loading for an IL4 building. Refer also to the HCG 67% IL4 strengthening memo dated 9th October 2015 and attached as Appendix E.

5.2.1 New Concrete Walls

From Level 2 and above the wall layout of the shear cores typically reduces from two walls coupled together on each face of the core to one cantilever wall on each face of the shear core, refer Figure 5-1 below. This reduces the stiffness of the building above Level 2 and increases the drifts expected in a seismic event. Currently the drifts at the roof level are predicted to be as high as 2.5% (the code limit) in specific locations. The increase in displacements due to the reduced stiffness above Level 2 can clearly be seen in Figure 5-2 below.

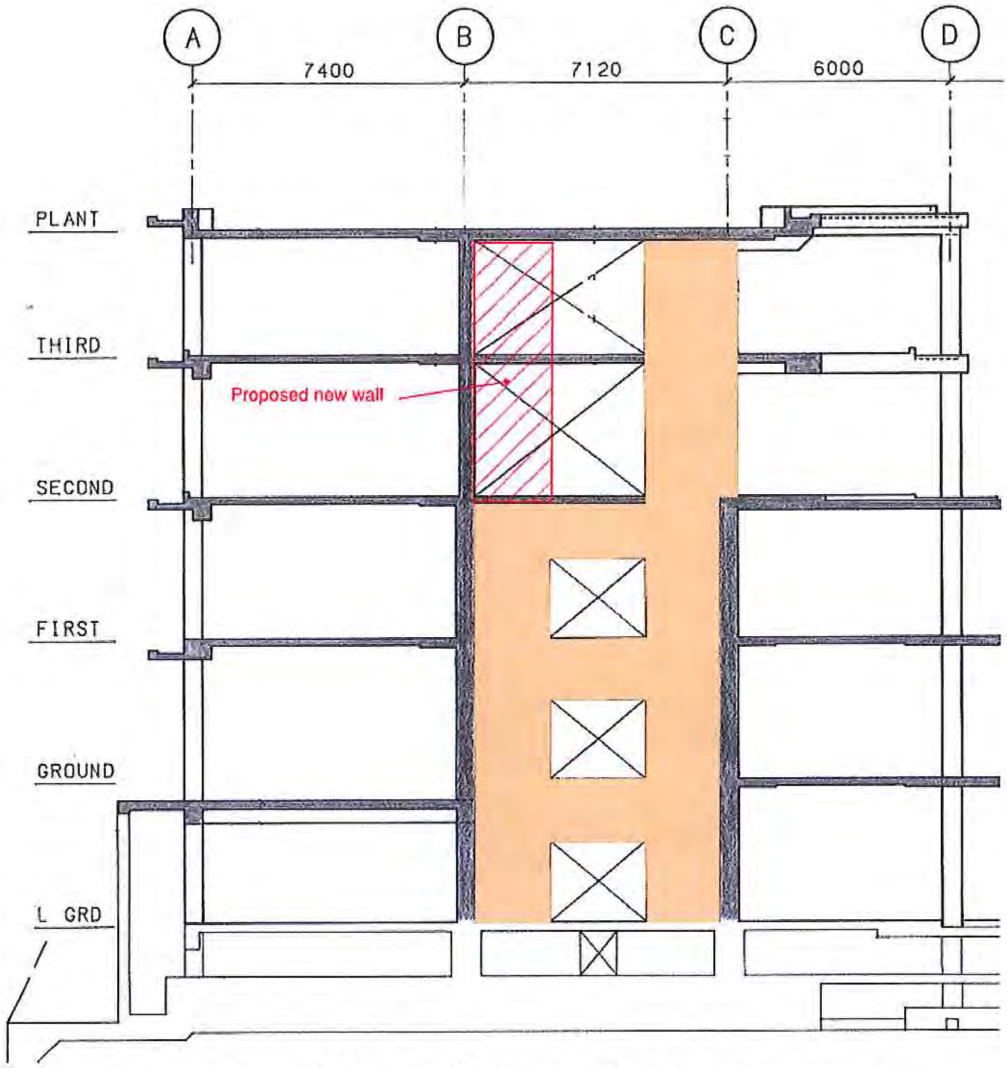


Figure 5-1:Typical Shear Wall Layout with Proposed New Wall.

The development of the strengthening options for the stairs and precast cladding panels has shown that strengthening to 67% IL4 can not be readily achieved above Level 2 due to the large drifts above Level 2. As such the building is required to be strengthened and stiffened in all Blocks to reduce these drifts and allow the stair and precast cladding panel strengthening solutions to work. Several solutions for strengthening the building above Level 2 have been proposed to date, some have been investigated and found to not produce the required reduction in drifts required by the stairs and precast panels. The solution investigated and proposed herein involves adding a second cantilever concrete wall into each shear core wall above Level 2, as shown in red in Figure 5-1. A steel braced frame option instead of concrete walls could be considered during Developed Design. This strengthening is required for all Parkside Blocks but further analysis may determine that Block C does not require additional walls due to the fact that it does not have an additional concrete plant level above third floor.

Adding the new walls to each side of the shear core above Level 2 increases the stiffness of the building and reduces the displacements. Figure 5-3 shows the reduced displacements when the building is strengthened above Level 2 with new walls. As can be seen, the interstorey drift of the upper levels is now in line with the drifts of the lower levels.

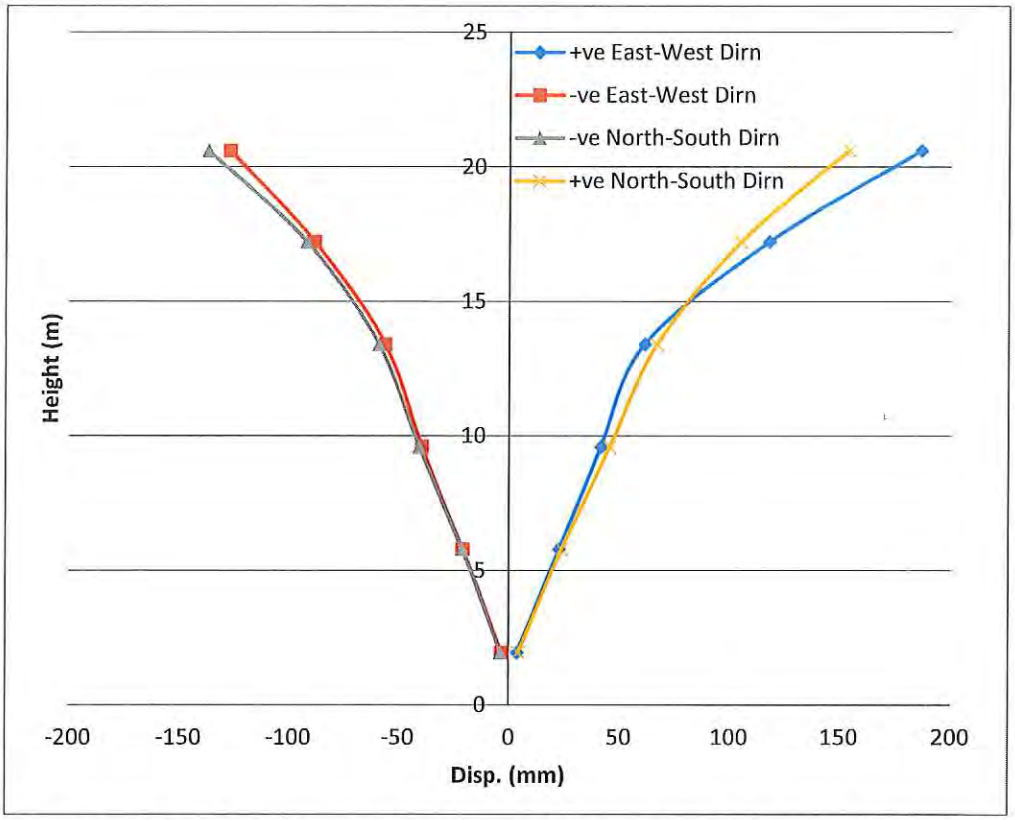


Figure 5-2:Displacements at the Centre of Mass in each direction for 67% IL4 level load, current building.

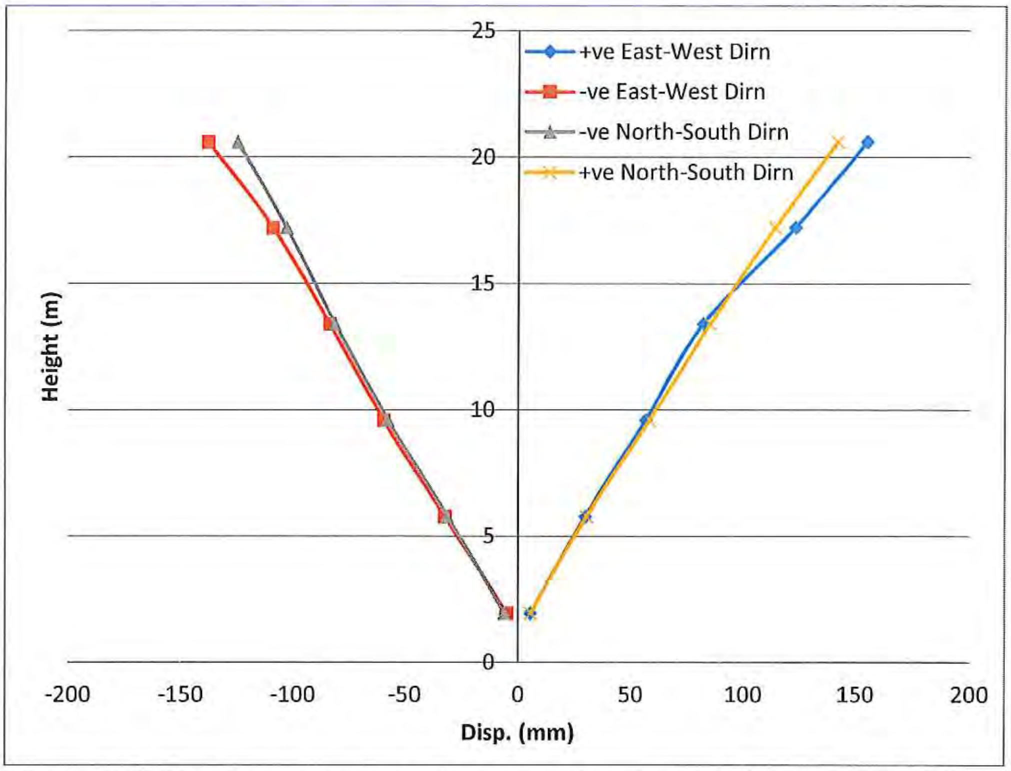


Figure 5-3:Displacements at the Centre of Mass in each direction for 67% IL4 level load, strengthened building.

5.2.2 Emergency Department Extension

The Emergency Department Extension (EDE) is a two level structure that extends out the north side of Block A. The NLTHA indicates that the capacity of this building is only around 30 % of Design Basis Earthquake loading (refer to Section 2.2.5.1). Review of this structure indicates that numerous elements fail at a similar capacity and therefore require strengthening.

To increase the capacity to above 67 % DBE loading for an IL4 structure would involve replacing the existing roof and wall cross bracing, as well as installing some additional bracing. By ensuring a desirable hierarchy of failure for the new elements it is possible to ensure Critical Structural Weaknesses do not govern the performance.

A concept strengthening scheme for pricing is shown in Figure 5-4 and described below:

1. Install new 250UB31 struts in the roof plane to replace the 150PFC's and DHS purlins currently utilised as part of the roof bracing system.

2. Install one bay of new 60x6 mm flat brace wall bracing and improve the welded end connections of the existing bay of wall bracing.

3. Improve connection from roof beams back into Parkside Block A, Level 1, floor. This will require new steel struts to transfer load from the roof plane down to the floor (around 500 mm below).

4. Replace existing 12 mm diameter roof cross bracing with 16 diameter rod bracing. Install an additional bay of 16 diameter roof cross bracing.

5. Increase capacity of cantilevered steel floor beams to resist over-strength actions generated by the wall bracing. This could be achieved by locally strengthening the two beams that have insufficient capacity by welding 40x6 mm flats to the beam flanges either side of the web. The flats would need to extend over several meters of the beam length. Alternatively, steel posts could be installed below these two beams which would mean they were no longer required to cantilever. The posts could be 75x75x5 SHS sections founded on shallow concrete foundations (600 mm square) tied into the Clinical Services building.

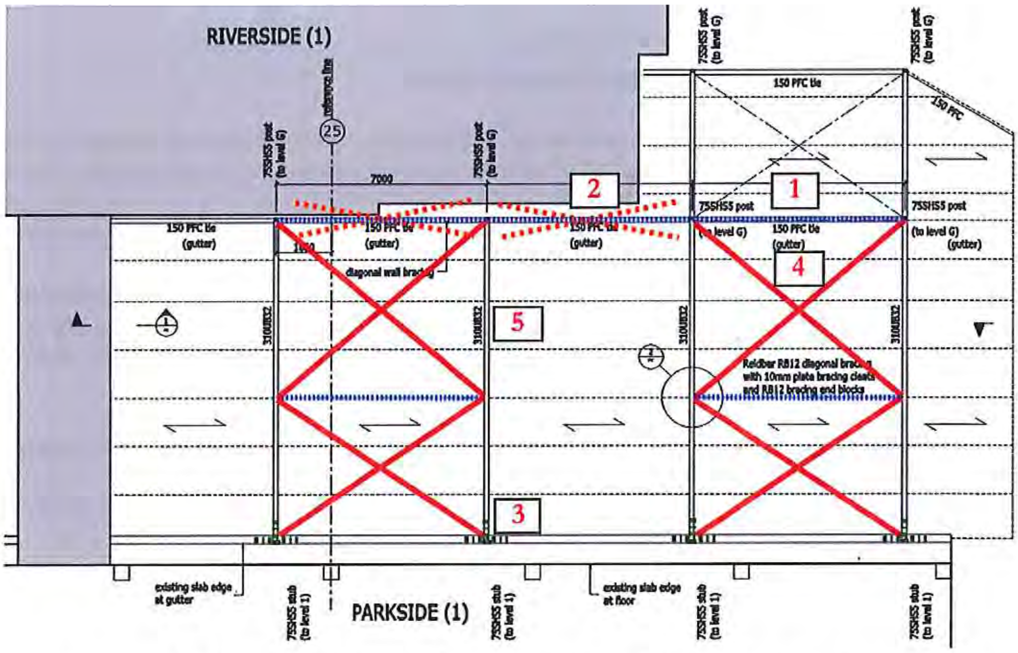


Figure 5-4: Concept 67 % DBE strengthening scheme for the Emergency Department Extension (IL4)

5.2.3 Link Bridges

The link bridges cantilever horizontally from the Parkside Blocks and have the capacity to resist approximately 33% of the IL4 Design Basis Earthquake loading, it has also been noted that the capacity of Link B is only 85 % under the maximum gravity load case (refer to Section 2.2.12). Multiple options have been investigated to strengthen the link bridges and a preferred scheme has been issued separately to the CDHB [28]. This scheme involves:

- Removing the concrete roofs and replacing these with a light-weight alternative that includes steel cross bracing.
- Removing the precast cladding panels and replacing these with a light-weight alternative.
- Strengthening the existing connection between the link bridges and the Parkside Blocks.
- Adding an external diagonal brace at the Ground and Level 1 floor levels which would brace the link bridges back to the Parkside building.
- Increasing the size of the seismic gap between the Parkside Link bridges and Clinical Services building upper levels to prevent pounding of the structures and potential damage to the link bridges. This can be achieved by cutting back the concrete slab and steel beams in the link and providing a longer seismic gap cover.

5.2.4 Precast Panels

The precast cladding panels have been constructed with a limited ability to sustain the expected interstorey movements. The movement allowance provided by oversized bolt holes within the panel connections has been shown to have been taken up by construction tolerance for a large number of panels. Consequently the interstorey drift capacity of the precast panels is assessed as being exceeded during loading less than 34% DBE. It is recommended that prior to strengthening, the panel connections be inspected to assess their likely interstorey drift capacity.

The panel strengthening concepts have been developed to bring the capacity of these precast panels above 67 % DBE. This will require increasing their ability to sustain interstorey deflections at all levels. The current interstorey drifts above Level 2 are predicted to be as high as 2.5% and this leads to large displacement requirements in the panel connections. These large displacements can not be accommodated because the large rotations cause the connections to bind up. As such, strengthening has been detailed for the demands associated with the strengthened Parkside building, ie approximately 1.0% interstorey drift.

The strengthening could be achieved by replacing the panel connections with similar connections designed to sustain the larger movement demand or removing the panels. Due to the hazard presented by panels potentially detaching when the interstorey drift capacity is exceeded, it is recommended that the strengthening of panels over egress routes be prioritised. Strengthening options for the precast panels have been outlined in the HCG Precast Cladding Panel Strengthening Options Memo dated 1 July 2015 [32].

The precast panels also need to sustain out-of-plane loading. There are around 40 panels that appear to have a capacity to resist only 50 % DBE loading. The panels with a capacity less than 67 % DBE loading are the largest cladding panels (solid panels greater than 2m width). It has been assumed that these cladding panels are connected back to the buildings in a similar manner to the remainder of the panels with one fixing per corner. Installing new fixings between the existing connections will increase the capacity of these panels to above 67 % DBE loading.

5.2.5 Pounding between the Main Blocks

Pounding between Block A and Block B is estimated to commence at 40 % DBE loading. This is primarily because of high interstorey drifts above Level 2 of the south wing of Block A. This will lead to increased accelerations at the upper levels of the building that could lead to slightly increased contents and services damage. We previously thought that adding structure and stiffening the building above Level 2 would be sufficient to prevent pounding at this load level, but this has been found not to be the case.

The pounding between Blocks A and B should not significantly affect the structure (due to the similar heights of the structures and the floors being at the same level) and therefore does not affect the overall percent DBE. However, we could complete an assessment using the Non-Linear Time History Analysis that has been prepared for the Block A that looks at the floor accelerations produced by Block A and B pounding together. These accelerations could then be used to create a specification for the securing of building plant, services and high value building contents.

5.2.6 Pounding with School of Medicine Building

Currently it is estimated that pounding between Block B and Block C with the School of Medicine Building commences at 50 % DBE loading. Full details of the pounding is outlined in the HCG memo titled CDHB – PARKSIDE – Seismic Gap Between Parkside and School of Medicine Buildings, issued 9 October 2015 [31].

The additional strengthening proposed to the Parkside Blocks above Level 2 (new walls) reduces the seismic gap requirements between the School of Medicine and Parkside Blocks B&C but the buildings are still likely to pound at levels of load less than 67% DBE. By introducing the new walls in Parkside Blocks B & C above Level 2 the buildings are estimated to pound at approximately 62% DBE loading It is recommended that pounding should be prevented until loads above the ULS. In addition damage due to pounding to the School of Medicine Building could impact the occupancy of the Parkside Buildings.

To prevent the Parkside Buildings pounding with the School of Medicine Building at 67 % DBE loading the gap between the buildings needs to be increased at the Parkside Plant Room Level and Level 3. Figure 5-5 below shows the required seismic gaps if no strengthening of the Parkside buildings is undertaken.

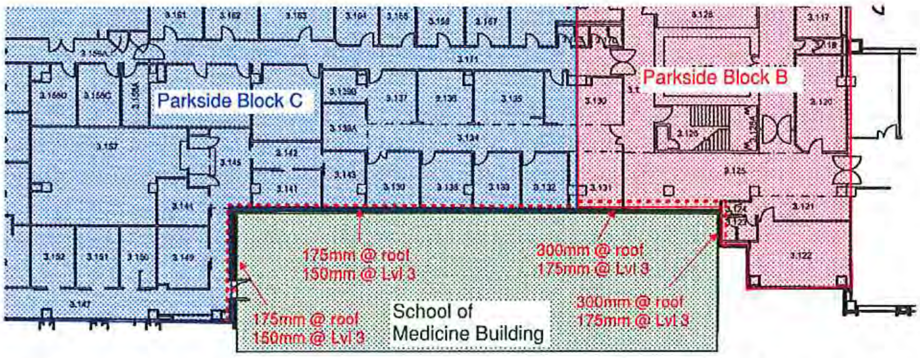


Figure 5-5: Seismic gap requirements between Parkside B&C and School of Medicine Building [31]

The seismic gap at Level 3 can be increased by cutting back either the edge of the Parkside Blocks or School of Medicine concrete slab by approximately 100 mm. It is our understanding that cutting back the School of Medicine Building is preferred but is likely to be more complicated to achieve. To achieve this, the concrete block walls along the north elevation of the building will be required to be removed. The easiest way to achieve the required seismic gap is to cut back the slab overhang in Parkside Blocks B & C. The main structural elements are set back from the edge of the building and hence cutting back the slab can be achieved without compromising the main structural elements of the building

At the Plant Level, the School of Medicine slab appears to be higher than the Parkside structure and therefore may not pound. Site investigations confirmed that the slab does indeed sit higher than the Parkside structure but that there is insufficient movement allowance between the two structures. Work will be required at this level to achieve the required seismic gaps. Waterproofing of the exterior façade at both levels will need to be addressed architecturally.

5.2.7 Retaining Walls

The main Parkside retaining walls have a capacity of 60% of the Design Basis Earthquake loading, for an IL2 structure. At this level of load the allowable soil bearing pressure is exceeded. This type of 'failure' is likely to cause rotation and/or settlement of the wall but not necessarily catastrophic failure of the wall itself.

The ground floor slab that spans the gap between the building and retaining walls has been designed to cantilever and has no positive connection to the retaining walls. Evidence of differential movement has been observed around the south and east sides of Block A. Investigations have been undertaken in Block D to determine whether there is any connection between the retaining walls and the slab in this location. No positive connection was found. It is possible that the slab has been poured on top of the walls without any separation layer and as such, in order for the two elements to move independently, the friction at the rough interface must be overcome. In Block D, this may not have occurred whereas it appears to have occurred in the south and east corners of Block A.

The NLTHA models indicate that the buildings move approximately 30mm at Ground floor level under 67% IL4 DBE loading. This increases to 60mm when 1.5 x the DBE level, i.e. CLS level loading, is considered. Therefore some work may be required to ensure that the buildings are free to move at this level, such as leaving a gap between the buildings and exterior hard surface finishes. Figure 5-3 below indicates the extent of retaining walls around Parkside and it is recommended that work is undertaken to provide a gap between the slab and exterior hard surfaces such that free access is maintained at all entranceways.

Potential strengthening of the crib walls has been considered and involves horizontal ties installed to the crib wall and back under the carpark behind. In addition, a reinforced concrete skin is proposed to be installed to the face of the crib wall. The extent of strengthening requirements depends on the level to which the crib wall is desired to be strengthened, which could differ from the strengthening level for the Parkside buildings.

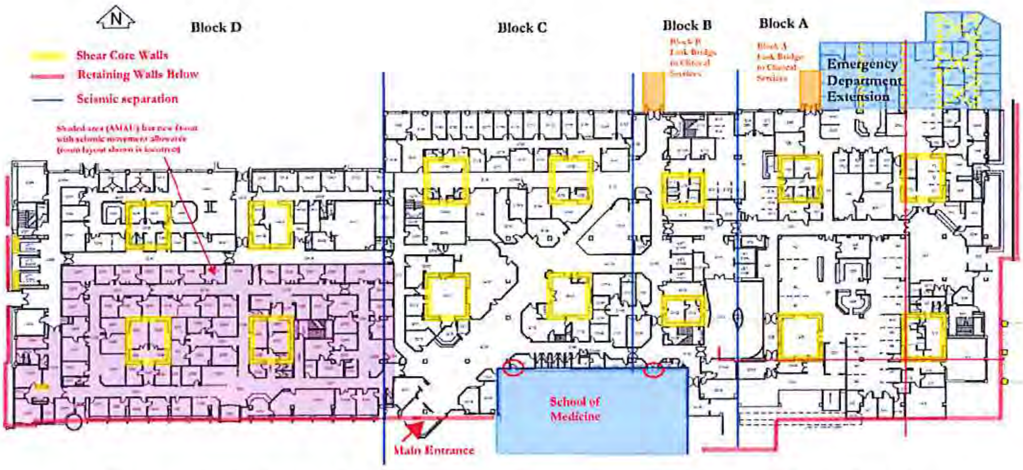


Figure 5-6: Lateral movement allowance at Ground Floor Level

5.2.8 Primary Structure of Main Blocks

The ULS capacity of the primary structure of the Main Blocks has been calculated as around 78 % DBE for an Importance Level 4 building. However, the onset of the Collapse Limit State (CLS) is estimated at only 89 % DBE loading. This results in a margin over ULS of only 1.14 and therefore the equivalent performance relative to a new building is only 49 - 59 %. To achieve the equivalent margin between ULS and CLS as a new building will require strengthening to increase the CLS capacity up to at least 117 % DBE loading.

This can be done by improving the detailing of the main concrete core walls between Lower Ground Floor and Ground Floor. If these were new buildings, more horizontal steel reinforcing ties would be included in the wall. To achieve the same performance from the existing walls we need to clamp horizontal steel bars each side of the core walls. This will contain the concrete in the walls and limit buckling to the vertical steel reinforcement. By doing this, the rotations the walls can sustain before losing gravity load carrying capacity increase.

5.3 STRENGTHENING TO ACHIEVE 100 % DBE (IL3) NEW BUILDING PERFORMANCE

If assessed as an Importance Level 3 (IL3) building the current capacity of the primary structure of the Main Blocks is around 108 % DBE loading. However, as discussed above, the margin between ULS and CLS is only 1.14 and therefore the equivalent performance relative to a new building is only 68 - 82 %.

To strengthen the building to have a capacity of 100 % DBE IL3 loading with a margin of 1.5 over collapse would require similar works as described in Section 5.2 to improve the building performance to 67 % DBE loading for an Importance Level 4 building.

5.4 STRENGTHENING TO ACHIEVE 100 % DBE (IL4) NEW BUILDING PERFORMANCE

To increase the Parkside performance to that of an equivalent new Importance Level 4 facility would require significant work. Possible schemes would likely include new concrete shear walls up the height of each Block in each direction. These walls will need to be sized to increase the structural capacity and reduce building drifts (to prevent the buildings pounding together below the ULS capacity). Further detailed analysis would be required to determine the feasibility of such schemes.

5.5 STRENGTHENING TO MEET SERVICEABILITY REQUIREMENTS

The building currently does not meet the SLS2 requirements of an IL4 facility due to the quantum of damage expected at this level of event and the large interstorey drifts. Strengthening to improve the building capacity will improve its serviceability performance; however, it is unlikely the full SLS2 requirements of a new building would be met without a significant amount of new structure such as that described above for strengthening to achieve 100 % DBE for an IL4 building. This strengthening would need to be sufficient to allow the ductility demand on the building to be reduced.

Although full SLS2 requirements are not met, it is recommended that future fit-outs be designed to sustain the anticipated drifts at the SLS2 load level (or as close as practical to it).

If the buildings are to be assessed as Importance Level 3 structures then there is no regulatory requirement to meet SLS2 limits. However, aiming to meet SLS2 requirements will reduce the quantum of damage the building experiences in earthquakes with a magnitude less than the building capacity. This will allow continued function in larger events than otherwise and reduce the time and cost of future repairs.

6. REFERENCES

1. *Christchurch Hospital Campus – Detailed Seismic Assessment Report – Base Report,*, Holmes Consulting Group, April 2011.

2. *Christchurch Hospital Campus – Detailed Seismic Assessment Report – Repair Specification*, Holmes Consulting Group, April 2011.

3. *Christchurch Hospital Redevelopment State III Structural Drawings for The Canterbury District Health Board*, Frederick Sheppard and Partners, 1984-1992.

4. *Canterbury District Health Board New Womens hospital and Day Surgery unit Christchurch Hospital, Riccarton Avenue*, Structural Drawings, Holmes Consulting Group, 2002.

5. *Christchurch Hospital Redevelopment Stage III, Main Building Contract 03. For the Canterbury Hospital Board*, Cutter Douglas Partnership Ltd. Architects, Christchurch, Folio 2, Set 13, October 1989.

6. *Christchurch Hospital Emergency Department Extensions for Canterbury District Health Board*, Structural Specification, structex metro ltd, 2007.

7. *Christchurch Hospital Emergency Department Extension Construction Drawings for Canterbury District Health Board*, structex, Project 10032, 18 October 2007.

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

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

10. *New Zealand Society for Earthquake Engineering Assessment and Improvement of the Structural Performance of Buildings in Earthquakes including Corrigendum No. 1*, Recommendations of a NZSEE Study Group on Earthquake Risk Buildings, June 2006.

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

12. *Code of Practice for General Structural | Design and Design Loadings for Buildings, NZS 4203:1984*, Standards New Zealand, 1984.

13 . *Concrete Structures Standard – Code of Practice for the Design of Concrete Structures, NZS 3101:1982*, Standards New Zealand, 1982.

14 . *Concrete Structures Standard – The Design of Concrete Structures, NZS 3101:Part 1:2006*, Standards New Zealand, 2006.

15 . *Practice Note – Design of Conventional Structural Systems Following the Canterbury Earthquakes,* Structural Engineering Society New Zealand (Inc.) (Sesoc), Version No.4, December 2011.

16 . *Structural Design Actions Part 0: General principles – Australian/New Zealand Standard™, A/NZS 1170.0:2002*, Standards New Zealand, 2002.

17 . *Preliminary Review of Suspended Ceiling in Parkside and Laboratory Buildings, Christchurch Hospital,* Holmes Consulting Group, 23 May 2011.

18 . *Non-Destructive Concrete Materials Testing for Parkside Hospital building, CDHB,* Holmes Solutions LP, Report 107155-1 (v1.0), August 2011.

19 . *Hardness Testing of Reinforcing Steel in Parkside, CDHB Campus, Christchurch,* Holmes Solutions LP, Report 10356 (v1.1), November 2012.

20 . *Steel reinforcing materials - Australian/New Zealand Standard™ (Amendment No1 Appended), AS/NZS 4671:2001*, Standards New Zealand, 2001.

21 . *Christchurch Public Hospital Building Survey Overall Campus Building Report,* Fox & Associates, 28 June 2011.

22 . *CDHB – Parkside - Floor Levels, Drawings 1 to 5,* Fox & Associates, Job No. 2434C/6, 24 May 2012.

23 . *CDHB – Christchurch Hospital Emergency Department – Elevations*, Fox & Associates, Job No. 2434C/6, 9 November 2012.

24 . *Canterbury District Health Board, Christchurch Hospital Campus, Preliminary Post Earthquake Geotechnical Assessment,* Tonkin & Taylor Ltd, T&T Ref:52135, June 2011.

25 . *Canterbury District Health Board, Christchurch Hospital, Phase 2 – Geotechnical Investigation and Analysis,* Tonkin & Taylor Ltd, September 2011.

26 . *Memo – To: Canterbury District Health Board, Subject: Christchurch Hospital – Parkside and Riverside Buildings – Post Earthquake Settlements*, Tonkin & Taylor, T&T Ref. 52135, 16 August 2012.

27 . *Earthquake Inspection, Christchurch Base Hospital, Parkside*, Goleman, 4 October 2011.

28 . *CDHB – Parkside – Strengthening Schemes for Parkside Link Bridges*, Holmes Consulting Group LP, 106186.20LECOL1112.003, 11 December 2012.

29. *CDHB – Parkside – 67%DBE Strengthening Concepts,* Holmes Consulting Group LP, CHCH.PARK.130813_HCG_67%DBE strengthening concepts, 16 August 2013.

30. *CDHB – Parkside – Stair Strengthening and Separations – Revision 2*, Holmes Consulting Group LP, 16 September 2015

31. *CDHB – Parkside – Seismic Gap Between Parkside and School of Medicine Buildings- Revision 2* Holmes Consulting Group LP, 9 October 2015

32. *CDHB – Parkside – Precast Cladding Panel Strengthening Concepts*, Holmes Consulting Group LP, 1 July 2015

33. *Christchurch Hospital, Parkside Building Basement Condition Survey Review*, Holmes Consulting Group LP, 3 April 2014

APPENDIX A

Record of Observations

APPENDIX A1 – RECORD OF OBSERVATIONS & REPAIRS - PARKSIDE BLOCK A

Inspection date: 20th & 21st April, 2011
Update: 29th September 2011
Update: 23th November 2011
Update 18th April 2012
Update: 30 May 2014
Update: 27 February 2015 Photos supplied by RCP of some repair work

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

Level	Room Number	Location (Ref Plan)	Building Element	Observations	Repair Required	Repair	Photo Reference
BM	B03A	A.BM.1	Concrete wall infill	21.04.11 - Cracking to wall to what appears to be a joint at a seismic gap. 29.9.11 - The "gap" is a cold joint. Rm B03A appears to be outside the Block A envelope	N		05.05.11 - Jenny 27
BM	B03	A.BM.2	Tunnel Walls	19.10.11 - 0.3mm horiz crack at 1.8m full length 30.5.14 - Part but not all complete	N		19.10.11 - 012
BM	B02	A.BM.3	Slab	19.10.11 - Spalling of slab adjacent to basement walkway. Slab has heaved generally in this area. 8.08.12 - Area of slab has been chipped out to investigate damage.	Y	Refer to site report	19.10.11 - 014, 015 18.04.12 - Building A 108039-402 12-08-08

Level	Room Number	Location (Ref Plan)	Building Element	Observations	Repair Required	Repair	Photo Reference
BM	B02	A.BM.4	Soffit of LG slab	03.12.12 Cracking to underside of LG slab identified from basement. Cracking varies from hairline to approximately 0.6mm in width. Appearance of crack age varies.	N		-

Level	Room Number	Location (Ref Plan)	Building Element	Observations	Repair Required	Repair	Photo Reference
BM	B01	east side	Retaining wall	6.11.12 - Movement residual observed between retaining walls and ground floor slab. Refer site report 27. Some spalling to top of retaining wall	N		6.11.12 - full folder and 8.11.12 full folder
BM	B02	A.BM.4	Column	18.04.12 - Crack in corner of column above walkway wall.	Y	Spalling repair	18.04.12 - Building A 108039-404
LG	L16	A.L.G.01	Concrete Beam/Shear Core	20.04.11 - Minor concrete spalling at concrete beam-shear core connection. Possibly existing. Hairline diagonal crack to shear core.	N		20.04.11 - Building A 001, 002
LG	L13	A.L.G.02	Shear Core	20.04.11 - Hairline diagonal crack to shear core.	C	Epoxy inject all cracks in shear cores as per HCG specification	20.04.11 - Building A 003
LG	L13	A.L.G.03	Coupling Beam	20.04.11 - Vertical crack out from door head.	C	Epoxy inject all cracks in shear cores as per HCG specification	20.04.11 - Building A 004
LG	L13B	A.L.G.04	Coupling Beam/Shear Core	20.04.11 - Gib damage- possible signs of movement in beam and shear core.	C	Expose shear core and epoxy inject all cracks as per HCG specification.	20.04.11 - Building A 005
LG	L17, L21	A.L.G.05	Ground Floor Slab	20.04.11 - Soffit of slab inspected locally in certain locations where possible. No damage observed.	N		20.04.11 - Building A 006
LG	L27a	A.L.G.06	Shear Core	20.04.11 - Top section of shear core inspected through ceiling. No damage observed.	N		

Level	Room Number	Location (Ref Plan)	Building Element	Observations	Repair Required	Repair	Photo Reference
LG	L29	A.L.G.07	Coupling Beam, Shear Core	20.04.11 - 0.2mm vertical crack from top left corner of door through beam. Hairline-0.1mm vertical crack from slab soffit through wall (No damage observed to slab soffit).	N		20.04.11 - Building A 007, 008
LG	L80, L81	A.L.G.08	Coupling Beam	20.04.11 - 0.2mm diagonal cracking from top corners of door head.	N		20.04.11 - Building A 009, 010
LG	L81	A.L.G.09	Shear Core	20.04.11 - No observed damage to shear core above ceiling level.	N		
LG	Various		Floor slab	27.11.12 Cracking in lower ground floor slab (suspended slab). Cracks up to 1.6mm in width identified throughout slab. Appears to be a mixture of shrinkage cracking and seismically induced cracking. Rooms in which cracking has been identified include L68, L72, L75, L89-L91	C	Epoxy inject. Refer site report 26	12-11-27 - cracking to LG68, 90 &91
LG	L17, L21	A.L.G.10	Floor Slab	20.04.11 - Cracking in lower ground floor slab (slab on ground). Cracks up to 2.5mm wide. One crack out from return wall across corridor. Cracks up to approximately 6m long, through lino but tapering to a crease in the lino. Adjacent seismic joint appears to have moved 3mm.	C	Remedial as per SK100, 100A (Appendix C)	20.04.11 - Building A 011, 012, 013, 14
LG	Various		Shear Cores	Shear cores are lined in light walls. The upper parts of the walls have been inspected through the ceiling where possible; however re-inspection will be required in order to determine the extent of damage to the walls. 30.5.14 - Crack Injection of LG core walls mostly complete (NE core wall still to complete)	C	Epoxy inject as per HCG Specification	

Refer to Table 4.1 and HCG Specification for repair details

Level	Room Number	Location (Ref Plan)	Building Element	Observations	Repair Required	Repair	Photo Reference
LG	Various		Columns	Columns are lined throughout. Re-inspection required in order to determine if they have sustained any damage.	F		
LG		A.L.G.11	Shear Core: South East	Linings removed. Cracking mapped 23/6/11: Hairline to 0.1mm cracks. No remedial requires	N		
LG		A.L.G.12	Shear Core: South West	22.9.11 - Linings removed on the south half of the core only, revealing 0.3/0.5mm cracks widespread. Balance of core needs to be exposed.	C	Epoxy inject as per HCG Specification	
LG		A.L.G.13	Shear Core: North East	29.09.11 - Limited sample opened in Rm L53. Widespread 0.1/0.2mm cracks identified. Balance of the core is inaccessible (Mortuary)	C	Epoxy inject as per HCG Specification	
LG	L80, L81	A.L.G.13a	Shear Core: North West	29.09.11 - Widespread 0.2/0.3mm	C	Epoxy inject as per HCG Specification	
LG	L121	A.L.G.14	Column	5.05.11 - Cover concrete has spalled from the corner of the column. Reinforcing exposed shows signs of rusting.	C		5.05.11 - Jenny 28
Throughout	L45A, L45B	A.L.G.15	Lift Shaft & Pit	29.09.11 - Some spalling at panel joints, water ingress in pit. General: No significant damage up lift shaft identified. Some gib cracking (Light weight fire rated walls). All steel framing OK. Concrete work around the lift over-run OK 01/8/2013 - Reinspection. 4mm crack along floor at East end construction joint. 4mm vertical crack up wall in west end of South wall at panel joint location. Other cracks 0.3mm - 3mm noted to propogate perpendicularly off the larger 4mm crack. Refer SR57 dated 02/08/2013	Y		

Refer to Table 4.1 and HCG Specification for repair details

Level	Room Number	Location (Ref Plan)	Building Element	Observations	Repair Required	Repair	Photo Reference
LG	L22	A.L.G.16	Soffit floor over	29.09.11 - Ceiling removed follow report of "uncomfortable" vibration in ED GL over. Widespread cracks identified at 200-250CRS. Refer SR 4/10/11	C	Epoxy inject as per HCG Specification	
LG	L121	A.L.G.17	Floor	19.10.11 - 0.3mm floor crack	C	Epoxy inject as per HCG Specification	19.10.11 - 008
LG	L121	A.L.G.18	Wall	19.10.11 - 0.2 diag crack from door head	N		19.10.11 - 007
LG	L46A	EW corridor	Floor Slab	27.03.13 - 1.5mm crack through floor slab Refer HCG SR46	C	Epoxy inject as per HCG Specification	
LG		North Elevation Adjacent Extension	Link Bridge Columns	19.10.11 - 0.2/0.3mm Horizontal cracks up to 300mm apart full height of column 30.5.14 - No work to be undertaken to link bridge columns at this stage.	F	Epoxy inject as per HCG Specification	
LG	L19, L22, L23, L24, L25	A.L.G 19	Floor Slab	04.04.13 - a large crack observed from the underside of the floor slab. Refer HCG SR48	C	Epoxy inject from above, as per HCG Specification	-
Throughout LG, G, L1			Floor Slab	22.05.13 - Open up representative areas as indicated in SR52 for inspection and instruction for repair.	C	Epoxy inject as per HCG specification if instructed by Engineer	

Level	Room Number	Location (Ref Plan)	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G52	A.G.01	Shear Core	21.04.11 - Hairline vertical cracking in shear core from the bottom of service penetration.	C	Epoxy Inject as per HCG Specification	15
G	G11	A.G.02	Coupling Beam	21.04.11 - Hairline crack top from top left corner of door head.	C	Epoxy Inject as per HCG Specification	16
G	G12	A.G.03	Stair Shaft	21.04.11 - Horizontal and vertical cracking through light walls in stair shaft, along length and height of walls. Damage also commonly observed from the bottom and top corners of doors & windows. Timber skirting along the bottom of stair flights, landings and at slab/stair connections shows signs of displacement.	N	Non-Structural	019, 020 (typical)
G		Ambulance Dock	Concrete Beam	21.04.11 - Minor damage to cantilever beam. Possibly existing.	N	Not Earthquake Damage	21.04.11 - Building A 004
G		Ambulance Dock	External Ground	Evidence of ground heave, Originally recommended re-levelling of the ground in this area. 30.5.14 - Client advises not re-levelling in this area	N		21.04.11 - Building A 005
G			Shear Cores, Columns	All shear cores and columns are lined on the inside and outside. Due to the nature of the ground floor (Emergency Ward) it was not practical to inspect through the ceiling at the time of inspection. Re-inspection required. 30.03.16 - All shear cores in the Emergency Department have been viewed and the cracks epoxy injected EXCEPT the shear core at G26.	Y	Expose and epoxy inject all cracks as per HCG specification.	

Level	Room Number	Location (Ref Plan)	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G27	A.G.06 ED counter, also Entry A.L.G.16	Suspended floor	29.09.11 - Staff reported "uncomfortable" vibration in the floor, experienced with walking foot traffic. Expose ceilings in Rm L22 below, check slab soffit & primary band beam load path and shear core connection. 4/10/11 Ceiling opened- 0.2mm cracking widespread in a N/S direction, appear to follow reinf bar lines @approx 200-250mm CRS Refer SR 5/10/11 090 folder Follow-up inspection 19/10/11: Perception is that it improved the vibration; Staff still concerned. Extend of epoxy work extended refer SR 26/10/11 090 folder Final inspection done: SR 23/11/11 090 folder	C		
G		A.G.08 Outside entry to Emergency Dept, over LG33 & LG33a	Planter & Paving on Suspended Concrete Floor	18.04.12 - The planter box and the paving immediately to the east of the entry to the Emergency Department have separated from the main building by up to 30mm. This is above the suspended slab that spans above LG33and LG33A out to the retaining wall to the east. Separation is visible in finishes at the entry door. Advised that when cleaner is hosing the external area, water pours into the building below. The leak is due to damage the sealant/waterproofing between the building and the external suspended slab due to movement of the retaining wall. Same movement but to a lesser degree is visible at the north end of the planter also.	C		18.04.12 - Building A 1080405-414
G	G73	North Extension (ED)	Floor beams	Steel floor beams supporting floor and wall bracing above were reviewed for damage. No damage observed. Refer site report #34	N		12-12-07_ED floor beams from below

Level	Room Number	Location (Ref Plan)	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G62	North Extension (ED)	Suspended floor	11.10.12 Cracking identified across joint between extension and Parkside Block A building. Damage has occurred as a result of insufficient separation between Parkside, basement tunnel and extension structures 30.5.14 - Repair to be addressed formally with ED extension repairs	Y	HCG to issue repair detail that instates seismic gap	12-10-11_ED opening up works
G	G73	North Extension (ED)	Suspended Floor	29.09.11 - Soffit inspected 27/10/11. Hibond spanning to steel primary beams, no apparent damage, even to beam connection back into Ground Floor slab of Block A	N		
Throughout		A.G.07	Stairs 1 & 2	29.09.11 - Fine soffit cracking identified	C	Stair remedial issued separately	
L1	1.15	A.1.01	Shear Core, Coupling Beam	20.04.11 - Damage to plaster wall adjacent coupling beam and shear core.	N	Non-structural	20.04.11 - Building A 017
L1	1.01, 1.01A	A1.02	Concrete Stairs/Slab Stair Shaft	20.04.11 - Damage to L2 slab soffit at stair/slab connection.	C	Stair remedial issued separately	20.04.11 - Building A 021, 022
L1	1.73A	A1.03	Shear Core	20.04.11 - Hairline horizontal and diagonal shear cracks in wall. Extent unknown due to electrical switchboard (other switchboard doors locked). 0.2mm cracking at wall/slab connection.	C C	Expose shear core and epoxy inject all cracks as per HCG specification.	20.04.11 - Building A 002, 003
L1	All		Shear Cores, Columns	No shear cores were inspected entirely. All cores and columns lined, no ceiling tiles could be lifted. No access to the north-eastern and north-western cores at time of inspection. Re-inspection required.	C	Expose shear core and epoxy inject all cracks as per HCG specification.	

Level	Room Number	Location (Ref Plan)	Building Element	Observations	Repair Required	Repair	Photo Reference
L1	1.104	A.1.04	Floor at Bridge	19.10.11 - 0.4mm crack at bridge/building line Steel stringer connections inspected 27/10/11- No apparent damage 30.5.14 - No repair work to be undertken to bridges at this stage.	F	Epoxy Inject as per HCG Specification	19.10.11 - 028
L1	1.41	A.1.05	Floor Slab	28.02.14 - Observations undertaken due to vibration issues noted by staff. Cracks of approximately 0.3mm wide noted to floor slab. Furhter investigations to other ICU department rooms to be undertaken as rooms become available	C	Epoxy inject cracks greater than 0.3mm as per HCG specification	-
L1	1.77	Above ceiling	Floor Slab	08.08.12 - Shrinkage cracking observed throughout floor slab	Y	Epoxy inject as per HCG Specificaion	12-08-08_Parkside Level 1 Rm1.77
L1	G62	Above ceiling	Steel post supporting EDE roof	11-10-12 - Connection of steel post to L1 floor viewed. One bolt appears to be bent. Also no grout under connection.	Y	Check if bolt bent and if so replace. Grout under connection. Check other similar connections	12-10-11 PA110401 - PA110411
L1	G62 G68		Roof Cross Bracing	11-10-12 - Roof cross-bracing in EDE viewed. No obvious damage noted	N		12-10-11 PA110408 12-10-03 IMG_0408 - IMG_0413

Refer to Table 4.1 and HCG Specification for repair details

Level	Room Number	Location (Ref Plan)	Building Element	Observations	Repair Required	Repair	Photo Reference
L2	2.42	A.2.01	Shear Wall	21.04.11 - Upper part of shear wall inspected through ceiling. No damage observed. 12.08.13 - Northern Shear walls inspected from corridor, epoxy injected cracks greater than 0.2mm 08.10.13 - Crack Maps and Photos provided by FCC show completed repairs to shear wall cores in Ward 16.	C	Expose shear core and epoxy inject all cracks as per HCG specification.	13.08.05 Ward 16 13.10.08 014-021
L2	2.16, 2.06	A.2.02	Coupling Beam	20.04.11 - Crease in wallpaper from top right side of door head- possible damage to beam beyond. Likely non-structural	N		20.04.11 - Building A 018
L2	2.09, 3.09	A.2.03	Concrete Stairs	20.04.11 - Flexural crack across bottom of L3 stair throat. 31.01.13 - spalling noted to mid-height landing	C	Stair remedial issued separately	20.04.11 - Building A 023
L2	2.57	A.2.04	Concrete Columns	20.04.11 - Cracking around perimeter of column near column head and associated plaster damage. Hairline-0.2mm. Applies to all columns in courtyard. Some spalling also. 27.02.15 Photos of repair undertaken provided by RCP	C	Epoxy inject and repair spalling as per HCG specification	20.04.11 - Building A 024, 025, 026 (Typical)
L2	2.57	A.2.05	Precast cladding Spandrel	20.04.11 - Minor spalling and cracking at column/beam connection.	C		20.04.11 - Building A 027
L2	2.73, 2.88	A.2.06	Concrete Column, Level 3 Slab	20.04.11 - Top of column and slab soffit adjacent seismic joint inspected locally through exposed ceiling. No damage observed.	N		20.04.11 - Building A 028, 029
L2	2.57	A.2.07	Spandrel Beam	20.04.11 - Hairline cracking to beam at beam/column connection.	N		20.04.11 - Building A 001

Refer to Table 4.1 and HCG Specification for repair details

Level	Room Number	Location (Ref Plan)	Building Element	Observations	Repair Required	Repair	Photo Reference
L2	All		Shear Walls, Columns	No shear walls or columns properly inspected. Walls and columns are lined. Access to the roof space is limited as roof tiles in the ward corridors cannot be easily lifted (likely due to services above), and it was not practical at the time of inspection to lift ceiling tiles in the hospital rooms. Re-inspection required.	C	Expose shear core and epoxy inject all cracks as per HCG specification.	
L2	2.01, 2.02, 2.05, 2.06	A.2.08	Precast cladding connections	02.05.14 - Top and bottom precast cladding panel fixings inspected. No damage noted but movement allowance is less than expected interstorey drift. Connections to be upgraded as timing allowed.	Y	New panel fixings, details and locations to be provided by HCG	
L3	3.16	A.3.01	Shear Wall	20.04.11 - Upper 30cm of wall inspected along corridor. No damage observed.	C	Expose shear core and epoxy inject all cracks as per HCG specification.	
L3		A.3.02	Columns	20.04.11 - No observed damage to exposed L3 columns when viewed from L2 courtyard (2.57).	N		
L3	Various		Shear Walls, Columns	No shear walls or columns properly inspected. Walls and columns are lined. Some roof tiles in the corridor were lifted however services obstructed the view of the shear wall beyond, and it was not practical at the time of inspection to lift ceiling tiles in the hospital rooms. Re-inspection required.	C	Expose shear core and epoxy inject all cracks as per HCG specification.	
G 1, 2, 3	Various		Stair Shaft	20.04.11 - Northern stair shaft displays damage typical of that described in the southern shaft.	C	Stair remedial issued separately	
G 1, 2, 3		Perimeter of Building	Concrete Columns	20.04.11 - No observed damage to exposed columns on the perimeter of the building when viewed from outside.	N		6

Level	Room Number	Location (Ref Plan)	Building Element	Observations	Repair Required	Repair	Photo Reference
L4	4.01	A.4.1	General Observation	20.04.11 - • No cracking in floor slab observed. • No cracking in perimeter up-stand beam observed. • No evidence of damage in steel roof structure other than alignment issue entered below. 04.12.12 - Revisited by LJO - same comments	N		04/12/12 - IMG_7009, 7010
L4	4.01	A.4.2	Seismic Joint	05.04.11 - Cover plate lifted & bent	C	Non-Structural	05.04.11 - 002, 003
L4	4.01	A.4.3	Seismic Joint	05.04.11 - Cover plates at the roof profile distorted	C	Non-Structural	05.04.11 - 004
L4	4.01	A.4.4	Roof Steel Portals	05.04.11 - Vertical portal legs out of alignment. Possibly inherent in construction.	Y	Not Earthquake Damage	05.04.11 - 005, 006
L4	4.01	A.4.5	Pipe Rack Support Frames	05.04.11 - Observation: Additional frames have been installed and damaged HD bolts have been replaced. No base plate grouting.	C	Grout under base plates	05.04.11 - 007
L4	4.01	A.4.6	Ceiling Lining	05.04.11 - Buckled flashings at steel rafter ceiling lining interface. Non structural	C	Non-Structural	05.04.11 - 008
L4	4.01	A.4.7	Wall Linings	05.04.11 - Fire wall & lining damage. Non structural	C	Non-Structural	05.04.11 - 009
L4	4.01	SE corner	Roof Steel Portals	04.12.12 - Damaged base plate connection. Appears that thread has been stripped off nut. 27.02.15 Photos of repair supplied by RCP, nut replaced.	C	Refer to SR33 - try replacing nut	04.12.12 - IMG_7007, IMG_7008
L4	4.01	All round	Roof Steel Portals	04.12.12 - many connections between the steel framing and concrete up stand beam have been damaged. Dry pack has been dislodged or crumbled.	C	Refer to SR33 - replace grout where loose	04.12.12 - IMG_7009 to IMG_7012
L4	4.04	A.4.8	Beams	05.04.11 - Cracks in transverse concrete beams 0.1mm	N		

Refer to Table 4.1 and HCG Specification for repair details

APPENDIX A2 – RECORD OF OBSERVATIONS & REPAIRS - PARKSIDE BLOCK B

Inspection date: 5th April, 2011
Update: 29th September 2011
Update: 18th October 2011
Update: 30th May 2014
Update: 27 February 2015 Photos supplied by RCP of some repair work
Update: 31st July 2015 Photos supplied by RCP of repair items

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

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
LG	L120	B.LG.1	Floor Slab	07.04.11 - Extensive cracking (0.2mm – 0.4mm) (NB: The concrete slab in room L120 is approximately 300mm above the typical lower ground floor level)	N		07.04.11 - Building B LG 1 & 2
LG	L118	B.LG.2 SS by entry B.LG.7	Shear Core	07.04.11 - Shear wall fully lined. The upper parts of the walls have been inspected through the ceiling where possible and no cracking was observed. Re-inspection will be required in order to determine the extent of damage.	C	Expose shear core and epoxy inject as per HCG specification	

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
LG	L122	B.LG.3	Lined Wall	07.04.11 - Significant damage to wall lining next to the shear core. Wall linings to be removed to allow further inspection of shear core walls.	C	Expose shear core and epoxy inject as per HCG specification	07.04.11 - Building B LG 3
LG	L114	SS by entry B.LG.7	Shear Core Wall	07.04.11 - Combination of Horizontal Vertical and Diagonal cracks (0.1mm - 0.2mm)	C	Epoxy inject as per HCG specification	07.04.11 - Building B LG 4
LG	L127 & L127a	B.LG.5	Seismic Joint	07.04.11 - Significant displacement between Building C and Building B along seismic joint. Varies in severity along the length of the joint	C	May be repaired during re-levelling	07.04.11 - Building B LG 5
LG		B.LG.6	Shear Core North	29.09.11 - Linings removed. Cracking mapped 30/6/11: Widespread cracking 0.2/0.3mm Work completed 10-15 lt of injection product	C	Epoxy inject as per HCG specification	
LG		B.LG.7	Shear Core: South	29.09.11 - Linings removed. Widespread cracking 0.2/0.3mm Work completed 10-15 lt of injection product	C	Epoxy inject as per HCG specification	
Throughout	L124A, B, C	B.LG.8	Lift Shaft & Pit	29.09.11 - General: No significant damage identified. Some gib cracking (Light weight fire rated walls). All steel framing OK. Concrete work around the lift over-run OK	N		
G	G125, G124	B.G.1	Shear Core Walls	5.04.11 - Shear wall fully lined. The upper parts of the walls have been inspected through the ceiling where possible and no cracking was observed. Re-inspection will be required in order to determine the extent of damage.	C	Expose shear core and epoxy inject as per HCG specification	
G	G122	B.G.2	Shear Core Walls	13.04.11 - Diagonal crack viewable from service duct room (0.3mm). Horizontal cracking also evident (0.1mm	C	Expose shear core and epoxy inject as per HCG specification	13.04.11 - Building B G1

Refer to Table 4.2 and HCG Specification for repair details

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G122	B.G.3	Beam/Columns by Lift Shaft	13.4.11 - Good view of the beams/columns from service duct room. No damage observed.	N		2
G	G129	B.G.4	Precast Wall Panel	13.4.11 - Horizontal crack at approximately mid height in precast panel (0.3mm)	N		3
G	G128	B.G.5	Concrete Column	13.4.11 - Reasonably widespread hairline flexural cracks (<0.1mm) in columns	N		4

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G128	B.G.6	Panel Connections	13.4.11 - Precast concrete panel connections appear to be in good condition. Some movement evident	N		5
G	G123	B.G.7	Floor Slab	13.4.11 - Superficial crack across marble floor built over a seismic joint	N		6
G	G118	B.G.8	Shear Core	13.4.11 - Widespread hairline cracking evident in shear core wall (0.1mm)	C	Expose shear core and epoxy inject as per HCG specification	7
G	G113	B.G.9	Shear Core Wall	13.4.11 - Minor cracks viewable in roof space (0.1)mm	C	Expose shear core and epoxy inject as per HCG specification	
G	G162		Floor slab	27.07.12 - Shrinkage cracking observed in suspended slab throughout room. Cracking observed in Rm G162.	N	Repair recommendations on case-by-case basis. Refer SR006	12.10.25_Block C+B_Lv 2+3 Parkside wards
Throughout		B.G.10	Stairs 3 & 4	29.09.11 - Fine soffit cracking identified. Significant crack in u/s of knee on flight coming from rising from mid-height landing Separation and sliding support of mid-height landing remedial currently in for BC	C	Stair remedial issued separately	

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
L1	1.136	B.1.1	Precast Panel	28.04.11 - Large flexural crack through panel approximately at midspan (0.4mm). Panel connections appear to be in good condition	Y	Epoxy inject as per HCG specification	28.04.11 - Building B L1 001
L1	1.136	B.1.2	Column/Beam	28.04.11 - No visible damage to column or beam	N		
L1	1.131/1.132	B.1.3	Shear Core	28.04.11 - Very minor hairline cracking found in roof space	N		
L1	1.13	B.1.4	Shear Core	28.04.11 - Minor hairline cracking found in roof space	N		2
L1	1.126	B.1.5	Shear Core	28.04.11 - Significant crack from service penetration (0.2mm). Minor flexural cracking also present across the wall (0.2mm)	C	Expose shear core and epoxy inject as per HCG specification	3
L1	1.117	B.1.6	Shear Core	19.10.11 - Diagonal crack from service penetration (0.3mm)	C	Expose shear core and epoxy inject as per HCG specification	4
L1	1.117	B.1.7	Floor at Bridge	19.10.11 - 0.3mm transverse crack on "park-side". Vert fine crack on bridge-side	Y	Epoxy inject as per HCG specification	19.10.11 - 025, 026
L1	1.118		Shear Core	28.04.11 - Diagonal cracking on all internal walls throughout shear core. Cracks up to 0.75mm	C	Expose shear core and epoxy inject as per HCG specification	

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
L2	2.117	B.2.1	Shear Core	28.04.11 - Very minor hairline cracking	C	Expose shear core and epoxy inject as per HCG specification	
L2	2.121	B.2.2	Column/Precast Panel	28.04.11 - Minor hairline cracking in column and across precast panel (<0.1mm)	N		
L2				28.04.11 - Suspended ceiling was removed at time of inspection allowing a thorough investigation of the above floor and supporting structure. No notable damage to the support system was discovered	N		
L2	2.125/2.102	B.2.3	Staircase	28.04.11 - Major cracking at the top of staircase through the first stair (<1.50mm)	C	Stair remedial issued separately	28.04.11 - Building B L2 1, 2
L3	3.105, 3.126	B.3.1	Staircase	28.04.11 - Major cracking at the top of staircase through the first stair (<1.50mm). Cracking evident in staircases right the way up the building. Photos show cracking at both staircases	C	Stair remedial issued separately	28.04.11 - Building B L3 1, 2
L3	3.101A	B.3.2	Seismic Joint	28.04.11 - Movement evident at seismic joint		Non structural repair	3
L3	3.128	B.3.3	Column	28.04.11 - Significant cracking and spalling around the beam column joint evident. Likely the consequence of poor construction 27.02.15 Photos of repair supplied by RCP	C	Epoxy inject and repair spalling as per HCG specification	4, 5, 6
L3	3.122	B.3.4	Precast Panel Connection	28.04.11 - Bolt in the connection of the precast panels to the concrete beam appears to have yielded. Other bolts along the wall line appear to be packed with steel plates therefore damage may be isolated. 09.10.11 - Remedial detail issued. Location and quantum needs to be confirmed	Y	HCG to provide repair detail	7

Refer to Table 4.2 and HCG Specification for repair details

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
L3	3.116		Column	20.07.12 - Spalled area of concrete at top of column on Grid D:20	C	Repair areas of spalling in accordance with the HCG specification.	12-07-20 RCP Block B Level 3 Rm 3116 Column Crack
L3	3.122	B.3.5	Column	28.04.11 - Minor hairline flexural cracking visible on the column. Cracking is widespread, but too fine to be required to be repaired	N		
L3	3.125	B.3.7	Column at Seismic Joint	28.04.11 - Minor cracking at column by seismic joint. 03.03.15 - Column inspected, no cracks greater than 0.2mm noted. No repair required.	N	No repair required	8
L3	3.175 3.178		Floor SLab	23.09.14 - Cracking ~0.3 - 0.7mm max. in width in L3 floor suspended slab. Cracking appears to be existing shrinkage cracking. Potentially jacked open by movement of adjacent shear wall elements. Photos provided by FCC following crack injection repairs	C	Epoxy injection repairs complete	14.09.23 L03 Floor Slab Injection Photos From FCC
L3	3.176	B.3.8	Cracking in Concrete Panel	28.04.11 - Vertical cracking in external concrete panel. Superficial damage as member is not structural but requires repair for durability. Panel has been removed, therefore no further repair required.	N	Epoxy inject as per HCG specification - panel may have been removed, if so no repair required	9, 10
L3	3.176/177 /179		Floor slab	28.10.11 - Cracking ~0.7mm max. in width in L3 floor suspended slab. Cracking appears to be existing shrinkage cracking.	N	Repair recommendations on case-by-case basis. Refer SR024	12.10.25_Block C+B_Lv 2+3 Parkside wards

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
L4	4.13	B.4.1	General Observation	28.04.11 - • No evidence of cracking in floor slab. • No evidence of cracking in perimeter up-stand beam. • No evidence of damage in steel roof structure. 04.12.12 - Revisited by LJO - same comments	**N**		
L4	4.13	B.4.2	Columns	05.04.11 - Fine horizontal cracks concentrated at the top of the columns. Typical all 4 columns supporting lift motor room over. 0.1mm	**C**	Epoxy inject as per HCG specification	05.04.11 - 011

APPENDIX A3 – RECORD OF OBSERVATIONS & REPAIRS - PARKSIDE BLOCK C

Inspection date: 5th April, 2011
Update: 29th September 2011
Update: 18th October 2011
Update: 18th April 2012
Update: 30 May 2014
Update: 27 February 2015. Photos of some repairs supplied by RCP

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

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
BM	B04	C.BM.2	Retaining Wall	18.04.12 - Vertical cracks in the retaining wall that forms the south side of the central east-west corridor through Block C. This supports the Lower Ground Floor raft slab. Cracks at 0.8 to 1.2m centres along the length of the wall and vary in width between 0.3mm and 1mm. Cracks are wider at the top of the wall and extend into the suspended Lower Ground raft slab. Horizontal crack at approximately 1.6m above the basement slab in what appears to be a repair to a construction joint. Crack up to 0.7mm in width	C	Epoxy Inject as per HCG specification	18.04.12 - 1080394-398

Refer to Table 4.3 and HCG Specification for repair details

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
BM	B04	C.BM.1	Floor Soffit Over	29.09.11 - Longitudinal crack in soffit 0.2mm	N		
LG	L152, L152a	C.LG.9	Shear Core	7.04.11 - Extensive cracking through the shear core (0.2mm – 0.4mm). Predominantly diagonal cracks visible on both North-South walls	C	Epoxy Inject as per HCG specification	7.04.11 - Building C LC 13 - 15
LG	L152, L152a	C.LG.8	Shear Core	7.04.11 - Extensive cracking through the shear core (0.1mm – 0.3mm). Predominantly horizontal and vertical cracks visible on both East-West walls	C	Epoxy Inject as per HCG specification	7.04.11 - Building C LC 10 - 12
LG	L154C	C.LG.7	Seismic Joint	7.04.11 - Significant displacement between Building C and Building D along seismic joint	C	May be repaired by re-levelling	7.04.11 - Building C LC 9
LG	L161, L158C	C.LG.6	Ground Floor Slab	7.04.11 - Diagonal cracking viewable along ground floor (0.3mm). Appears to have originated from existing construction joint. Crack continues through support beam	N		7.04.11 - Building C LC7, 8
LG	L129, L131	C.LG.5	Wall Lining	7.04.11 - Vertical cracking through wall lining. Superficial only – roof space inspected and crack was apparently not continuous into concrete shear core	C	Expose shear wall and epoxy Inject as per HCG specification	7.04.11 - Building C LC 6
LG	L126	C.LG.4	Shear Core	7.04.11 - Diagonal Cracking observed in roof space (0.2mm). Wall lined below therefore extent of the cracking not known	C	Expose shear wall and epoxy Inject as per HCG specification	7.04.11 - Building C LC 4, 5
LG	L113	C.LG.3	Shear Core	7.04.11 - Diagonal Cracking observed in roof space (0.2mm). Wall lined below therefore extent of the cracking not known	C	Expose shear wall and epoxy Inject as per HCG specification	7.04.11 - Building C LC 3

Refer to Table 4.3 and HCG Specification for repair details

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
LG	L150, 151a	C.LG.2	Shear Core	7.04.11 - Diagonal cracking observed in roof space (0.3mm). Wall lined below therefore extent of the cracking not known	C	Expose shear wall and epoxy Inject as per HCG specification	7.04.11 - Building C LC 2
LG	L127, 127A	C.LG.15	Suspended Floor	29.09.11 - Lifted seismic joint cap & 15mm step across the join into block B. Grind back raised screed. Grout fill de-lamination. Re-connect capping plate Remedial instructed 11/5/11. Work complete	C		
LG	L147	C.LG.14	Suspended Floor	23.08.11 - 20mm level change identified. Soffit of floor inspection identified 2x 0.2/0.3mm cracks (23/8/11)	C	Epoxy Inject as per HCG specification	
LG		C.LG.13	Shear Core: South East	29.09.11 - Core cracking not observed by HCG. Widespread cracking was identified by FCC. Injection work was carried out base on previous remedial criteria. Work complete 10-15 lt of injection product	C	Epoxy Inject as per HCG specification	
LG		C.LG.12	Shear Core: South West	29.09.11 - Core cracking not observed by HCG. Widespread cracking was identified by FCC. Injection work was carried out base on previous remedial criteria. Work complete 10-15 lt of injection product	C	Epoxy Inject as per HCG specification	
LG		C.LG.11	Sear Core: North East	29.09.11 - Only the north and west outside faces exposed. Hairline cracking identified no work considered necessary. Exposure was limited due to lined server room L151a	C	Expose shear wall and epoxy Inject as per HCG specification	

Refer to Table 4.3 and HCG Specification for repair details

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
LG	L152, L152a	C.LG.10	Shear Core: North West	29.09.11 - Cracking mapped 4/7/11: Widespread 0.2/0.3mm cracking Work completed approx. 55 lt of injection product	C	Epoxy Inject as per HCG specification	
LG	L145	C.LG.1	Shear Core	7.04.11 - Diagonal crack observed in roof space (0.2mm). Wall lined below therefore extent of the cracking not known	C	Expose shear wall and epoxy Inject as per HCG specification	7.04.11 - Building C LC 1
LG	L137 & L145	C.L.G 16	Floor Slab	23.04.13 - Cracking up to 1.2mm wide in Block C and 3.0mm wide in Block B side. Refer SR49	C	Epoxy inject as per HCG specification	
G	G165	C.G.9	Precast Wall Panels	13.04.11 - Some hairline cracking (<0.1mm) in precast panels. Panel connections appear to be in good condition	N		13.04.11 - Building C G 4 - 5
G	G195	C.G.8	Shear Core	13.04.11 - Walls lined from suspended ceiling down to the ground. Diagonal crack found running from service penetration (0.1mm) Minor hairline shear cracks also present (<0.1mm)	C	Expose shear wall and epoxy Inject as per HCG specification	13.04.11 - Building C G 10 - 11
G	North Wall	C.G.7	Precast Panel Connections	13.04.11 - No cracking detected in concrete around the precast panel connection	N		
G	G170	C.G.6	Shear Core	13.04.11 - Walls lined from suspended ceiling down to the ground. Cracking detected in roof space both diagonal and vertical (0.2mm)	C	Expose shear wall and epoxy Inject as per HCG specification	13.04.11 - Building C G 3
G	G167	C.G.5	Shear Core	13.04.11 - Diagonal crack visible in roof space (0.3mm). Crack has initiated from the corner of the service duct penetration	C	Expose shear wall and epoxy Inject as per HCG specification	13.04.11 - Building C G 2

Refer to Table 4.3 and HCG Specification for repair details

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G162	C.G.4	Column/Floor Beam	13.04.11 - Columns/beams in good condition. No cracking found in roof space	N		
G	G157	C.G.3	Shear Core	13.04.11 - Walls lined from suspended ceiling down to the ground. cracking found in roof space	C	Expose shear wall and epoxy Inject as per HCG specification	
G	G154	C.G.2	Shear Core	13.04.11 - Walls lined from suspended ceiling down to the ground. evidence of cracking in roof space	C	Expose shear wall and epoxy Inject as per HCG specification	
G	G210	C.G.14	Shear Core	13.04.11 - Walls lined from suspended ceiling down to the ground. Diagonal crack found running from service penetration (0.2mm)	C	Expose shear wall and epoxy Inject as per HCG specification	13.04.11 - Building C G 14
G	G210	C.G.13	Shear Core	13.04.11 - Walls lined from suspended ceiling down to the ground. Diagonal crack found running from service penetration (0.1mm). Minor hairline shear cracks also present (<0.1mm)	C	Expose shear wall and epoxy Inject as per HCG specification	13.04.11 - Building C G 12 - 13
G	G198	C.G.12	Shear Core	13.04.11 - Walls lined from suspended ceiling down to the ground. Diagonal crack found running from service penetration (0.1mm)	C	Expose shear wall and epoxy Inject as per HCG specification	13.04.11 - Building C G 9
G	G201	C.G.11	Stairwell	13.04.11 - Larger extensive cracking right through the stairwell (0.1mm-0.4mm). Damage isolated to staircase closer to shear core	C	Stair remedial issued separately	13.04.11 - Building C G 8

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G187	C.G.10	Shear Core	13.04.11 - Spalled concrete around beam/column joint. Cracks found in roof space (0.2mm)	C	Epoxy inject and repair spalled concrete as per HCG specification	13.04.11 - Building C G 6 - 7
G	G156	C.G.1	Shear Core	13.04.11 - Walls lined from suspended ceiling down to the ground. Cracking evident in roof space (0.2mm)	C	Expose shear wall and epoxy Inject as per HCG specification	13.04.11 - Building C G 1
G	Various		Floor slab	27.07.12 - Extensive shrinkage cracking throughout Ground Floor suspended slab. Rm. G.161-163, G167-172, G198. See RCP crack map email dated 10/10/12	N	-	
Throughout		C.G.15	Stair 5	16.9.11 - Fine soffit cracking identified Separation and sliding support of the mid-height landing to be detailed. Schematic issue to Project Manager 16/9/11 Due to misalignment of support structure there is a proposed departure from the standard remedial scheme. Upstand stiffening beams support stair load at each floor level, rather that a continuous column to foundation	C	Stair remedial issued separately	

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
L1		C.1.1	General Comment	13.04.11 - Level one of Parkside Building C is the main surgery area for the hospital. Inspection of the area was not possible at the time of investigation. Floors below and above this level have be assessed. Further investigation is required to assess the damage present at Level 1. 29.01.13 - Walk through completed with RCP and Fletchers no significant structural damage noted. Only limited structure exposed for viewing.			
L1			Shear cores	Limited structure available for review	Y	Expose shear wall and epoxy Inject as per HCG specification	
L1	T2, 4, 6, 7		Theatre Light Pendants	23.11.11 - Access provided. Inverted cantilever bracket and connection plate and bolting to soffit of floor observed. There is a gap between the soffit and the plate with spacer nuts that are generally not engaged (i.e. hard-on) Recommend a simple load test on the mechanical fixing when theatre renovation is done in January	F		23.11.11.-032 to 038
L2	2.186, 2.215, 2.191, 2.177	C2.13	Shear Core	24.09.13 - Wall linings removed to allow shear walls to be observed. 0.4-0.6mm cracking typical on each wall (refer to SR64). Obervations from one side of wall only, cracks grinded prior to inspection.	C	Expose shear wall and epoxy Inject as per HCG specification	13-09-27 13-09-24 Ward 15 Shear Walls

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
L2	2.185a	C.2.9	Seismic Joint	28.4.11 - Significant vertical displacement between buildings evident 30.5.14 - Repair to joint complete, no re-levelling proposed.	C	Re-levelling may repair	28.4.11 - Building C L2 7
L2	2.185	C.2.8	Staircase	28.4.11 - Minor horizontal cracking across staircase (<0.1mm)	C	Stair remedial issued separately	28.4.11 - Building C L2 6
L2	2.203	C.2.7	Shear Core	28.4.11 - Minor vertical and diagonal cracking evident (<0.2mm)	C	Expose shear wall and epoxy Inject as per HCG specification	28.4.11 - Building C L2 5
L2		C.2.6	External Columns	28.4.11 - Widespread hairline shrinkage cracking found in external columns. Horizontal flexural cracking also evident (<0.1mm)	N		28.4.11 - Building C L2 4
L2	2.156	C.2.5	Shear Core	28.4.11 - Minor vertical and diagonal cracking	C	Expose shear wall and epoxy Inject as per HCG specification	
L2	2.152	C.2.4	Shear Core	28.4.11 - Minor vertical and diagonal cracking evident (0.2mm)	C	Expose shear wall and epoxy Inject as per HCG specification	28.4.11 - Building C L2 3
L2	2.155	C.2.3	Shear Core	28.4.11 - Minor horizontal and diagonal cracking evident (0.2mm). Shear core to floor connection ok – no cracking evident	C	Expose shear wall and epoxy Inject as per HCG specification	28.4.11 - Building C L2 2

Refer to Table 4.3 and HCG Specification for repair details

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
L2	2.149	C.2.2	Column at Seismic Joint	28.4.11 - Minor hairline cracking evident in column	**N**		28.4.11 - Building C L2 1
L2	2.181	C.2.12	Precast Panel Connections	1.10.13 -Local area of wall linings between windows removed to allow inspection of precast panel connections (refer to SR61 and 62). No damage observed. Not able to tell if bolt to panel central to steel bracket. Bracket appears to be same as page PP-9 in calculations.	**N**		13-10-01 - DSCF5884-88
L2	2.181	C.2.12	Precast Panel Connections	1.10.13 -Ceiling tiles removed to allow partial inspection of precast panel connections (refer to SR61 and 62). Only bracket connection to beam soffit observed, no damage observed. Not able to tell if bolt to panel central to steel bracket. Bracket appears to be same as page PP-11 in calculations.	**N**		13-10-01 - DSCF5889
L2	2.177	C.2.11	Precast Panel Connections	1.10.13 -Local area of wall linings between windows removed to allow inspection of precast panel connections (refer to SR61 and 62). No damage observed. Not able to tell if bolt to panel central to steel bracket. Bracket appears to be same as page PP-9 in calculations.	**N**		13-10-01 - DSCF5876-83
L2	2.175	C.2.10	Seismic Joint	28.4.11 - Significant horizontal displacement between buildings at seismic joint 30.5.14 - Repair to joint complete, no re-levelling proposed	**C**	Re-levelling may repair	
L2	2.138	C.2.1	Beam Column Joint	28.4.11 - No evidence of damage in beam or column	**N**		

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
L2			Floor slab	25.10.12 - Extensive cracking throughout L2 floor suspended slab. Both seismically induced and existing shrinkage cracking varying from hairline up to 1.5mm in width.	N	Refer site report 106186.20SR2910.025 for HCG comments.	12.10.25_Block C+B_Lv 2+3 Parkside wards
L3	3.168	C.3.3	Seismic Joint	28.4.11 - Significant movement evident at seismic joint	C	Non-structural repair	
L3	3.143	C.3.2	Column	28.4.11 - Minor horizontal/flexural cracking observed in column	C	Epoxy Inject as per HCG specification	
L3	3.217	C.3.1	Seismic Joint	28.4.11 - Significant damage to seismic joint. Noticeable difference in floor height from Building D – Building C 30.5.14 - Repair to joint complete, no re-levelling proposed	C	Re-levelling may repair	
L3			Floor slab	25.10.12 - Extensive cracking throughout L3 floor suspended slab. Both seismically induced and existing shrinkage cracking varying from hairline up to 1.5mm in width.	N	Refer site report 106186.20SR2910.025 for HCG comments.	12.10.25_Block C+B_Lv 2+3 Parkside wards
L3			Precast Wall Panels	25.10.12 - Vertical Cracking (~0.4mm in width) to precast panels along southern elevation of courtyard. Horizontal cracking to precast panels along southwest elevation.	N	Panels removed as part of the New Parkside Wards project	12.10.25_Block C+B_Lv 2+3 Parkside wards

Refer to Table 4.3 and HCG Specification for repair details

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
L4		C.4.6	Parapet Panel	29.09.11 - Parapet panels, Grout lifted in HD pocket	Y	Refer SK102 Appendix C	
L4		C.4.5	Parapet Panel	29.09.11 - Dislodged parapet panel, bent bolt	N	Refer SK102 Appendix C	
L4	4.15	C.4.4	Concrete Beams	5.04.11 - Cracking to beams 27.02.15 Repair photos supplied by RCP	C	Epoxy Inject as per HCG specification	5.04.11 - 015
L4	4.16	C.4.3	Seismic Joints	5.04.11 - Dislodged parapet panel, bent bolt	N	Refer SK102 Appendix C	5.04.11 - 014
L4	4.16	C.4.2	Seismic Joints	5.04.11 - Floor capping plates lifted & bent	C	Architects Schedule	5.04.11 - 013
L4	4.16	C.4.1	Architectural Walls	5.04.11 - Lining & fire wall damage. Non structural	C	Non-structural repair	5.04.11 - 012
L4	4.15	Around perimeter	Steel frame base connections	4.12.12 - Damage to dry pack under multiple connections between steel roof framing and concrete up-stand beams 30.5.14 Bolts tightened and drypack repaired. Further investigation of bolted joints required, refer SR 77	Y	Refer site report 33 - Reinstate grout Refer site report 77 for further work	4.12.12 - IMG_7011

APPENDIX A4 – RECORD OF OBSERVATIONS & REPAIRS - PARKSIDE BLOCK D

Inspection date: 5thApril, 2011, 2 May 2011, 3 May 2011
Update: 29th September 2011
Update: 18th October 2011
Update:December 2012
Update: 30 May 2014
Update: 27 February 2015. Photos supplied by RCP of some completed repair work
Update: 31 July 2015. Photos supplied by RCP of some completed repair work

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

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
BM	B09	D.BM.1	Internal Shear Cores	06.04.11 - Widespread horizontal and vertical cracking 0.2 to 0.4mm	C	Epoxy Inject as per HCG Specification	
BM	B09	D.BM.2	Retaining Walls	06.04.11 - Widespread vertical cracking 0.2 to 0.4mm 30.5.14 - Most repair complete but not all.	N		06.05.11 - 028, 030, 031, 032
BM	B09	D.BM.3	Stub Column	06.04.11 - No evidence of cracking	N		
BM	B09	D.BM.4	Full Height Columns	06.04.11 - No evidence of cracking	N		
BM	B09	D.BM.5	Perimeter Walls	06.04.11 - No evidence of cracking from sample inspected. Further inspection required to confirm	F		
BM	B09	D.BM.6	Shear Wall, West Elevation Behind Lift Shaft	06.04.11 - Full inspection required from inside shaft Car ride done 23/8/11 No evidence of cracking in lift pit or west shear wall up the building	N		

Refer to Table 4.4 and HCG Specification for repair details

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
BM	B09	D.BM.7	Trench Slab	06.04.11 - Cracking & evidence of silt ingress (Likely to be older shrinkage cracking)	N		
BM	B09	D.BM.8	Upper Raft Slab	06.04.11 - Random slab cracking (fine)	N		
BM	B09	D.BM.9	Interface Between D & C	06.04.11 - Cracking & spalling between the two structures from pounding	Y	Epoxy inject cracks and repair spalled concrete as per HCG Specification.	06.04.11 - 033, 038
BM	B09	D.BM.10	South Wall West Lift Shaft	06.04.11 - Widespread diagonal cracks 0.4mm Not evident from inside lift pit ? (23/8/11) Recommend follow up inspection	N		
BM		D.BM.11	North Wall West Lift Shaft	06.04.11 - Widespread diagonal cracks 0.4mm Not evident from inside lift pit ? (23/8/11) Recommend follow up inspection	N		06.04.11 - 029
BM	B09	D.BM.12	"Wing Wall" North Elevation	06.04.11 - Diagonal crack 0.2mm	N		06.04.11 - 027
BM		D.BM.13	Raft Slab - South Side Crawl Space	29.09.11 - 0.3mm cracking on top of raft slab identified by contractor. Injection work is been carried out	N		

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
LG	L214	D.LG.1	North-East Shear Core- West Elevation	02.04.11 - Numerous diagonal cracks <0.2mm predominantly running from re-entrant corners towards the North (NB. Observations for this shear core coupling beams have only been made above ceiling level. No observations have been made of the walls due to wall linings.)	C	Expose shear wall and epoxy Inject as per HCG specification	02.05.11 - 004, 005, 006
LG	L210	D.LG.2	North-East Shear Core - North Elevation	02.04.11 - Few hairline cracks <0.2mm (NB. Cracking on North and South elevations observed was generally to a lesser extent than that of the East and West elevations)	N		
LG	L210	D.LG.3	North-East Shear Core - East Elevation	02.04.11 - Fine cracks <0.2mm	N		
LG	L229	D.LG.4	North-East Shear Core - South Elevation	02.04.11 - Few hairline cracks <0.2mm (Similar to North elevation)	N		
LG	L225	D.LG.5	South-East Shear Core - All Elevations	02.04.11 - No access due to fully lined ceilings. Further inspection required to confirm damage. 30.5.14 - Crack injection to LG floor shear cores complete	C		
LG	L200	D.LG.6	North-West Shear Core - West Elevation	02.04.11 - Numerous diagonal cracks <0.2mm predominantly running from re-entrant corners towards the North. Similar to corresponding wall at North-East shear wall. (NB. Observations for this shear core coupling beams have only been made above ceiling level. No observations have been made of the walls due to wall linings.)	N		

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
LG	L200	D.LG.7	North-West Shear Core - North Elevation	02.04.11 - Few hairline cracks <0.2mm	N		
LG	L233	D.LG.8	North-West Shear Core - East, South Elevation	02.04.11 - Unable to remove ceiling tiles	N		
LG	L240	D.LG.9	South-West Shear Core - West Elevation, Looking West	02.04.11 - Numerous diagonal cracks <0.2mm predominantly running up towards the South. Numerous cracks <0.2mm running from soffit of coupling beam up. Diagonal cracks <0.2mm running to re-entrant corners of service penetrations.	C	Epoxy Inject as per HCG Specification	
LG	L240	D.LG.10	South-West Shear Core - North Elevation, Looking North	02.04.11 - Numerous vertical cracks <0.2mm from soffit of coupling beam. Diagonal crack <0.2mm from re-entrant corner of shear wall to service penetration. (NB. Cracking to lesser extent than West elevation)	C	Epoxy Inject as per HCG Specification	
LG	L240	D.LG.11	South-West Shear Core - East Elevation, Looking West	02.04.11 - Extensive diagonal cracking to wall and coupling beam. Crack width <0.2mm predominantly running up towards the South (similar to West elevation). Diagonal cracks <0.2mm to service penetration re-entrant corner.	C	Epoxy Inject as per HCG Specification	
LG	L240	D.LG.12	South-West Shear Core - South Elevation, Looking North	02.04.11 - Numerous cracking <0.2mm throughout wall and coupling beam. Cracking to a lesser extent than West and East wall.	C	Epoxy Inject as per HCG Specification	

Refer to Table 4.4 and HCG Specification for repair details

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
LG	L244	D.LG.13	Stair Well	02.04.11 - Fine flexural cracks to underside of flight at stair tread throats <0.2mm.	C	Stair remedial issued separately	
LG	L224	D.LG.14	Stair Well	02.04.11 - Fine flexural cracks to underside of flight at stair tread throats <0.2mm.	C	Stair remedial issued separately	
LG	L240	D.LG.15 Follow up to entry D.L.G 9 and 12	Shear Core: South West	04.07.11 - Cracking mapped 4/7/11: Widespread 0.2/0.3mm cracking Work completed 10-15 lt of injection product	C	Epoxy Inject as per HCG Specification	
LG	L225	D.LG.17	Shear Core: South East	04.07.11 - Widespread 0.2/0.3mm cracking Work completed 10-15 lt of injection product	C	Epoxy Inject as per HCG Specification	
LG		D.LG.17	Shear Core- North West & North East	04.07.11 - Not yet open for inspection	C	Expose shear wall and epoxy Inject as per HCG specification	
Throughout		D.LG.18	Lift Shaft & Pit	04.07.11 - General: No significant damage identified. Some gib cracking (Light weight fire rated walls). All steel framing OK. Concrete work around the lift over-run OK	N		
LG	L242, 242A	D.LG.19	Floor slab	04.07.11 - Floor cracking. One crack up to 2mm. Adjacent CHCH Woman's. This is an infill floor. Structural drawing indicate a service void Soffit inspection recommended	F	Soffit inspection recommended	
LG	L211		Blockwork walls	22.11.12 - Cracking to top and second from top course of block work wall.	N	-	12.11.22_C racking to block wall block D LG, Rm L212

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
LG			Floor Slab	09.10.12 - Large (>2.0mm), straight crack in suspended slab through a number of rooms (Rm. L236, 235, 233, 232, 231, 229, 228). Strain hardening testing has been undertaken to reinforcement over crack indicate some strain hardening	C	Repair recommendations on case-by-case basis. To reinstate strain capacity will require replacing bars crossing cracks	12-10-09_Cracks to LG Rm232D
LG			Floor slab	09.10.12 - Shrinkage cracking throughout floor suspended slab. Shrinkage cracking observed in Rm. L232, L216	C	Repair recommendations on case-by-case basis.	12-10-09_Cracks to LG Rm232D
LG	L242C		south retaining wall	12.11.12 - Interface between Ground floor slab and south retaining wall investigated. No obvious movement between the two but no reinforcement crosses interface. Ref SR27 & SR28	N		12-11-13_Block D retaining wall….

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G399	D.G.1	North-West Shear Core - West Elevation	03.04.11 - 3 vertical cracks from coupling beam soffit <0.2mm 08.10.13 - Photos and crack maps provided by FCC show 0.3mm diagonal and horizontal cracking. The cracking has been sinse repaired by epoxy injection	C	Expose shear wall and epoxy Inject as per HCG specification	08.10.13 008-009
G	G399	D.G.2	North-West Shear Core - East Elevation	03.04.11 - Numerous vertical cracks from coupling beam soffit and re-entrant corner <0.2mm 08.10.13 - Photos and Crack Maps provided by FCC show 0.3-0.4mm diagonal cracking evenly distributed across face of shear wall. Cracking greater than 0.3mm has been epoxy injected.	C	Expose shear wall and epoxy Inject as per HCG specification	08.10.13 005-007
G	G399	D.G.3	North-West Shear Core - North, South Elevations	03.04.11 - No observations made due to sealed plasterboard ceiling and ceiling tiles unable to be lifted. 08.10.13 - Photos and Crack Maps provided by FCC show 0.3mm diagonal cracking to each face of the north-west corner of the shear wall core. Cracking greater than 0.3mm has been epoxy injected.	C	Expose shear wall and epoxy Inject as per HCG specification	08.10.13 001-004
G	G433	D.G.4	North-East Shear Core - North Elevation	03.04.11 - Few vertical cracks <0.2mm from soffit of coupling beam	C	Expose shear wall and epoxy Inject as per HCG specification	03.04.11 - 002
G	G433	D.G.5	North-East Shear Core - South Elevation	03.04.11 -2 vertical cracks from re-entrant corner < 0.2mm	C	Expose shear wall and epoxy Inject as per HCG specification	

Refer to Table 4.4 and HCG Specification for repair details

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G433	D.G.6	North-East Shear Core - East, West Elevation	03.04.11 -No observations made due to sealed plasterboard ceiling and ceiling tiles unable to be lifted.	C	Expose shear wall and epoxy Inject as per HCG specification	
G	G360	D.G.7	South-East Shear Core - South Elevation	03.04.11 -Few vertical cracks from re-entrant corners of coupling beam < 0.2mm	C	Expose shear wall and epoxy Inject as per HCG specification	
G	G309	D.G.8	South-East Shear Core - West Elevation	03.04.11 -Few vertical cracks from re-entrant corners of coupling beam < 0.2mm	C	Expose shear wall and epoxy Inject as per HCG specification	

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G357	D.G.9	South-East Shear Core - North, East Elevation	03.04.11 -No observations made due to sealed plasterboard ceiling and ceiling tiles unable to be lifted.	C	Expose shear wall and epoxy Inject as per HCG specification	
G	G294	D.G.10	South-West Shear Core - All Elevations	03.04.11 -No observations made due to no access and sealed ceilings	C	Expose shear wall and epoxy Inject as per HCG specification	
G	G387	D.G.11	Stair Well	03.04.11 -Fine flexural cracks to underside of flight at stair tread throats <0.2mm. Typical of stairwell down to Ground.	C	Stair remedial issued separately	
G	G351	D.G.12	Stair Well	03.04.11 -Fine flexural cracks to underside of flight at stair tread throats <0.2mm. Longitudinal crack along bottom edge of flight.	C	Stair remedial issued separately	
G	G331, 336, 337		Floor slab	12.09.12 - Cracking (~0.6mm max) in suspended slab adjacent seismic gap with Block C.	C	Repair complete	12-09-12_Block D ground floor strip out
G	G374, 375		Floor slab	12.09.12 - Cracking (~0.6mm max) in slab-on-grade in southwest corner.	C	Repair complete	12-09-12_Block D ground floor strip out
Throughout		D.G.13	Stair 6 & 7	16.09.11 - Fine soffit cracking identified Separation and sliding support of the mid-height landing to be detailed. Schematic issue to Project Manager	C	Stair remedial issued separately	

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
L1			General Comments	03.04.11 - Access to L1 was limited to due to operating theatres etc. 29.01.13 - Walk through completed with RCP and Fletchers no significant structural damage noted. Only limited structure exposed for viewing.			
L1			Shear cores	Limited structure available for review	Y	Expose shear wall and epoxy Inject as per HCG specification	
L1	1.410	D.1.1	Stair Well	03.04.11 - Fine flexural cracks to underside of flight at stair tread throats <0.2mm. Typical of stairwell down to Ground.	C	Stair remedial issued separately	
L1	1.306	D.1.2	Column/Slab Soffit	03.04.11 - No damage observed	N		
L1	1.384	D.1.3	Stair Well\	03.04.11 - Fine flexural cracks to underside of flight at stair tread throats <0.2mm. Longitudinal crack at bottom edge of flight <0.2mm	C	Stair remedial issued separately	

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
L2	2.358	D.2.1	Shear Wall/Slab Soffit	03.04.11 - End of shear wall. No damage observed.	C	Expose shear wall and epoxy Inject as per HCG specification	
L2	2.361	D.2.2	Column	03.04.11 - No observations due to sealed ceiling	N		
L2	2.424	D.2.3	Column/Slab Soffit	03.04.11 - No damage observed	N		
L2	2.363	D.2.4	Column/Slab Soffit	03.04.11 - No damage observed	N		
L2	2.364	D.2.5	Slab Soffit	03.04.11 - No damage observed	N		
L2	2.410	D.2.6	Shear Wall	03.04.11 - Wall lining down to floor level to be removed to allow for further inspection	C	Expose shear wall and epoxy Inject as per HCG specification	
L2	2.342	D.2.7	Column	03.04.11 - Poor compaction of concrete at Corbel-column joint. No earthquake damage observed.	N	Not Earthquake Damage	03.04.11 - 008
L2	2.425	D.2.8	Stair Well	03.04.11 - Fine flexural cracks to underside of flight at stair tread throats <0.2mm. Typical of stairwell down to Ground.	C	Stair remedial issued separately	
L2	2.401	D.2.9	Stair Well	03.04.11 - Fine flexural cracks to underside of flight at stair tread throats <0.2mm. Longitudinal cracks at bottom edge of flight < 0.2mm	C	Stair remedial issued separately	22.01.13 - P1220019 P12200209
L2	2.400A	Courtyard	Precast spandrel panel	22.01.13 - Corner section of panel on south side of courtyard has spalled (was loose and removed by Golemans)	C	Repair spalled concrete as per HCG specification	
L2	2.411	D.2.10	Floor	03.04.11 - 04/01/12 - Tear in vinyl floor covering and discolouration of vinyl indicate that there is a crack in the slab that extends from the corner of the shear core. The underside of the slab was not able to be viewed due to the fixed gib ceiling.	F	lift vinyl to inspect floor	

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
L3	3.393	D.3.1	Seismic Joint	03.04.11 - Major damage to seismic joint and plywood ramp. Refer to photo	C	Repair to be confirmed	
L3	3.384	D.3.2	Shear Wall	03.04.11 - Wall lining down to floor level to be removed to allow for further inspection	C	Expose shear wall and epoxy Inject as per HCG specification	
L3	3.400	D.3.3	Stair Well	03.04.11 - Fine flexural cracks to underside of flight at stair tread throats <0.2mm. Typical of stairwell down to Ground.	C	Stair remedial issued separately	03.04.11 - 009
L3	3.375	D.3.4	Stair Well	03.04.11 - Fine flexural cracks to underside of flight at stair tread throats <0.2mm.	C	Stair remedial issued separately	
L3	3.312 & 3.378	D.3.5	Stair Well	29.09.11 - Fine flexural cracks to underside of flight at stair tread throats <0.2mm.	C	Stair remedial issued separately	

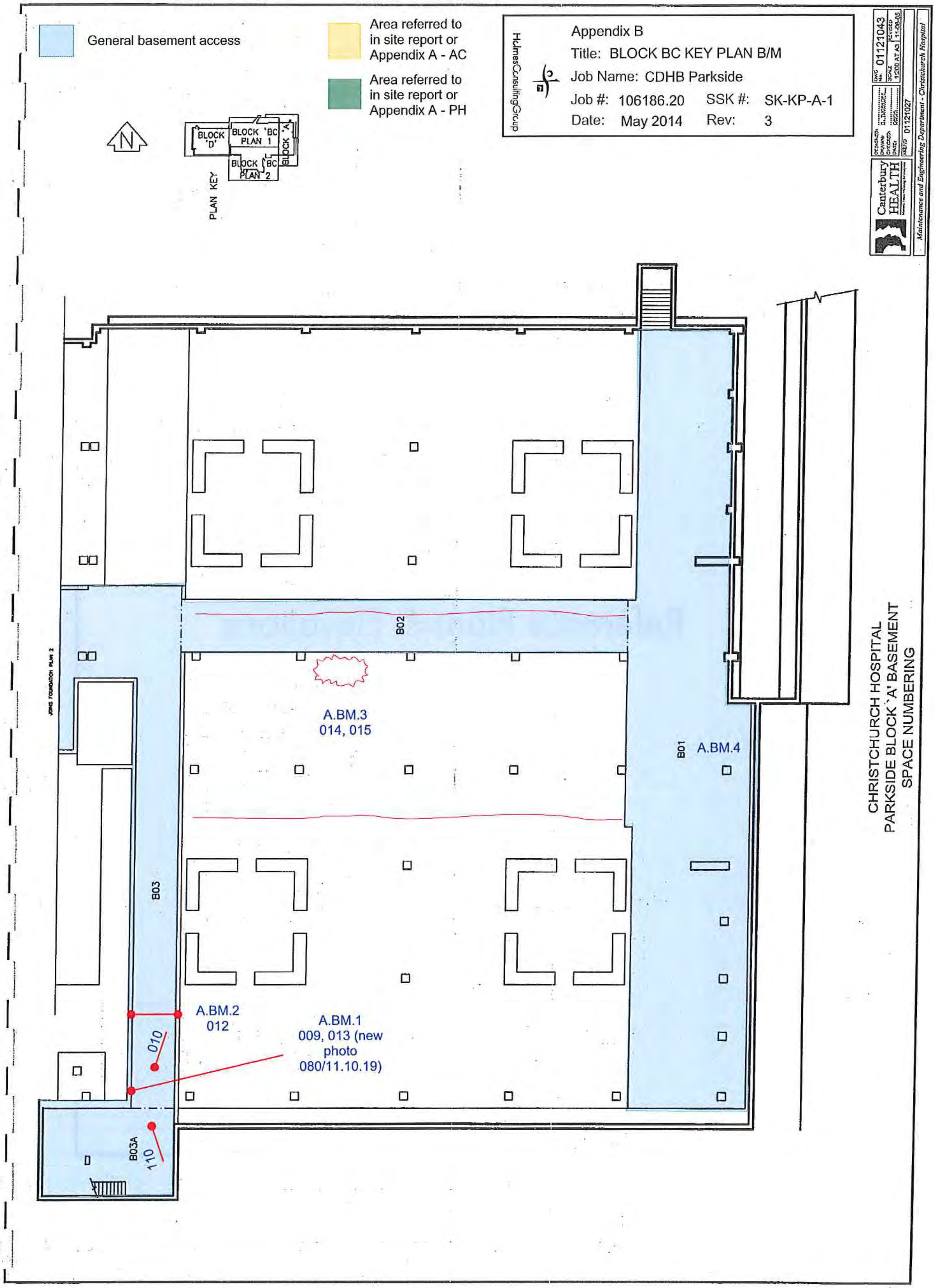
Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
L4	4.18	D.4.1	General Observation	05.04.11 - • No evidence of cracking in floor slab. (two exceptions below) • No evidence of cracking in perimeter up-stand beam, with one exception entered below. • No evidence of damage in steel roof structure other.	N		
L4	4.18	D.4.2	Up-Stand Beam	05.04.11 - Crack in top of beam 0.2-0.4mm.	C	Epoxy Inject as per HCG Specification	05.04.11 - 017
L4	4.18	D.4.3	Columns	05.04.11 - Horizontal cracking at the tops of columns, 0.1mm	N		05.04.11 - 018
L4	4.18	D.4.4	Beam/Column Joint	05.04.11 - Diagonal crack on the eastern side of the beam/column joint, 0.1mm	N		05.04.11 - 019
L4	4.18	D.4.5	Beam/Column Joint	05.04.11 - Diagonal crack on the south side of the beam/column joint, 0.1mm	N		
L4	4.18	D.4.6	Beam	05.04.11 - Widespread cracks in beam supporting lift motor room. 0.1mm	N		05.04.11 - 020
L4	4.18	D.4.7	Beam/Column Joint	05.04.11 - Diagonal crack on the north side of the beam/column joint, 0.1mm	N		05.04.11 - 021
L4	4.18	northern side of 4.18	Up-Stand Beam	28.02.13 - Flexural and diagonal cracks to soffit up-stand beam - up to 0.3 mm wide (refer site report #43) 27.02.15 Repair photos supplied by RCP	C		
L4	4.18	east side	Up-Stand Beam	04.12.12 - Flexural and diagonal cracks to up-stand beam - up to 0.3 mm wide	N		4.12.12 - 2012-12-04 11.50.44 - 2012-12-04 11.53.05

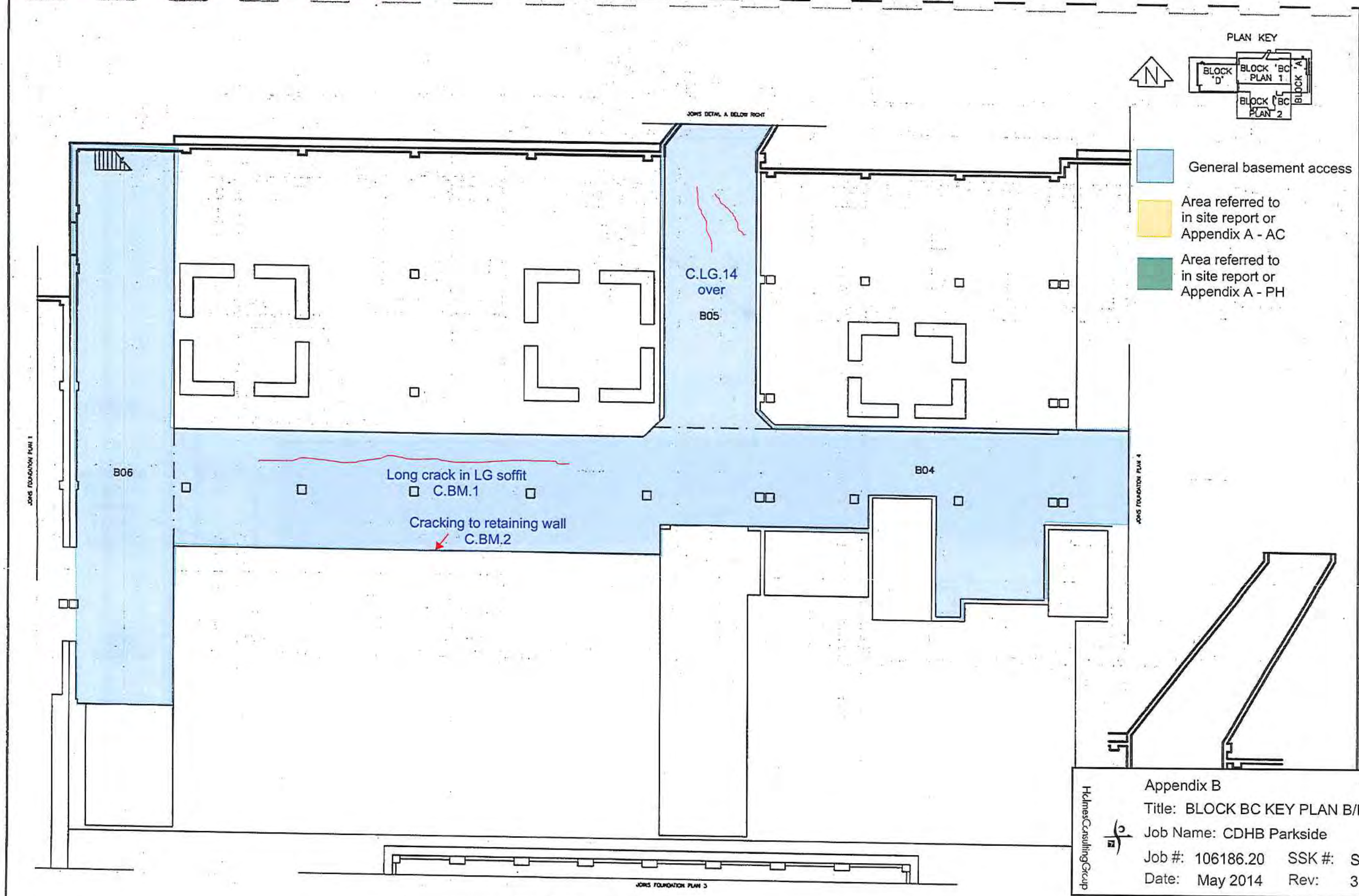
Refer to Table 4.4 and HCG Specification for repair details

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
L4	4.18	north end by door to 4.27	Up-Stand Beam	04.12.12 - Flexural cracks to up-stand beam - up to 0.6 mm wide 27.02.15 Repair photos supplied by RCP	C	Epoxy Inject as per HCG Specification	4.12.12 - IMG_7013
L4	4.18	south-east corner	slab	04.12.12 - Diagonal crack across corner of slab - up to 0.3 mm wide	N		
L4	4.18	south-west corner	slab	04.12.12 - Diagonal crack across corner of slab - up to 0.3 mm wide	N		
L4	4.19	west wall	Precast panels	4.12.12 - hairline cracks noted to precast panels, no movement noted to panel connections	N		4.12.12 - IMG_7014 - 7016
L4	4.26	D.4.8	Concrete Landing	05.04.11 - Spalling concrete from edge of cantilever landing	C	Epoxy inject and repair spalling as per HCG specification	05.04.11 - 022
L4	4.17	D.4.9	Stair Well	05.04.11 - Fine flexural cracks to underside of flight at stair tread throats <0.2mm.	C	Stair remedial issued separately	
L4	4.18	South side of courtyard	Precast panel	22.01.13 - Panel above Room 3.37 appears to have rotated. Further investigation required. Panel also noted in Goleman report.	F		13-01-22 - P1220021-23

APPENDIX B

Reference Plans & Elevations
Sk-KP-A-1 to 7
Sk-KP-BC-1 to 7
Sk-KP-D-1 to 7


General basement access
Area referred to in site report or Appendix A - AC
Area referred to in site report or Appendix A - PH
HolmesConsultingGroup
Appendix B
Title: BLOCK BC KEY PLAN B/M
Job Name: CDHB Parkside
Job #: 106186.20
SSK #: SK-KP-A-1
Date: May 2014
Rev: 3
01121043
Canterbury HEALTH
Maintenance and Engineering Department - Christchurch Hospital
BLOCK 'D'
BLOCK 'BC' PLAN 1
BLOCK 'BC' PLAN 2
BLOCK 'A'
PLAN KEY
N
B02
A.BM.3
014, 015
B01
A.BM.4
B03
A.BM.2
012
A.BM.1
009, 013 (new photo 080/11.10.19)
010
B03A
110
CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK 'A' BASEMENT
SPACE NUMBERING



CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK 'BC' BASEMENT PLAN 1
SPACE NUMBERING

HolmesConsultingGroup

Appendix B

Title: BLOCK BC KEY PLAN B/M

Job Name: CDHB Parkside

Job #: 106186.20 SSK #: SK-KP-BC-1

Date: May 2014 Rev: 3

Canterbury HEALTH

DWG No. 01121044

SCALE 1:200 AT A3 REVISED 11-08-05

01121025

Maintenance and Engineering Department - Christchurch Hospital

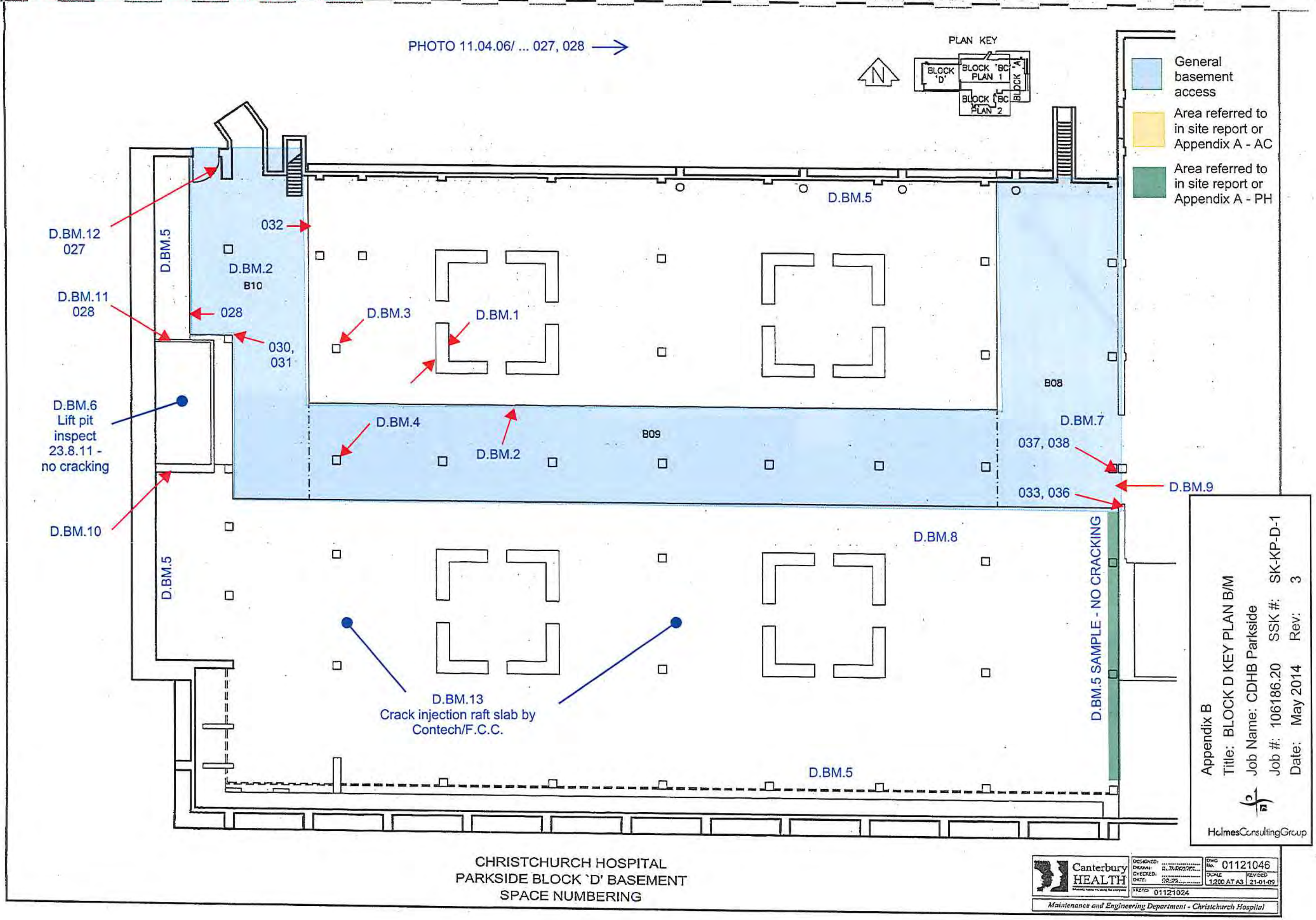
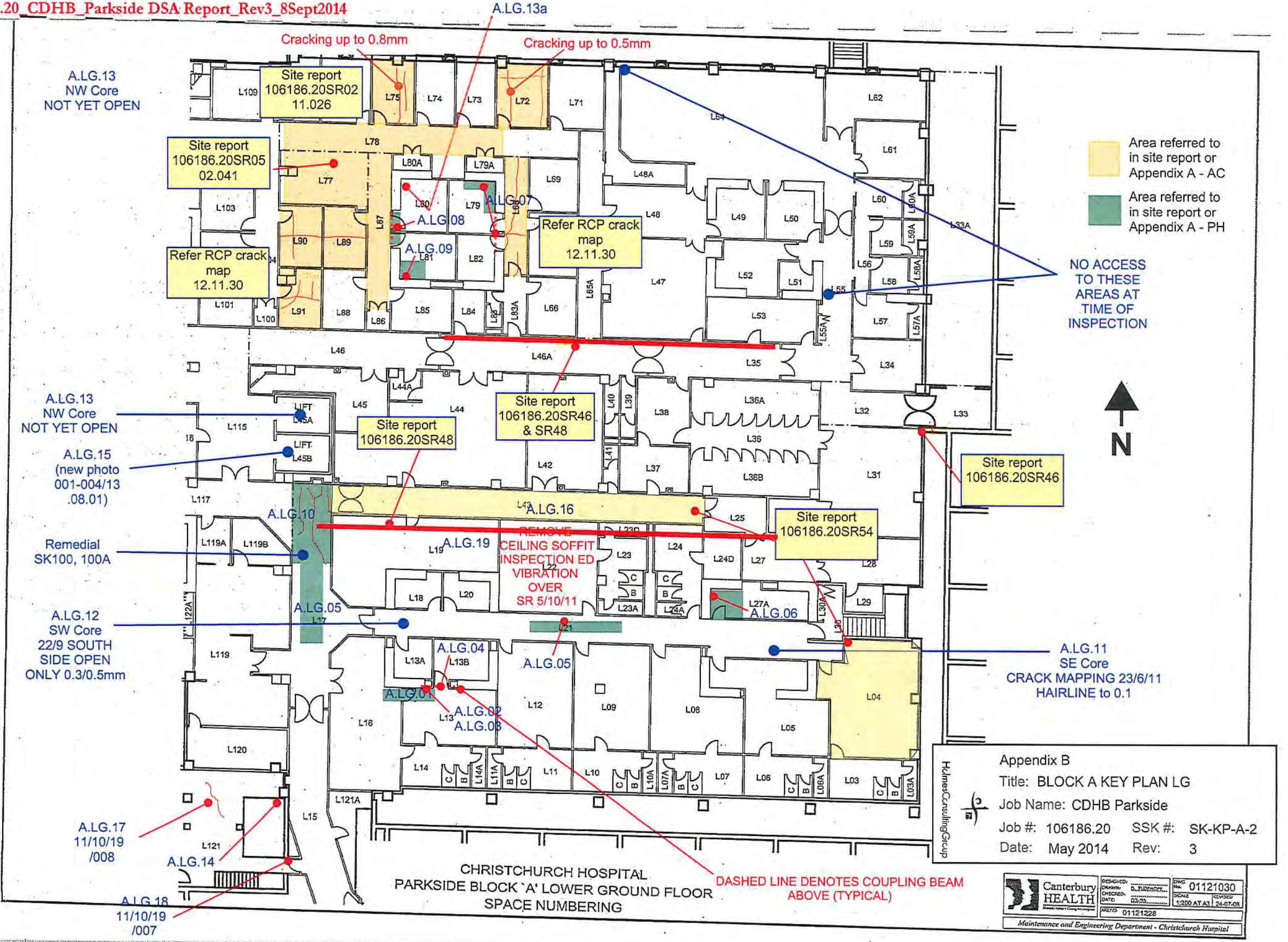


PHOTO 11.04.06/ ... 027, 028
PLAN KEY
BLOCK 'D'
BLOCK 'BC' PLAN 1
BLOCK 'BC' PLAN 2
BLOCK 'A'
N
General basement access
Area referred to in site report or Appendix A - AC
Area referred to in site report or Appendix A - PH
D.BM.12
027
D.BM.11
028
D.BM.6
Lift pit
inspect
23.8.11 -
no cracking
D.BM.10
D.BM.5
032
D.BM.2
B10
028
030, 031
D.BM.3
D.BM.1
D.BM.4
D.BM.2
B09
B08
D.BM.7
037, 038
D.BM.9
033, 036
D.BM.8
D.BM.5 SAMPLE - NO CRACKING
D.BM.13
Crack injection raft slab by
Contech/F.C.C.
D.BM.5
Appendix B
Title: BLOCK D KEY PLAN B/M
Job Name: CDHB Parkside
Job #: 106186.20 SSK #: SK-KP-D-1
Date: May 2014 Rev: 3
HolmesConsultingGroup
CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK `D' BASEMENT
SPACE NUMBERING
Canterbury HEALTH
01121046
1:200 AT A3
21-01-09
01121024
Maintenance and Engineering Department - Christchurch Hospital



Appendix B

Title: BLOCK A KEY PLAN LG

Job Name: CDHB Parkside

Job #: 106186.20 SSK #: SK-KP-A-2

Date: May 2014 Rev: 3

HolmesConsultingGroup

Canterbury HEALTH

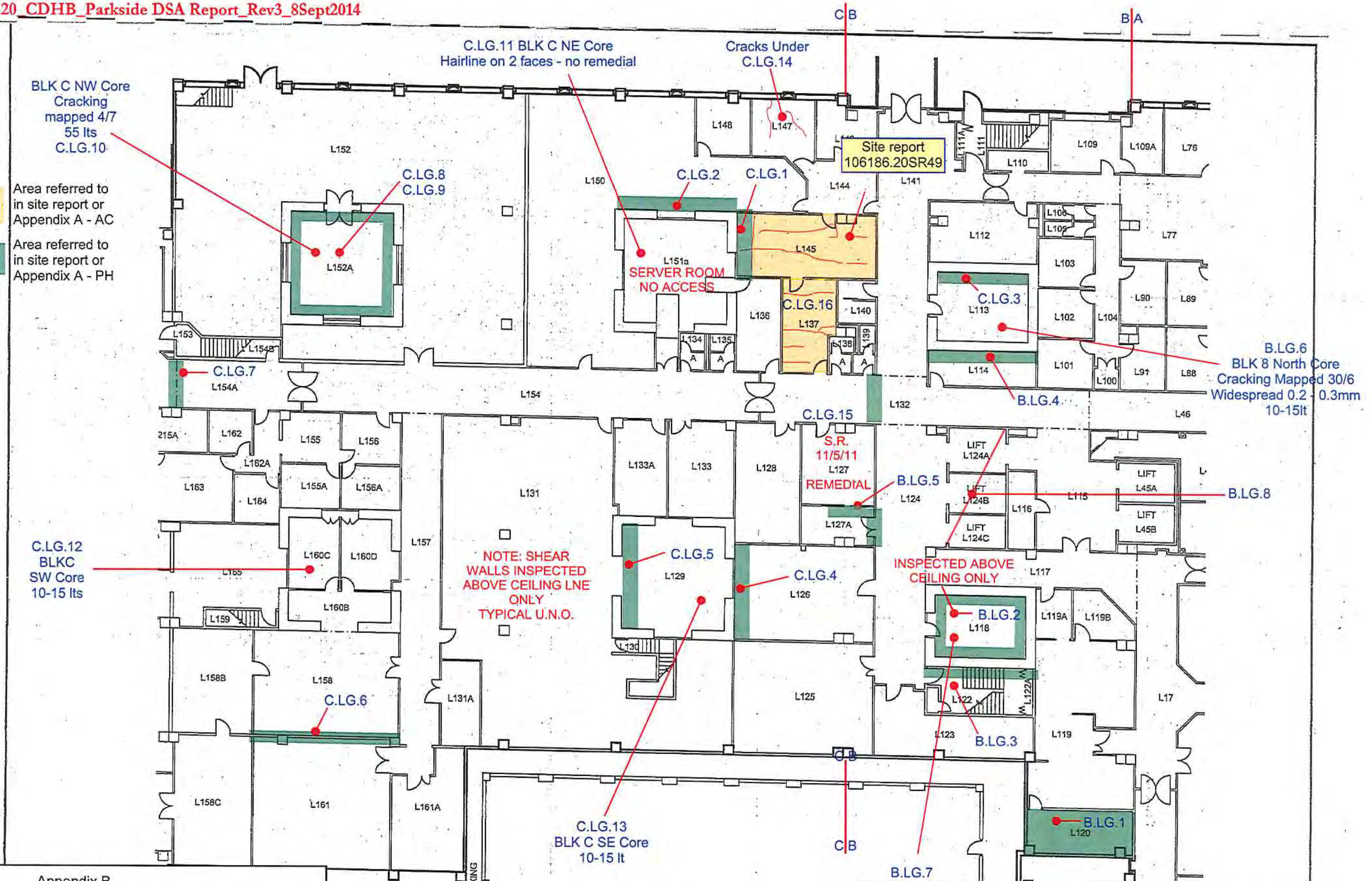
DWG No. 01121030

SCALE 1:200 AT A3

REVISED 24-07-08

01121228

Maintenance and Engineering Department - Christchurch Hospital



Area referred to in site report or Appendix A - AC

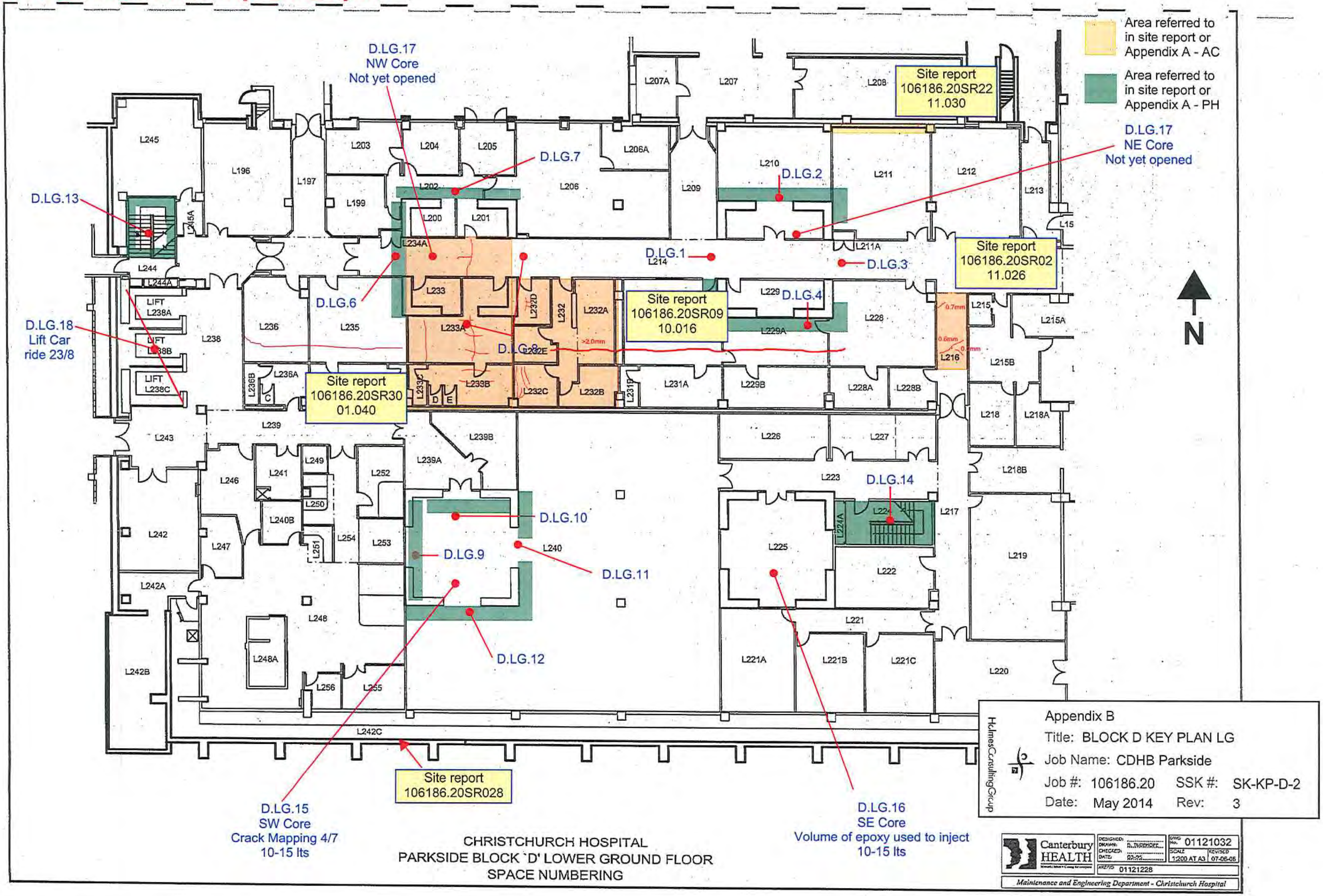
Area referred to in site report or Appendix A - PH

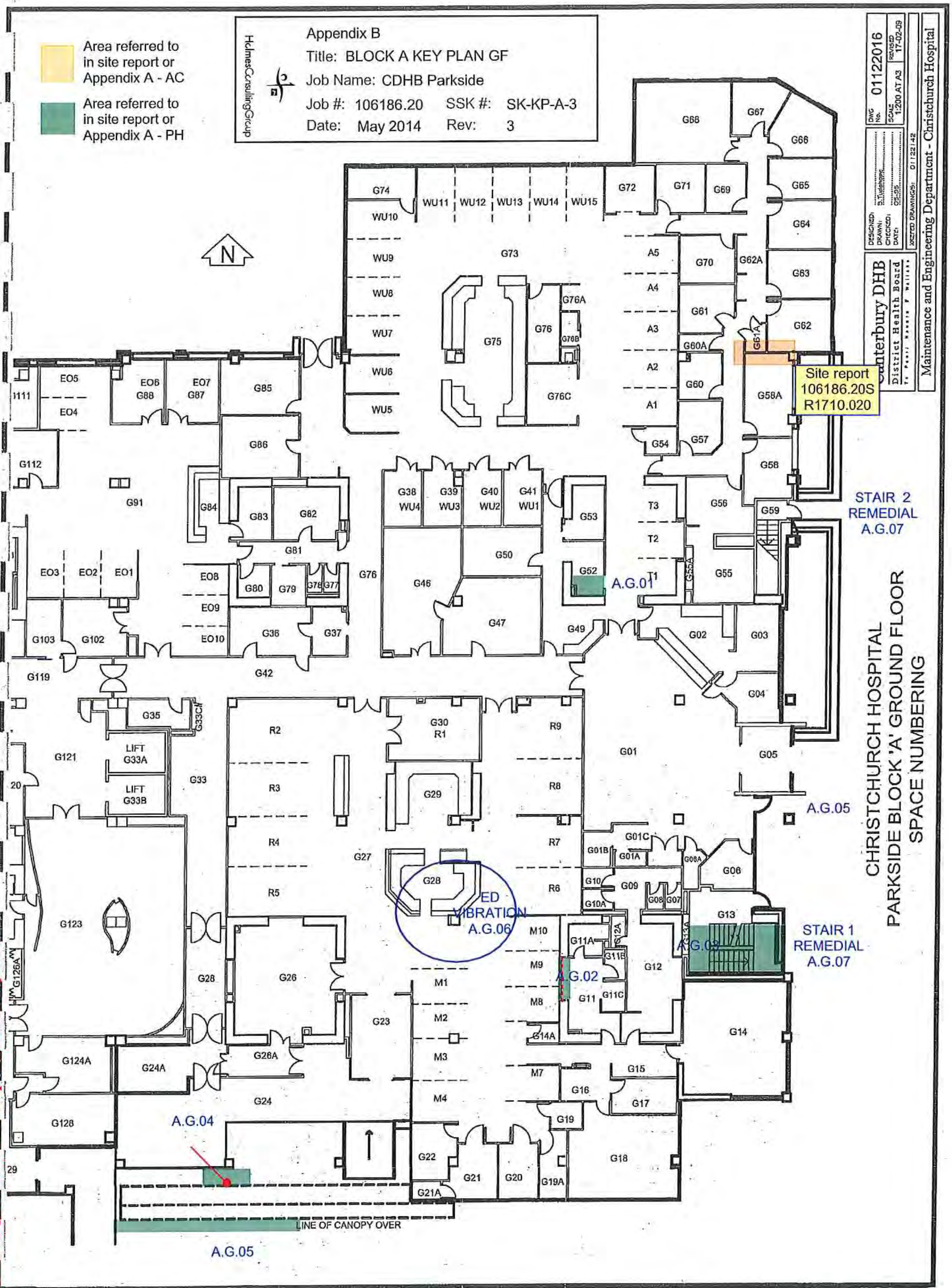
CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK 'B-C' LOWER GROUND FLOOR
SPACE NUMBERING

Canterbury HEALTH
DWG No. 01121031
SCALE 1:200 AT A3
REVISED 24-07-08
01121228
Maintenance and Engineering Department - Christchurch Hospital

HolmesConsultingGroup

Appendix B
Title: BLOCK BC KEY PLAN LG
Job Name: CDHB Parkside
Job #: 106186.20 SSK #: SK-KP-BC-2
Date: May 2014 Rev: 3


Area referred to in site report or Appendix A - AC
Area referred to in site report or Appendix A - PH
D.LG.17 NW Core Not yet opened
D.LG.17 NE Core Not yet opened
Site report 106186.20SR22 11.030
Site report 106186.20SR02 11.026
Site report 106186.20SR09 10.016
Site report 106186.20SR30 01.040
Site report 106186.20SR028
D.LG.1
D.LG.2
D.LG.3
D.LG.4
D.LG.6
D.LG.7
D.LG.8
D.LG.9
D.LG.10
D.LG.11
D.LG.12
D.LG.13
D.LG.14
D.LG.15 SW Core Crack Mapping 4/7 10-15 lts
D.LG.16 SE Core Volume of epoxy used to inject 10-15 lts
D.LG.18 Lift Car ride 23/8
N
CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK 'D' LOWER GROUND FLOOR
SPACE NUMBERING
Appendix B
Title: BLOCK D KEY PLAN LG
Job Name: CDHB Parkside
Job #: 106186.20
SSK #: SK-KP-D-2
Date: May 2014
Rev: 3
HolmesConsultingGroup
Canterbury HEALTH
01121032
1:200 AT A3
01121228
Maintenance and Engineering Department - Christchurch Hospital



Area referred to in site report or Appendix A - AC
Area referred to in site report or Appendix A - PH
HolmesConsultingGroup
Appendix B
Title: BLOCK A KEY PLAN GF
Job Name: CDHB Parkside
Job #: 106186.20
SSK #: SK-KP-A-3
Date: May 2014
Rev: 3
Site report
106186.20S
R1710.020
STAIR 2
REMEDIAL
A.G.07
STAIR 1
REMEDIAL
A.G.07
A.G.01
A.G.02
A.G.03
A.G.04
A.G.05
ED
VIBRATION
A.G.06
LINE OF CANOPY OVER
CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK 'A' GROUND FLOOR
SPACE NUMBERING
Canterbury DHB
District Health Board
Maintenance and Engineering Department - Christchurch Hospital
01122016

CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK 'B-C' GROUND FLOOR
SPACE NUMBERING

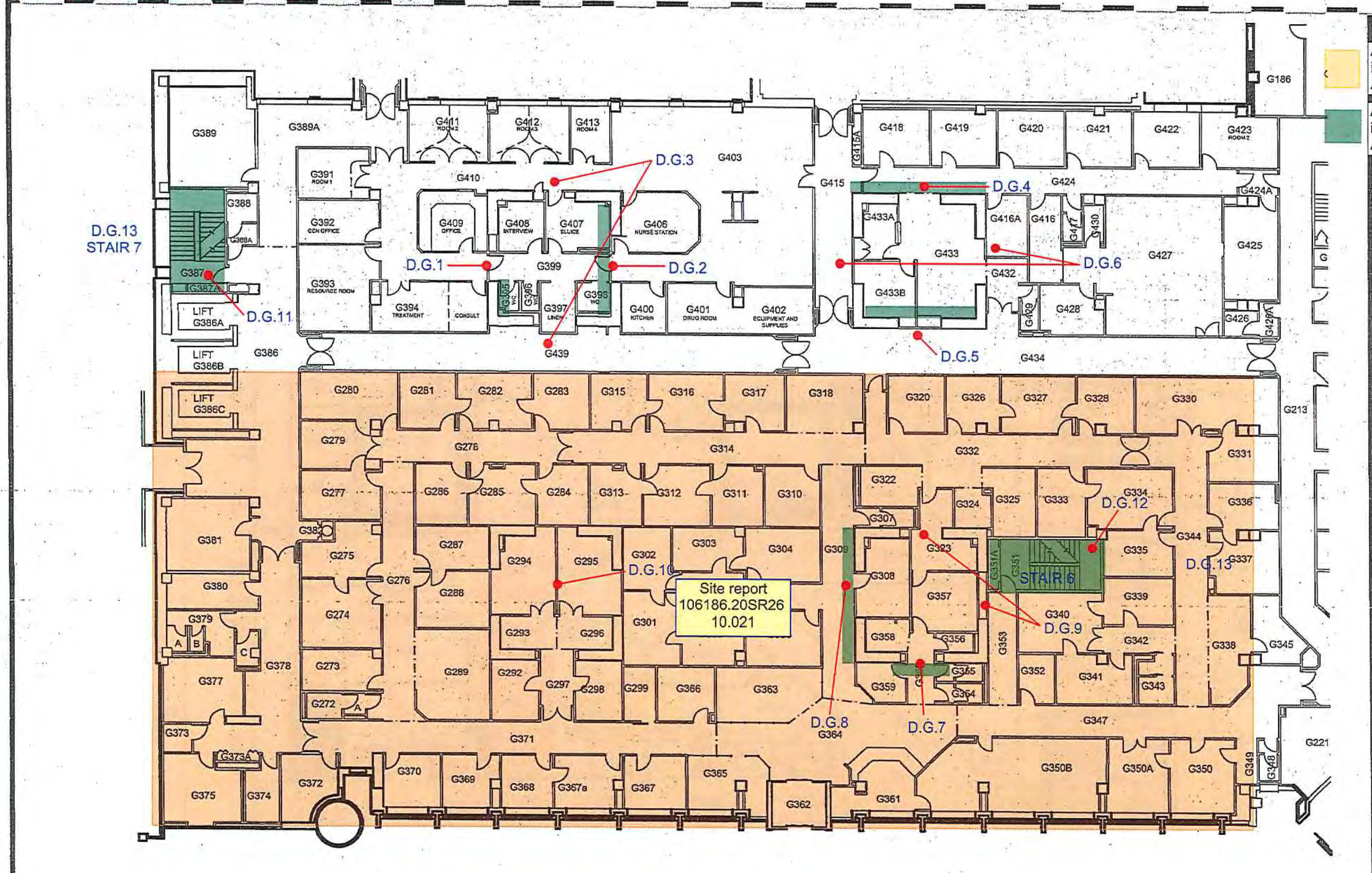
Canterbury DHB
District Health Board

DWG No. 01122017
SCALE 1:200 AT A3
REVISED 15-12-08
XREFED DRAWINGS: 01122142

Maintenance and Engineering Department - Christchurch Hospital

Appendix B
Title: BLOCK BC KEY PLAN GF
Job Name: CDHB Parkside
Job #: 106186.20 SSK #: SK-KP-BC-3
Date: May 2014 Rev: 3

HolmesConsultingGroup



Area referred to in site report or Appendix A - AC

Area referred to in site report or Appendix A - PH

Appendix B

Title: BLOCK D KEY PLAN GL

Job Name: CDHB Parkside

Job #: 106186.20 SSK #: SK-KP-D-3

Date: May 2014 Rev: 3

HolmesConsultingGroup

CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK 'D GROUND FLOOR
SPACE NUMBERING

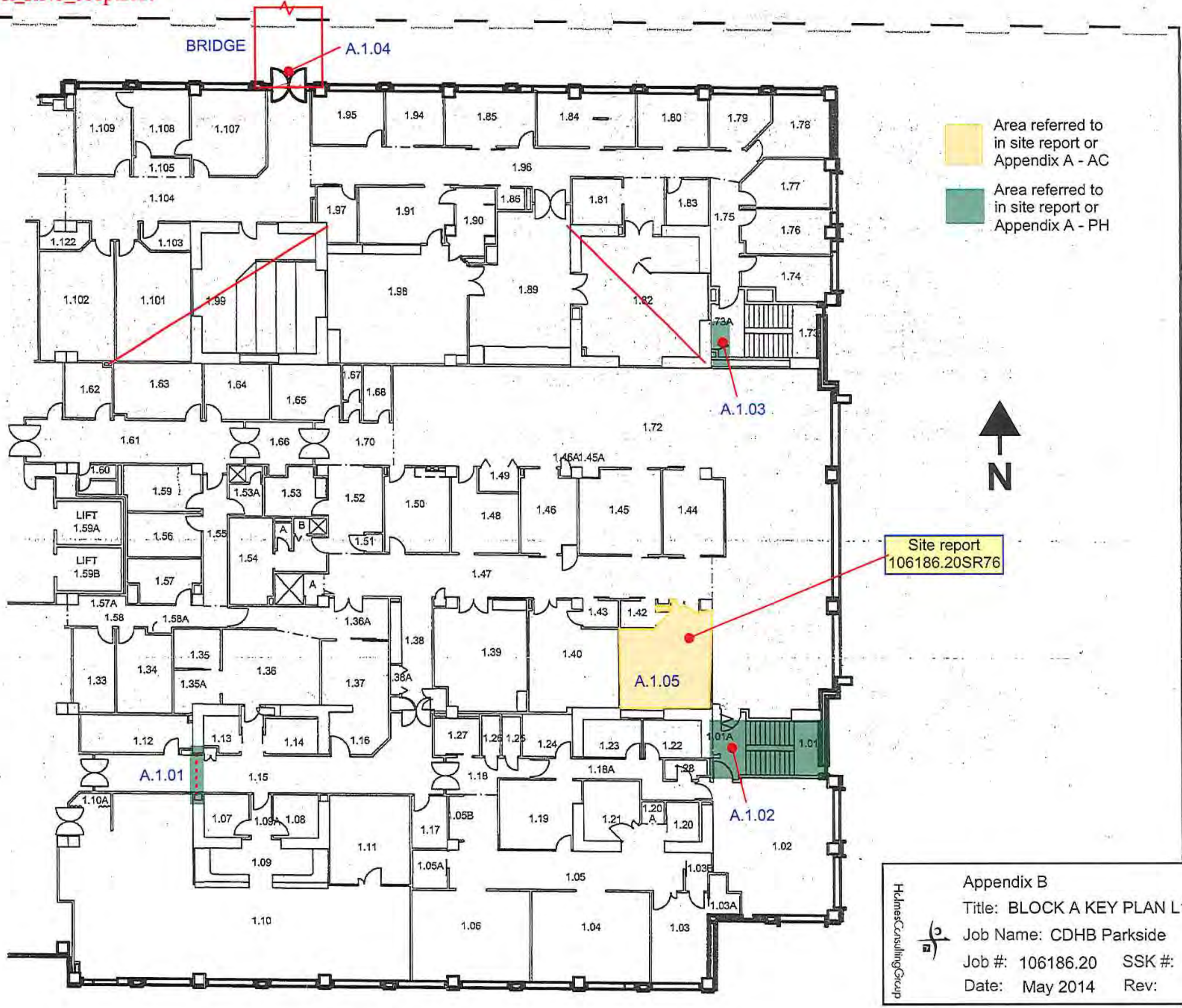
Canterbury DHB
District Health Board

DESIGNED:
DRAWN: J.Dendle
CHECKED:
DATE: 05-96

DWG No. 01122018
SCALE 1:200 AT A3
REVISED 26-08-09

XREFED DRAWING/S: 01122142

Maintenance and Engineering Department - Christchurch Hospital



CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK 'A' FIRST FLOOR
SPACE NUMBERING

Appendix B

Title: BLOCK A KEY PLAN L1

Job Name: CDHB Parkside

Job #: 106186.20 SSK #: SK-KP-A-4

Date: May 2014 Rev: 3

HolmesConsultingGroup

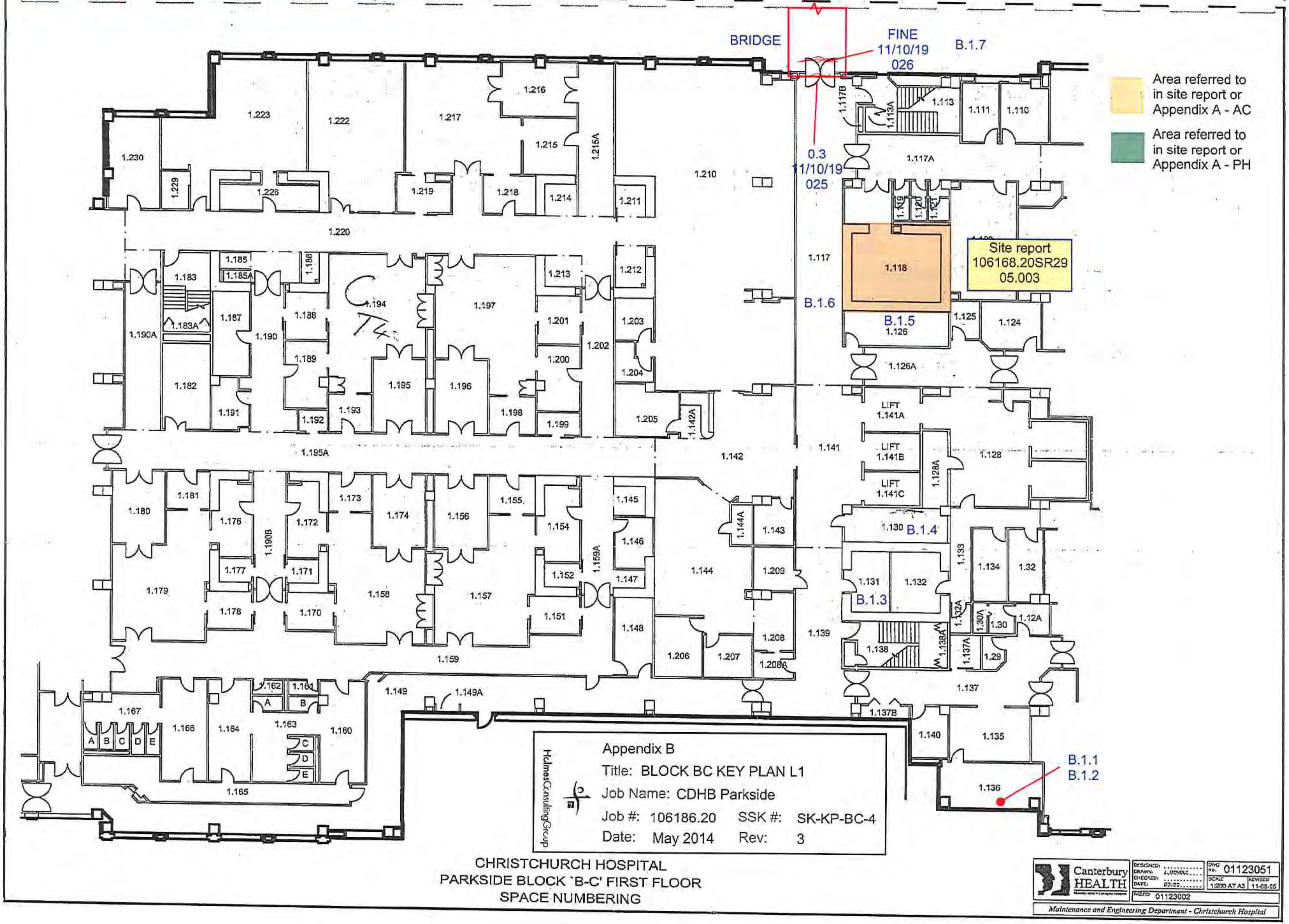
Canterbury HEALTH

01123050

1:200 AT A3 11-08-05

01123001

Maintenance and Engineering Department - Christchurch Hospital


BRIDGE
FINE
11/10/19
026
0.3
11/10/19
025
B.1.7
B.1.6
B.1.5
B.1.4
B.1.3
B.1.1
B.1.2
Site report
106168.20SR29
05.003
Area referred to in site report or Appendix A - AC
Area referred to in site report or Appendix A - PH
Appendix B
Title: BLOCK BC KEY PLAN L1
Job Name: CDHB Parkside
Job #: 106186.20 SSK #: SK-KP-BC-4
Date: May 2014 Rev: 3
HolmesConsultingGroup
CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK 'B-C' FIRST FLOOR
SPACE NUMBERING
Canterbury HEALTH
01123051
1:200 AT A3
01123002
Maintenance and Engineering Department - Christchurch Hospital

Area referred to in site report or Appendix A - AC

Area referred to in site report or Appendix A - PH

CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK 'D' FIRST FLOOR
SPACE NUMBERING

Appendix B

Title: BLOCK D KEY PLAN L1

Job Name: CDHB Parkside

Job #: 106186.20 SSK #: SK-KP-D-4

Date: May 2014 Rev: 3

HolmesConsultingGroup

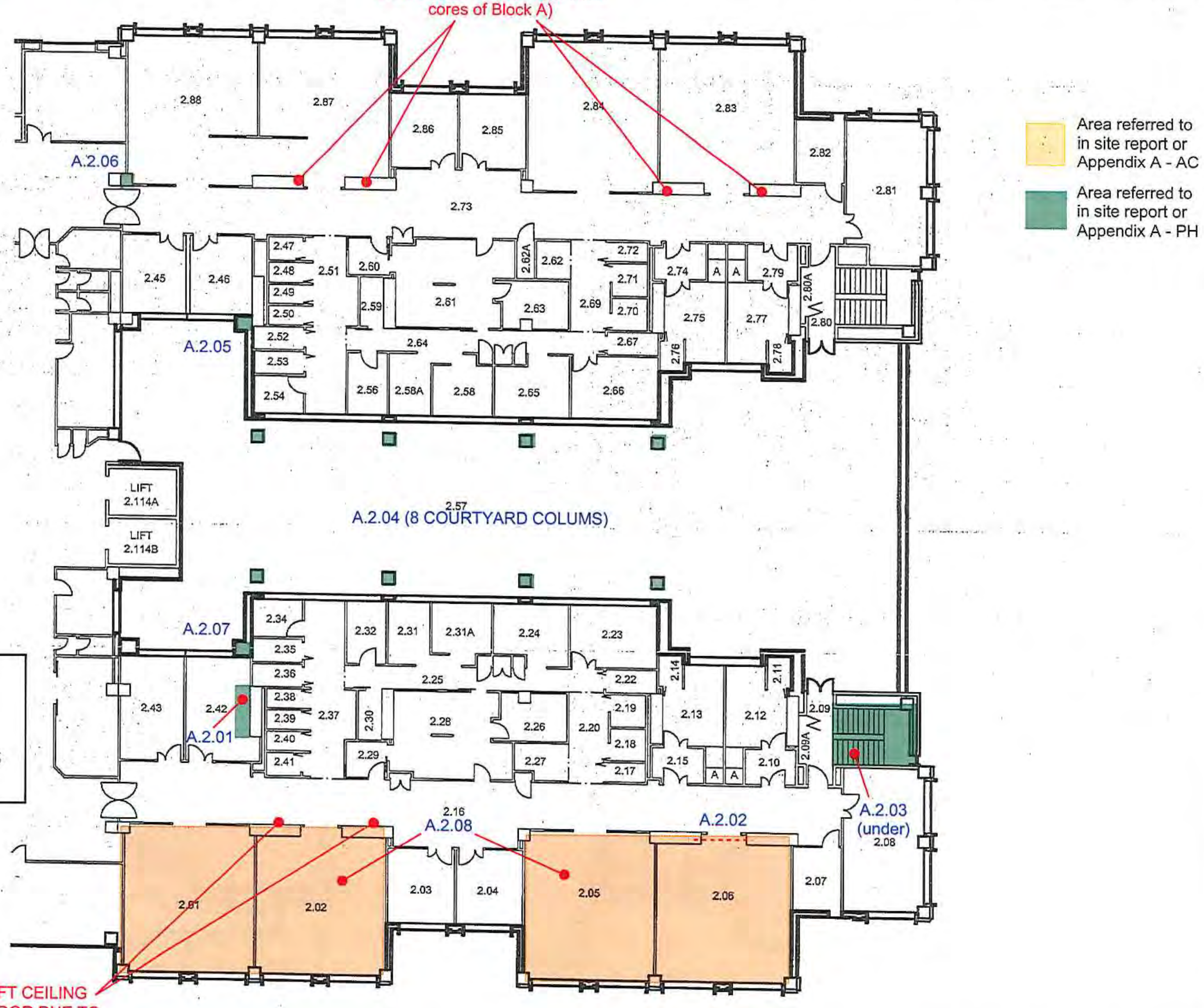
Canterbury DHB
District Health Board

DWG No. 01123052

SCALE 1:200 AT A3 REVISED 07-06-06

XREFED DRAWINGS: 0112303C

Maintenance and Engineering Department - Christchurch Hospital

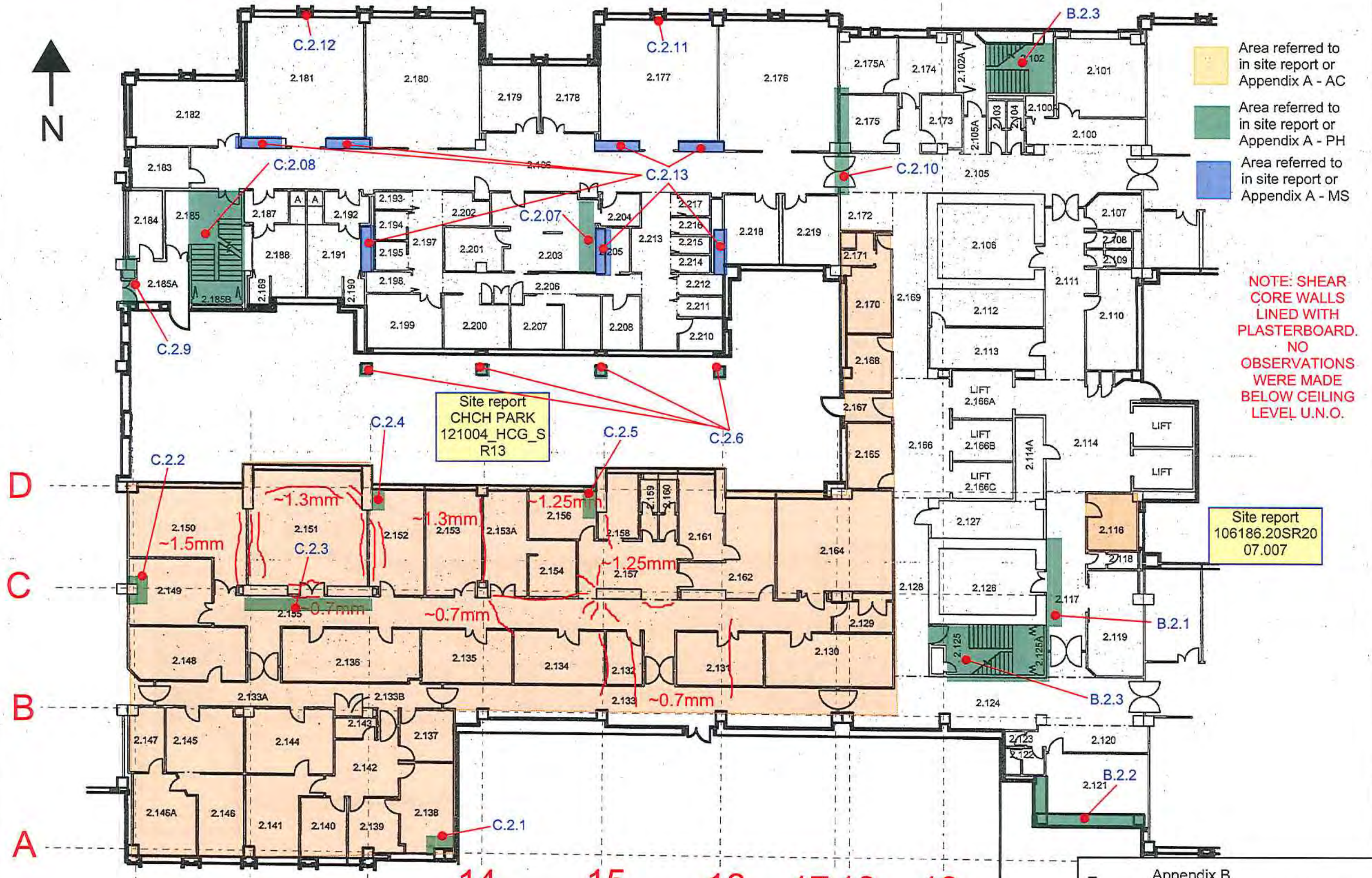


CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK 'A' SECOND FLOOR
SPACE NUMBERING

Canterbury HEALTH
DWG No. 01124013
SCALE 1:200 AT A3
REVISED 11-08-05
01124001
Maintenance and Engineering Department - Christchurch Hospital

HolmesConsultingGroup

Appendix B
Title: BLOCK A KEY PLAN L2
Job Name: CDHB Parkside
Job #: 106186.20 SSK #: SK-KP-A-5
Date: May 2014 Rev: 3



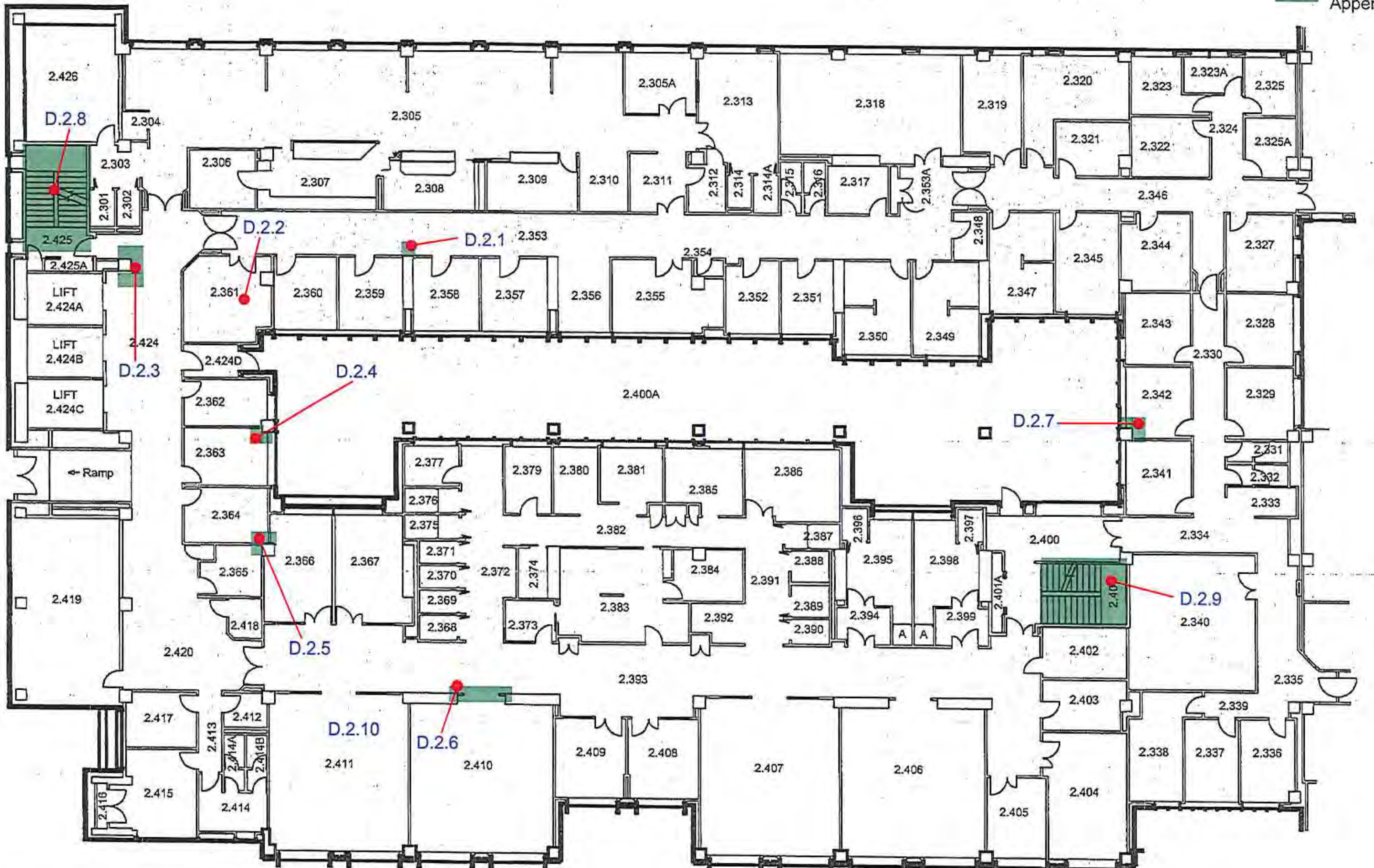
CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK 'B-C' SECOND FLOOR
SPACE NUMBERING

HolmesConsultingGroup

Appendix B
Title: BLOCK BC KEY PLAN L2
Job Name: CDHB Parkside
Job #: 106186.20 SSK #: SK-KP-BC-5
Date: May 2014 Rev: 3

Area referred to in site report or Appendix A - AC

Area referred to in site report or Appendix A - PH



Appendix B

Title: BLOCK D KEY PLAN L2

Job Name: CDHB Parkside

Job #: 106186.20 SSK #: SK-KP-D-5

Date: May 2014 Rev: 3

HolmesConsultingGroup

CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK 'D' SECOND FLOOR
SPACE NUMBERING

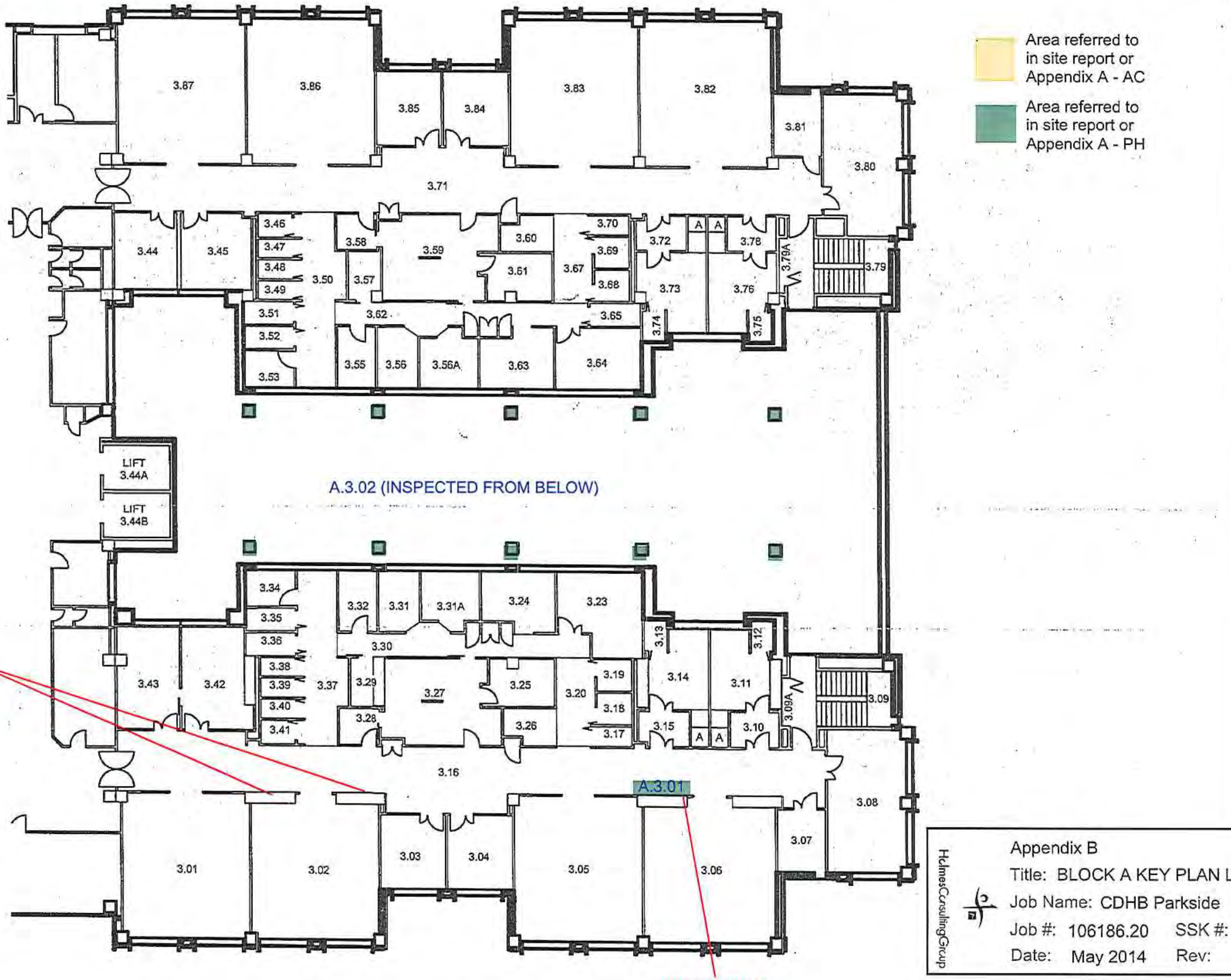
Canterbury HEALTH

DWG No. 01124015

SCALE 1:200 AT A3 REVISED 05-07-10

01124009

Maintenance and Engineering Department - Christchurch Hospital

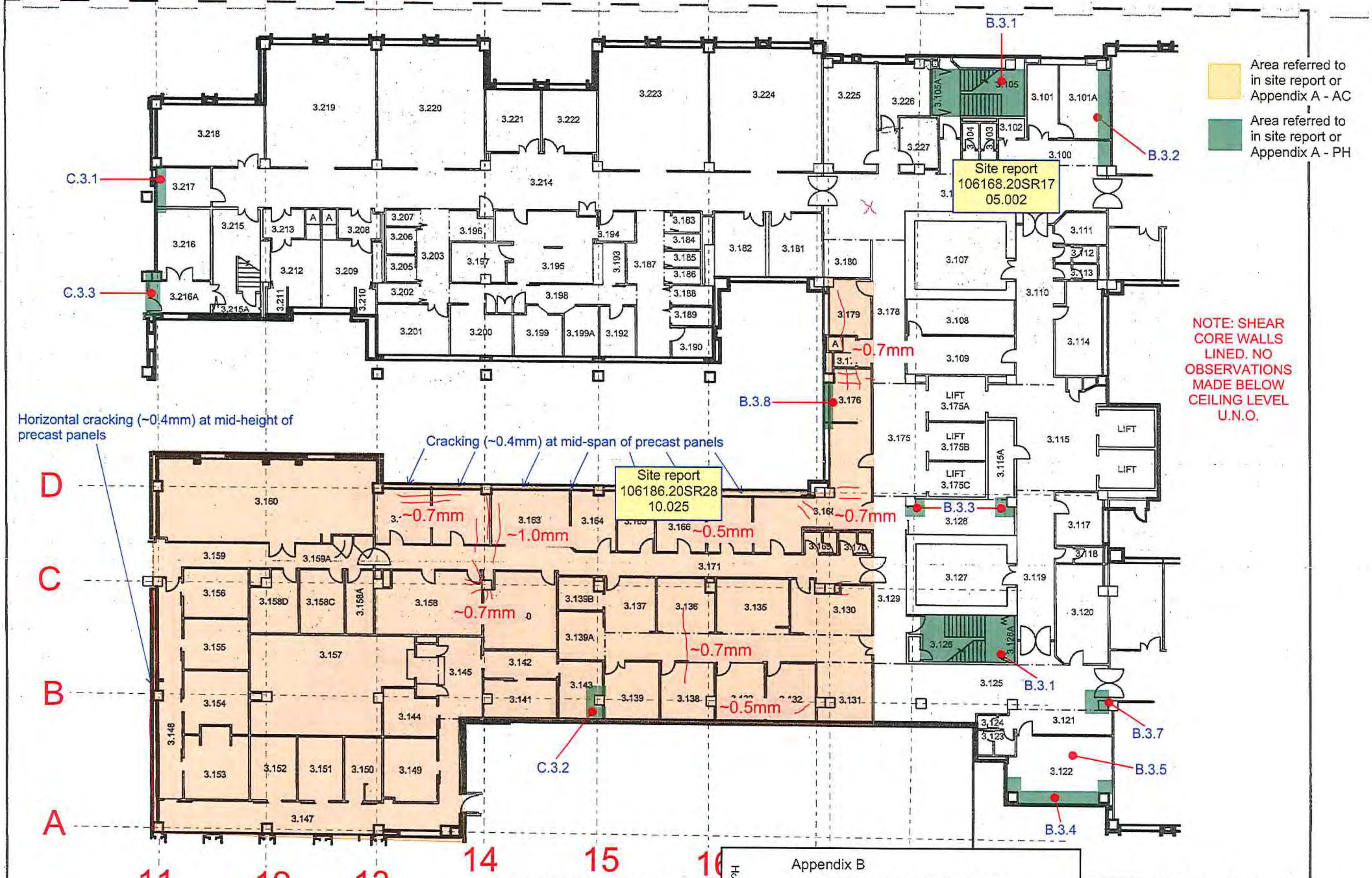


CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK 'A' THIRD FLOOR
SPACE NUMBERING

HolmesConsultingGroup

Appendix B
Title: BLOCK A KEY PLAN L3
Job Name: CDHB Parkside
Job #: 106186.20 SSK #: SK-KP-A-6
Date: May 2014 Rev: 3

Canterbury HEALTH
DWG No. 01125008
SCALE 1:200 AT A3
REVISED 11-08-05
01125001
Maintenance and Engineering Department - Christchurch Hospital



CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK 'B-C' THIRD FLOOR
SPACE NUMBERING

Appendix B

Title: BLOCK BC KEY PLAN L3

Job Name: CDHB Parkside

Job #: 106186.20 SSK #: SK-KP-BC-6

Date: May 2014 Rev: 3

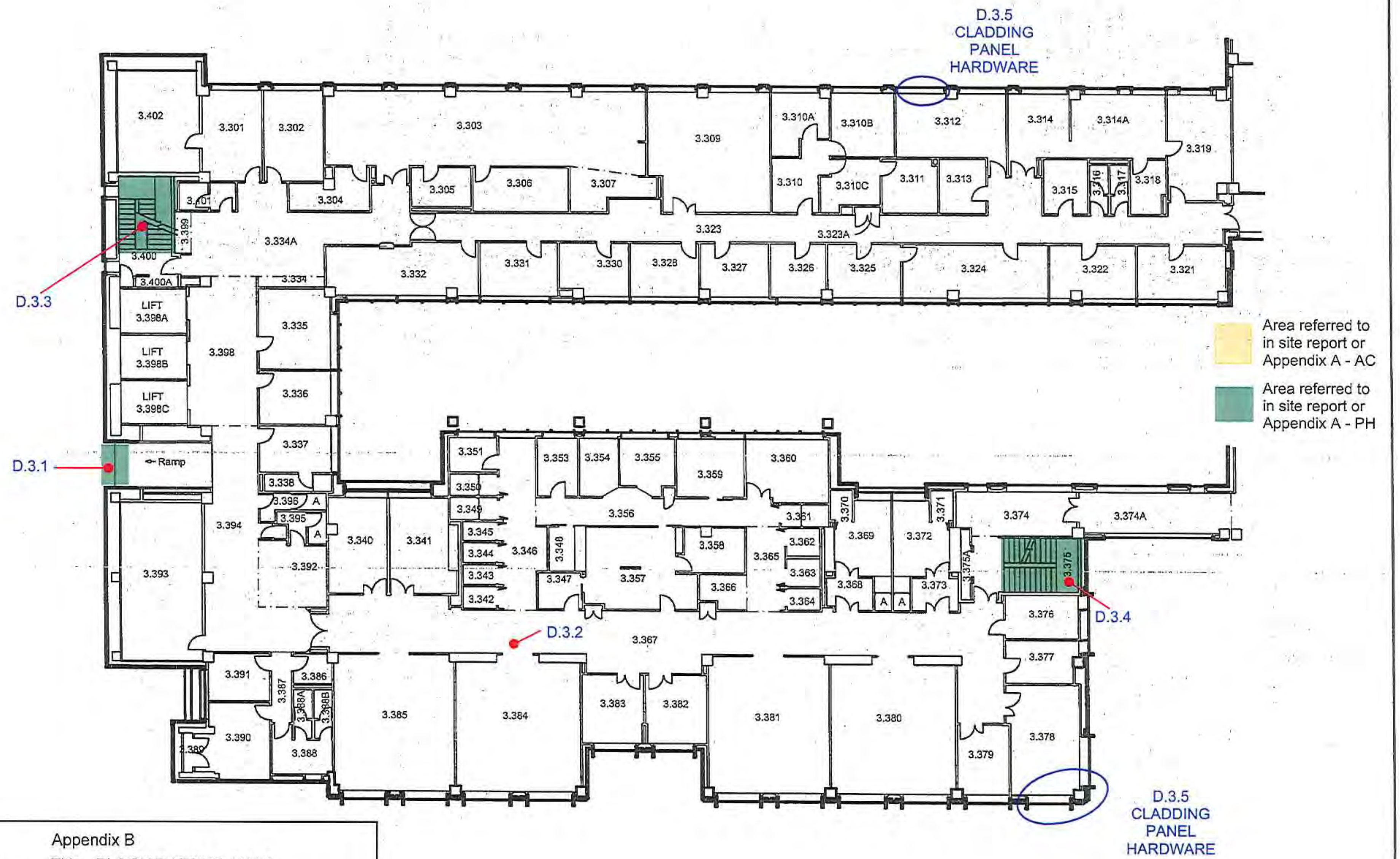
HolmesConsultingGroup

Canterbury HEALTH

DWG No. 01125009 SCALE 1:200 AT A3 REVISED 07-06-06

01125002

Maintenance and Engineering Department - Christchurch Hospital



HolmesConsultingGroup

Appendix B

Title: BLOCK D KEY PLAN L3

Job Name: CDHB Parkside

Job #: 106186.20 SSK #: SK-KP-D-6

Date: May 2014 Rev: 3

CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK 'D' THIRD FLOOR
SPACE NUMBERING

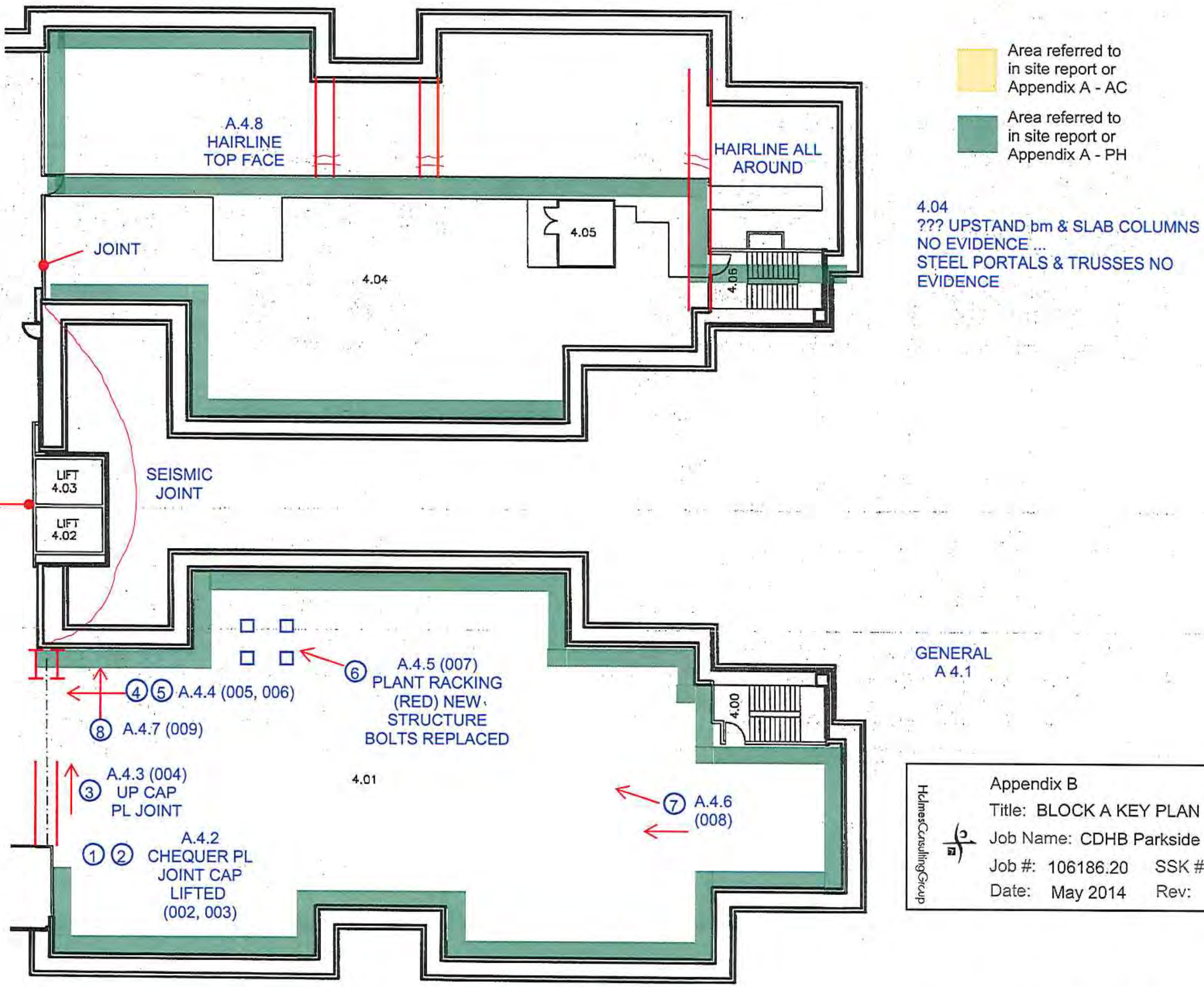
Canterbury HEALTH

DWG No. 01125010

SCALE 1:200 AT A3 REVISED 10-10-06

01125003

Maintenance and Engineering Department - Christchurch Hospital

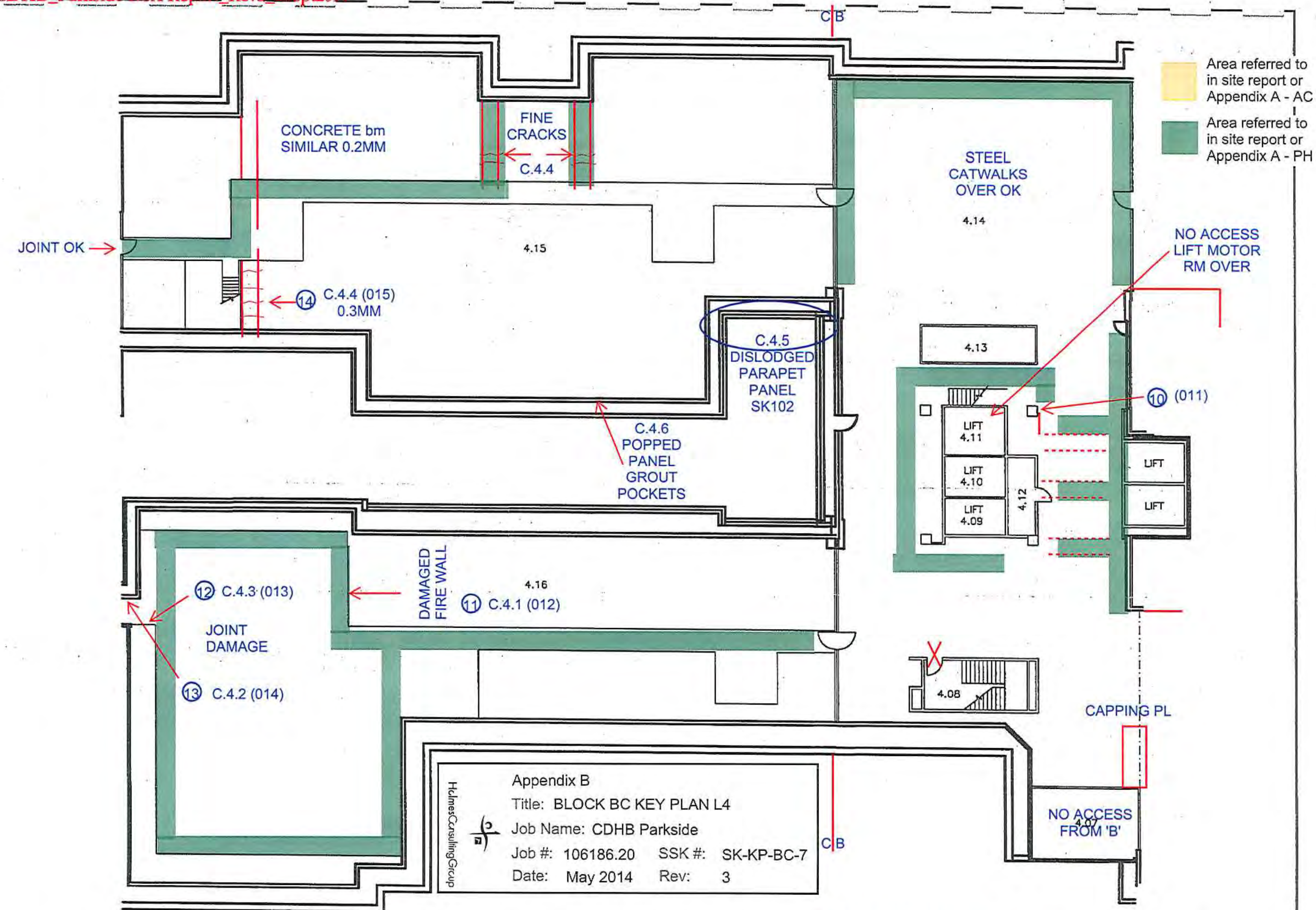


CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK 'A' FOURTH FLOOR PLANT ROOM
SPACE NUMBERING

Appendix B
Title: BLOCK A KEY PLAN L4
Job Name: CDHB Parkside
Job #: 106186.20 SSK #: SK-KP-A-7
Date: May 2014 Rev: 3

HolmesConsultingGroup

Canterbury HEALTH
DWG No. 01126005
SCALE 1:200 AT A3 REVISED 11-08-05
REF/S 01126001
Maintenance and Engineering Department - Christchurch Hospital

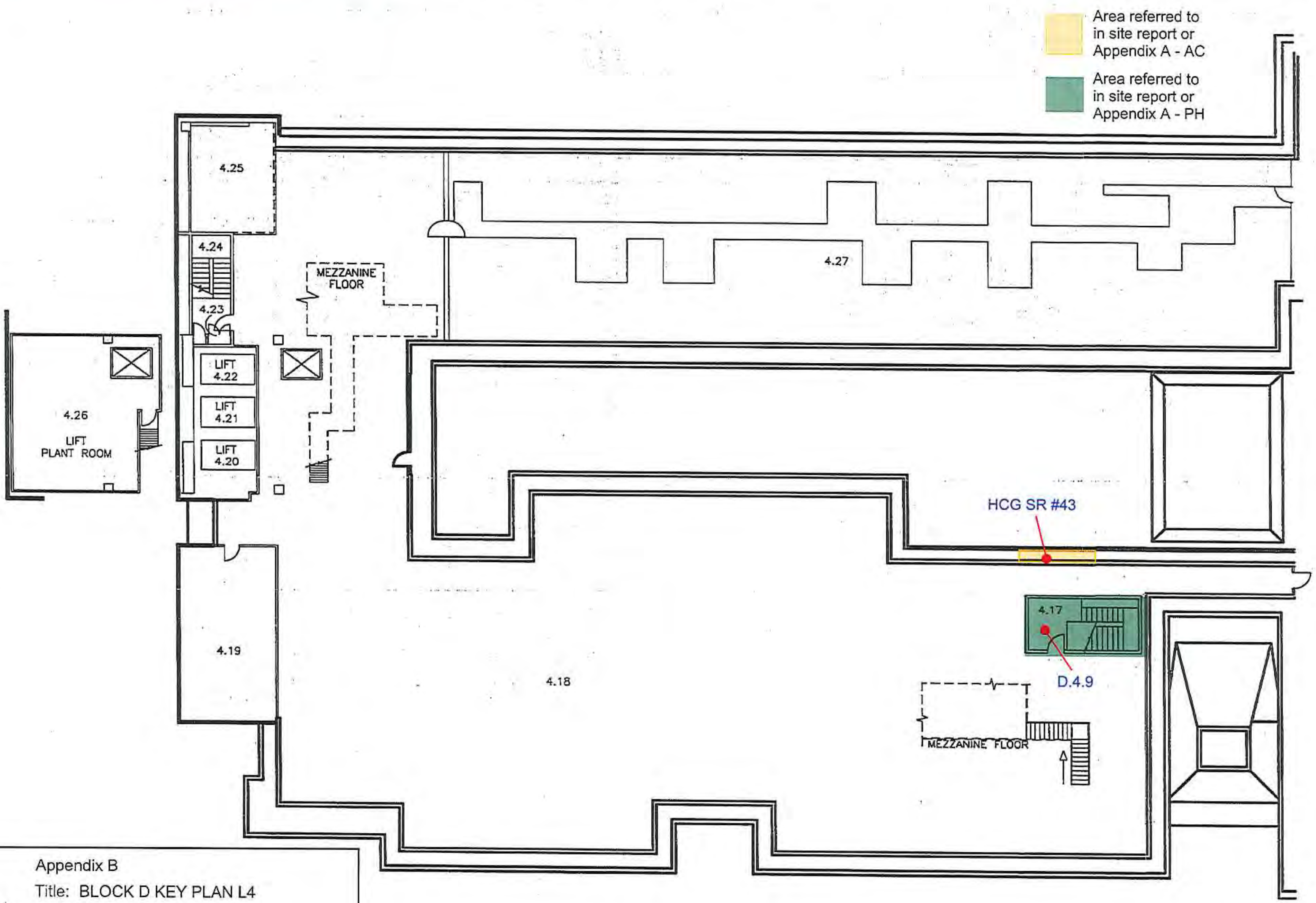


Appendix B
Title: BLOCK BC KEY PLAN L4
Job Name: CDHB Parkside
Job #: 106186.20 SSK #: SK-KP-BC-7
Date: May 2014 Rev: 3
HolmesConsultingGroup

CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK 'B-C' FOURTH FLOOR PLANT ROOM
SPACE NUMBERING

9 PARKSIDE 'B' 5/4/11

Canterbury HEALTH
DWG No. 01126006
SCALE 1:200 AT A3
REVISED 11-08-05
REF 01126002
Maintenance and Engineering Department - Christchurch Hospital



Appendix B
Title: BLOCK D KEY PLAN L4
Job Name: CDHB Parkside
Job #: 106186.20 SSK #: SK-KP-D-7
Date: May 2014 Rev: 3

HolmesConsultingGroup

CHRISTCHURCH HOSPITAL
PARKSIDE BLOCK 'D' FOURTH FLOOR PLANT ROOM
SPACE NUMBERING

Canterbury HEALTH
DWG No. 01126007
SCALE 1:200 AT A3
REVISED 11-08-05
01126003
Maintenance and Engineering Department - Christchurch Hospital

APPENDIX C

Repair Sketches
Sk100, 102

HolmesConsultingGroup

Project Name: CDHB PARKSIDE

Project No: 106186·20

Calcs By:

Date: OCT 2011

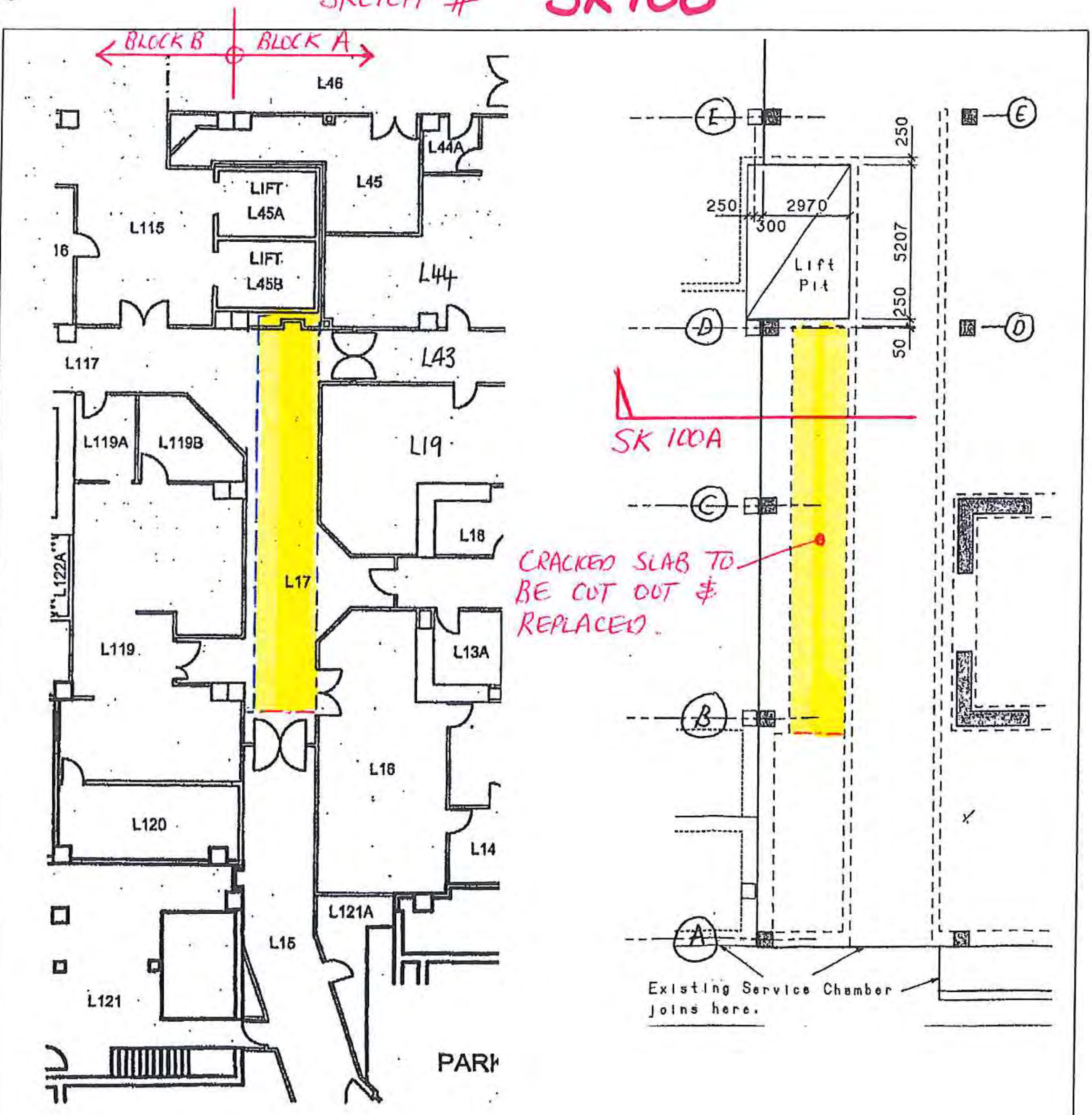
Page No:

Sketch No:

Revision: A

CALCS/SKETCHES

SKETCH # SK100



ROOM # PLAN

STRUCTURAL PLAN

BLOCK A
LOWER GROUND

SLAB REPLACEMENT

HolmesConsultingGroup

Project Name: CDHB PARKSIDE

Project No: 106 186·20

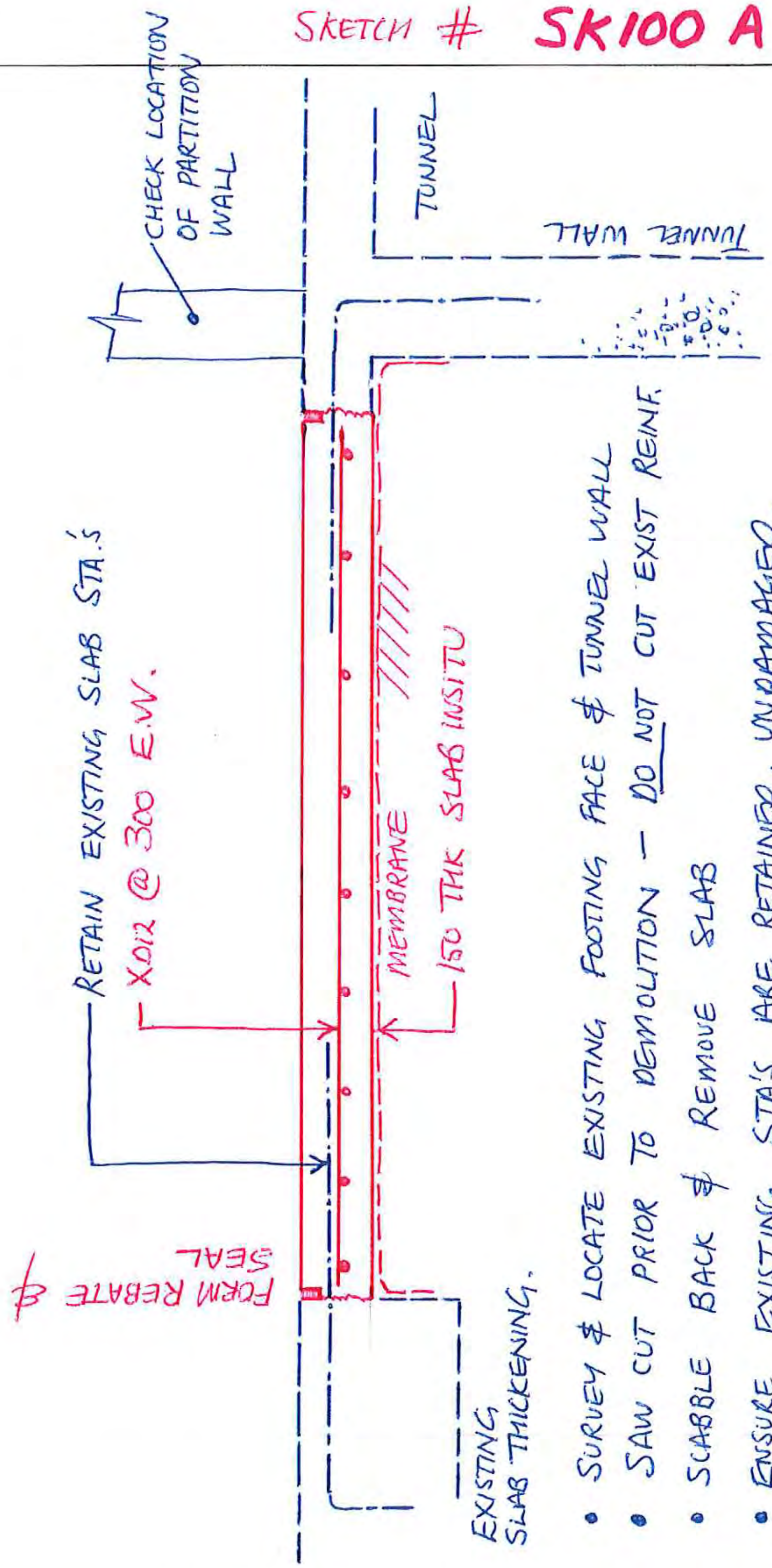
Calcs By:

Date: OCT 2011 Page No:

Sketch No: Revision: A

CALCS/SKETCHES

SKETCH # SK100 A



- SURVEY & LOCATE EXISTING FOOTING FACE & TUNNEL WALL
- SAW CUT PRIOR TO DEMOLITION – DO NOT CUT EXIST REINF.
- SCABBLE BACK & REMOVE SLAB
- ENSURE EXISTING STA'S ARE RETAINED, UNDAMAGED. NOTIFY ENGINEER FOR INSPECTION & MEASURE OF STA'S, SO EFFECTIVE BAR LAP CAN BE ACHIEVED.
- REINSTATE MEMBRANE.

HolmesConsultingGroup

Project Name: COMB PARKSIDE

Project No: 106186·20

Calcs By:

Date: OCT 2011

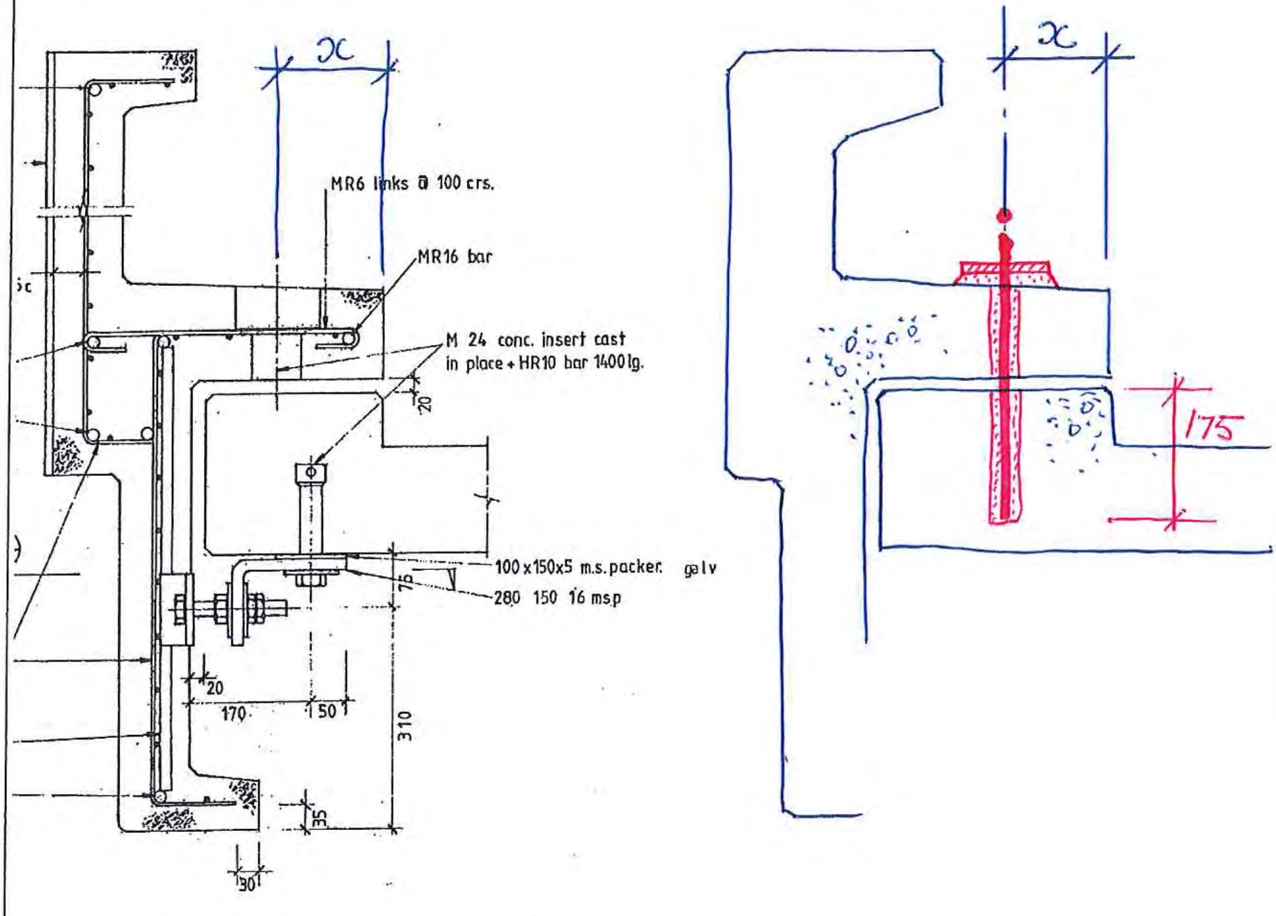
Page No:

Sketch No:

Revision: A

CALCS/SKETCHES

SKETCH # **SK 102**



(3) Section
PANEL TYPE 543 OR SIM

EXISTING DETAIL @ PARAPET

REMEDIAL FIXING TO BE APPLIED BESIDE DAMAGED FIXING.

- M20 GALV THREADED ROD DRILL & EPOXIED.
- 100x100x10 PL WASHER H.D. GALV. ON GROUT BED.
- PAINT HARDWARE POST INSTALLATION.

APPENDIX D

Holmes Solutions
Materials Testing Reports

106186.20_CDHB_Parkside DSA Report_Rev3_8Sept2014

HOLMESSOLUTIONS

REPORT 107155-1 (v1.0)

PREPARED FOR HOLMES CONSULTING GROUP

AUGUST 2011

NON-DESTRUCTIVE CONCRETE MATERIALS TESTING FOR PARKSIDE HOSPITAL BUILDING, CDHB

HOLMESSOLUTIONS

DISCLAIMER

This document was prepared by Holmes Solutions Ltd (HSL) under contract. The information presented in this document relates to non-destructive structural load testing and does not address any other related or un-related issues, including but not limited to environmental durability of the product, nor applications for the tested product. It is the responsibility of the user to assess relevant performance of the product and determine suitable applications.

This document does not constitute a standard, specification, or regulation. In undertaking the testing described in this report, Holmes Solutions have exercised the degree of skill, care, and diligence normally expected of a competent testing agency. The name of specific products or manufacturers listed herein does not imply endorsement of those products or manufacturers.

Report Produced by:

Dr Chris Allington, B.E (Hons), PhD (Civil)
MANAGING DIRECTOR

Report Reviewed by:

Wouter Van Toor, B.E (Hons)
SENIOR TEST ENGINEER

REV NO.	DATE	REVISION
V1.0	5/10/11	Issued for client review

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1.0 EXECUTIVE SUMMARY

Following the recent Christchurch earthquakes a series of detailed investigations were completed into the material properties used to construct the Hospital buildings on the CDHB campus. Testing primarily focused on completion of non-destructive testing of reinforced concrete elements.

As part of this investigation, Holmes Solutions was contracted to undertake concrete hardness testing on building elements from within the Parkside Hospital Building. A series of measurements were taken in 4 locations from within room L113 using a Silverschimdt rebound hammer. At each test location, a total of 25 individual measurements were taken. The individual measurements were analysed using the recommendations of ASTM C805 to obtain a single measurement of concrete strength for each tested location.

The average concrete hardness measurements completed for the Parkside Hospital Building indicated that the concrete strength (when measured as a concrete cylinder) varied between 25 MPa and 28 MPa. The obtained results were found to be consistent between measurements at each test site, with very little statistical variation.

2.0 TEST METHODOLOGY

Concrete hardness is often used as a non-destructive means of determining the compressive strength of concrete. The most common method employed is the rebound hardness, obtained from a portable Schmidt Hammer. The Schmidt hammer works using a similar principle to the Leeb Hardness measurements, whereby a weight is impacted on the surface of the material and the change in velocity between the impact speed and rebound speed is determined. Correlations are then applied to convert the change in speed to hardness and compressive strength.

An increased accuracy in the obtained results are achieved if the hardness measurements can be directly correlated against the specific material being tested, by completing destructive materials testing on samples of the material. This is typically achieved by removing core samples from the structure and subjecting them to compressive testing. However, if no materials testing is completed, standard conversion tables can be used to form the correlations, with an associated reduction in accuracy.

All correlations for this project were completed using the standard lower 10 percentile strength curves specifically developed for the instrument used in the testing. The curves were derived from testing of over 2,300 discrete locations. Use of the lower 10 percentile curve is recommended by the leading Standards, EN 13791 and ASTM C805/ACI 228.1.

In each tested location, a grid of readings was recorded. The results from the grid of readings were then averaged to provide the concrete hardness and associated concrete strength of that location. This testing method is endorsed by most International Testing Standards, and the manufacturers of the test equipment.

Concrete strength results are reported as Concrete cylinder strengths. All conversions between concrete cube strength and cylinder strength were completed in accordance with the ACI recommendations.

3.0 TEST EQUIPMENT

3.1. SILVERSCHIMDT HAMMER

A Proceq Silverschimdt Rebound Hammer was used to undertake all field based concrete hardness testing for concretes of compressive strength ranging from 10 to 100 MPa. This device and methodology generally accepted as the industry leading device for determining the compressive strength of concrete in-situ.

The Proceq Silverschimdt was fitted with the N-Type rebound hammer providing test impact energy of 2.207 Nm.

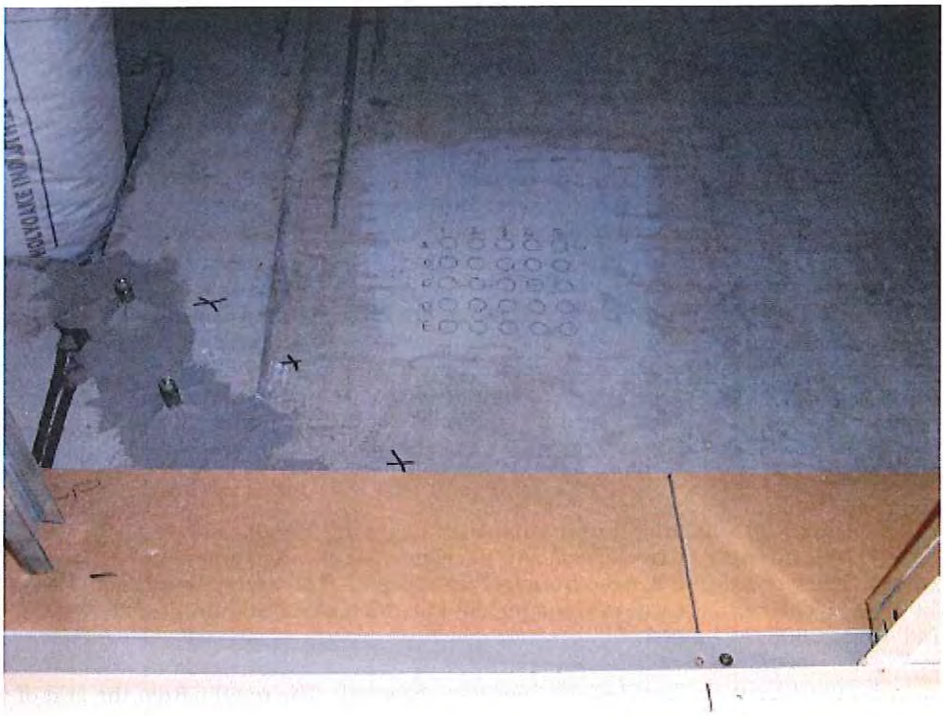

Figure 1 Prepared Test Location L113A

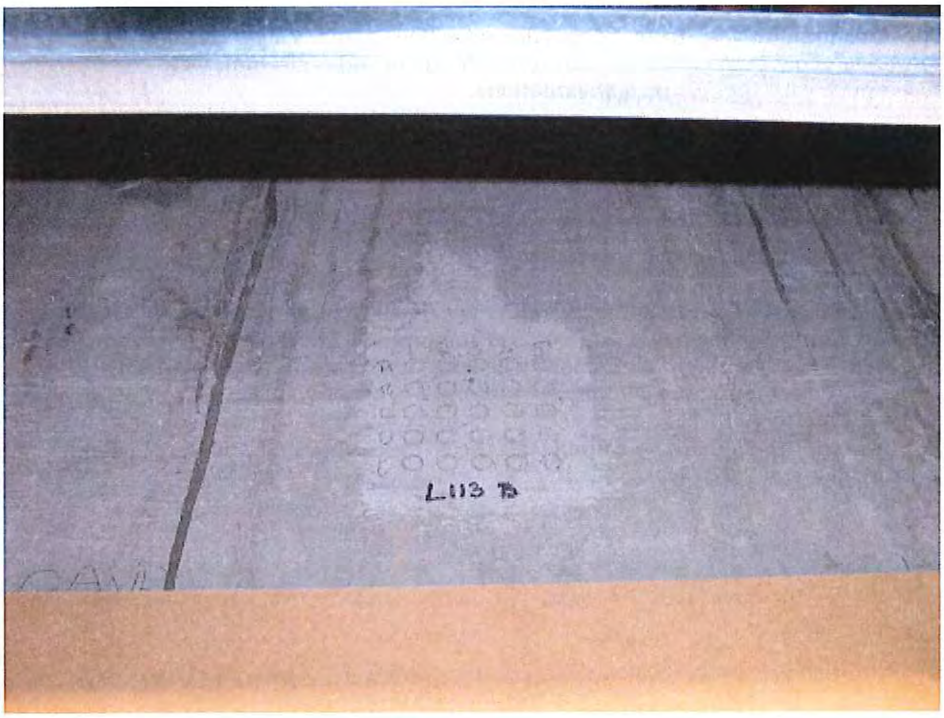

Figure 2 Prepared Test Location L113B

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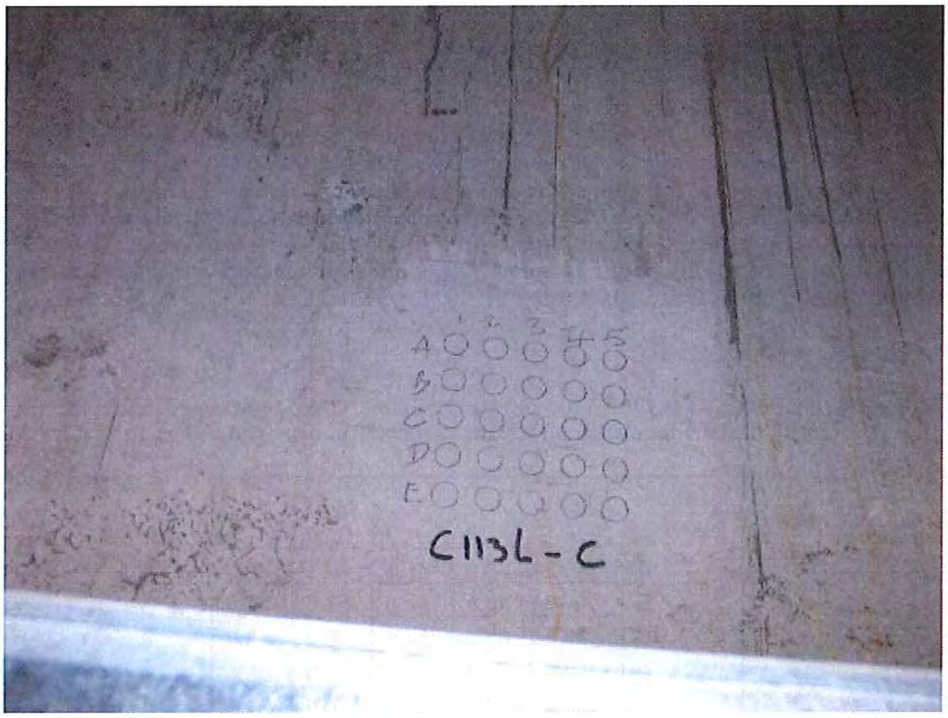

Figure 3 Prepared Test Location L113C

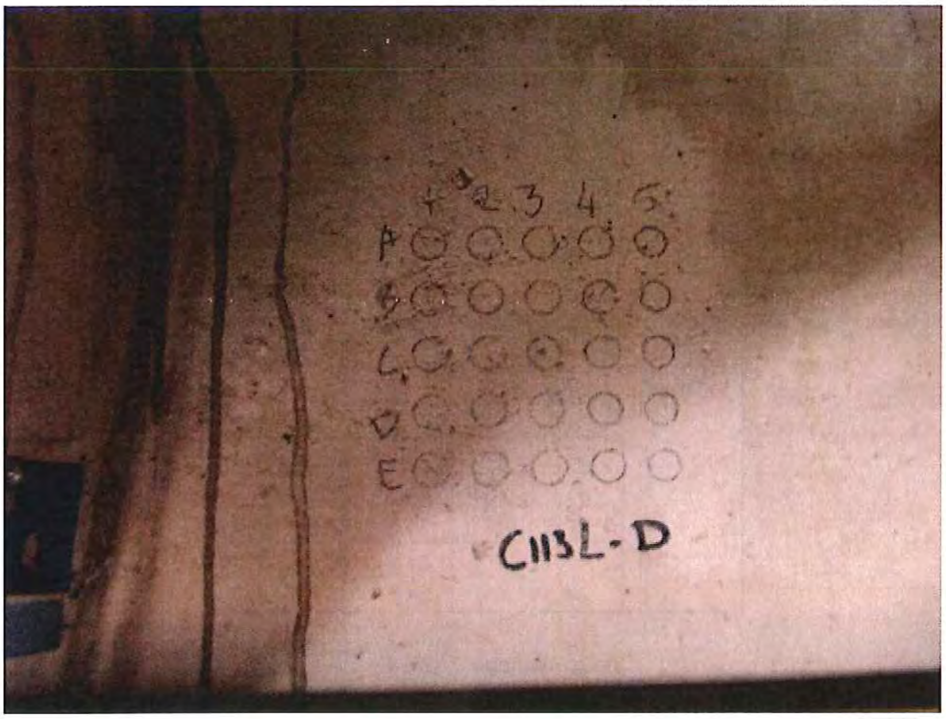

Figure 4 Prepared Test Location L113D

4.0 IN-SITU TEST RESULTS

Concrete hardness results were recorded in four discrete locations within room L113 of the Parkside Hospital building. The results from the testing are presented below.

In each tested location, a grid of readings was recorded, as shown in Figure 1 through Figure 4. The results from the grids were then statistically averaged, using the procedures detailed in ASTM C805 to provide a single concrete hardness result for each test location. It should be noted that this testing method is endorsed by the manufacturers of the test equipment.

LOCATION L113A

The results from Location L113A are presented below. Test 113A was located on the eastern sidewall of the room, with all testing completed above the ceiling line.

location: L113A

	1	2	3	4	5
A	60.2	52.0	47.5	52.0	49.5
B	54.5	48.5	47.5	53.5	55.0
C	52.5	51.0	48.0	46.0	48.5
D	52.5	53.0	49.5	49.0	45.5
E	50.5	51.0	53.0	50.0	32.5

Correct Average: 50.7
Cube Strength: 31.6 MPa

Cylinder Strength, f'c: 25.5 MPa

LOCATION L113B

Test L113B was located on the southern wall of room L113. The test grid of measurements was completed in the centre of the wall panel (lengthwise) and above the ceiling panels.

The results from Location L113B are presented below:

location: L113B

	1	2	3	4	5
A	46.0	61.0	45.0	48.0	46.0
B	47.0	58.0	61.5	45.5	53.0
C	47.5	46.0	51.0	44.5	48.0
D	47.5	47.5	54.5	46.0	60.5
E	44.0	46.0	45.0	46.5	47.0

Correct Average: 48.4
Cube Strength: 28.5 MPa

Cylinder Strength, f'c: 23.0 MPa

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LOCATION L113C

The results from Location L113C are presented below. Test L113C was located on the Northern end wall of the room, with the testing completed in the ceiling cavity above the removable ceiling tiles.

location: L113C

	1	2	3	4	5
A	56.5	58.0	54.5	53.0	47.5
B	42.5	54.0	58.5	55.5	52.0
C	48.0	45.0	56.0	53.0	61.0
D	51.5	51.0	47.5	54.5	48.0
E	48.0	53.0	53.5	52.0	43.5

Correct Average: 52.4
Cube Strength: 34.4 MPa

Cylinder Strength, f'c: 27.5 MPa

LOCATION L113D

Test L113D was located on the Western wall of room L113, on the northern side of the door opening. All testing was completed in the ceiling cavity.

The results from Location L113D are presented below:

location: L113D

	1	2	3	4	5
A	43.0	52.5	46.0	52.0	48.5
B	54.0	50.5	43.5	56.5	55.5
C	53.0	49.0	57.0	53.5	60.0
D	55.5	54.0	51.5	54.0	57.5
E	61.5	60.5	40.5	48.5	50.5

Correct Average: 52.9
Cube Strength: 35.2 MPa

Cylinder Strength, f'c: 28.0 MPa

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HOLMESSOLUTIONS

REPORT 109356 (V1.1)

PREPARED FOR HOLMES CONSULTING GROUP

NOVEMBER 2012

HARDNESS TESTING OF REINFORCING STEEL IN BLOCK D PARKSIDE, CDHB CAMPUS, CHRISTCHURCH

HOLMESSOLUTIONS

DISCLAIMER

This document was prepared by Holmes Solutions LP (HSLP) under contract. The information presented in this document relates to non-destructive structural load testing and does not address any other related or un-related issues, including but not limited to environmental durability of the product, nor applications for the tested product. It is the responsibility of the user to assess relevant performance of the product and determine suitable applications.

This document does not constitute a standard, specification, or regulation. In undertaking the testing described in this report, Holmes Solutions have exercised the degree of skill, care, and diligence normally expected of a competent testing agency. The name of specific products or manufacturers listed herein does not imply endorsement of those products or manufacturers.

Report Produced by:

Dr Chris Allington, B.E (Hons), PhD (Civil)
CEO

Report Reviewed by:

W von Toor
SENIOR MECHANICAL ENGINEER

REV NO.	DATE	REVISION
V1.0	30/11/12	Issued for internal review
V1.1	05/12/12	Issued for client review – DRAFT ONLY

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1.0 EXECUTIVE SUMMARY

Following the recent Christchurch earthquakes a series of cracks were observed in a number of structural elements in Block D building, Parkside Building on the Canterbury District Health Board building in Christchurch. Concerns were raised about potential damage of the reinforcing steel in areas near the observed cracking. Holmes Solutions were engaged to undertake testing on structural elements of the building. To minimise further loss of capacity of the structure, the testing of the reinforcing steel was requested to be non-destructive in nature. The results from this project are reported herein.

In order to provide the required non-destructive testing, Holmes Solutions utilised a new form of testing protocol that provides an estimation of the strain in the reinforcing steel by measuring the hardness of the steel at zones of damage in the building. This form of testing can be conducted in-situ and requires minimum preparation of the reinforcement beyond removal of the cover concrete to expose the reinforcing bars surface.

A section of steel from an undamaged region of the building was firstly removed and used to derive a correlation response between the level of imposed steel stress (and the corresponding strain) in the steel and the hardness of the material. This was achieved by applying a predetermined level of load to the steel sample and then measuring the hardness. At the completion of the hardness tests, the sample was subjected to a further increment of stress and the hardness testing repeated. This process was repeated until a suitable number of measurements had been obtained to allow a full correlation between imposed engineering stress and material hardness to be derived. The level of imposed strain was then derived using the material stress-strain response for the steel.

The aforementioned correlation testing was undertaken on a deformed steel bar with a diameter of 12 mm. The yield strength of the material was found to be 347 MPa, with a strain corresponding to the attainment of peak stress (defined as the peak strain) equal to 20.5%. The measured material properties suggest the steel was manufactured from hot rolled material.

All in-situ testing was completed on steel sections crossing noted concrete crack in the concrete members. The cover concrete was first removed (to expose the steel) before the steel samples were prepared for testing. If the steel sample was of a small diameter, a layer of additional supporting grout was placed around all bars prior to testing in an attempt to provide consistency in the support conditions of the steel. Testing was completed by undertaking a series of 6 hardness tests across each of the 15 mm increments measured along the length of the exposed steel sections. The obtained in-situ hardness results were normalised for support conditions and then using the derived correlations, an estimate of the average potential level of imposed strain in the steel bar was calculated.

The obtained test results typically showed an increase in hardness at or near the location of the cracks in the concrete elements. The derived correlations between Leeb Hardness and imposed steel stresses, in conjunction with the material stress strain curves were used to calculate the level of average potential induced strain in the steel samples. Furthermore, the level of induced strain was then used to determine the potential lost strain capacity of the steel, as a percentage of the materials uniform elongation capacity.

The maximum values of Average Potential Induced Strain and the estimated Potential Lost Strain Capacity for each of the tested locations are presented in the following table. Potential lost strain capacities are derived as a percentage of the uniform elongation capacity of the material, where uniform elongation is defined as the elongation of the steel corresponding with the attainment of peak stress, and is considered to be the end of the usable strain capacity of the steel.

Reference	Element	Location	Average Potential Induced Strain (%)	Potential Lost Strain Capacity (%)
109356-1	Concrete Wall	Room L152	2.73	12-16
109356-2	Concrete Wall	Room L152	3.75	14-24
109356-3	Concrete Wall	Room L113	6.41	24-41
109356-4	Concrete Wall	Room L240	5.56	18-41
109356-5	Concrete Wall	Room L240	5.03	20-30
109356-12A	Concrete Floor	Top side of lower ground floor	5.75	22-35
109356-12B	Concrete Floor	Top side of lower ground floor	3.63	22-27
109356-13	Concrete Floor	Bottom side of lower ground floor	3.61	15-21
109356-14	Concrete Floor	Bottom side of lower ground floor	4.22	16-27

2.0 TEST METHODOLOGY

Hardness testing has long been used as to determine the material properties of steel samples, however it has only recently been used as a non-destructive tool to verify the likely change in material properties of steel component due to damage caused under seismic (or other) applied loads [L1, K1, N2, M1]. There is limited published information regarding this technique, particular relating to reinforcing steel in concrete elements. The primary difficulty with the technique has been determining a suitable correlation between the measured hardness state and the material properties [M2, S1, P1, N1, T1, T2].

Recent research in New Zealand and overseas has shown that hardness can be used as an indicator of the current strain state of steel samples [L1, N2, M1, S2, Y1]. Relating the hardness of steel samples to the stress-strain properties of the base material allows an understanding of likely damage (or loss of strain capacity) that the steel sample has sustained and therefore to determine how much residual strain capacity the sample retains. This form of direct comparison can only be achieved if suitable correlations are developed between the measured hardness and the strain state of the specific steel sample.

The majority of hardness measurements are not suitable for field application. Portable electronic Leeb hardness testers have been developed to accurately measure the dynamic hardness of a steel sample in-situ with a high degree of repeatability [L1, N1, S1, M2, Y1]. Leeb hardness provides a measure of the dynamic hardness of the material, which is a combination of the materials elastic and plastic hardness characteristics. Research has shown that both the elastic hardness and plastic hardness of a material alter as a material is damaged due to induced plastic strains; however the change in the elastic hardness is minor. As such Leeb hardness provides a convenient method of measuring for use in-situ and has been shown to provide a measure of the likely material damage, albeit a relatively small change. Other hardness method that provide a greater measure of change in the material properties with damage are less repeatable in the field and therefore not used.

Holmes Solutions has completed extensive research into the correlation between Leeb hardness and the steel samples strain state for a range of different reinforcing steels. The results from the research have been developed into a normalisation technique that can be use to provide an indication as to the current strain state of the tested steel sample.

To accurately derive suitable correlations it is necessary to undertake materials testing on a non-damaged sample of steel. The steel is subjected to a uniaxial tension test and the hardness measured at a series of predefined stress and strains. Hardness testing is completed when the sample is both under load and with the applied load removed. Care must be taken to closely match the support conditions of the sample tested in the laboratory to the material tested in-situ. Any error in support conditions has been shown to significantly influence the accuracy of the obtained results. The resulting correlations are used, in conjunction with the normalisation techniques derived from obtaining numerous hardness readings in the area surrounding the expected zone of damage, to determine the value of imposed steel stress and corresponding strain in the steel from the recorded Leeb measurements. These results are then directly compared to the properties of the parent material to estimate the potential reduction in strain capacity that has been sustained in the steel sample.

Leeb readings are collected from in-situ reinforcing bars. The surface of the bars is carefully prepared to specific requirements prior to testing. Readings are obtained at critical locations along the length of the reinforcing bar to allow the strain profile of the steel to be determined and to assist in the normalisation procedures.

The overall estimation of strain degradation for the tested steel samples is achieved by using the derived strain damage from the Leeb testing in conjunction with engineering knowledge of the particular application.

Wherever possible, in-situ hardness testing is completed in accordance with ASTM A956-06 Standard Test Methods for Leeb Hardness Testing of Steel Products [A2]. For all locations, a minimum of six individual hardness tests are completed and the truncated mean of the recorded results is used to obtain a value for the reported average Leeb value [A1]. All recorded values are then corrected for variation in support condition and normalised using the derived correlation factors.

A detailed uncertainty analysis is completed on the obtained results, in accordance with the guidance of ISO/IEC Guide 98-3 [I1] with due consideration provided to both Type A and Type B errors in the measurement chain. The reported uncertainties are computed as expanded uncertainties using a coverage factor of 2 thereby providing a level of confidence of 95% in the reported results (assuming a normal distribution of results).

The measurement and analysis procedure used in this project has been used extensively by Holmes Solutions in reviewing the performance of damaged buildings and structures following the recent earthquakes in Christchurch. By means of validation, a number of tests have been completed where the predicted results obtained from the hardness testing have been validated by destructive testing of the steel element. In all occasions, the degree of strain damage determined from the destructive testing correlated well with the predicted results from the hardness testing programme.

2.1. LEEB HARDNESS

The term hardness may be defined as the ability of a material to resist permanent indentation or deformation when in contact with an indenter under load. Generally, a hardness test consists of pressing an indenter of known geometry and mechanical properties under predefined conditions into the test material.

Different hardness test methods have been developed and adopted for various test problems, with each test method measuring a different aspect of the materials hardness, such as static hardness, micro-hardness, nano-hardness, and dynamic hardness. As a result, a key concern with hardness testing has been the comparison of test results on an unequivocal base to an accepted reference scale. Given that Hardness is not a fundamental (physical) property of a material but rather a combination of variables associated with the material, it is highly dependent on not only the test method used but to a lesser extent on the quality of the equipment used in the testing. As such it is very important to clearly define the instrument that is being used and the associated test parameters when reporting hardness values.

Leeb hardness is a direct measure of a materials dynamic hardness and is considered to be accurately measuring the materials elastic and plastic hardness characteristics. Leeb hardness is obtained by firing an impact body containing a permanent magnet and a very hard indenter sphere towards the surface of the test material and measuring the velocity of the impact body. The velocity is measured in three main test phases;

- Pre-impact phase, where the impact body is accelerated by spring force towards the surface of the test piece.
- Impact phase, where the impact body and the test piece are in contact. The hard indenter tip deforms the test material elastically and plastically and is deformed itself elastically. After the impact body is fully stopped, elastic recovery of the test material and the impact body takes place and causes the rebound of the impact body.

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- Rebound phase, where the impact body leaves the test piece with residual energy, not consumed during the impact phase.

The Leeb hardness is determined by calculation, relating the three recorded velocities. The velocities are measured in a contact-free means via the induction voltage generated by the moving magnet through a defined induction coil mounted on the guide tube of the device. The induced voltage is directly proportional to the velocity of the magnet and therefore used to determine the hardness of the steel sample.

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3.0 TEST EQUIPMENT

3.1. LEEB HARDNESS TESTER

A Proceq Equotip 3 portable hardness tester was used to test all material hardness values. The device is generally acknowledged as the industry standard for the determination of Leeb hardness. The hardness tester was installed with a DL impact device, allowing measurements on smaller diameter steel samples than the conventional D device.

The Equotip 3 has a reported accuracy of ±4 HL and is traceably calibrated to NIST standards.

3.2. UNIVERSAL TEST MACHINE

A UH600 Shimazu servo-controlled Universal Test Machine (UTM) with a 600 kN capacity was used to undertake all laboratory based materials testing. The UTM has a maximum stroke of 250 mm and a peak table velocity of 150 mm/min.

Steel Elongation was recorded using a strain gauge based digital extensometer with a gauge length of 50 mm. Applied loads were recorded directly using the internal pressure transducer of the Shimazu control system.

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4.0 TEST RESULTS

4.1. STRESS-HARDNESS CORRELATIONS

Previous researchers have determined, using experimental investigations, that the correlation between steel Hardness and imposed plastic stress is relatively unaffected by imposed strain history or loading rate [M2]. Furthermore, in most structures, the critical section of reinforcing steel is likely to have been located at or near the extreme fibres of the concrete element. Given that the neutral axis of the element is expected to have been located near the location of the reinforcing steel during the compression load cycle, the steel is typically only subjected to small induced compressive strains. During the reverse loading cycle the steel located at or near a crack in the concrete section is likely to have been subjected to disproportionately larger tensile strains, thereby significantly skewing the strain profile into the tension domain. Due to the skewed strain profile, it is believed that the unidirectional cyclic tensile test provides an adequate representation of the strains induced in the steel during a seismic event.

Testing of the reinforcing bars was completed by applying a predetermined level of stress to the steel bar while installed in the UTM. At the attainment of the desired stress the load was removed, and the sample taken from the test machine before being embedded into a cement matrix. Once the matrix has cured sufficiently, the steel was subjected to a series of 6 Leeb Hardness tests at a minimum of three locations along the length (18 tests in total). The sample was then extracted from the cement, cleaned, and placed back into the UTM where a further increment of stress was applied. This process was repeated until a suitable number of measurements had been recorded to adequately describe the response of the steel. The truncated average of the measured Leeb Hardness results for each stress increment was then used to derive the general correlation of hardness to applied stress.

4.1.1. HOT ROLLED DEFORMED REINFORCING BAR

A section of 12 mm diameter reinforcing bar was removed from an area in the building that was noted to be free form any cracking or visual damage. The collected sample was considered to be a representative sample of the parent steel material used in all deformed reinforcing bars tested in the report. The steel sample was subjected to uni-directional cyclic tensile testing using a Universal Testing Machine (UTM) as per the Holmes Solutions testing protocol described below.

During the abovementioned correlation procedure a direct measure of the steels stress-strain response was obtained, presented in Figure 1 below. This response was used to obtain the material properties of the tested steel and to assist in the conversion from the level of derived stress (predicted from the in-situ hardness measurements to a value) to the predicted level of strain.

The baseline Leeb value hardness for the steel was determined to be 608 HLDL. This value was used as the baseline for all hot rolled, deformed reinforcing bars tested in this report.

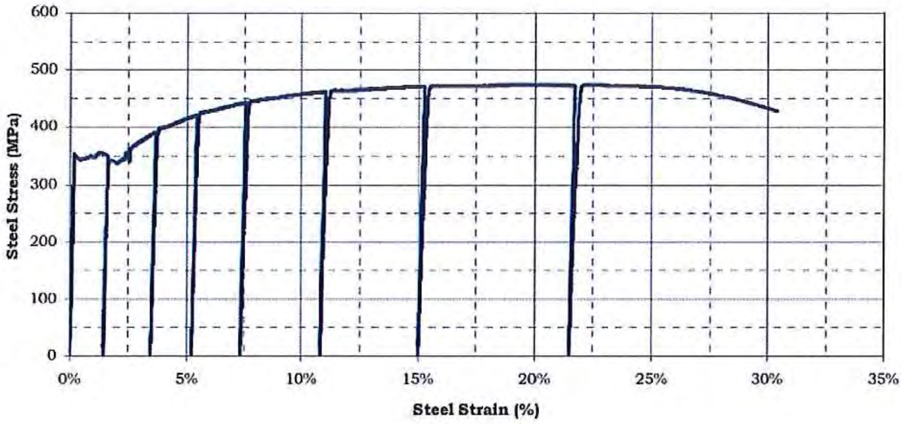

Figure 1 Materials Stress Strain response for the baseline material

Table 1 Material Properties of the baseline material

Yield Stress, f_y:	347	(MPa)
Strain Hardening Stress, f_{sh}:	347	(MPa)
Ultimate Stress, f_u:	473	(MPa)
Stress Ration, f_u/f_y:	1.36	
Yield Strain, ε_y:	0.3	(%)
Strain Hardening Strain, ε_{sh}:	2.2	(%)
Peak Strain, ε_u:	20.5	(%)
Baseline Leeb Hardness:	608	(HLDL)

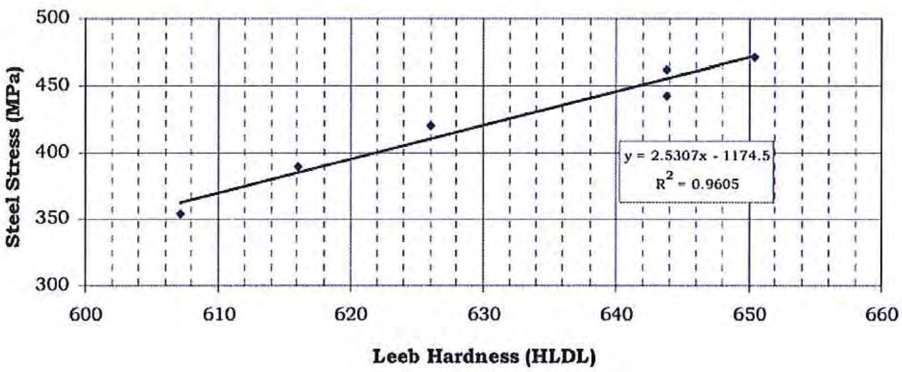

Figure 2 Correlation between Leeb Hardness and Steel Stress for the baseline material

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4.2. TEST RESULTS

The recorded results from all in-situ hardness testing completed in this testing programme are reported below. All test locations are defined using a naming sequence representing the reference number for the building (108418), the reference location from within the building where the testing was undertaken (1-20), the reference bar label from the test location (A, B and C), and the individual test location along the length of the bar. Testing was typically completed at approximately 15 mm intervals along the length of each exposed bar, with each test location being defined as its own reference location on the steel (numeric reference). Unless otherwise noted, all steel segments are labelled from right to left for horizontal steel sections and top to bottom for all vertical steel elements, and are marked on the photographs with an arrow on the test bar reference label.

Prior to undertaking Leeb hardness testing it was necessary to remove the cover concrete and prepare the surface of the steel for testing. Care was taken not to damage the surface of the steel during the removal process. Wherever possible, the exposed length of steel projected approximately 250 mm either side of the centre of the test region, allowing the baseline properties for the steel section to be captured.

During on-site collection of the Leeb Hardness data, care was taken to note the support conditions of the reinforcing bar at the location of each series of readings. Variations in the support conditions have been found to influence the recorded hardness results through vibrations of reinforcing steel during the impact of the Leeb weight. Reinforcing bars with poor support conditions can result in artificially low Leeb Hardness results. If it was considered the bar to be tested had poor support conditions, an additional grout layer was applied to the steel after the cover concrete was removed in an attempt to normalise the support conditions along the bars and between bars.

The results from the Leeb Hardness testing are presented in both Tabular and graphical form. The Graphs have been included to provide a visual reference as to how the hardness varied along the length of the reinforcing steel. The following colour coded keys are used on all graphs

- All locations with viable Leeb hardness readings are presented with red columns.
- All locations non-viable Leeb hardness reading or locations where no readings were undertaken are presented as blank columns.
- The location of the concrete crack in the concrete member relative to the tested steel sample is shown as a Yellow column. If no crack was evident, all of the column elements in the graphs will remain red.
- Any locations of known damaged to the reinforcing steel sample (such as those damaged during the removal of the cover concrete) are highlighted with a Green column on the graphs.

It should be noted that often the hardness is seen to vary at the ends of the tested region. This variation is typically caused by changes in the support conditions of the reinforcing steel in these locations rather than actual variations in the steel properties. Variations in the measured Leeb hardness can also be found to occur in the general proximity of other reinforcing bars that cross the test region. In general, it is noted that test locations with a significant number of reinforcing bars crossing the test region have more scatter in the obtained Leeb Hardness results.

The tabulated results for the Leeb hardness testing includes the Average Recorded Leeb values (derived from the truncated mean of the test samples) and the associated Normalised Leeb hardness values for each testing location along each reinforcing bar. In all circumstances the normalised Leeb values are used in the analysis of the influence of the potential induced strains. The bar support factor, is used in the normalisation process to correct for variations in the support condition along the length of all tested steel samples. The uncertainty in the Leeb Hardness was computed as an expanded uncertainty with a coverage factor of 2, providing a level of confidence of 95% in the reported results (assuming a normal distribution of results).

The minimum and maximum potential induced strains values are derived directly from the upper and lower bounds of the normalised Leeb hardness values. As such, both strain figures are reported with a level of confidence of 95%. It should be noted that due to the non-linear relationship between imposed stress and the associated strain that the average value of potentially induced strain does not typically occur at the mid point between the reported minimum and maximum strain values and is likely skewed towards the lower end of the reported range.

Given the correlation between Leeb Hardness and imposed steel stress is derived for strain hardening region of the steel stress-strain response, it is difficult to determine the value of imposed strain at strain levels below the onset of strain hardening (ε_{sh}). As such, any values of derived strain below the onset of strain hardening are reported as zero. Additionally, due to the shape of the steel stress strain curve near the attainment of the peak stress, a small variation in stress results in a significant increase in associated strain. To maintain the required level of confidence in the reported results, any potentially induced strains greater than 80% of the peak strains are only reported as such.

All bars were prepared for testing by smoothing the front face to the required (and consistent) surface roughness prior to dividing the test region into a series of 15 mm long test increments. If the bar was considered to have inadequate support from the surrounding concrete, additional cement grout was placed around the bar. With the inclusion of the additional grout placed around the steel prior to testing, the support conditions were typically found to be consistent along the length of the bar.

4.2.1. TEST LOCATION 109356-1

Test Location: 109356-1

Element: Concrete Wall

Floor: L152

Bar Reference 24 mm deformed bar

Average Potential Induced Strain (%): 2.73

Maximum Potential Loss of Strain Capacity (% peak): 12-16

Observation and Notes:

Figure 3 Test Location 109356-1

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Table 2 Test Results for Location 109356-1

Reference	Location along bar (mm)	Bar Support Factor	Average Recorded Leeb (HLDL)	Normalised Leeb (HLDL)	Leeb Hardness Uncertainty (+/- HLDL)	Average Potential Induced Strain (%)	Potential Lost Strain Capacity (%)
109356-01-01	15	Normal	598	606	6	2.20	1 - 11%
109356-01-02	30	Normal	605	613	4	2.40	11 - 13%
109356-01-03	45	Normal	590	598	7	0.00	1 - 1%
109356-01-04	60	Normal	593	601	7	0.00	1 - 11%
109356-01-05	75	Normal	609	617	7	2.73	11 - 19%
109356-01-06	90	Normal	599	607	6	2.20	1 - 12%
109356-01-07	105	Normal	606	614	6	2.46	11 - 15%
109356-01-08	120	Normal	607	615	8	2.54	11 - 17%
109356-01-09	135	Normal	595	603	1	0.00	1 - 1%
109356-01-10	150	Medium	576	595	4	0.00	1 - 1%
109356-01-11	165	Normal	609	617	4	2.73	12 - 16%
109356-01-12	180	Normal	608	616	7	2.63	11 - 18%
109356-01-13	195	Normal	596	604	10	0.00	1 - 12%
109356-01-14	210	Normal	593	601	6	0.00	1 - 11%

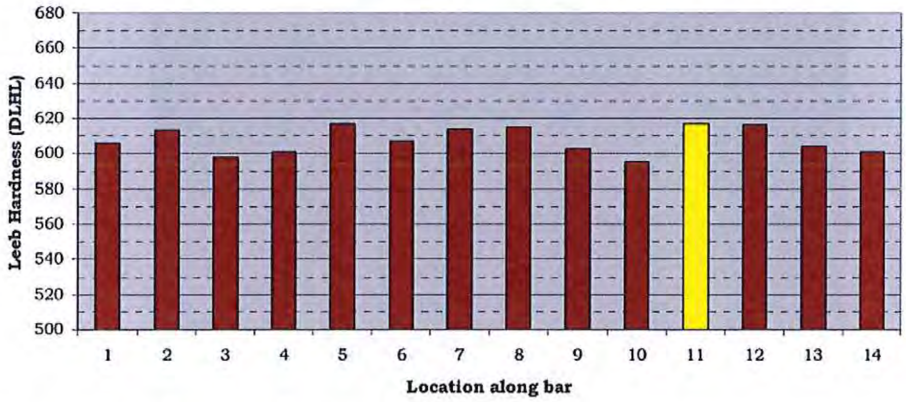

Figure 4 Leeb Hardness plot for Location 109356-1

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4.2.2. TEST LOCATION 109356-2

Test Location: 109356-2

Element: Concrete Wall

Floor: Room L125

Bar Reference 20 mm deformed bar

Average Potential Induced Strain (%): 3.75

Maximum Potential Loss of Strain Capacity (% peak): 14-24

Observation and Notes:

Figure 5 Test Location 109356-2

Table 3 Test Results for Location 109356-2

Reference	Location along bar (mm)	Bar Support Factor	Average Recorded Leeb (HLDL)	Normalised Leeb (HLDL)	Leeb Hardness Uncertainty (+/- HLDL)	Average Potential Induced Strain (%)	Potential Lost Strain Capacity (%)
109356-02-01	15	Normal	601	609	3	2.23	11 - 11%
109356-02-02	30	Normal	605	613	7	2.40	11 - 15%
109356-02-03	45	Normal	605	613	3	2.40	11 - 13%
109356-02-04	60	Normal	605	613	1	2.40	12 - 12%
109356-02-05	75	Normal	590	598	9	0.00	1 - 11%
109356-02-06	90	Normal	599	607	12	2.20	1 - 14%
109356-02-07	105	Normal	605	613	4	2.40	11 - 13%
109356-02-08	120	Normal	599	607	5	2.20	1 - 11%
109356-02-09	135	Normal	576	584	4	0.00	1 - 1%
109356-02-10	150	Normal	606	614	4	2.46	11 - 14%
109356-02-11	165	Normal	586	594	3	0.00	1 - 1%
109356-02-12	180	Normal	594	602	4	0.00	1 - 11%
109356-02-13	195	Normal	588	596	8	0.00	1 - 1%
109356-02-14	210	Normal	607	615	6	2.54	11 - 16%
109356-02-15	225	Normal	596	604	5	0.00	1 - 11%
109356-02-16	240	Normal	616	624	5	3.75	14 - 24%
109356-02-18	#REF!	High	615	611	5	2.28	1 - 13%

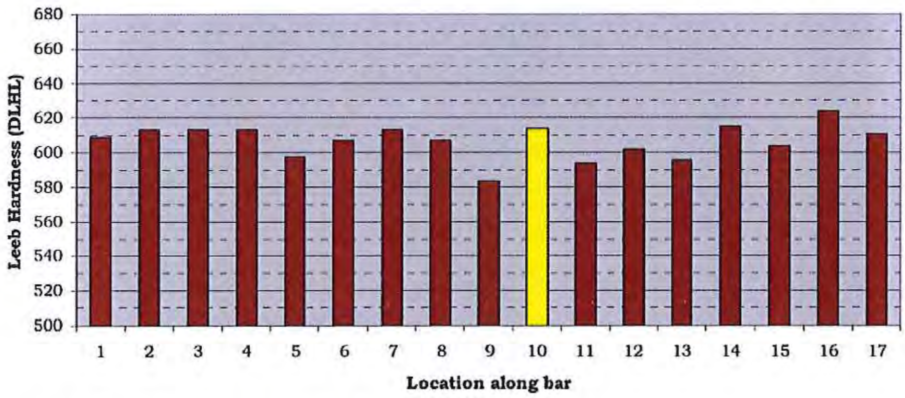

Figure 6 Leeb Hardness plot for Location 109356-2

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4.2.3. TEST LOCATION 109356-3

Test Location:	109356-3
Element:	Concrete Wall
Floor:	Room L113
Bar Reference	24 mm deformed bar

Average Potential Induced Strain (%): 6.41

Maximum Potential Loss of Strain Capacity (% peak): 24-41

Observation and Notes: Results showed a general saw-tooth action of highs and lows, common for steel with a number of transverse steel sections preventing continuous readings

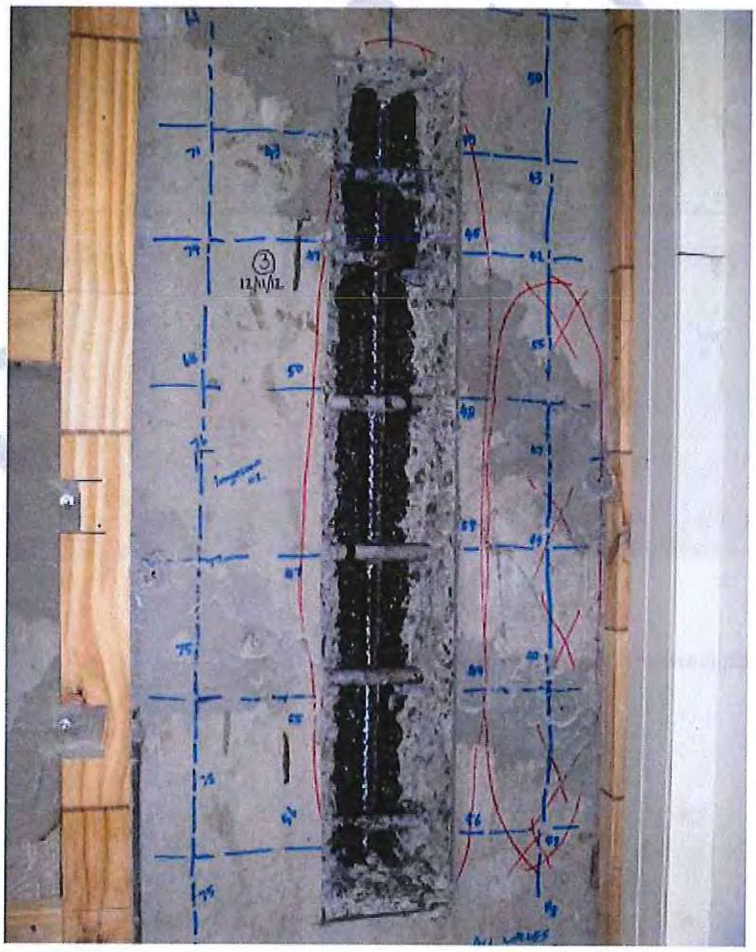

Figure 7 Test Location 109356-3

Table 4 Test Results for Location 109356-3

Reference	Location along bar (mm)	Bar Support Factor	Average Recorded Leeb (HLDL)	Normalised Leeb (HLDL)	Leeb Hardness Uncertainty (+/- HLDL)	Average Potential Induced Strain (%)	Potential Lost Strain Capacity (%)
109356-03-01	15	Normal	588	605	5	0.00	1 - 11%
109356-03-02	30	Medium	576	604	7	0.00	1 - 11%
109356-03-03	45	Normal	590	607	11	2.20	1 - 14%
109356-03-04	60	Normal	593	610	4	2.26	11 - 12%
109356-03-05	75	Normal	601	618	5	2.85	12 - 18%
109356-03-06	90	Normal	610	628	11	4.44	13 - 37%
109356-03-07	105	Normal	606	623	2	3.61	16 - 19%
109356-03-08	120	Normal	597	614	5	2.47	11 - 15%
109356-03-09	135	Normal	583	600	12	0.00	1 - 11%
109356-03-10	150	Normal	588	605	9	0.00	1 - 12%
109356-03-11	165	Normal	596	613	4	2.40	11 - 13%
109356-03-12	180	Normal	613	631	6	5.20	18 - 35%
109356-03-13	195	Normal	617	635	6	6.41	24 - 41%
109356-03-14	210	Normal	596	613	6	2.40	11 - 14%
109356-03-15	225	Medium	581	610	4	2.25	1 - 12%
109356-03-16	240	Normal	592	609	6	2.23	1 - 13%
109356-03-17	255	Normal	592	609	8	2.23	1 - 13%
109356-03-18	270	Normal	598	615	2	2.55	12 - 13%
109356-03-19	285	Medium	587	616	2	2.61	12 - 14%
109356-03-20	300	Medium	566	594	14	0.00	1 - 11%
109356-03-21	315	Medium	583	612	9	2.33	1 - 16%
109356-03-22	330	Medium	583	612	7	2.33	1 - 14%
109356-03-23	345	Medium	568	596	3	0.00	1 - 1%
109356-03-24	360	Medium	585	614	9	2.45	1 - 17%
109356-03-25	375	Medium	579	608	6	2.21	1 - 12%
109356-03-26	390	Medium	566	600	11	0.00	1 - 11%
109356-03-27	405	Medium	563	597	4	0.00	1 - 1%
109356-03-28	420	Medium	576	604	5	0.00	1 - 11%
109356-03-29	435	Medium	586	615	4	2.52	11 - 15%
109356-03-30	450	Medium	578	607	6	2.20	1 - 12%
109356-03-31	465	Medium	577	605	3	0.00	1 - 11%
109356-03-32	480	Normal	603	620	8	3.12	11 - 23%
109356-03-33	495	Normal	592	609	6	2.23	1 - 12%
109356-03-34	510	Normal	579	596	2	0.00	1 - 1%
109356-03-35	525	Normal	594	611	5	2.30	1 - 13%
109356-03-36	540	Normal	611	629	7	4.68	16 - 33%
109356-03-37	555	Normal	604	621	7	3.27	12 - 22%
109356-03-38	570	Normal	609	627	4	4.21	17 - 26%
109356-03-39	585	Normal	596	613	11	2.40	1 - 18%
109356-03-40	600	Normal	600	617	5	2.74	11 - 17%
109356-03-41	615	Normal	586	603	4	0.00	1 - 11%
109356-03-42	630	Normal	597	614	1	2.47	12 - 13%

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109356-03-43	645	Normal	586	603	8	0.00	1 - 11%
109356-03-44	660	Normal	590	607	7	2.20	1 - 12%
109356-03-45	675	Normal	596	613	6	2.40	11 - 14%
109356-03-46	690	Normal	596	613	2	2.40	11 - 12%
109356-03-47	705	Normal	592	609	8	2.23	1 - 13%
109356-03-48	720	Normal	583	600	6	0.00	1 - 1%
109356-03-49	735	Normal	599	616	6	2.64	11 - 17%
109356-03-50	750	Normal	586	603	5	0.00	1 - 11%
109356-03-51	765	Normal	588	605	8	0.00	1 - 12%
109356-03-52	780	Normal	594	611	6	2.30	1 - 13%
109356-03-53	795	Normal	593	610	2	2.26	11 - 12%
109356-03-54	810	Normal	595	612	5	2.34	11 - 13%
109356-03-55	825	Normal	610	628	10	4.44	14 - 36%
109356-03-56	840	Normal	603	620	5	3.12	13 - 19%
109356-03-57	855	Normal	599	616	6	2.64	11 - 16%
109356-03-58	870	Normal	595	612	5	2.34	11 - 13%

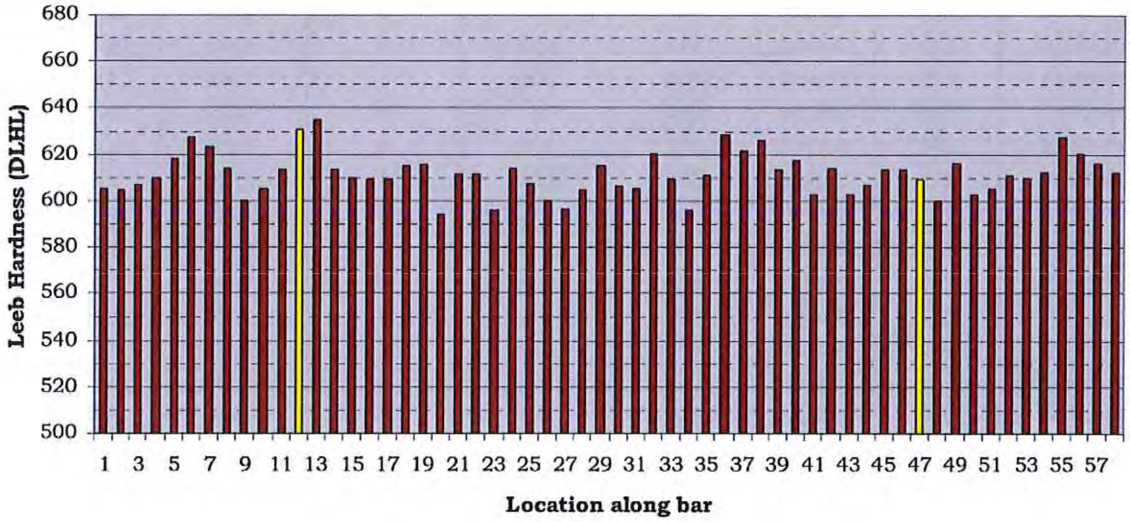

Figure 8 Leeb Hardness plot for Location 109356-3

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4.2.4. TEST LOCATION 109356-4

Test Location: 109356-4

Element: Concrete Wall

Floor: Room L240

Bar Reference 20 mm deformed bar

Average Potential Induced Strain (%): 5.56

Maximum Potential Loss of Strain Capacity (% peak): 18-41

Observation and Notes: Results showed a general saw-tooth action of highs and lows, common for steel with a number of transverse steel sections preventing continuous readings

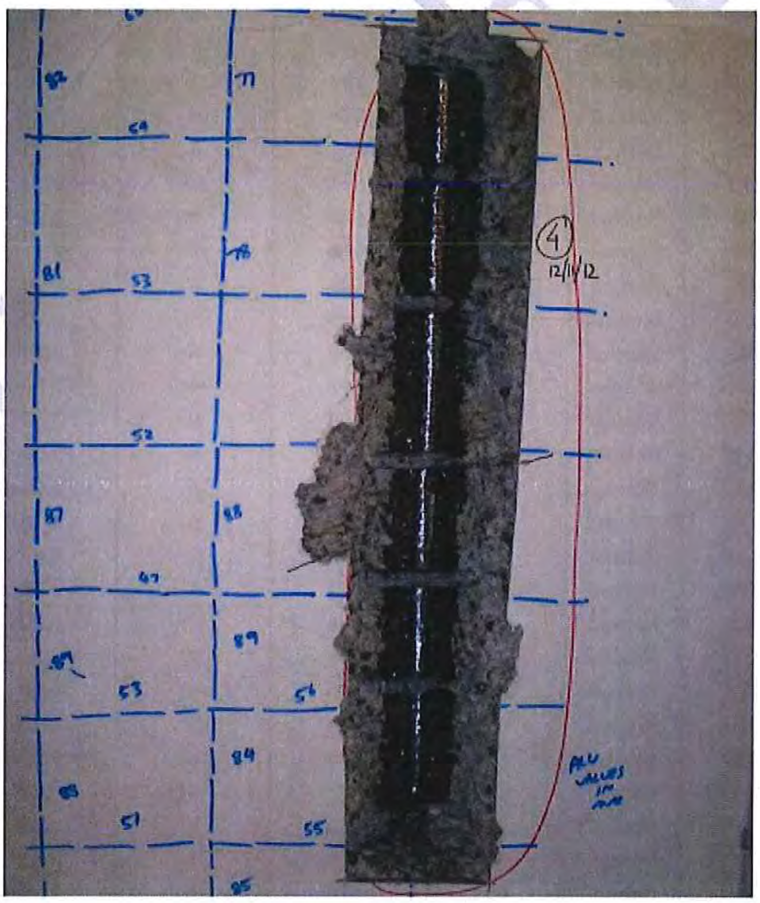

Figure 9 Test Location 109356-4

Table 5 Test Results for Location 109356-4

Reference	Location along bar (mm)	Bar Support Factor	Average Recorded Leeb (HLDL)	Normalised Leeb (HLDL)	Leeb Hardness Uncertainty (+/- HLDL)	Average Potential Induced Strain (%)	Potential Lost Strain Capacity (%)
109356-04-01	15	High	627	612	7	2.36	1 - 15%
109356-04-02	30	Normal	612	610	5	2.26	1 - 12%
109356-04-03	45	High	629	614	3	2.48	11 - 13%
109356-04-04	60	High	631	616	2	2.65	12 - 14%
109356-04-05	75	High	634	619	6	2.98	12 - 19%
109356-04-06	90	Normal	620	618	5	2.82	12 - 17%
109356-04-07	105	Normal	632	630	3	5.01	21 - 29%
109356-04-08	120	Normal	627	625	2	3.89	17 - 21%
109356-04-09	135	Normal	633	631	6	5.28	19 - 35%
109356-04-10	150	Normal	634	632	8	5.56	18 - 41%
109356-04-11	165	Normal	614	612	13	2.34	1 - 19%
109356-04-12	180	Normal	630	628	6	4.53	17 - 29%
109356-04-13	195	Normal	624	622	6	3.36	13 - 22%
109356-04-14	210	Normal	629	627	3	4.30	18 - 24%
109356-04-15	225	Normal	626	624	5	3.70	15 - 23%
109356-04-16	240	Normal	620	618	8	2.82	11 - 20%
109356-04-17	255	Normal	626	624	3	3.70	16 - 21%
109356-04-18	270	Normal	623	621	10	3.21	11 - 25%
109356-04-19	285	Normal	611	609	8	2.23	1 - 13%
109356-04-20	300	Normal	618	616	7	2.61	11 - 17%
109356-04-21	315	Normal	608	606	8	2.20	1 - 12%
109356-04-22	330	Medium	583	593	14	0.00	1 - 11%
109356-04-23	345	Normal	611	609	6	2.23	1 - 12%
109356-04-24	360	Medium	587	597	4	0.00	1 - 1%
109356-04-25	375	Normal	607	605	9	0.00	1 - 12%
109356-04-26	390	Normal	615	613	4	2.39	11 - 13%
109356-04-27	405	Normal	609	607	8	2.20	1 - 12%
109356-04-28	420	Normal	631	629	6	4.76	17 - 31%
109356-04-29	435	Normal	620	618	2	2.82	13 - 15%
109356-04-30	450	Normal	613	611	4	2.29	11 - 12%
109356-04-31	465	Normal	600	598	12	0.00	1 - 11%
109356-04-32	480	Normal	610	608	2	2.21	1 - 11%
109356-04-33	495	Normal	597	595	6	0.00	1 - 1%
109356-04-34	510	Normal	616	614	5	2.45	11 - 14%
109356-04-35	525	Normal	610	608	5	2.21	1 - 12%
109356-04-36	540	Normal	623	621	4	3.21	13 - 19%
109356-04-37	555	Normal	615	613	1	2.39	11 - 12%
109356-04-38	570	Normal	618	616	5	2.61	11 - 15%
109356-04-39	585	Normal	612	610	6	2.26	1 - 13%
109356-04-40	600	Normal	613	611	4	2.29	11 - 12%
109356-04-41	615	Normal	600	598	11	0.00	1 - 11%
109356-04-42	630	Normal	623	621	5	3.21	13 - 20%

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109356-04-43	645	Normal	614	612	2	2.34	11 - 12%
109356-04-44	660	Normal	608	606	8	2.20	1 - 12%
109356-04-45	675	Normal	631	629	6	4.76	17 - 32%
109356-04-46	690	Normal	620	618	7	2.82	11 - 19%
109356-04-47	705	Normal	606	604	4	0.00	1 - 11%
109356-04-48	720	High	639	624	7	3.74	14 - 26%
109356-04-49	735	Normal	617	615	1	2.53	12 - 13%
109356-04-50	750	Normal	604	602	2	0.00	1 - 1%
109356-04-51	765	Normal	606	604	19	0.00	1 - 17%
109356-04-52	780	High	618	610	4	2.25	11 - 12%
109356-04-53	795	High	624	610	6	2.24	1 - 13%
109356-04-54	810	High	638	623	7	3.56	13 - 24%
109356-04-55	825	High	628	613	9	2.42	1 - 17%
109356-04-56	840	High	637	616	1	2.60	12 - 13%

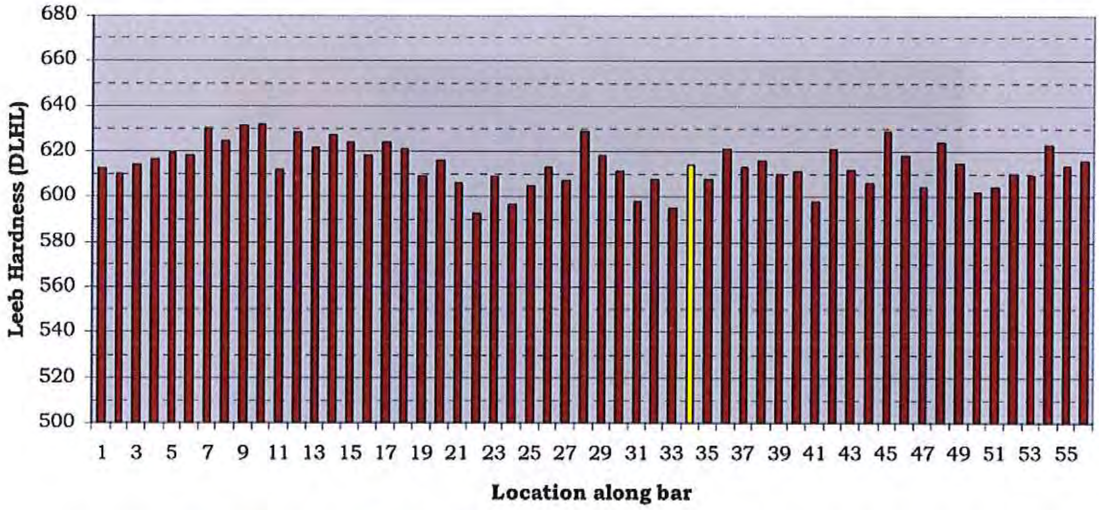

Figure 10 Leeb Hardness plot for Location 109356-4

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4.2.5. TEST LOCATION 109356-5

Test Location: 109356-5

Element: Concrete Wall

Floor: Room L240

Bar Reference 24 mm deformed bar

Average Potential Induced Strain (%): 5.03

Maximum Potential Loss of Strain Capacity (% peak): 20-30

Observation and Notes:

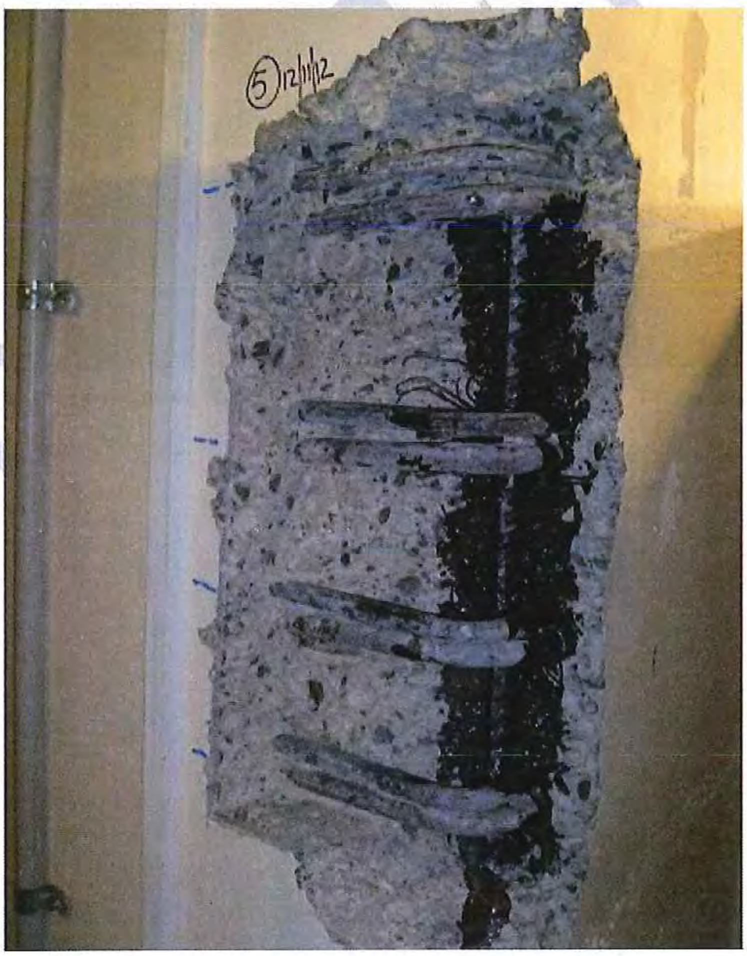

Figure 11 Test Location 109356-5

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Table 6 Test Results for Location 109356-5

Reference	Location along bar (mm)	Bar Support Factor	Average Recorded Leeb (HLDL)	Normalised Leeb (HLDL)	Leeb Hardness Uncertainty (+/- HLDL)	Average Potential Induced Strain (%)	Potential Lost Strain Capacity (%)
109356-05-01	15	Normal	600	600	3	0.00	1 - 1%
109356-05-02	30	Normal	603	603	4	0.00	1 - 11%
109356-05-03	45	Normal	603	603	3	0.00	1 - 1%
109356-05-04	60	Normal	618	618	4	2.82	12 - 16%
109356-05-05	75	Normal	615	615	3	2.53	11 - 14%
109356-05-06	90	Normal	621	621	6	3.21	12 - 21%
109356-05-07	105	Normal	623	623	4	3.53	15 - 21%
109356-05-08	120	Normal	630	630	4	5.03	20 - 30%
109356-05-09	135	Normal	614	614	6	2.46	11 - 15%
109356-05-10	150	Normal	606	606	5	2.20	1 - 11%
109356-05-11	165	Normal	611	611	3	2.29	11 - 12%
109356-05-12	180	Normal	619	619	2	2.94	13 - 16%
109356-05-13	195	Normal	620	620	2	3.07	14 - 17%
109356-05-14	210	Medium	595	607	3	2.20	1 - 11%
109356-05-15	225	Medium	596	608	4	2.21	1 - 11%
109356-05-16	240	Normal	614	614	3	2.46	11 - 13%
109356-05-17	255	Normal	611	611	6	2.29	1 - 13%
109356-05-18	270	Normal	621	621	8	3.21	12 - 24%
109356-05-19	285	Normal	629	629	4	4.78	19 - 29%
109356-05-20	300	Normal	626	626	5	4.10	15 - 26%
109356-05-21	315	Normal	635	635	7	6.49	22 - 44%
109356-05-22	330	Normal	622	622	3	3.37	14 - 19%
109356-05-23	345	Normal	621	621	4	3.21	13 - 19%

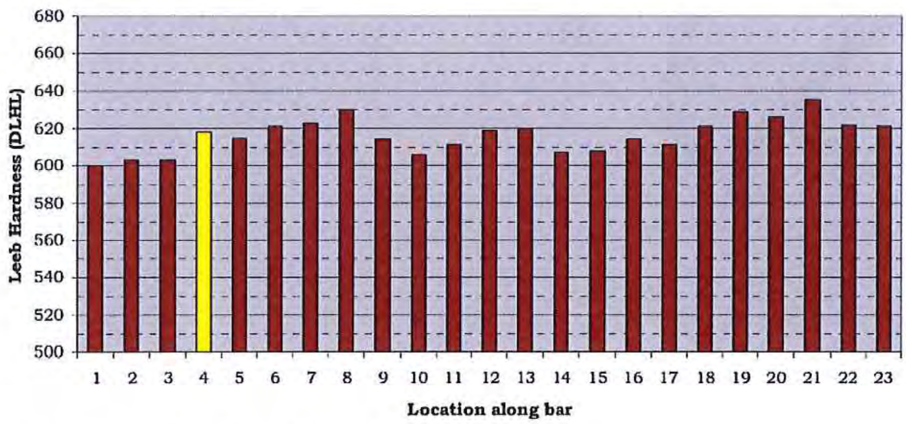

Figure 12 Leeb Hardness plot for Location 109356-5

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TEST LOCATION 109356-12A

Test Location: 109356-12A

Element: Concrete Floor

Floor: Upper side of Lower Ground Floor slab

Bar Reference 16 mm deformed bar

Average Potential Induced Strain (%): 5.75

Maximum Potential Loss of Strain Capacity (% peak): 22-35

Observation and Notes:

Figure 13 Test Location 109356-12A

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Table 7 Test Results for Location 109356-12A

Reference	Location along bar (mm)	Bar Support Factor	Average Recorded Leeb (HLDL)	Normalised Leeb (HLDL)	Leeb Hardness Uncertainty (+/- HLDL)	Average Potential Induced Strain (%)	Potential Lost Strain Capacity (%)
109356-12A-01	15	Normal	628	616	5	2.60	11 - 15%
109356-12A-02	30	Normal	627	615	5	2.52	11 - 15%
109356-12A-03	45	Normal	626	614	6	2.45	11 - 15%
109356-12A-04	60	Normal	645	633	4	5.73	22 - 35%
109356-12A-05	75	Normal	615	603	5	0.00	1 - 11%
109356-12A-06	90	Normal	623	611	2	2.29	11 - 12%
109356-12A-07	105	Normal	626	614	5	2.45	11 - 14%
109356-12A-08	120	Normal	635	623	3	3.48	15 - 20%
109356-12A-09	135	Normal	630	618	8	2.80	11 - 20%
109356-12A-10	150	Normal	635	623	5	3.48	14 - 22%
109356-12A-11	165	Normal	622	610	6	2.26	1 - 13%
109356-12A-12	180	Normal	625	613	4	2.39	11 - 13%
109356-12A-13	195	Normal	621	609	7	2.23	1 - 13%
109356-12A-14	210	Normal	638	626	6	4.03	15 - 26%
109356-12A-15	225	Normal	639	627	6	4.23	16 - 28%
109356-12A-16	240	Medium	607	607	5	2.20	1 - 11%
109356-12A-17	255	Medium	609	609	7	2.23	1 - 13%
109356-12A-18	270	Medium	607	607	1	2.20	11 - 11%
109356-12A-19	285	Medium	607	607	7	2.20	1 - 12%

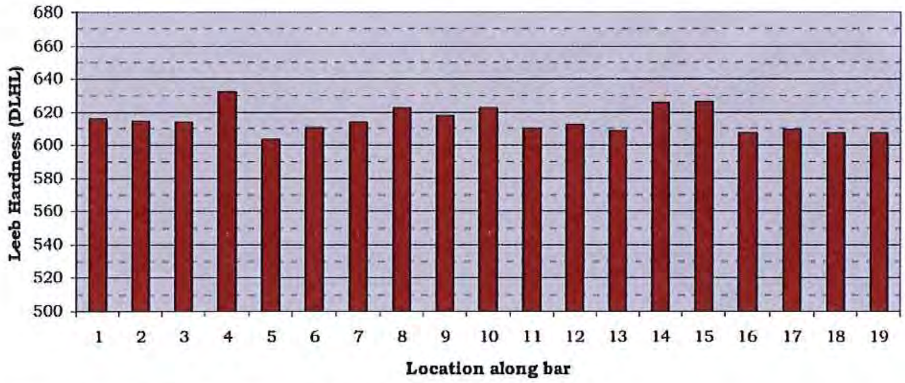

Figure 14 Leeb Hardness plot for Location 109356-12A

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4.2.6. TEST LOCATION 109356-12B

Test Location: 109356-12B

Element: Concrete Floor

Floor: Upper side of Lower Ground Floor slab

Bar Reference 16 mm deformed bar

Average Potential Induced Strain (%): 3.63

Maximum Potential Loss of Strain Capacity (% peak): 12-27

Observation and Notes:

Figure 15 Test Location 109356-12B

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Table 8 Test Results for Location 109356-12B

Reference	Location along bar (mm)	Bar Support Factor	Average Recorded Leeb (HLDL)	Normalised Leeb (HLDL)	Leeb Hardness Uncertainty (+/- HLDL)	Average Potential Induced Strain (%)	Potential Lost Strain Capacity (%)
109356-12B-01	15	Normal	632	615	7	2.52	11 - 16%
109356-12B-02	30	Normal	630	613	5	2.38	11 - 14%
109356-12B-03	45	Normal	618	601	5	0.00	1 - 1%
109356-12B-04	60	Normal	627	610	3	2.26	11 - 12%
109356-12B-05	75	Normal	628	611	5	2.29	1 - 13%
109356-12B-06	90	Normal	631	614	2	2.44	11 - 13%
109356-12B-07	105	Normal	632	615	5	2.52	11 - 15%
109356-12B-08	120	Normal	618	601	5	0.00	1 - 11%
109356-12B-09	135	Normal	631	614	5	2.44	11 - 14%
109356-12B-10	150	Normal	621	604	10	0.00	1 - 12%
109356-12B-11	165	Normal	626	609	5	2.23	1 - 12%
109356-12B-12	180	Normal	630	613	5	2.38	11 - 14%
109356-12B-13	195	Normal	629	612	0	2.33	11 - 11%
109356-12B-14	210	Normal	636	619	11	2.90	11 - 24%
109356-12B-15	225	Normal	634	617	2	2.69	12 - 14%
109356-12B-16	240	Normal	626	609	11	2.23	1 - 15%
109356-12B-17	255	Normal	631	614	1	2.44	12 - 12%
109356-12B-18	270	Normal	632	615	6	2.52	11 - 15%
109356-12B-19	285	Normal	629	612	3	2.33	11 - 12%
109356-12B-20	300	Normal	641	624	8	3.63	12 - 27%
109356-12B-21	315	Normal	641	624	2	3.63	16 - 19%
109356-12B-22	330	Normal	632	615	8	2.52	11 - 17%
109356-12B-23	345	Normal	640	623	8	3.46	12 - 25%
109356-12B-24	360	Normal	629	612	8	2.33	1 - 15%
109356-12B-25	375	Medium	608	603	5	0.00	1 - 11%

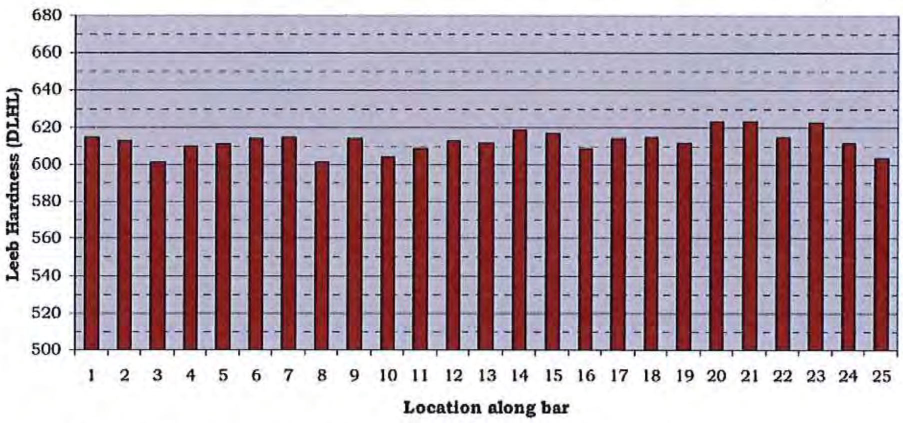

Figure 16 Leeb Hardness plot for Location 109356-12B

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4.2.7. TEST LOCATION 109356-13

Test Location: 109356-13

Element: Concrete Floor

Floor: Underside of Lower Ground Floor Slab

Bar Reference 16 mm deformed bar

Average Potential Induced Strain (%): 3.61

Maximum Potential Loss of Strain Capacity (% peak): 15-21

Observation and Notes:

Figure 17 Test Location 109356-13

Table 9 Test Results for Location 109356-13

Reference	Location along bar (mm)	Bar Support Factor	Average Recorded Leeb (HLDL)	Normalised Leeb (HLDL)	Leeb Hardness Uncertainty (+/- HLDL)	Average Potential Induced Strain (%)	Potential Lost Strain Capacity (%)
109356-13-01	15	Normal	589	607	6	2.20	1 - 12%
109356-13-02	30	Normal	582	600	8	0.00	1 - 11%
109356-13-03	45	Normal	579	597	7	0.00	1 - 1%
109356-13-04	60	Normal	591	609	10	2.23	1 - 14%
109356-13-05	75	Normal	589	607	3	2.20	1 - 11%
109356-13-06	90	Normal	583	601	6	0.00	1 - 11%
109356-13-07	105	Normal	570	587	4	0.00	1 - 1%
109356-13-08	120	Normal	578	596	8	0.00	1 - 1%
109356-13-09	135	Normal	583	601	2	0.00	1 - 1%
109356-13-10	150	Normal	578	596	5	0.00	1 - 1%
109356-13-11	165	Normal	567	584	5	0.00	1 - 1%
109356-13-12	180	Normal	575	593	5	0.00	1 - 1%
109356-13-13	195	Normal	575	593	5	0.00	1 - 1%
109356-13-14	210	Normal	572	589	8	0.00	1 - 1%
109356-13-15	225	Normal	588	606	5	2.20	1 - 11%
109356-13-16	240	Normal	605	623	4	3.61	15 - 21%
109356-13-17	255	Normal	587	605	11	0.00	1 - 13%
109356-13-18	270	Normal	590	608	4	2.21	1 - 11%
109356-13-19	285	High	591	603	9	0.00	1 - 11%
109356-13-20	300	High	599	605	7	0.00	1 - 11%
109356-13-21	315	High	602	608	15	2.21	1 - 18%

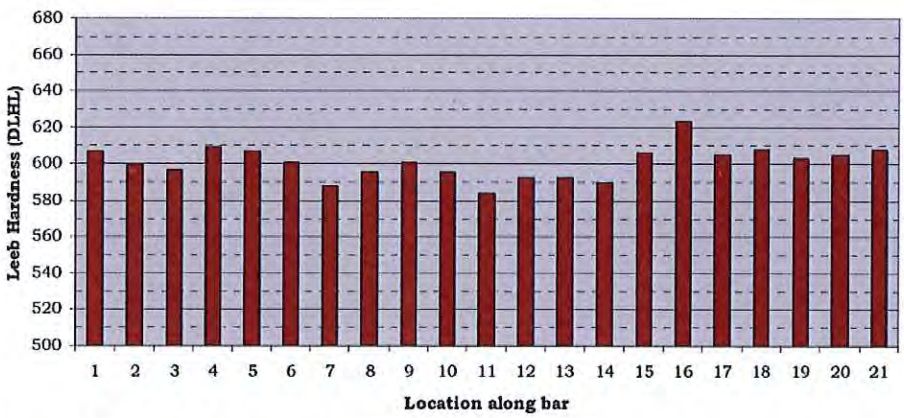

Figure 18 Leeb Hardness plot for Location 109356-13

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4.2.8. TEST LOCATION 109356-14

Test Location: 109356-14

Element: Concrete Floor

Floor: Underside of Lower Ground Floor Slab

Bar Reference 16 mm deformed bar

Average Potential Induced Strain (%): 4.22

Maximum Potential Loss of Strain Capacity (% peak): 16-27

Observation and Notes: Bar was noted to have variable support conditions along the length of the bar resulting in increased scatter to the recorded results and less certainty in the derived results

Figure 19 Test Location 109356-14

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Table 10 Test Results for Location 109356-14

Reference	Location along bar (mm)	Bar Support Factor	Average Recorded Leeb (HLDL)	Normalised Leeb (HLDL)	Leeb Hardness Uncertainty (+/- HLDL)	Average Potential Induced Strain (%)	Potential Lost Strain Capacity (%)
109356-14-01	15	Normal	594	612	3	2.34	11 - 13%
109356-14-02	30	Normal	587	605	4	0.00	1 - 11%
109356-14-03	45	Normal	581	599	6	0.00	1 - 1%
109356-14-04	60	Normal	584	602	5	0.00	1 - 11%
109356-14-05	75	Medium	563	592	6	0.00	1 - 1%
109356-14-06	90	Medium	564	593	8	0.00	1 - 1%
109356-14-07	105	Normal	568	585	3	0.00	1 - 1%
109356-14-08	120	Medium	569	598	3	0.00	1 - 1%
109356-14-09	135	Medium	562	591	7	0.00	1 - 1%
109356-14-10	150	Medium	560	589	5	0.00	1 - 1%
109356-14-11	165	Normal	580	598	9	0.00	1 - 11%
109356-14-12	180	Medium	561	590	5	0.00	1 - 1%
109356-14-13	195	Normal	578	596	9	0.00	1 - 1%
109356-14-14	210	Normal	571	588	9	0.00	1 - 1%
109356-14-15	225	Normal	575	593	8	0.00	1 - 1%
109356-14-16	240	Normal	599	617	7	2.74	11 - 19%
109356-14-17	255	Normal	602	620	5	3.12	13 - 19%
109356-14-18	270	Medium	574	603	7	0.00	1 - 11%
109356-14-19	285	Normal	595	613	7	2.40	1 - 15%
109356-14-20	300	Normal	608	627	5	4.22	16 - 27%
109356-14-21	315	Normal	584	602	7	0.00	1 - 11%

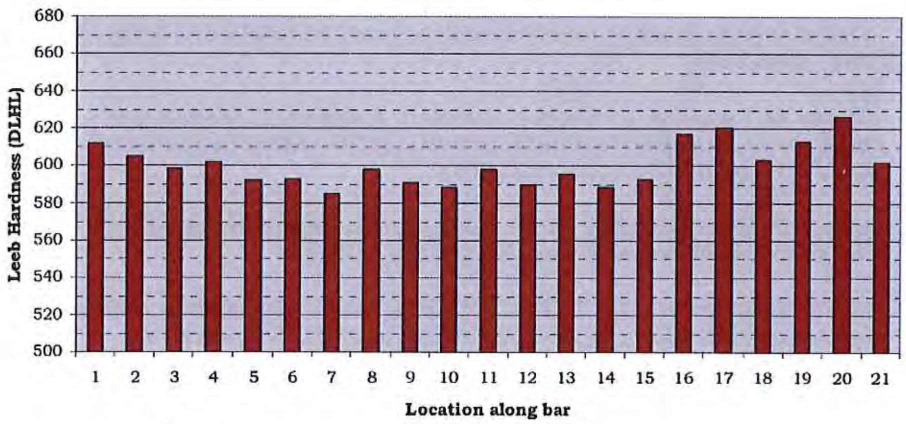

Figure 20 Leeb Hardness plot for Location 109356-14

5.0 REFERENCES

[A1] ASTM International, *"Standard Test Method for Indentation Hardness of Metallic Materials by Portable Hardness Testers"*, ASTM Designation E11010, 2010

[A2] ASTM International, *"Standard Test Method for Leeb Hardness Testing of Steel Products"*, ASTM Designation A956-06, 2006

[E1] Eun-chae. J, Park. J, Choi.D, Kim.K, Kwon. D, *"A method for estimating Uncertainty of Indentation Tensile Properties in Instrumented Indentation Tests"*, Journal of Engineer Materials and Technology, Vol 131, July 2009

[I1] ISO/IEC Guide 98-3, *"Uncertainty in Measurement – Part 3: Guide to the expression of Uncertainty in Measurement"*, International Organization for Standardization, Geneva, Switzerland

[K1] Kapoor, *"A re-evaluation of the life to rupture of ductile metals by cyclic plastic strain"*, Fatigue and Fractures of Engineering Materials and Structures, Vol 17, No 2, pp 201-219, 1994

[L1] Lemaitre Dufailly, *"Damage Measurements"*, Engineering Fracture Mechanics, Vol 28, pp 643-661, 1987

[M1] Mkaddem, Gassara, Hambli, *"A new procedure using micro-hardness technique for sheet material damage characterisation"*, Journal of Material Processing Technology, 178, 2006, pg 111-118

[M2] Matsumoto Y. *"Study on the residual deformation capacity of plastically strained steel"*, Taylor Francis Group, London, 2009

[N1] Nishikawa, Soyama, *"Two step method to evaluate equibiaxial residual stress of metal surface based on micro-indentation tests"*, Materials and Design 32, 2011, 3240-3247

[N2] Nakane. M, Kanno. S, Kurosaki.Y, Komotori. J, Ogawa. T, *"Accuracy of Plastic Strain Estimated by Hardness to Assess Remaining Fatigue Lives"*, Material Properties Measurements, 2011.

[P1] Pirondi, Bonora, *"Modelling ductile damage under fully reversed cycling"*, Computational Materials Science 26, 2003, pg 129-141

[S1] Suzi. Y, Tokinoya. H, Asai. H, Eto. H, *"Damage estimation of shear yielding type hysteretic dampers by hardness measurements"*, Summaries of Technical Papers of Annual Meeting, AIJ, c-1: 557-558, 2003

[T1] Tasan, Hoefnagels, *"A critical assessment of indentation based ductile damage quantification"*, Acta Materialia, Volume 57, Issue 17, October 2009, Pages 4957, 4966

[T2] Tasan, Hoefnagels, Geers, *"Indentation-based damage quantification revisited"*, Scipta Materialia 63, 2010, pg 316-319

[Y1] Yabe. Y, Ishikawa. N, *"Experimental study on the estimation of plastic strain damage in steel structures due to earthquakes by hardness measurements"*, Summaries of Technical Papers Annual Meeting AIJ: 1079-1080, 1985

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APPENDIX E

67% DBE Strengthening Schemes Memorandum

HolmesConsultingGroup

STRUCTURAL AND CIVIL ENGINEERS

MEMORANDUM

To:	Graham Bugler
Company:	Canterbury District Health Board
From:	Renee Brook/Lisa Oliver
Date	9 October 2015 Project No: 106186.20
Subject:	CDHB - PARKSIDE - 67 % IL4 STRENGTHENING SCHEMES - REVISION 2

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We have been working on Strengthening Scheme Concepts for the Parkside buildings at the Christchurch Hospital. The brief was to investigate four different schemes based on Importance Level 4 (IL4) loading:

a) 67 % IL4 Design Basis Earthquake (DBE) Loading (equivalent to 93 % IL3 DBE loading)
b) 67 % IL4 DBE (93 % IL3 DBE) plus increase in redundancy.
c) 67 % IL4 DBE (93 % IL3 DBE) plus increase in redundancy and CDHB IL3 - SLS2 requirements.
d) 100 % IL4 DBE plus increase in redundancy.

Because of the similarities between the buildings we have been concentrating our efforts looking at Block A, these concepts have then been extrapolated out to all the Blocks. When the strengthening design develops past concept stage we will assess each building individually. In particular, Block B will need to be analysed separately due to the difference in geometry and wall detailing.

This memo expands on what is likely required for the first two schemes to allow these to be priced. This memo has been updated in Revision 2 to include the new drift requirements for the stairs and updated strengthening recommendations.

1 STRENGTHENING CONCEPT A - 67 % IL4 DESIGN BASIS EARTHQUAKE (DBE) LOADING (EQUIVALENT TO 93 % IL3 DBE LOADING)

1.1 Primary Structure

The current analysis indicates that the Ultimate Limit State (ULS) capacity of the primary structure is 78% DBE loading. Therefore, technically to achieve this strengthening objective, no strengthening is required to the primary structure.

The analyses of the primary structures assume that the buildings are not restrained laterally at the Ground Floor Level. We have observed evidence that this is the case for Block A (movement between the east and south retaining walls and the Ground Floor has been observed); however, this condition will need to be confirmed for the other

Blocks (especially Block D which also has retaining walls up to the Ground Floor on the west and south sides).

The models indicate that the buildings move around 30 mm at Ground floor under 67 % IL4 DBE loading, therefore some work might be required to ensure the buildings are free to move at this level, such as leaving a gap between the buildings and exterior hard surface finishes. The following sketches indicate where this gap is required.

ME10 – SK001 - Lateral Movement allowance at Ground Floor Level - Typical Sections through Block A

ME10 – SK002 - Lateral Movement allowance at Ground Floor Level – Lower Ground Floor Plan

ME10 – SK003 - Lateral Movement allowance at Ground Floor Level – Ground Floor Plan

1.2 New Concrete Walls

From Level 2 and above the wall layout of the shear cores typically reduces from two walls coupled together on each face of the core to one cantilever wall on each face of the shear core, refer Figure 1-1 below. This reduces the stiffness of the building above Level 2 and increases the drifts expected in a seismic event. Currently the drifts at the roof level are predicted to be as high as 2.5% (the code limit) in specific locations.

The development of the strengthening options for the stairs and precast cladding panels has shown that strengthening to 67% IL4 can not be readily achieved above Level 2 due to the large drifts above Level 2. As such the building is required to be strengthened and stiffened to reduce these drifts and enable the stairs and precast cladding panels to be strengthened. Several solutions for strengthening the building above Level 2 have been proposed to date, some have been investigated and found to not produce the required reduction in drifts required by the stairs and precast panels. The solution investigated and proposed herein involves adding a second cantilever concrete wall into each shear core wall above Level 2, as shown in red in Figure 1-1. A steel braced frame option instead of concrete walls could be considered during Developed Design.

Adding the new walls to each face of the shear core above Level 2 increases the stiffness of the building and reduces the displacements to a level where the stair and panel strengthening solutions can accommodate the interstorey drift.

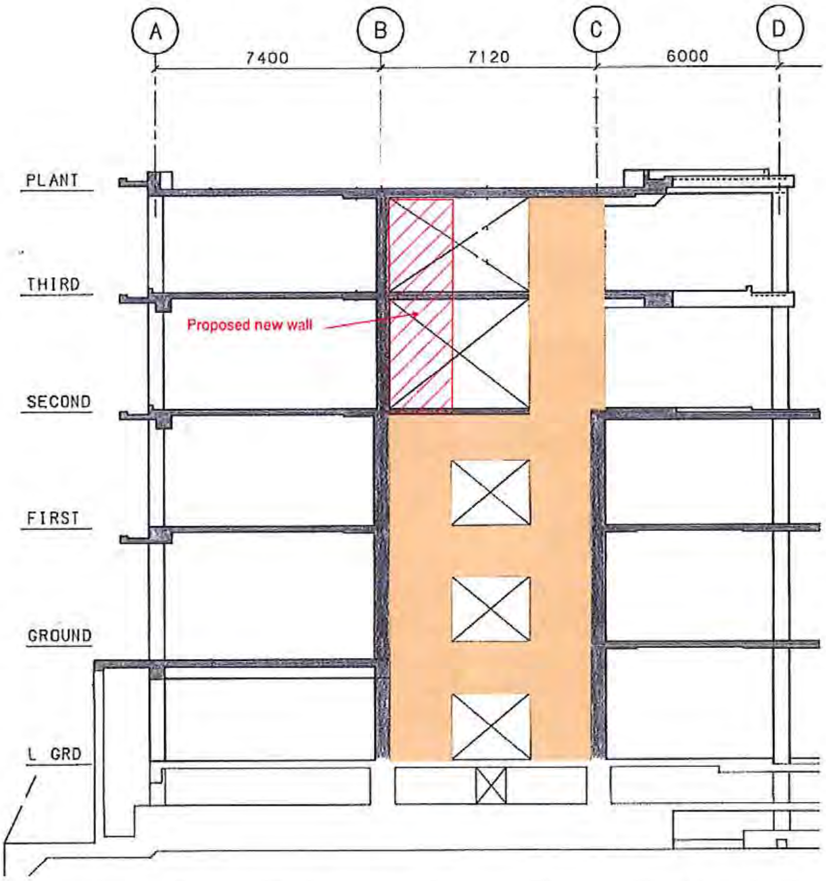


Figure 1-1:Typical Shear Wall Layout with Proposed New Wall.

Sketches showing the new concrete walls in Block A are shown in SK017, similar wall layouts will be required in Blocks C and D. Block B has a different wall layout (that does not reduce above Level 2) and further analysis work will be required to determine if any strengthening work is required here. Further analysis may show that new walls are not required for Block C as it does not have the additional concrete plant level of Blocks A and D.

ME10 – SK017 – Locations of new Concrete Walls – Typical All Blocks.

1.3 Emergency Department Extension (EDE)

Although the primary structure appears to generally achieve ULS at 67 % DBE loading, strengthening is still required to the Emergency Department Extension. The EDE is a two level structure that extends out the north side of Block A. The NLTHA indicates that the capacity of this building is only around 30 % of Design Basis Earthquake loading. Review of this structure indicates that numerous elements fail at a similar capacity and therefore require strengthening.

To increase the capacity to above 67 % DBE loading for an IL4 structure would involve replacing the existing roof and wall cross bracing, as well as installing some additional bracing. By ensuring a desirable hierarchy of failure for the new elements it is possible to ensure Critical Structural Weaknesses do not govern the performance.

A concept strengthening scheme is indicated in the following sketches.

ME10 – SK004 – Emergency Department Extension – Roof Plan

ME10 – SK005 – Emergency Department Extension – Ground Floor Plan

ME10 – SK006 – Emergency Department Extension – New Roof Connection Detail

1.4 Link Bridges

The link bridges cantilever horizontally from the Parkside Blocks and have the capacity to resist approximately 33% of the IL4 Design Basis Earthquake loading, it has also been noted that the capacity of Link B is only 85 % under the maximum gravity load case. Multiple concept options have been investigated to strengthen or replace the link bridges; we understand these have been priced separately.

1.5 Precast Cladding Panels

The precast cladding panels were designed withconnections with oversized holes to allow for some interstorey movements. The installation of the panels has however limited the ability of the panels to sustain the expected interstorey movements. The interstorey drift capacity of the panel connections is determined by the extent to which the oversized bolt hole tolerance has been taken up by construction tolerance. Where the oversized bolt hole tolerance is taken up by construction tolerance, it is likely that the panels will be damaged at an earthquake loading of less than 34% DBE. HCG recommend all of the Parkside Building cladding panels on the ground floor and above are considered during the strengthening work.

To allow for sufficient interstorey movements, we propose that the connections of the precast cladding panels be strengthened or replaced. The interstorey movement demands on the panel connections at ULS have been assessed for the existing buildings drift as approximately 95mm in each direction above Level 2. Detailing new panel connections to sustain this magnitude of movement is unlikely to be feasible due to the new Code requirements that the panels be detailed to allow for the inter-storey drifts factored up by 1.5/Sp (Sp is the structural performance factor in NZS1170.5). Consequently it is recommended that the panel strengthening be undertaken in conjunction with stiffening of the primary structure. To reduce the drifts above Level 2, the concrete core walls will be required to be extended above Level 2 as per section 1.2. Further analysis may determine that Block C does not require additional walls due to fact that it does not have an additional concrete plant level above third floor. This option is detailed in SK-017 in the attached documentation.

The strengthening concept for the precast panel connections is detailed to allow for the 67% IL4 inter-storey drift based on the building being strengthened above Level 2 to reduce the inter-storey drifts in the upper levels. A full concept for the remediation of the precast panels is outlined in the memo dated 01 July 2015 (HCG Job Number 113711).

The consequences of panel detachemtn during earthquake loading are a fundction of the panel location and this indicates the priority in which the panel strengthening should be addressed. The order of priority for precast panel strengthening are shown in the following sketch:

> ME10 – SK018 – Indicative priority levels for strengthening of precast panel connections

The programme for the works is governed by the priority order above, the timing of carrying out the works and access requirements. At the completion of the work, the overall capacity of the cladding system is likely to be greater than 67% DBE for an IL4 building, including a suitable margin (>1.5) between ULS and CLS behaviour, provided the building has been stiffened. A significantly improved SLS and SLS2 performance is likely to be able to be achieved during the stage 2 works.

1.6 Pounding with School of Medicine

Currently it is estimated that pounding between Block B and Block C with the School of Medicine Building commences at 50 % DBE loading. Full details of the pounding is outlined in the HCG memo titled CDHB – Parkside – Seismic Gap Between Parkside and School of Medicine Buildings, issued 13 April 2015.

The additional strengthening proposed to the Parkside Blocks above Level 2 (new concrete walls – refer section 1.2 above) reduces the seismic gap requirements between the School of Medicine and Parkside Blocks B&C but the buildings are still likely to pound at levels of load less than 67% DBE. By introducing the new walls in Parkside Blocks B & C above Level 2 the buildings are estimated to pound at approximately 62% DBE loading It is recommended that pounding should be prevented until loads above the ULS. In addition damage due to pounding to the School of Medicine Building could impact the occupancy of the Parkside Buildings.

To prevent the Parkside Buildings pounding with the School of Medicine Building at 67 % DBE loading the gap between the buildings needs to be increased at the Parkside Plant Room Level and Level 3. Figure 1-2 below shows the required seismic gaps if no strengthening of the Parkside buildings is undertaken.

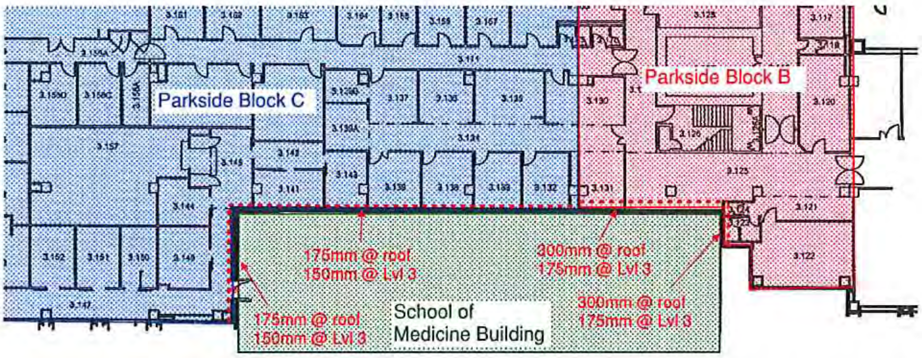


Figure 1-2: Seismic gap requirements between Parkside B&C and School of Medicine Building

If the Parkside buildings are strengthened the pounding is expected at approximately 62% IL4 DBE loading This reduces the requirements in Parkside B to approximately those of Parkside C. The reduction in seismic gap requirements to Parkside C is small and within the margin of error and safety factors already included in the calculations. These numbers are based on the analysis model of Parkside A extrapolated to the other Parkside Blocks. Specific analysis of Parkside C may show a reduction in the seismic gaps required.

The required separations are illustrated in the attached concept sketches.

ME10-SK011 – Section through the School of Medicine and Parkside Block B

ME10-SK012 – Level 3 Plan and edge details at Parkside Block B

ME10-SK013 – Plant Level Plan and edge details at Parkside Block B

ME10-SK014 – Section through the School of Medicine and Parkside Block C

ME10-SK015 – Level 3 Plan at Parkside Block C

ME10-SK016 – Plant Level Plan at Parkside Block C

The seismic gap at Level 3 can be increased by cutting back either the edge of the Parkside Blocks or School of Medicine concrete slab by approximately 100 mm. It is our understanding that cutting back the School of Medicine Building is preferred but is likely to be more complicated to achieve. To achieve this, the concrete block walls along the north elevation of the building will be required to be removed. The easiest way to achieve the required seismic gap is to cut back the slab overhang in Parkside Blocks B & C. The main structural elements are set back from the edge of the building and hence cutting back the slab can be achieved without compromising the main structural elements of the building. The work will involve alterations to consulting rooms and corridors shown in Figure 1-2.

At the Plant Level, the School of Medicine slab appears to be higher than the Parkside structure and therefore may not pound. Site investigations confirmed that the slab does indeed sit higher than the Parkside structure but that there is insufficient movement allowance between the two structures. The School of Medicine slab is supported by downstand cantilever beams. These beams have minimal clearance to the edge beam structure at the roof level of Parkside Blocks B and C. Cutting back the downstand beam to provide the required clearance would be the simplest option.

These sketches consider only the structural elements; it is likely that there will be architectural elements that also need to be modified to create the required separations. Waterproofing of the exterior façade at both levels will need to be addressed architecturally.

1.7 Pounding between adjacent Parkside Blocks

It is likely that the south sides of Blocks A and B will pound together at the upper levels below 67 % DBE loading. This will lead to increased accelerations at the upper levels of the building that could lead to increased damage to building services and contents.

The pounding between Blocks A and B should not significantly affect the structure (due to the similar heights of the structures and the floors being at the same level) and therefore does not affect the overall percent DBE. However, we could complete an assessment using the Non-Linear Time History Analysis that has been prepared for the Block A that looks at the floor accelerations produced by Block A and B pounding together. These accelerations could then be used to create a specification for the securing of building plant, services and high value building contents.

1.8 Stairs

The stairs were originally connected floor to floor such that there was no allowance for inter-storey drift. The work to seismically separate the stairs at the mid-height landing has been carried out for stairs 1, 3-7. The stairs are detailed to allow for the 67%IL4 inter-storey drift based on the building being strengthened above Level 2 to reduce the inter-storey drifts in the upper levels.

Code changes are underway that will require that the stairs be detailed to allow for the inter-storey drifts factored up by 1.5/Sp. Stair 2 separation has been completed with these new requirements.

To reduce the drifts above Level 2, the proposed strengthening scheme is to extend the core walls above Level 2 in all of the Blocks as detailed in Section 1.2.

2 STRENGTHENING CONCEPT B - 67 % IL4 DBE (93 % IL3 DBE) PLUS INCREASE IN REDUNDANCY

The ULS capacity of the un-strengthened primary structure is estimated to be 78 % DBE loading with a margin of 1.14 to the Collapse Limit State (CLS). To meet the objectives of this strengthening scheme, the margin to CLS would be increased to at least 1.5.

The difference between a margin of 1.14 and 1.5 seems small and therefore we have been attempting to develop a strengthening scheme that achieves this objective without causing too much disruption to the current hospital operations. This has not proved to be simple and more strengthening is required to achieve this than we estimated in our previous strengthening update memo (dated 2 August 2013).

2.1 Primary Structure

2.1.1 Lateral Movement Allowance at Ground Floor

As for Scheme A, we need to ensure that the buildings are not restrained laterally at the Ground Floor Level. For this scheme we need to achieve this at the load level of 1.5 times the ULS capacity. At this load level the models indicate the buildings move laterally around 60 mm at the Ground Floor Level. Therefore, works around the building to ensure this movement is accommodated are likely to be slightly more extensive than discussed above.

2.1.2 Clamping of Concrete Core Walls at Lower Ground Floor Level

We also need to improve the detailing of the main concrete core walls between Lower Ground Floor and Ground Floor. If these were new buildings, more horizontal steel reinforcing ties would be included in the wall. To achieve the same performance from the existing walls we need to clamp horizontal steel bars each side of the core walls. This will contain the concrete in the walls and limit buckling to the vertical steel reinforcement. By doing this, the rotations the walls can sustain before losing gravity load carrying capacity increase.

The location and detailing for the proposed new wall ties is shown in the following concept sketches:

ME10 – SK021 – Location of walls that require new clamping detail at Lower Ground Floor Level

ME10 – SK022 – Concrete Core Wall Clamping Details

2.1.3 Reducing Interstorey Drift above Level 2 in Block A

The other elements that exceed collapse limit state criteria prematurely are some of the concrete columns in the south wing of Block A above Level 2 (also the south of Block D and in Block B – due to the additional level of concrete and reduced core wall layout). The interstorey drifts in these areas are above acceptable limits and the columns can not sustain these drifts and maintain their gravity load carrying capacity.

Strengthening the building, as outlined above in Section 1.2, by extending the core walls above Level 2 to the top of each building reduces the drifts and therefore the demands on the concrete columns. As such, the new walls increase the resilience of the building and would be designed to achieve a load level of 1.5 times the ULS level whilst allowing the columns to remain below critical levels and continue to carry gravity loads.

2.2 Emergency Department Extension

The new structure required above to achieve strengthening scheme A, as per Section 1.3, should be sufficient to also increase margin between ULS and CLS for the Emergency Department Extension.

2.3 Link Bridges

The new structure required above to achieve strengthening scheme A, as per Section 1.4, should be sufficient to also increase margin between ULS and CLS for the Link Bridges.

2.4 Precast Cladding Panels

The new details and structures required above to achieve strengthening concept A, as per Section 1.5, should be sufficient to also increase the margin between ULS and CLS to approximately 1.5 for the Cladding Panels.

2.5 Pounding with School of Medicine

The new increased seismic gap required above to achieve strengthening scheme A, as per Section 1.6, should be sufficient for this strengthening scheme.

2.6 Stairs

The new seismic gaps required to achieve strengthening scheme A, as per Section 1.6, for stairs 1, 3-7 will need to be increased by 1.5/Sp.

The new structure required above to achieve strengthening scheme A will be sufficient to also increase the margin between ULS and CLS in conjunction with the increase stair gaps.

Renee Brook
PROJECT ENGINEER

106186.20ME1408.010.doc

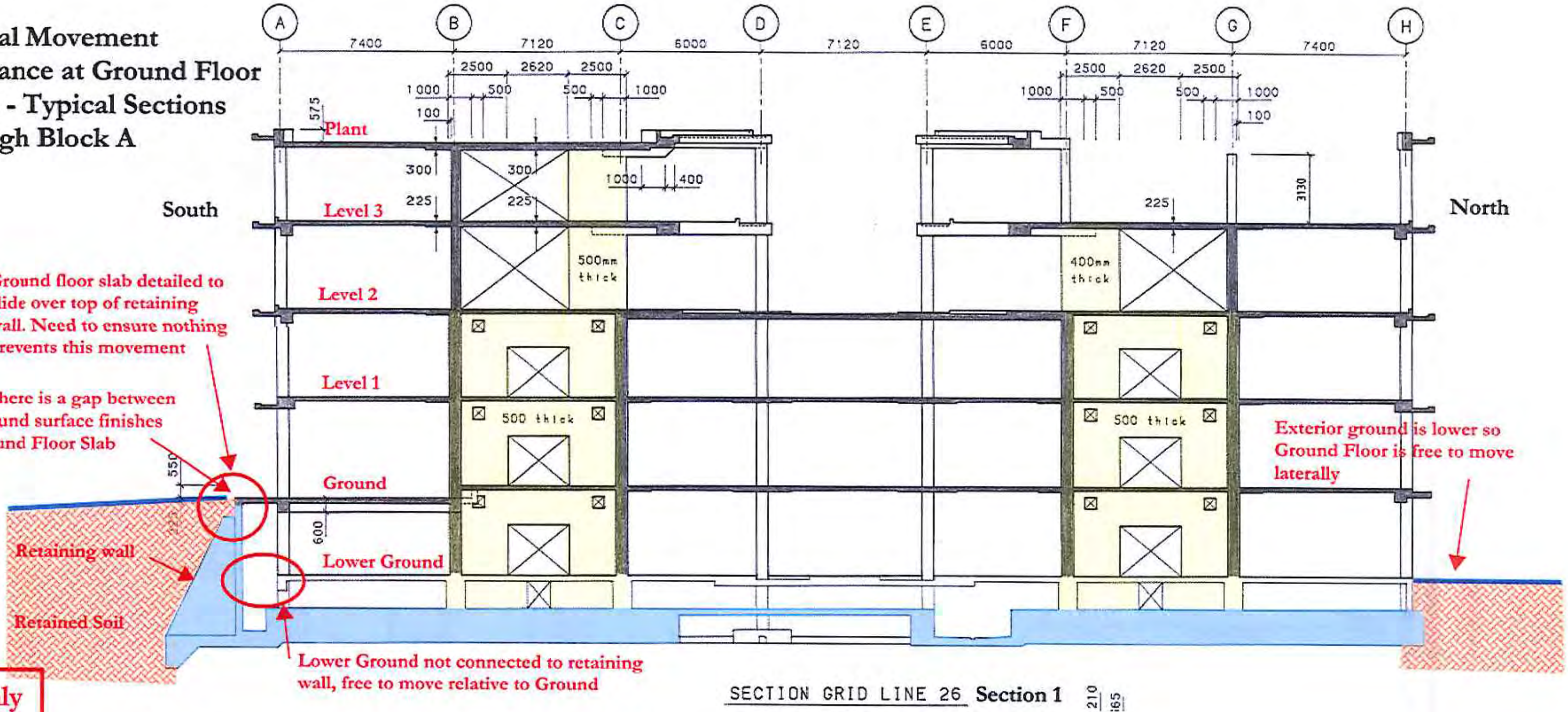
Lateral Movement allowance at Ground Floor Level - Typical Sections through Block A

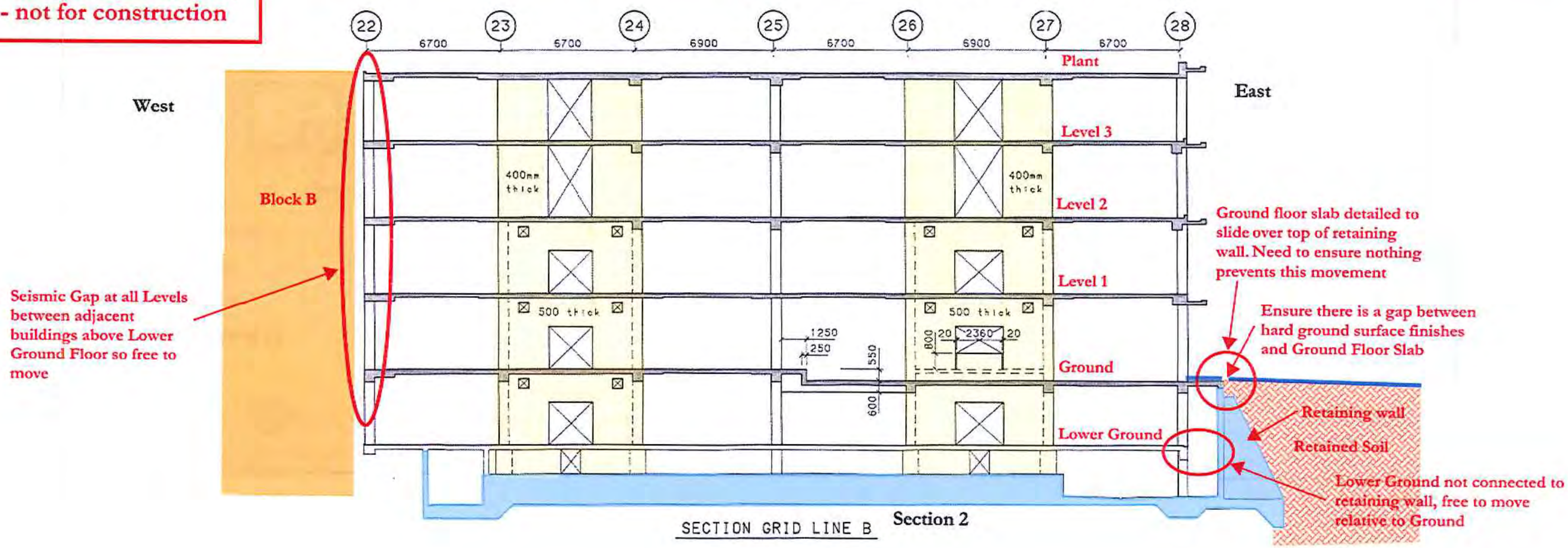
HolmesConsultingGroup

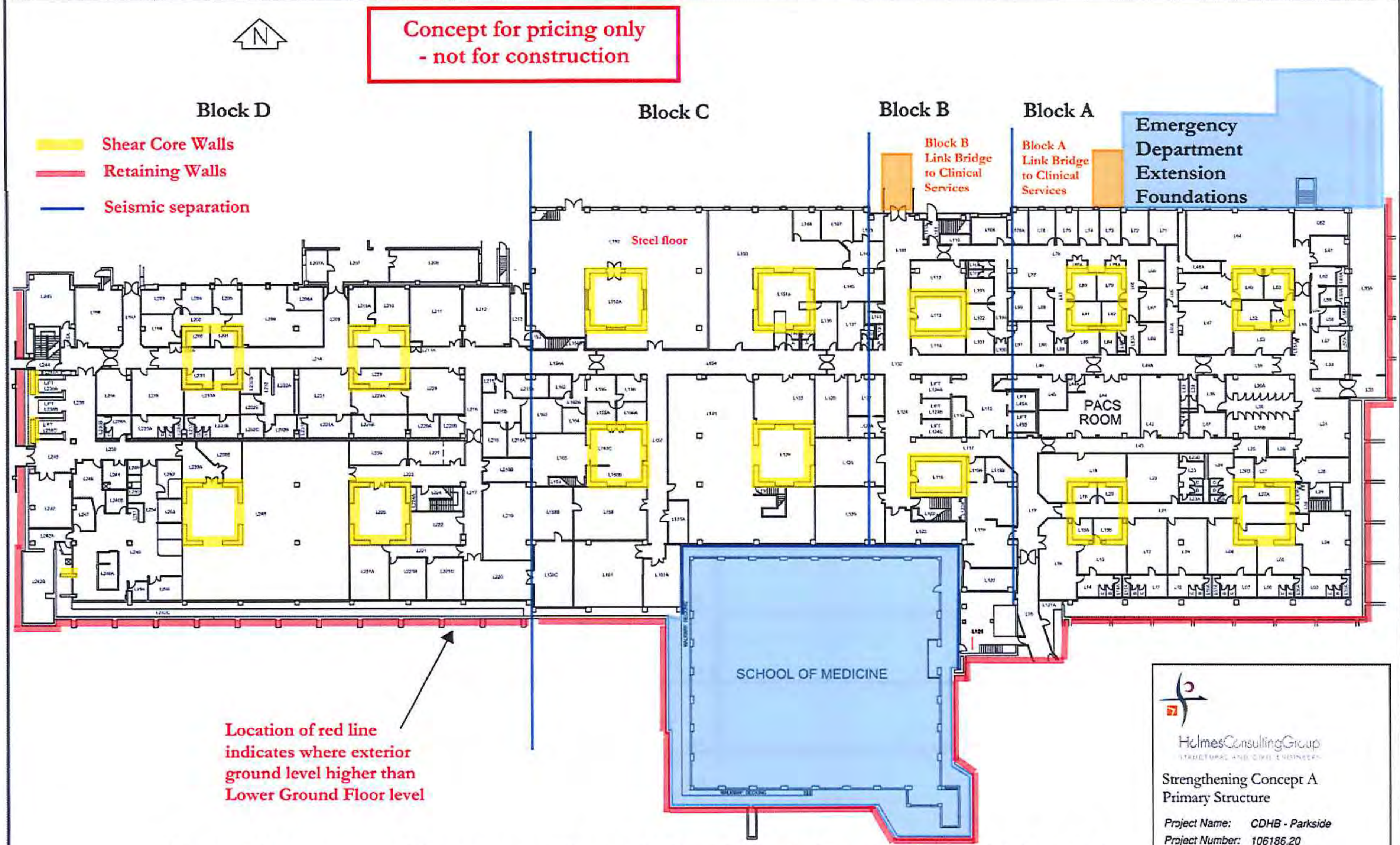
Strengthening Concept A)
Primary Structure

Project Name:	CDHB - Parkside
Project Number:	106186.20
Sketches By:	LJO
Date:	Aug 2013
Sketch Number:	ME10 - SK001

Concept for pricing only - not for construction







Lateral Movement allowance at Ground Floor Level - Lower Ground Floor Plan

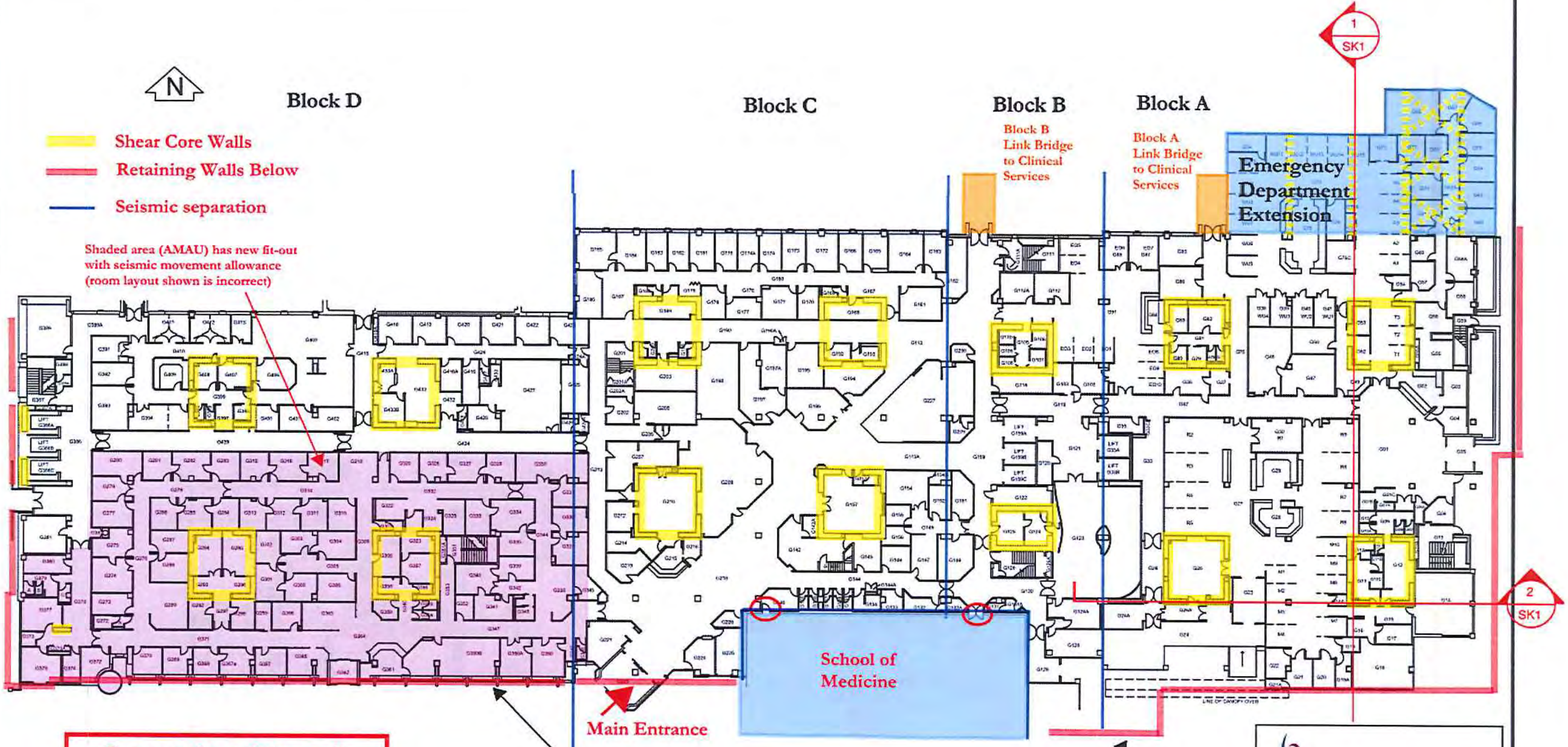
AMENDMENT	DRAWN	CHECKED	DATE
SCHOOL OF MEDICINE ADDED	B.T.		10-11-99
LINKS TO WOMENS HOSPITAL ADDED	B.T.		24-01-05
UPDATED	B.T.		09-05-05
UPDATED	B.J.T.		08-12-06

CHRISTCHURCH HOSPITAL
PARKSIDE COMBINED FLOOR PLAN
LOWER GROUND FLOOR

HolmesConsultingGroup
STRUCTURAL AND CIVIL ENGINEERS

Strengthening Concept A
Primary Structure

Project Name: CDHB - Parkside
Project Number: 106186.20
Sketches By: LJO
Date: Aug 2013
Sketch Number: ME10 - SK002



Concept for pricing only
- not for construction

Approximate location of edge of Ground Floor slab. There needs to be gap in hard surface finishes at this point to allow the Ground Floor to move laterally relative to the surrounding ground

Lateral Movement allowance at Ground Floor Level - Ground Floor Plan

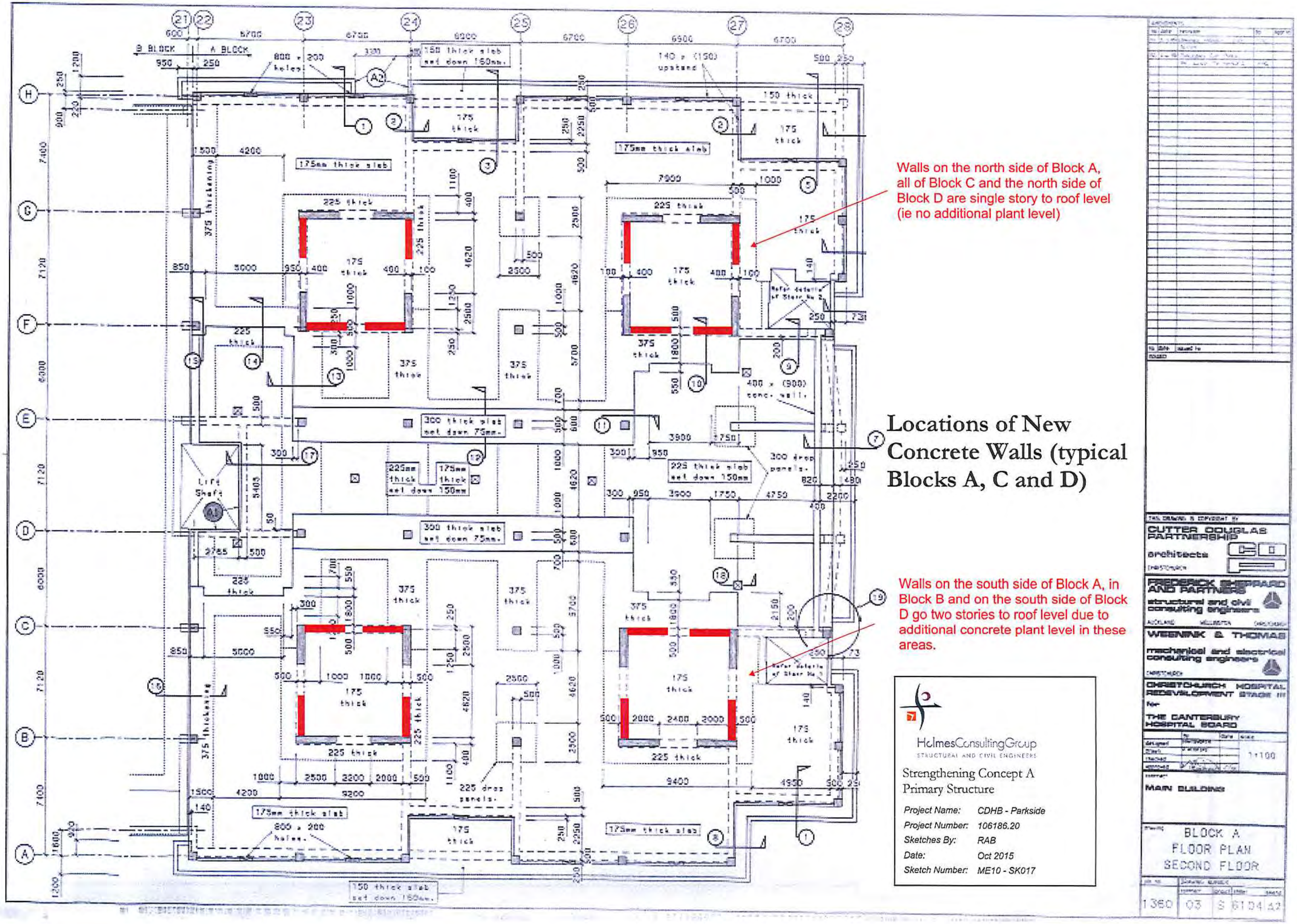
AMENDMENT	DRAWN	CHECKED	DATE
MEDICAL DAY UNIT ALTERATIONS ADDED	BT		26-08-08
UPDATED	BT		09-05-08
UPDATED	BT		25-01-07
UPDATED	DJT		09-12-08
UPDATED	BT		03-03-08

CHRISTCHURCH HOSPITAL
PARKSIDE COMBINED FLOOR PLAN
GROUND FLOOR

HolmesConsultingGroup
STRUCTURAL AND CIVIL ENGINEERS

Strengthening Concept A)
Primary Structure

Project Name: CDHB - Parkside
Project Number: 106186.20
Sketches By: LJO
Date: Aug 2013
Sketch Number: ME10 - SK003

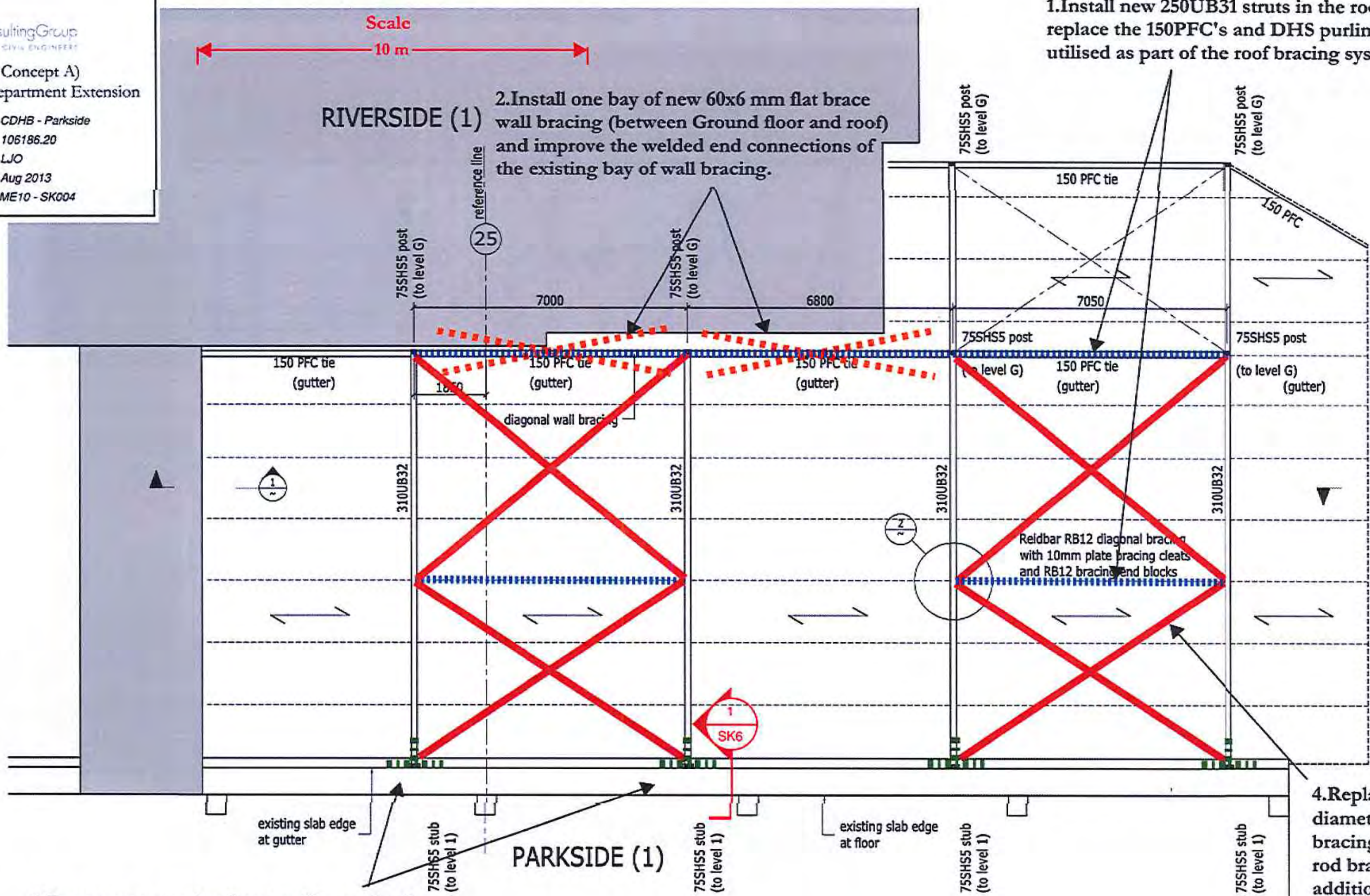

Walls on the north side of Block A, all of Block C and the north side of Block D are single story to roof level (ie no additional plant level)
Locations of New Concrete Walls (typical Blocks A, C and D)
Walls on the south side of Block A, in Block B and on the south side of Block D go two stories to roof level due to additional concrete plant level in these areas.
HolmesConsultingGroup
STRUCTURAL AND CIVIL ENGINEERS
Strengthening Concept A
Primary Structure
Project Name: CDHB - Parkside
Project Number: 106186.20
Sketches By: RAB
Date: Oct 2015
Sketch Number: ME10 - SK017
CUTTER DOUGLAS PARTNERSHIP
architects
WEBNINK & THOMAS
mechanical and electrical consulting engineers
CHRISTCHURCH HOSPITAL REDEVELOPMENT STAGE III
THE CANTERBURY HOSPITAL BOARD
1:100
MAIN BUILDING
BLOCK A
FLOOR PLAN
SECOND FLOOR
1360
03
S 6104 A2
Lift Shaft
175mm thick slab
300 thick slab set down 75mm.
225 thick slab set down 150mm
150 thick slab set down 150mm.

HolmesConsultingGroup

Strengthening Concept A)
Emergency Department Extension

Project Name:	CDHB - Parkside
Project Number:	106186.20
Sketches By:	LJO
Date:	Aug 2013
Sketch Number:	ME10 - SK004

Emergency Department Extension - Roof Plan



1.Install new 250UB31 struts in the roof plane to replace the 150PFC's and DHS purlins currently utilised as part of the roof bracing system.

2.Install one bay of new 60x6 mm flat brace wall bracing (between Ground floor and roof) and improve the welded end connections of the existing bay of wall bracing.

3.Improve connection from roof beams back into Parkside Block A, Level 1, floor. This will require new steel struts to transfer load from the roof plane down to the floor (around 500 mm below). Refer to details on sheet ME10 - SK006.

4.Replace existing 12 mm diameter roof cross bracing with 16 diameter rod bracing. Install an additional bay of 16 diameter roof cross bracing.

Concept for pricing only
- not for construction

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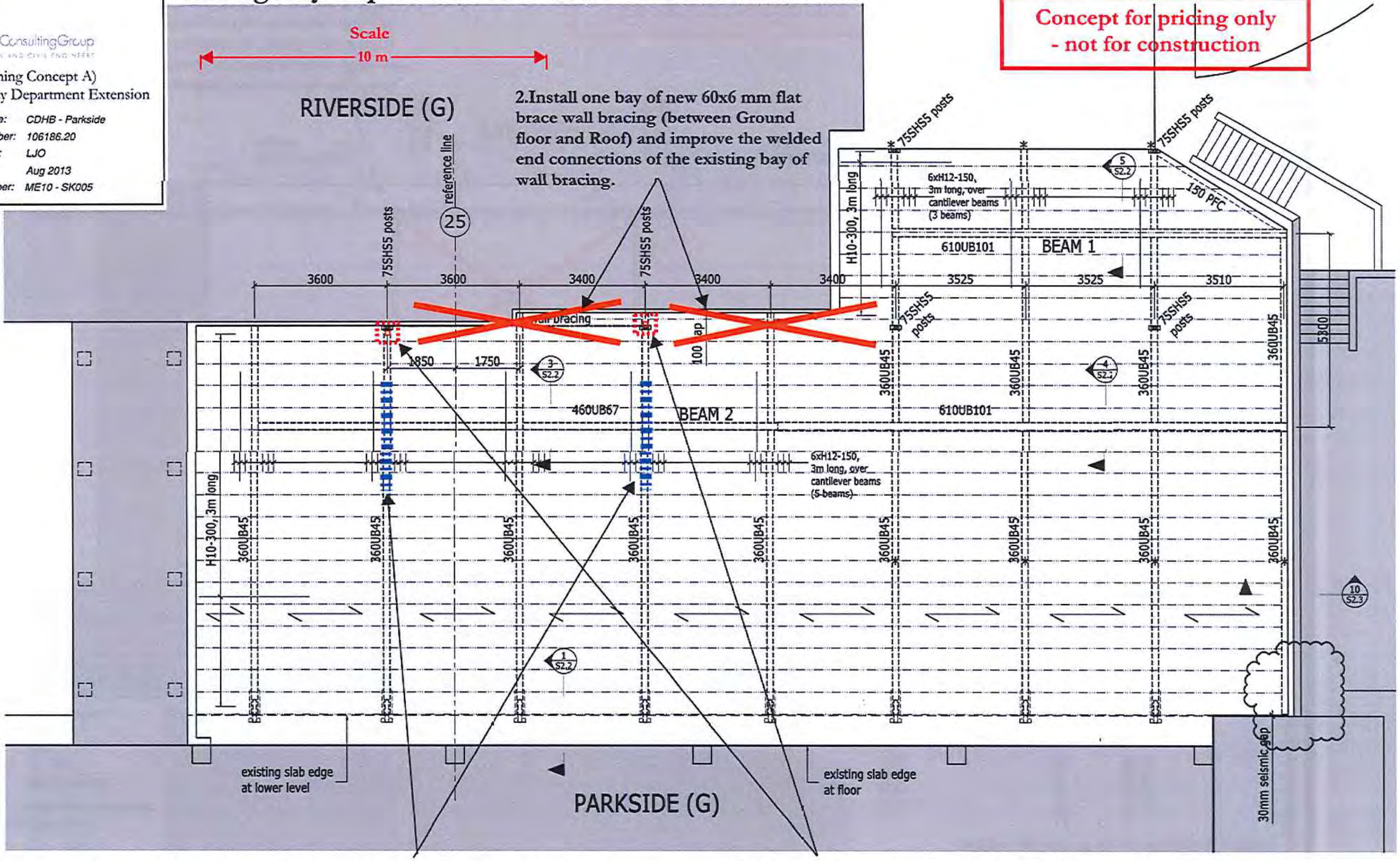
Strengthening Concept A)
Emergency Department Extension

Project Name:	CDHB - Parkside
Project Number:	106186.20
Sketches By:	LJO
Date:	Aug 2013
Sketch Number:	ME10 - SK005

Emergency Department Extension - Ground Floor Plan

Concept for pricing only
- not for construction



2.Install one bay of new 60x6 mm flat brace wall bracing (between Ground floor and Roof) and improve the welded end connections of the existing bay of wall bracing.

5.Increase capacity of cantilevered steel floor beams to resist over strength actions generated by the wall bracing. This could be achieved by locally strengthening the two beams that have insufficient capacity by welding 40x6 mm flats to the beam flanges either side of the web. The flats would need to extend over several meters of the beam length.

5. Alternatively, steel posts could be installed below these two beams which would mean they were no longer required to cantilever. The posts could be 75x75x5 SHS sections founded on shallow concrete foundations (600 mm square) tied into the Clinical Services building

Project Name:	CDHB - Parkside
Project Number:	106186.20
Sketches By:	LJO
Date:	Aug 2013
Sketch Number:	ME10 - SK006

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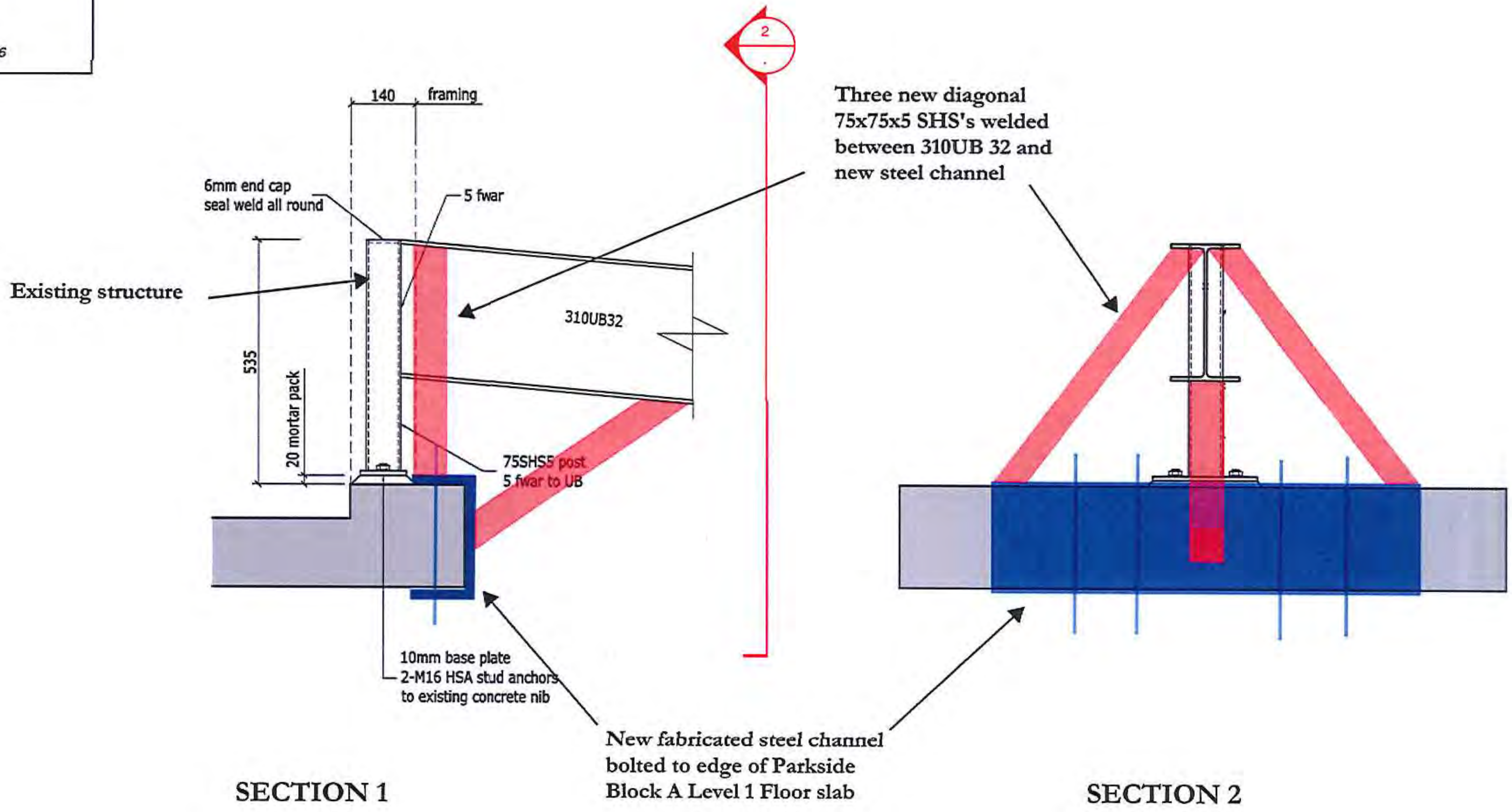
Strengthening Concept A)
Emergency Department Extension

Emergency Department Extension
- New roof connection detail

Concept for pricing only
- not for construction



5.Increase capacity of cantilevered steel floor beams to resist over strength actions generated by the wall bracing. This could be achieved by locally strengthening the two beams that have insufficient capacity by welding 40x6 mm flats to the beam flanges either side of the web. The flats would need to extend over several meters of the beam length.

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SKETCHES

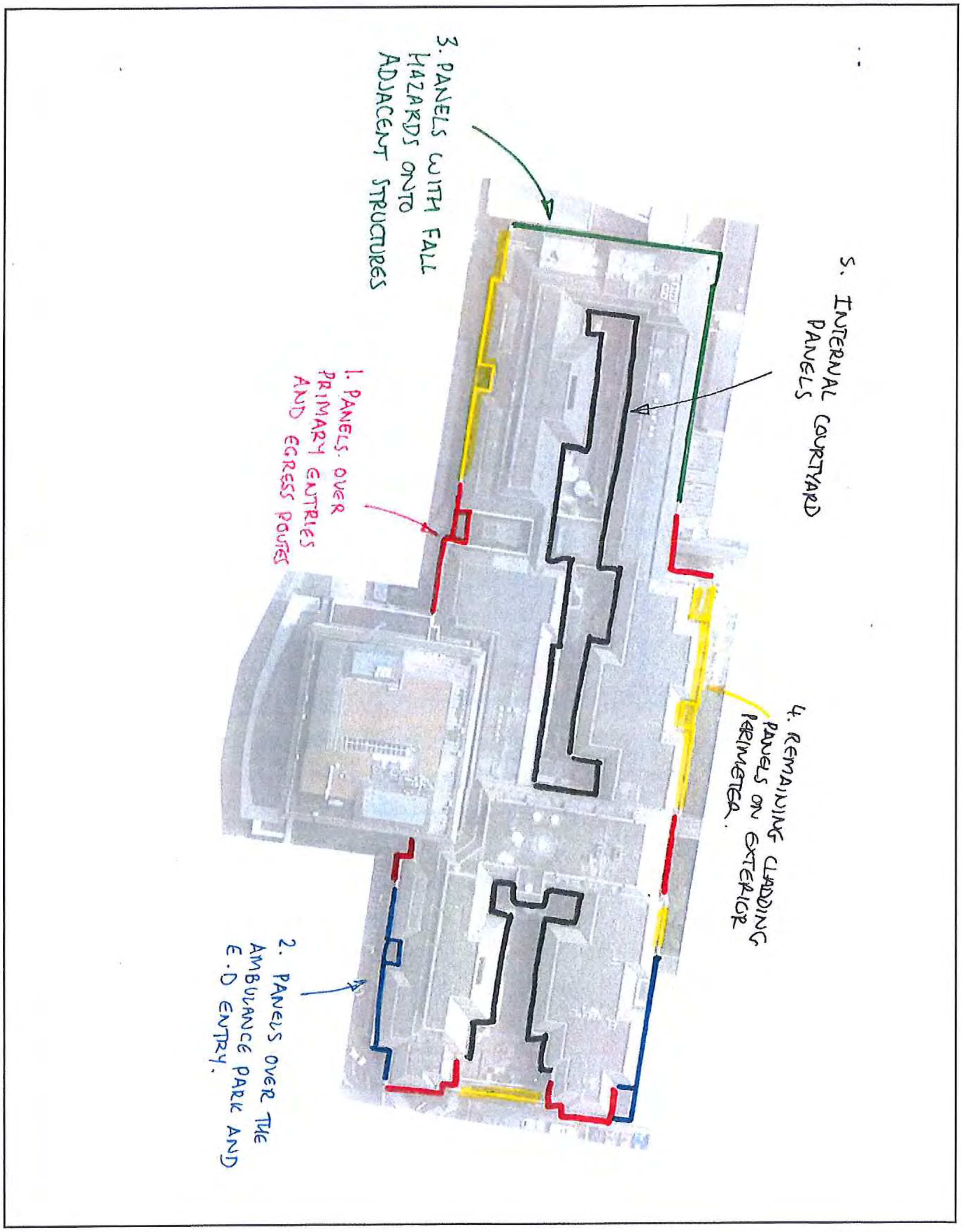
Project Name: PARKSIDE HOSPITAL – CLADDING

Project No: 106186.20

Calcs By: MNS

Date: OCTOBER 2014 Page No: 9

Sketch No: ME10 - SK018 Revision: 1



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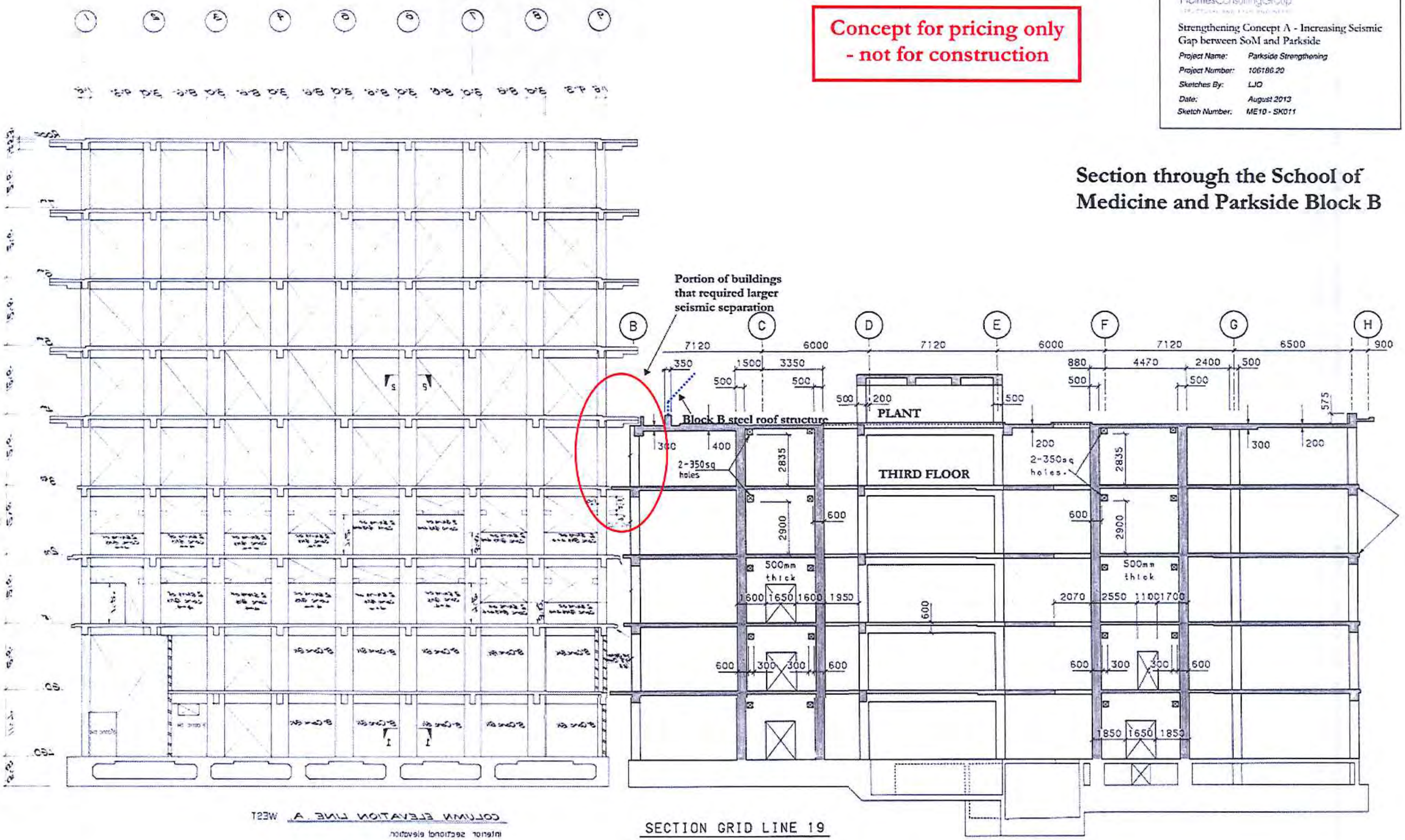
Strengthening Concept A - Increasing Seismic Gap between SoM and Parkside

Project Name:	Parkside Strengthening
Project Number:	106186.20
Sketches By:	LJO
Date:	August 2013
Sketch Number:	ME10 - SK011

Concept for pricing only - not for construction

Section through the School of Medicine and Parkside Block B



School of Medicine

Parkside Block B

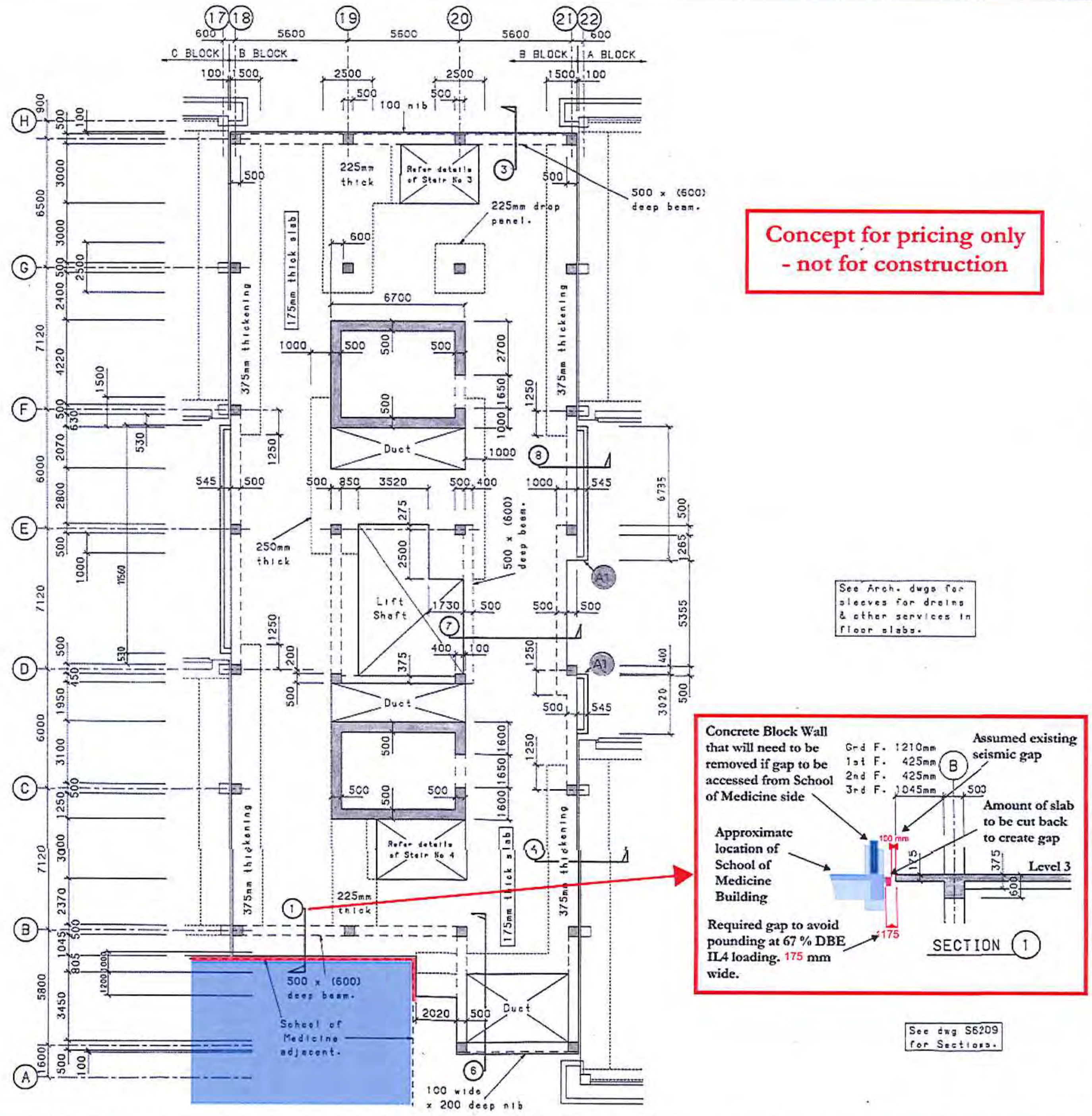
Holmes Consulting Group

Strengthening Concept A - Increasing Seismic Gap between SoM and Parkside

Project Name:	Parkside Strengthening
Project Number:	106186.20
Sketches By:	LJO
Date:	August 2013
Sketch Number:	ME10 - SK012

Level 3 Plan and edge details at Parkside Block B

unstrengthened building



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FREDERICK SHEPPARD AND PARTNERS structural and civil consulting engineers AUCKLAND WELLINGTON CHRISTCHURCH

WEENINK & THOMAS mechanical and electrical consulting engineers CHRISTCHURCH

CHRISTCHURCH HOSPITAL REDEVELOPMENT STAGE III

for THE CANTERBURY HOSPITAL BOARD

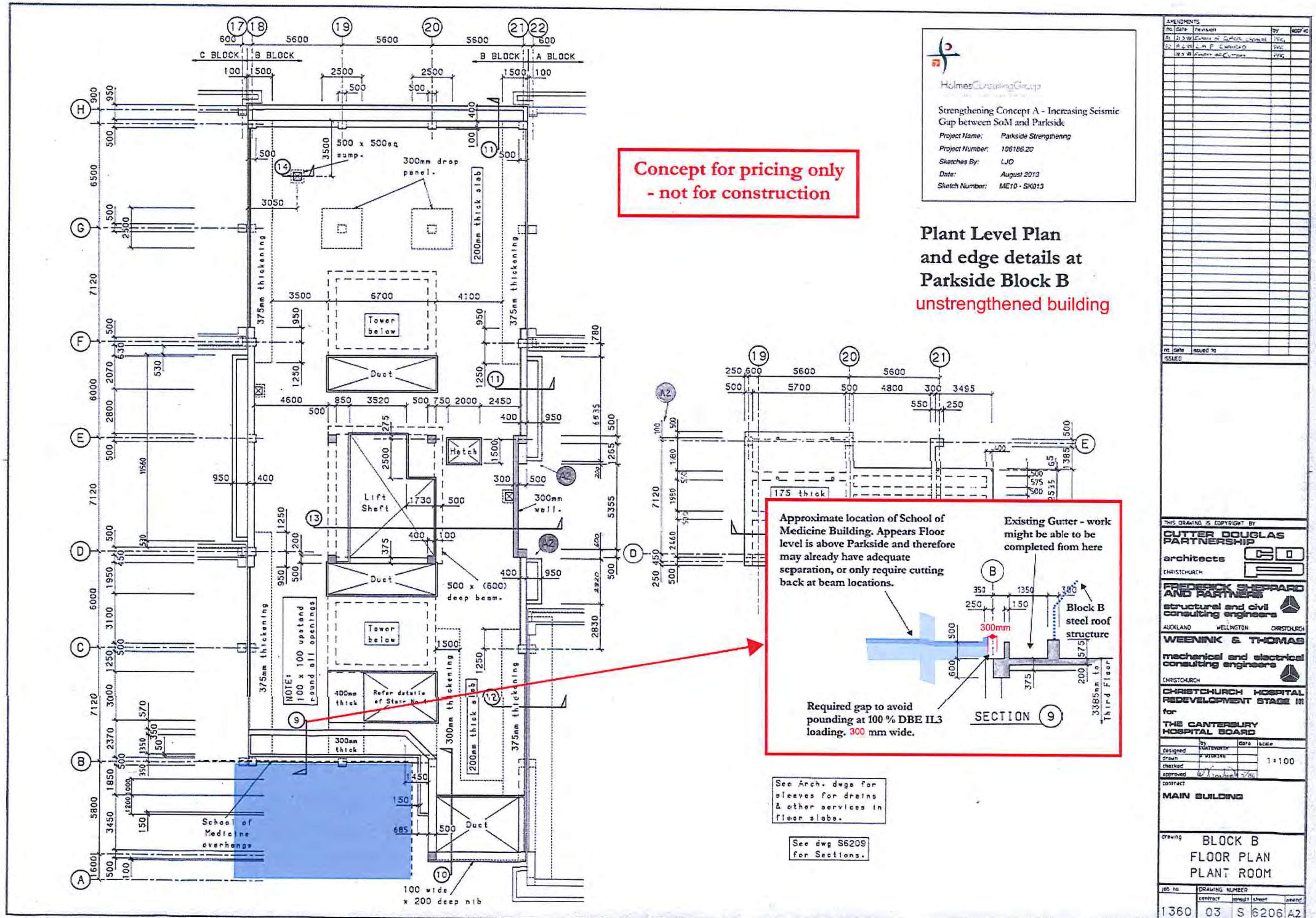
scale 1:100

MAIN BUILDING

drawing BLOCK B FLOOR PLAN THIRD FLOOR

job no	contract	consult	sheet	amend
1360	03	S	6205	A1

TIME 00:05:53 <90 METRES OF LINE 3485 LINES OF PLOT CODE



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Strengthening Concept A - Increasing Seismic Gap between SoM and Parkside
Project Name: Parkside Strengthening
Project Number: 106186.20
Sketches By: LJO
Date: August 2013
Sketch Number: ME10 - SK013
Concept for pricing only - not for construction
Plant Level Plan and edge details at Parkside Block B
unstrengthened building
Approximate location of School of Medicine Building. Appears Floor level is above Parkside and therefore may already have adequate separation, or only require cutting back at beam locations.
Existing Gutter - work might be able to be completed from here
Block B steel roof structure
300mm
Required gap to avoid pounding at 100 % DBE IL3 loading. 300 mm wide.
SECTION 9
3385mm to Third Floor
C BLOCK
B BLOCK
A BLOCK
500 x 500sq sump.
300mm drop panel.
200mm thick slab
375mm thickening
300mm thickening
Tower below
Duct
Lift Shaft
Hatch
300mm wall.
500 x (600) deep beam.
NOTE: 100 x 100 upstand round all openings
400mm thick
300mm thick
Refer details of Stair No 4
175 thick
School of Medicine overhangs
100 wide x 200 deep nib
See Arch. dwgs for sleeves for drains & other services in floor slabs.
See dwg S6209 for Sections.
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WEENINK & THOMAS
mechanical and electrical consulting engineers
CHRISTCHURCH
CHRISTCHURCH HOSPITAL REDEVELOPMENT STAGE III
for
THE CANTERBURY HOSPITAL BOARD
scale 1:100
MAIN BUILDING
BLOCK B
FLOOR PLAN
PLANT ROOM
1360 03 S 6206 A2

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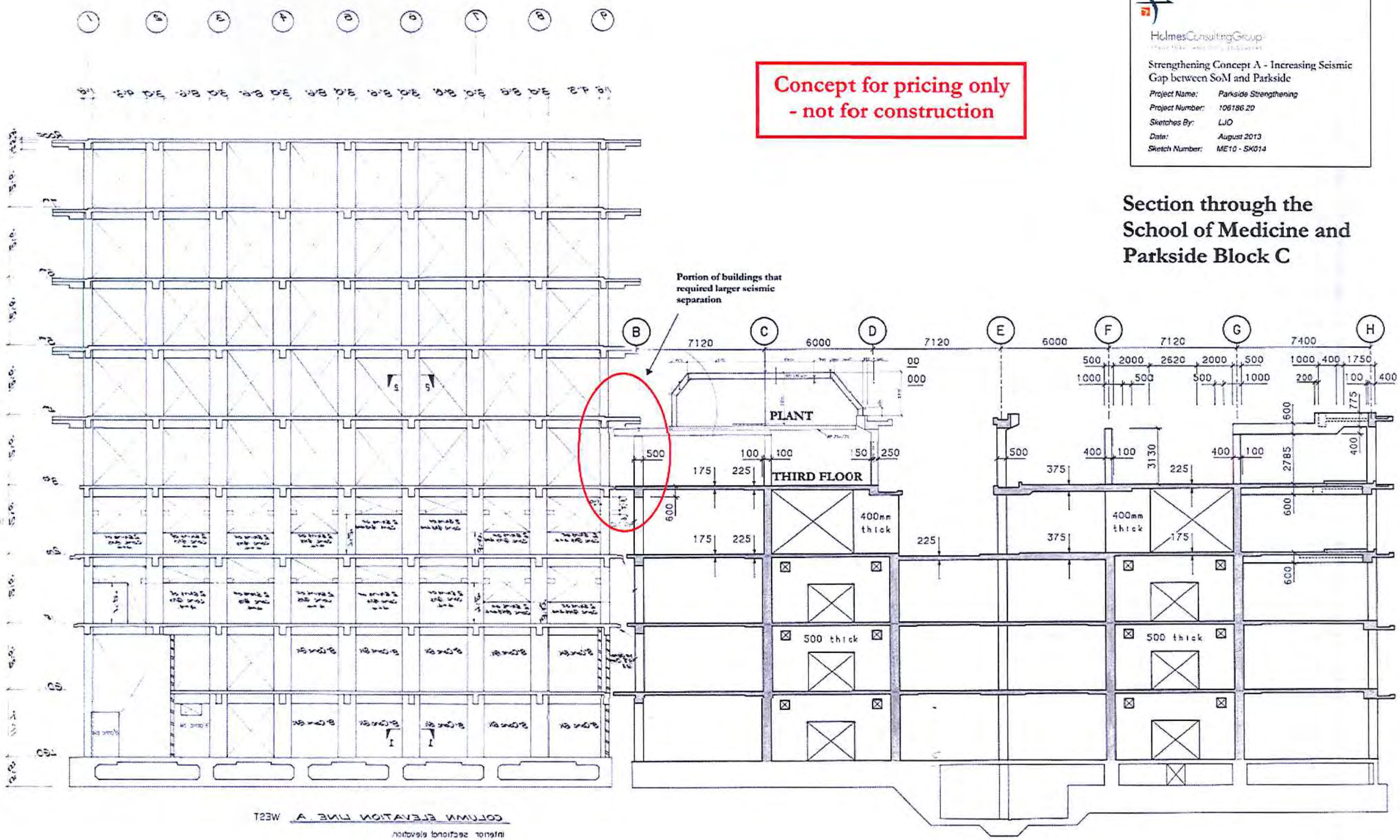
Strengthening Concept A - Increasing Seismic Gap between SoM and Parkside

Project Name:	Parkside Strengthening
Project Number:	106186.20
Sketches By:	LJO
Date:	August 2013
Sketch Number:	ME10 - SK014

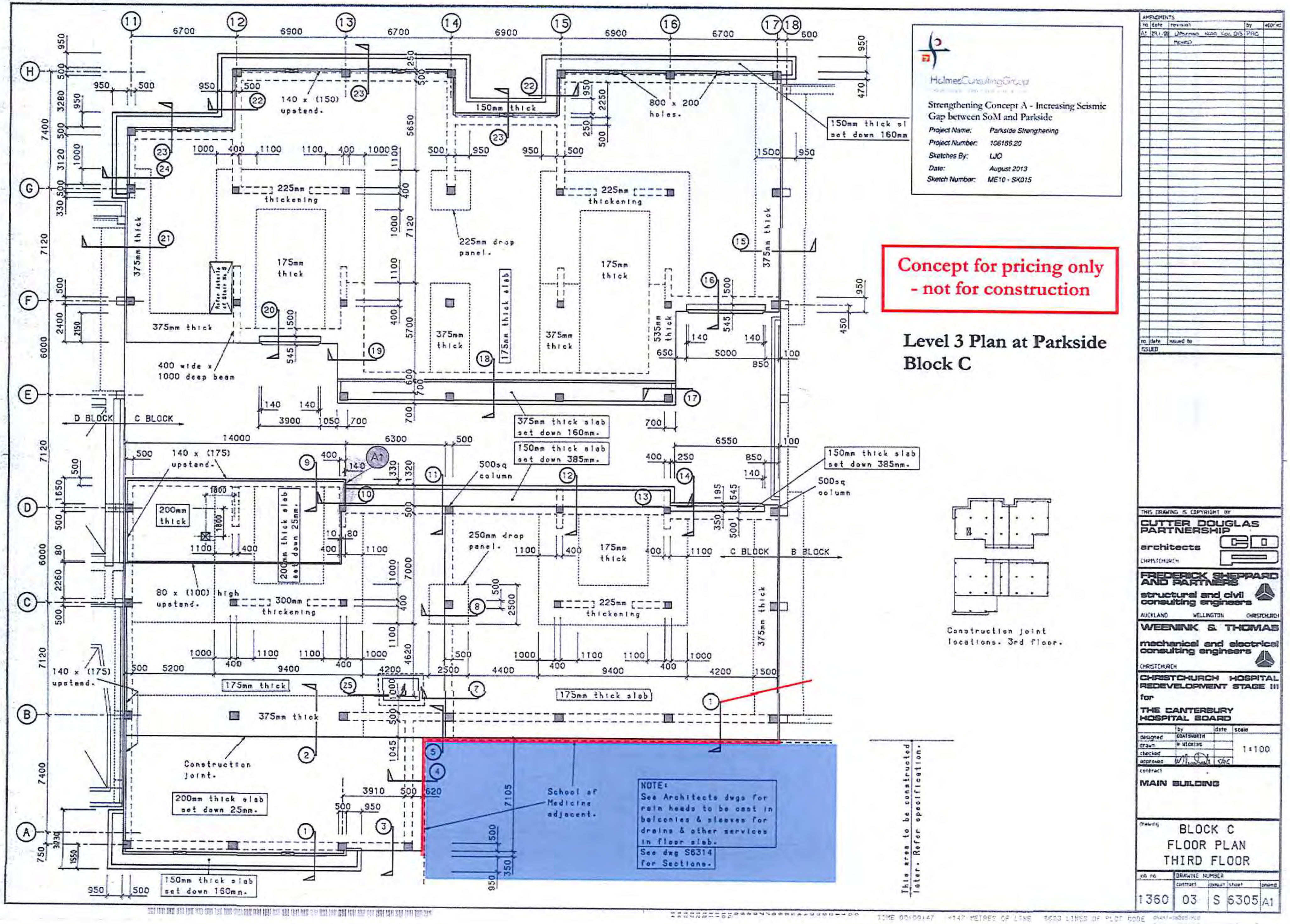
Concept for pricing only - not for construction

Section through the School of Medicine and Parkside Block C



School of Medicine

Parkside Block C



HolmesConsultingGroup
Strengthening Concept A - Increasing Seismic Gap between SoM and Parkside
Project Name: Parkside Strengthening
Project Number: 106186.20
Sketches By: LJO
Date: August 2013
Sketch Number: ME10 - SK015
Concept for pricing only - not for construction
Level 3 Plan at Parkside Block C
140 x (150) upstand.
150mm thick
800 x 200 holes.
150mm thick sl set down 160mm
225mm thickening
225mm drop panel.
175mm thick
375mm thick
375mm thick
375mm thick
175mm thick slab
535mm thick
400 wide x 1000 deep beam
D BLOCK
C BLOCK
375mm thick slab set down 160mm.
150mm thick slab set down 385mm.
150mm thick slab set down 385mm.
500sq column
500sq column
140 x (175) upstand.
200mm thick
200mm thick slab set down 25mm.
250mm drop panel.
C BLOCK
B BLOCK
80 x (100) high upstand.
300mm thickening
225mm thickening
375mm thick
Construction joint locations. 3rd floor.
140 x (175) upstand.
175mm thick
175mm thick slab
375mm thick
Construction joint.
200mm thick slab set down 25mm.
School of Medicine adjacent.
NOTE: See Architects dwgs for rain heads to be cast in balconies & sleeves for drains & other services in floor slab. See dwg S6314 for Sections.
150mm thick slab set down 160mm.
This area to be constructed later. Refer specification.
AMENDMENTS
ISSUED
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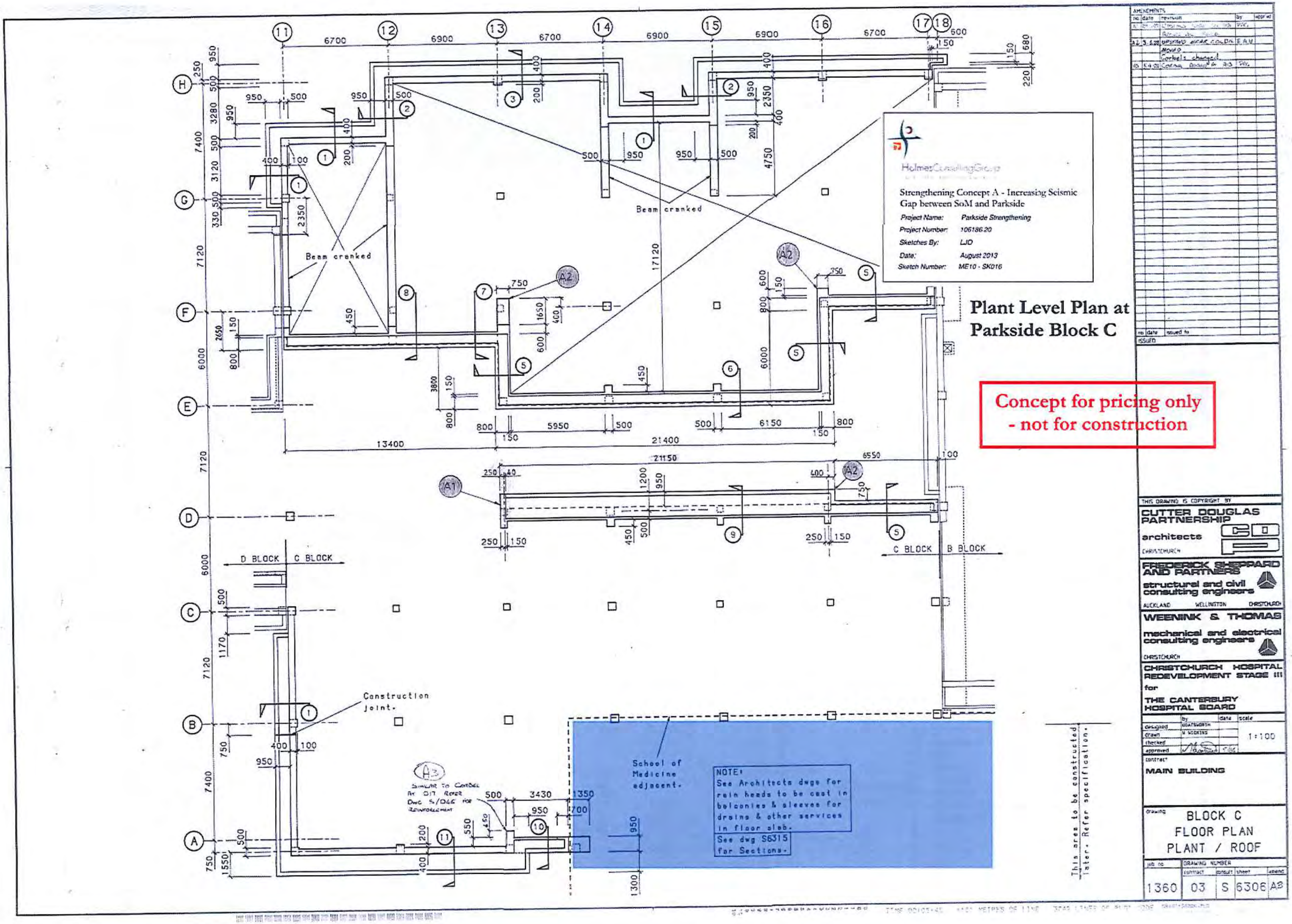
FREDERICK SHEPPARD AND PARTNERS structural and civil consulting engineers AUCKLAND WELLINGTON CHRISTCHURCH
WEENINK & THOMAS mechanical and electrical consulting engineers CHRISTCHURCH
CHRISTCHURCH HOSPITAL REDEVELOPMENT STAGE III
for
THE CANTERBURY HOSPITAL BOARD
1:100
MAIN BUILDING
BLOCK C FLOOR PLAN THIRD FLOOR
1360 03 S 6305 A1



Plant Level Plan at Parkside Block C
Concept for pricing only - not for construction
HolmesConsultingGroup
Strengthening Concept A - Increasing Seismic Gap between SoM and Parkside
Project Name: Parkside Strengthening
Project Number: 106186.20
Sketches By: LJO
Date: August 2013
Sketch Number: ME10 - SK016
Beam cranked
Construction joint.
School of Medicine adjacent.
NOTE: See Architects dwgs for rain heads to be cast in balconies & sleeves for drains & other services in floor slab.
See dwg S6315 for Sections.
D BLOCK
C BLOCK
B BLOCK
This area to be constructed later. Refer specification.
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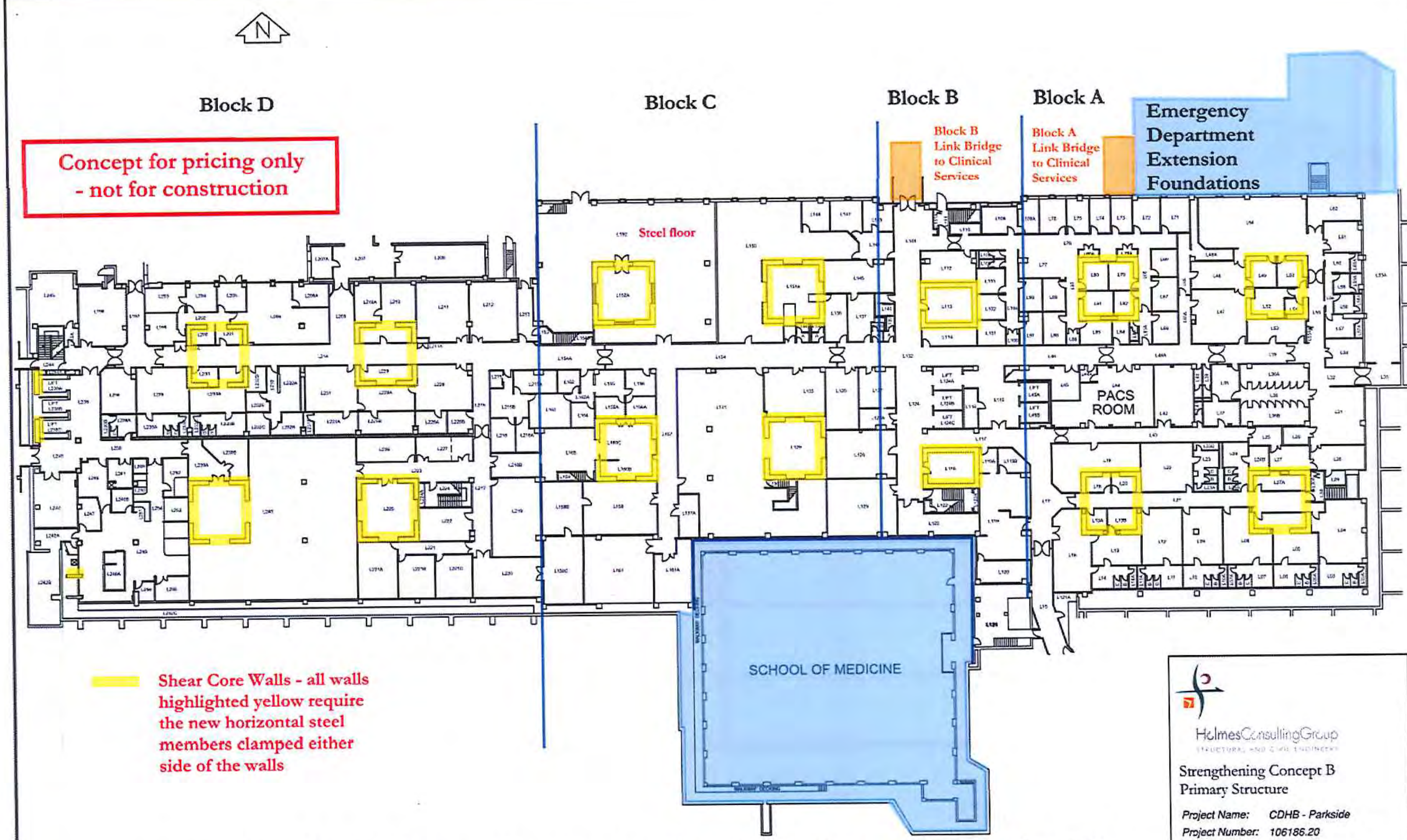
mechanical and electrical consulting engineers
CHRISTCHURCH
CHRISTCHURCH HOSPITAL REDEVELOPMENT STAGE III
for
THE CANTERBURY HOSPITAL BOARD
1:100
MAIN BUILDING
BLOCK C
FLOOR PLAN
PLANT / ROOF
1360 03 S 6306 A3



Location of walls that require new clamping detail - Lower Ground Floor Plan

HolmesConsultingGroup

Strengthening Concept B
Primary Structure

Project Name: CDHB - Parkside
Project Number: 106186.20
Sketches By: LJO
Date: Aug 2013
Sketch Number: ME10 - SK021

AMENDMENT	DRAWN	CHECKED	DATE
SCHOOL OF MEDICINE ADDED	B.T.		10-11-09
LINKS TO WOMENS HOSPITAL ADDED	B.T.		24-01-05
UPDATED	B.T.		09-05-05
UPDATED	B.A.T.		08-12-08

CHRISTCHURCH HOSPITAL
PARKSIDE COMBINED FLOOR PLAN
LOWER GROUND FLOOR

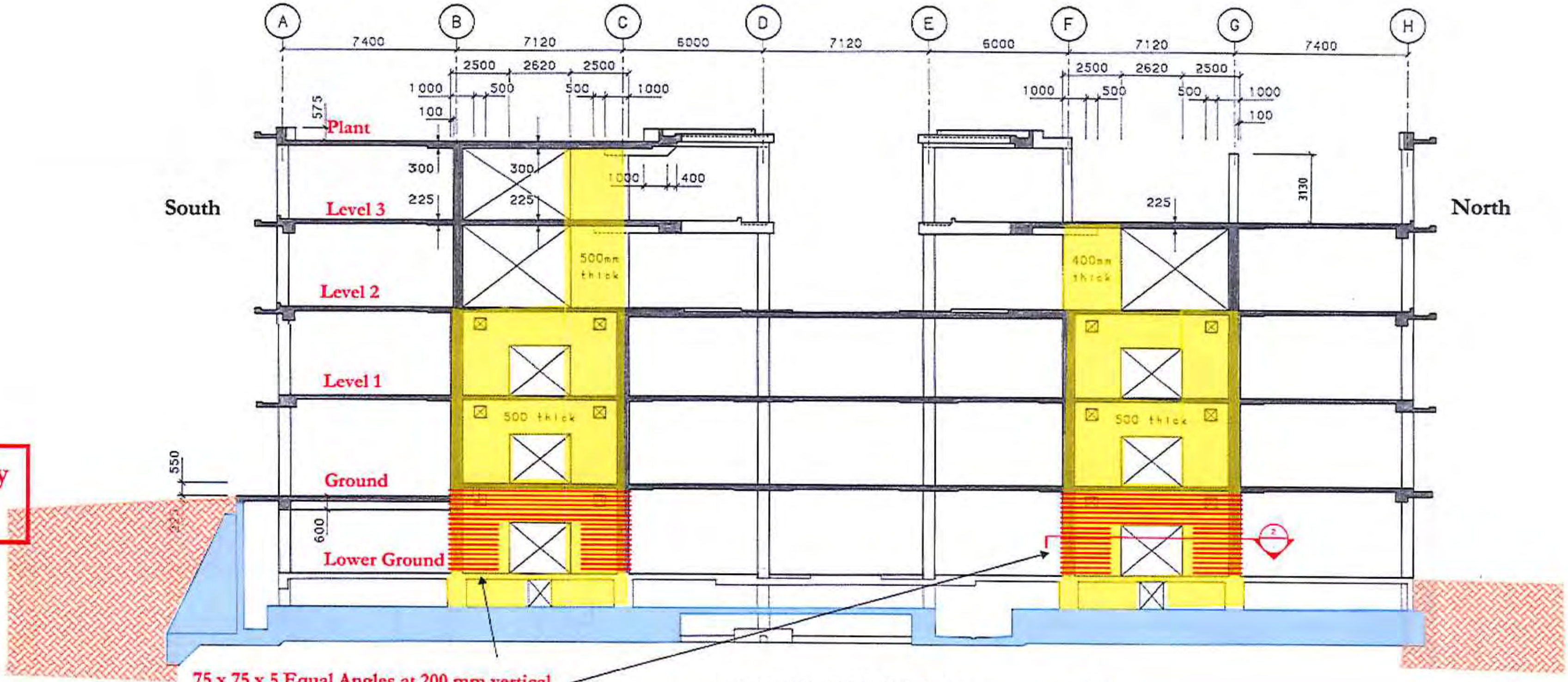
HolmesConsultingGroup

Strengthening Concept B)
Primary Structure

Project Name:	CDHB - Parkside
Project Number:	106186.20
Sketches By:	LJO
Date:	Aug 2013
Sketch Number:	ME10 - SK022

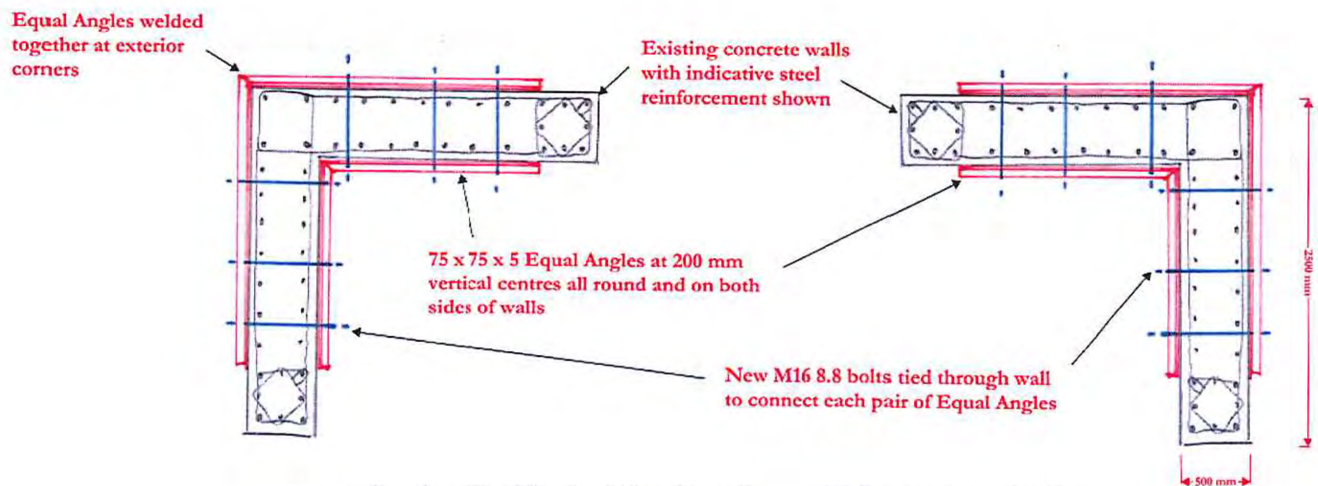
Concrete Core Wall External Clamping Details

Concept for pricing only - not for construction



SECTION GRID LINE 26

Typical Section through Block A



Section 2 - Typical Section through Shear Core Walls

Detailed Seismic Assessment Update
Riverside Central Building

Riverside Central
Riccarton Ave, Christchurch Central
Christchurch

Detailed Seismic Assessment Report

Holmes Consulting

Revision 1
20 December 2017
110628.60

Holmes Consulting

Level 2, 254 Montreal Street
Christchurch Central
PO Box 6718
Christchurch 8442
holmesgroup.com

Report

Seismic Assessment Update of the Riverside Central Building

Prepared For:
Ministry of Health c/o Bryan Spinks & Antony Manners

Date:	20 December 2017
Project No:	110628.60
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EXECUTIVE SUMMARY

Background

Holmes Consulting LP has been engaged by the Ministry of Health (MOH) to reassess the capacity of the primary structure of the Riverside Central building, on the Christchurch Hospital Campus, and provide high level strengthening options. The reassessment has been carried out to in accordance with "The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments, July 2017, Version 1" (NZSEE, 2017). This document is referred to as the "**Engineering Assessment Guidelines**" within this report.

This report updates in part a previous Detailed Seismic Assessment (DSA) which we provided in 2013 (Holmes Consulting, 2013). That report identified some detailing deficiencies which limited assessed dependable performance to 35-40% of the Importance Level 3 (IL3) loads which would be used to design a similar new building. Some improvements were implemented to target the most critical of these deficiencies, including work to strengthen a deficiency in short columns at Level 6 near the building setback (by introducing infill concrete walls locally).

The earthquake damage from the Canterbury Earthquake Sequence and repair work required is not included in this report. These are outlined in the previous DSA report (Holmes Consulting, 2013).

The current assessment of the Riverside Central Building, incorporates changes in Non-Linear Time History Analysis (NLTHA) modelling, seismic input and the margin to collapse that has been defined in the new Engineering Assessment Guidelines. The findings of this reassessment are then used to support a high-level review of the potential impact on the originally assessed seismic performance for a number of other buildings on the Christchurch Hospital Campus (Holmes Consulting, 2017).

The Assessment of Riverside Central has been updated to comply with the 2017 Engineering Assessment Guidelines

The 2013 assessment has been updated for the project in order to incorporate learnings from recent earthquakes (most notably Canterbury 2010 and 2011, and Kaikoura 2016), and where possible, recommendations from the Royal Commission of Inquiry into performance of buildings in the Canterbury Earthquakes. The latter generally take the form of recently published guidance and updated standards.

The assessment has been completed in line with the Engineering Assessment Guidelines. Under the Earthquake-prone Buildings Amendment Act 2016 enacted in July 2017, the new guidelines must now be used for assessing whether a building is or is not Earthquake-prone. While the structure and process has been completed in accordance with the recently issued Engineering Assessment Guidelines the output has been reported in accordance with the specific DA definition to demonstrate compliance. This is because the DA seismic capacity definition predates the new Engineering Assessment Guidelines.

Building Description

Riverside Central consists of seven suspended levels, with a two storey rooftop plantroom plus a partial basement containing services tunnels. Suspended levels are supported by a combination of reinforced concrete walls, internal gravity columns and a north perimeter reinforced concrete frame. The walls and columns are founded at basement level on a combination of strip footings and isolated pad foundations. Un-sealed service trenches run the length of the building above the foundation structure.

The Riverside Buildings (Central, West and East) were designed and constructed in the late 1960's. The three buildings are separated by seismic gaps of 100mm and are also separated from the adjacent Clinical Services Block. Riverside Central links the West and East buildings. The Riverside Block is currently designated as an Importance Level 3 (IL3) structure by the CDHB.

Figure 1: North elevation of the Riverside Block (Central, East & West pictured)

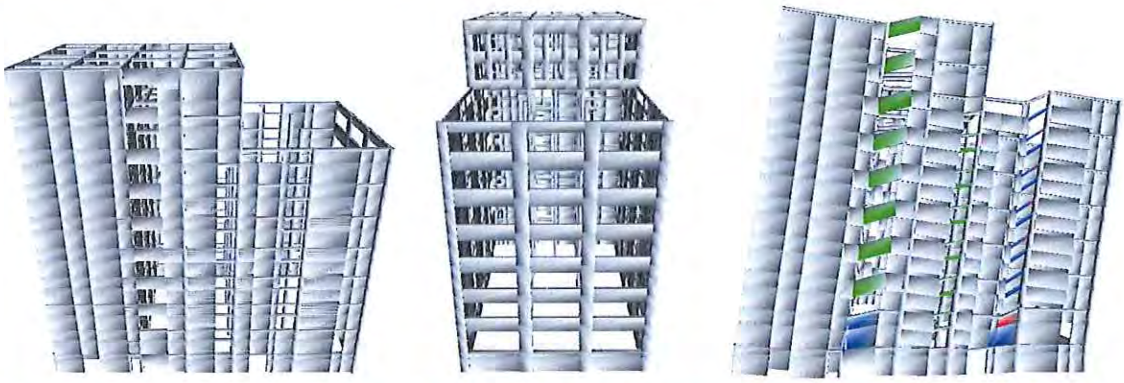


Figure 2: Graphical representation of the building's earthquake lateral system, and an exaggerated view of its lateral mechanism

Assessed Seismic Rating

Assessment to the 2017 NZSEE Engineering Assessment Guidelines

Under the Earthquake-prone Buildings Amendment Act 2016 enacted in July 2017, the new guidelines must now be used for assessing whether a building is or is not Earthquake-prone.

A building with an earthquake rating less than 34% NBS fulfils one of the requirements for the Territorial Authority to consider it to be an Earthquake-prone Building (EPB) in terms of the Building Act 2004. A building rating less than 67% NBS is considered as an Earthquake Risk Building (ERB) by the New Zealand Society for Earthquake Engineering. Riverside Central could be considered Earthquake-prone for the IL3-50yr classification under the assessment framework.

The building is a Grade D building following the NZSEE grading scheme. Grade D buildings represent a medium to high risk exposure relative to a new building if a large earthquake occurs.

Potential Structural Weaknesses Identified in the DSA

The assessment identified the following Potential Structural Weaknesses in the building:

Table 1: Potential Structural Weaknesses identified in the DSA

Building Element	Potential Structural Weakness	% NBS (IL3)
Spandrel beams and supporting structure for Level 7 Plant Room water tanks – considered as full	Excessive plastic rotations under lateral loading in beams detailed with inadequate shear reinforcing. Onset of inelastic demands leading to strength degradation and potential loss of vertical load carrying capacity (MCE[2])	<34%
Wall Panels on the east elevation, south-east corner, around the services duct	Out-of-plane performance of the these panels, unrestrained over the height of the building, Flexural performance limited under inertial force face loading, resulting in potential local collapse hazard (ULS[1])	40%
Columns on the north perimeter RC frame	Excessive plastic rotations under lateral loading. Inelastic demand leading to strength degradation and potential loss of vertical load carrying capacity (MCE[2])	45%
Wall panel shear strength – various locations in the lower storeys	Wall panel shear strains in excess of Life Safety criteria below Level 2. Strength degradation leading to wall instability (MCE[2])	50%
Coupling beams on the east and west perimeter	Excessive plastic rotation under lateral loading. Degrading beam strength leading to local collapse hazard (MCE[2])	50%

[1] ULS is an abbreviation for Ultimate Limit State Earthquake shaking.

[2] MCE is an abbreviation for Maximum Considered Earthquake shaking.

The Critical Structural Weakness is the lowest scoring structural weakness, the failure of which would result in a significant life safety hazard. The lowest scoring potential structural weaknesses had Ultimate Limit State (ULS) capacities of <34% NBS (IL3). These were found to be excessive plastic rotations in the spandrel beams and supporting structure of the Level 7 Plant Room, leading to strength degradation and a potential for loss of vertical load carrying capacity.

The following secondary structural and non-structural aspects were considered in the assessment of the seismic rating:

- South core egress stairs
- Out-of-plane capacity of slender wall panels

The assessment of other secondary structural and non-structural elements of the building such as heavy plant, ceilings and appendages have not been included in this reassessment of the building. The capacity of these elements may govern the capacity of the building. It is recommended that an assessment of these elements be carried out.

Basis for the Assessment

The Detailed Seismic Assessment Update has been completed in accordance with "The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments, July 2017, Version 1" (NZSEE, 2017). The method used is Non-Linear Time History Analysis (NLTHA), in accordance with ASCE 41-13 (ASCE/SEI, 2014) as cited by Section C1.6.2 of the Engineering Assessment Guidelines. The loadings standard used is NZS 1170.5:2004.

The assessment is based on the original structural drawings (dated 1967-1969). The assessment also uses information from the previous 2013 Detailed Seismic Assessment and subsequent strengthening projects, following the 2010-2011 Canterbury Earthquake sequence.

Seismic Strengthening to Meet 45% NBS IL3

The structural work required to strengthen the building to 45% NBS comprises:

- Emptying the storage water tanks at Level 7 or relocating to the base of the structure, to eliminate detrimental effects on supporting structure due to the imposed loadings. Tanks were considered full in analysis, with a total weight of 75 tonnes.

- Bracing of the unrestrained perimeter wall panels. Walls around east lift core and service riser are currently unrestrained over the height of the building and present an out-of-plane deficiency under face loading. Coordination with services will be required in implementation of any upgrade works.

The structural work required to strengthen the building to 52% (IL3) and 67% (IL3) are outlined in the report.

Technical Summary

For the assessed seismic rating in accordance with the NZSEE 2017 Engineering Assessment Guidelines definition, a tabulated Assessment Summary Report is required to be prepared in accordance with the requirements of those guidelines. The report completed for the primary structure only is included in Appendix A.

1 INTRODUCTION AND SCOPE

Holmes Consulting LP has been engaged by the Ministry of Health (MOH) to reassess the capacity of the primary structure of the Riverside Central building, on the Christchurch Hospital Campus, and provide high level strengthening options. The reassessment has been carried out to in accordance with "The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments, July 2017, Version 1" (NZSEE, 2017). This document is referred to as the "**Engineering Assessment Guidelines**" within this report. The Engineering Assessment Guidelines are referenced in the Earthquake-prone Building methodology, for the identification of Earthquake-prone Buildings, as set by the Chief Executive of the Ministry of Business Innovation and Employment under section 133AV of the Building Act 2004.

The earthquake damage from the Canterbury Earthquake Sequence and repair work required is not included in this report. These are outlined in the previous DSA report (Holmes Consulting, 2013).

This report updates in part a previous Detailed Seismic Assessment (DSA) which we provided in 2013 (Holmes Consulting, 2013). That report identified some detailing deficiencies which limited assessed dependable performance to 35-40% of the Importance Level 3 (IL3) loads which would be used to design a similar new building. Some improvements were implemented to target the most critical of these deficiencies, including work to strengthen a deficiency in short columns at Level 6 near the building setback (by introducing infill concrete walls locally). The current assessment of Riverside Central incorporates changes in Non-Linear Time History Analysis (NLTHA) modelling, seismic input and the margin to collapse that has been defined in the new Engineering Assessment Guidelines. The findings of this reassessment are then used to support a high-level review of the potential impact on the originally assessed seismic performance for a number of other buildings on the Christchurch Hospital Campus (Holmes Consulting, 2017).

A Detailed Seismic Assessment quantifies the risk posed to people by existing buildings from earthquakes

The objective of a DSA is to inform users about the risk posed to people by existing buildings under earthquake actions. A DSA specifically considers life safety, egress and protection of adjacent property by assessing strength and deformation capacity. A DSA quantifies the seismic performance/behaviour of the building as a seismic rating. This is expressed as a percentage of the standard achieved from application of the building code requirements, or % NBS (percent New Building Standard). The rating provides a measure of the expected performance from a life safety point of view, compared with the minimum performance that the Building Code would require for a similar new building on the same site.

A DSA is also used to determine whether or not a building is Earthquake-prone in accordance with the definition for an Earthquake-prone building (EPB) in the New Zealand Building Act.

The DSA has been updated to comply with the 2017 Engineering Assessment Guidelines

The 2013 DSA has been updated in order to incorporate learnings from recent earthquakes (most notably Canterbury 2010 and 2011, and Kaikoura 2016), and where possible, recommendations from the Canterbury Earthquakes Royal Commission. The latter generally take the form of recently published guidance and updated standards.

The DSA has also been updated to comply with the Engineering Assessment Guidelines. The Earthquake-prone Buildings Amendment Act 2016, enacted in July 2017, requires these guidelines to be used for assessing whether a building is or is not Earthquake-prone. The guidelines also represent a new consistent framework for the engineering profession to use when assessing, rating and communicating seismic performance of buildings of all types (as it relates to life safety).

This reassessment of Riverside Central is of the primary structure only. An assessment of all secondary structural and non-structural elements of the building such as heavy plant, ceilings and appendages have not been included in this reassessment. The capacity of these elements may govern the capacity of the building. It is recommended that an assessment of these elements is competed.

Specific guidance on damage limitation or the ability to occupy a building after an earthquake event is not generally discussed, unless specifically appended to the scope of a normal DSA. We have not included that scope in this DSA.

Our scope of work for the Riverside Central DSA

The scope of work for this project included the following:

- Undertake a Detailed Seismic Assessment (DSA) of the primary structure of the building using a 3D computer analysis model with supporting calculations and evaluation (by updating the model and calculations used in the previous DSA to comply with the 2017 Engineering Assessment Guidelines).
- Generate a revised DSA report outlining the assessed seismic rating, summary of likely seismic performance and risks, and also describing analysis methodology, modelling parameters.
- Include in the assessment the strengthening required (and proposed) to achieve the seismic performance of 52% NBS (IL3-50yr), equivalent to 67% ULS (IL3-25yr) for the primary structure.
- If other deficiencies are identified in the updated assessment, provide concept level advice for potential further strengthening measures as appropriate.
- Present the report findings and recommendations.

Terminology used to communicate earthquake risk and building performance

The communication of seismic risk and assessed seismic behaviour of the building use the terminology defined in the Engineering Assessment Guidelines. The terminology has been clarified and standardised in these new guidelines.

Key terms and acronyms used in this report include:

- **Earthquake rating** – The rating given to a building as a whole to indicate the seismic standard achieved in regard to human life safety compared with the minimum seismic standard required of a similar new building on the same site. Expressed in terms of % NBS.
- **Ultimate Limit State (ULS) shaking demand** – the shaking demand (loading or displacement) defined for the ULS design of a new building and/or its members for the same site.
- **Percent New Building Standard (% NBS)** – The ratio of the ultimate capacity of a building as a whole or of an individual member/element and the ULS shaking demand for a similar new building on the same site, expressed as a percentage.
- **Structural Weakness (SW)** – An aspect of the building structure and/or foundation soils that scores less than 100% NBS.
- **Severe Structural Weakness (SSW)** – A defined SW that is potentially associated with catastrophic collapse and for which the capacity may not be reliable assessed based on current knowledge.
- **Critical Structural Weakness (CSW)** – The lowest scoring structural weakness determined from a DSA.

2 LIMITATIONS

Findings presented as a part of this project are issued pursuant to our contract with the Ministry of Health and for the sole use of the MOH and CDHB in their evaluation of the subject property. The findings are not intended for use by other parties and Holmes Consulting assumes no liability to any party other than the MOH.

Our observations have been restricted to structural aspects only, are limited to visual review of representative samples. 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.

3 STATUTORY REQUIREMENTS

When working with existing buildings, there are statutory requirements that must be considered, these vary depending on whether the building is considered to be potentially Earthquake-prone.

The relevant sections of the New Zealand Building Act 2004 that need to be considered in relation to the building's structure and strength are:

- Section 112: Alterations to existing buildings. Section 112 of the Building Act requires that a building subject to an alteration continue to comply with the relevant provisions of the Building Code to at least the same extent as before the alteration. Essentially this section means that the building may not be made any weaker than it was as a result of any alteration.
- Section 115: Change of Use. Section 115 of the Building Act requires that the territorial authority be satisfied that the building in its new use will comply with the relevant sections of the Building Code "as nearly as is reasonably practicable". In relation to building earthquake strength, this does not necessarily require the building to comply in full with the current Building Code, provided that it can be shown that full compliance is impractical, or that the cost (by any relevant measure of value) is unreasonable under the circumstances. Interpretations would relate to the specific circumstance and would need to be agreed in dialogue with the territorial authority on a case by case basis.
- Section 122: Meaning of earthquake prone building. Section 122 of the Building Act 2004 deems a building to be earthquake prone if its ultimate capacity (strength) would be exceeded in a "moderate earthquake" and it would be likely to collapse causing injury or death, or damage to other property. The Building Regulations (2005) define a moderate earthquake as one that would generate loads 33% as strong as those used to design an equivalent new building.
- Section 124: Powers of Territorial Authority. If a building is found to be earthquake prone, the territorial authority has the power under Section 124 of the Building Act to require strengthening work to be carried out, or to close the building and prevent occupancy.
- Section 131: Earthquake prone building policy. Section 131 of the Building Act requires all territorial authorities to adopt a specific policy on dangerous, earthquake prone, and unsanitary buildings.

4 BUILDING DESCRIPTION

Riverside Central consists of seven suspended levels, with a two storey rooftop plantroom plus a partial basement containing services tunnels. Suspended levels are supported by a combination of reinforced concrete walls, internal gravity columns and a north perimeter reinforced concrete frame. The walls and columns are founded at basement level on a combination of strip footings and isolated pad foundations. Un-sealed services trenches run the length of the building above the foundation structure.

The Riverside Buildings (Central, West and East) were designed and constructed in the late 1960's. The three buildings are separated by seismic gaps of 100mm and are also separated from the adjacent Clinical Services Block. Riverside Central links the West and East buildings. The Riverside Block is currently designated as an Importance Level 3 (IL3) structure by the CDHB.

Figure 3: North Elevation of the Riverside Block (Central, East & West pictured)

4.1 Gravity Resisting System

The vertical load resisting structure consists of two-way spanning cast-insitu reinforced concrete waffle slabs, spanning between internal columns and the perimeter walls and frames. The plantroom slab extends over the southern portion of the building with concrete slabs forming both sections of the roof.

4.2 Earthquake (Lateral) Resisting System

The lateral load resisting structure consists of reinforced concrete walls, primarily located around the perimeter of the building. Significant lengths of wall are provided on the west and east elevations, with a concentration of walls surrounding the services shafts at the southern end of the building. In contrast, the northern elevation is heavily punctured to reflect the piers and spandrels of the West and East buildings. The suspended concrete floors act as structural diaphragms to distribute lateral forces to the walls.

The walls and columns are founded at basement level on a combination of strip footings and isolated pad foundations. Un-sealed services trenches run the length of the building above the foundation structure.

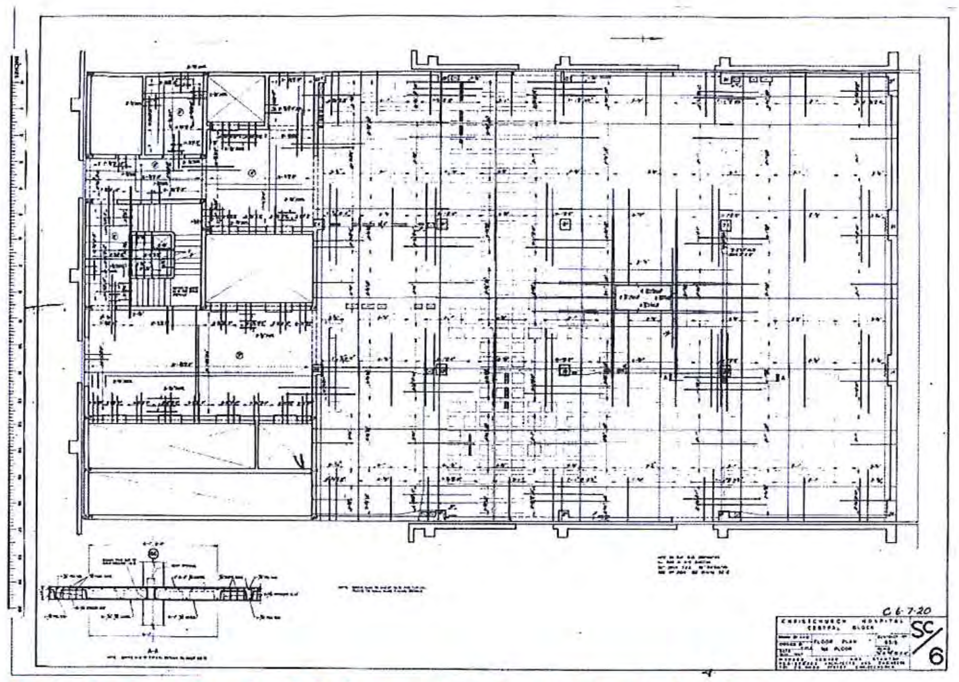


Figure 4: Typical plans levels 2-5 from original structural drawings

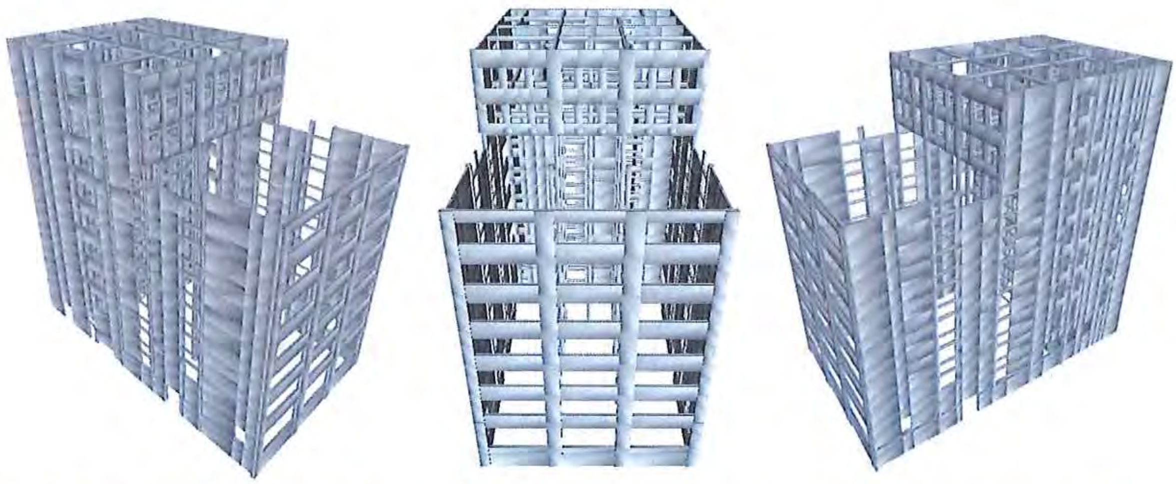


Figure 5: Earthquake (lateral) resisting system comprising concrete shear walls and RC frame (north elevation) on basement walls

4.3 Foundations and Subsoil

The walls and columns are founded at basement level on a combination of strip footings and isolated pad foundations. Un-sealed services trenches run the length of the building above the foundation structure.

The site has been mapped to be Subsoil Class D in accordance with NZS1170.5:2004, which agrees with geotechnical and geophysical testing that has been completed on adjacent sites on the hospital campus. There is sufficient confidence to use this classification for the DSA.

The seismic response is considered to be structurally dominated in accordance with the Engineering Assessment Guidelines. In structurally dominated buildings, the structural response is unlikely to be significantly influence by geohazards, foundation soil nonlinearity or Soil Structure Interaction (SSI) up to the capacity of the structure.

5 ASSESSED SEISMIC RATING

The seismic rating is intended to provide a measure of the seismic safety of a building relative to the minimum that would meet the performance objectives of the Building Code, for the category of building under consideration, that is, with respect to its form and use. The seismic rating is expressed conventionally as the ratio of the assessed seismic capacity of the building and the seismic demand for an equivalent new building, termed %NBS.

The design working life for a new building is 50 years. Hence the demand used to establish the %NBS must be assessed against a nominal 50 year life in order to maintain consistent relativity with new buildings. In the case of the Riverside Central building this is presented in terms of an Importance Level 3 (IL3) building classification, for a 50 year life span. This implies actions 30% greater than for a regular (IL2) building such as would be used for office space, for example.

We have provided more commentary on the method of assessment in Section 9 of this report.

5.1 Consideration of a Reduced Lifespan of 25 years

It has been advised that Riverside Central may be considered to have a reduced lifespan of 25 years based on the Masterplanning for the Christchurch Hospital Campus. Clearly this reduces the aggregate risk over the total life remaining. However, the consideration of risk over an aggregate of 25 or 50 years does not change the risk of an earthquake occurring on any one day, or the consequences of an earthquake, which will be dependent on the actual shaking at the site for a given event. Considering risk for a reduced building life is not recognised by either the Buildings Act, or the *Seismic Assessment of Existing Buildings* (NZSEE, 2017) that is used to assess the seismic rating.

Therefore, although an assessment of a structure with a 25 year life may be useful in assessing the aggregate risk for an individual building, it should not be used for determining the requirement for a building with respect to adjacent buildings or for comparing one building against another.

If considering aggregate risk, NZS1170.0 provides the annual probability of exceedance for Ultimate Limit State for a design working life of 25 years. This relates to the aggregate risk over the 25 year design working life. The hazard level at the IL3-25yr classification is equivalent to IL2-50yr performance, and corresponds to 77% of the loadings used to design a similar new building at an Importance Level 3, 50 year design life (IL3-50yr).

Conceptual strengthening objectives to achieve an expected seismic performance of approximately 45% NBS (IL3-50yr) are provided in Section 6. Further strengthening to achieve an assessed seismic performance in the order of 52% NBS (IL3) is also presented. 52% NBS (IL3) for a 50 year design life has the same aggregate risk as 67% (IL3) for a 25 year life.

5.2 Assessed Seismic Rating to the 2017 NZSEE Engineering Assessment Guidelines

For the rating definition in the 2017 NZSEE Engineering Assessment Guidelines—required for confirmation that the building is not Earthquake-prone—an additional level of assessment is introduced. It requires the seismic evaluation to be repeated using 1.8 times the seismic loads used for the Ultimate Limit State assessment, but reduced factors of safety are permitted for capacities. Refer to Section 9 for further discussion.

The results of the DSA indicate the building's current assessed seismic rating for the primary structure to be <34% NBS (IL3). This rating assumes the IL3 classification for a standard life span of 50 years in accordance with the Joint Australian/New Zealand Standard – Structural Design Actions Part 0, AS/NZS 1170.0:2002.

The assessment of other secondary structural and non-structural elements of the building such as heavy plant, ceilings and appendages have not been included in this reassessment of the building. The capacity of these elements may govern the capacity of the building. It is recommended that an assessment of these elements be carried out.

Earthquake-prone status of Riverside Central

A building with an earthquake rating less than 34% NBS would fulfil one of the two tests for building to be considered Earthquake-prone Building (EPB) in terms of the Building Act 2004. A building rating less than 67% NBS is considered as an Earthquake Risk Building by the New Zealand Society for Earthquake Engineering. Riverside Central could be considered Earthquake-prone for the IL3 classification as described above.

This is a Grade D building following the NZSEE grading scheme, as shown in Table 2 below. Grade D buildings represent a risk to occupants 10-25 times greater than those expected for a new building, indicating a medium to high risk exposure relative to a new building if a large earthquake occurs.

Table 2: Grading system for earthquake risks

Percentage of New Building Standard (% NBS)	Seismic rating	Approx. risk relative to a new building	Life-safety risk description
>100	A+	Less than or comparable to	Low risk
80-100	A	1-2 times greater	Low risk
67-79	B	2-5 times greater	Low to Medium risk
35-66	C	5-10 times greater	Medium risk
20-34	D	10-25 times greater	High risk
<20	E	25 times greater	Very high risk

5.3 Identified Structural Weaknesses

A Structural Weakness (SW) is any aspect of the building structure and/or foundation soils that scores less than 100% NBS. Table 3 lists the SWs identified in the assessment. All other elements that could present a significant life safety hazard were assessed as having strength and deformation capacities above 100% NBS (IL3).

Items have been tabulated separately for their evaluation at ULS, and MCE.

Table 3: Structural Weaknesses identified in the DSA

Item	Building Element	Structural Weakness	% NBS (IL3)
1	Spandrel beams and supporting structure for Level 7 Plant Room water tanks – considered as full	Excessive plastic rotations under lateral loading in beams detailed with inadequate shear reinforcing. Onset of inelastic demands leading to strength degradation and potential loss of vertical load carrying capacity (MCE[2])	<34%

Item	Building Element	Structural Weakness	% NBS (IL3)
2	Wall Panels on the east elevation, south-east corner, around the services duct	Out-of-plane performance of the these panels, unrestrained over the height of the building, Flexural performance limited under inertial force face loading, resulting in potential local collapse hazard (ULS[1])	40%
3A	Columns on the north perimeter frame	Excessive plastic rotations under lateral loading. Inelastic demand leading to strength degradation and potential loss of vertical load carrying capacity (ULS[1])	60%
4A	Wall panel shear strength - various locations in the lower storeys	Wall panel shear strains in excess of Life Safety criteria below Level 2. Strength degradation leading to wall instability (ULS[1])	80%
5A	Coupling beams on the east and west perimeter	Excessive plastic rotation under lateral loading. Degrading beam strength leading to local collapse hazard (ULS[1])	80%
3B	Columns on the north perimeter frame	Excessive plastic rotations under lateral loading. Inelastic demand leading to strength degradation and potential loss of vertical load carrying capacity (MCE[2])	45%
4B	Wall panel shear strength - various locations in the lower storeys	Wall panel shear strains in excess of LS criteria below Level 2. Strength degradation leading to wall instability (MCE[2])	50%
5B	Coupling beams on the east and west perimeter (MCE[2])	Excessive plastic rotation under lateral loading. Degrading beam strength leading to local collapse hazard (MCE[2])	50%

[1] ULS is an abbreviation for the Ultimate Limit State evaluation, with loading determined in accordance with NZS 1170.5:2004, used for both the DA and NZSEE 2017 assessed seismic ratings.

[2] MCE is an abbreviation for Maximum Considered Earthquake shaking. These structural weaknesses relate to a lower margin of resilience to very strong shaking (beyond the loads for which a building is designed to according to NZS 1170.5:2004) when compared to the margin which a new similar building should possess. These SWs apply only to the NZSEE 2017 rating.

The critical or governing structural weakness

The Critical Structural Weakness (CSW) is the lowest scoring structural weakness, the failure of which would result in a significant life safety hazard. The CSW was found to be excessive plastic rotations in the spandrel beams and supporting structure of the Level 7 Plant Room, leading to strength degradation and potential for the loss of vertical load carrying capacity.

Severe structural weaknesses

A Severe Structural Weakness (SSW) is defined as a structural weakness that is potentially associated with catastrophic collapse and for which the probable capacity may not be reliably assessed based on current knowledge. The assessment did not identify any SSWs in the building.

Secondary structural and non-structural elements

The following secondary structural and non-structural aspects were considered in the assessment of the seismic rating, as reported above:

- South core egress stairs
- Out-of-plane sensitivity of slender wall panels

The assessment of other secondary structural and non-structural elements of the building such as heavy plant, ceilings and appendages have not been included in this reassessment of the building. The capacity of these elements may govern the capacity of the building. It is recommended that an assessment of these elements be carried out.

6 SEISMIC STRENGTHENING WORKS

6.1 Seismic Improvement Works Already Completed

Level 6 improvements to the short columns as recommended in the 2013 DSA report have been completed, as indicated in Figure 6, and have been included in the assessment.

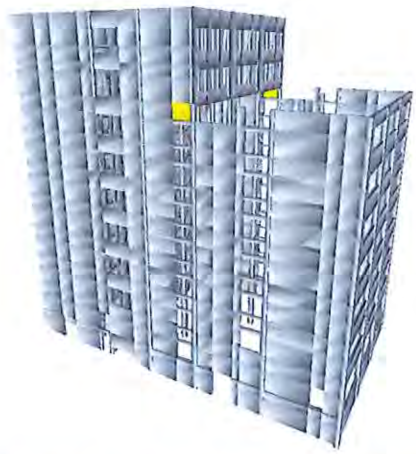


Figure 6: Level 6 strengthening works - infill walls highlighted in yellow

6.2 Strengthening Required to Achieve >34% NBS (IL3)

The seismic improvements listed below are required to achieve a global seismic rating greater than 34% NBS (IL3).

- Emptying water tanks at Level 7 or relocating to the basement or Lower Ground Floor of the structure. Current seismic performance is governed by the load carrying capacity of spandrel beams supporting the water tanks, and can be directly improved with a reduction in seismic mass and vertical load carried by the beams.
- Bracing of unrestrained perimeter wall panels. Walls around lift core and service riser are currently unrestrained over the high of the building and present an earthquake prone deficiency under face loading.

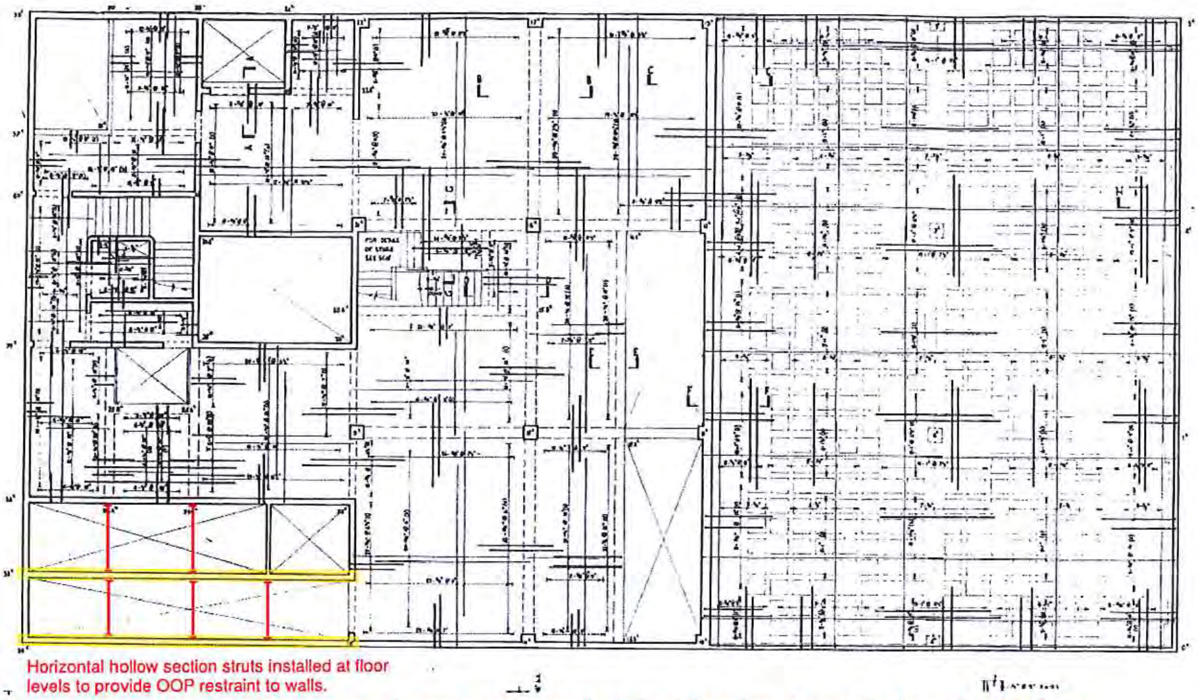


Figure 7: Typical floor plan showing proposed strengthening to upgrade out-of-plane deficiency to walls as highlighted

Proposed strengthening to unrestrained wall panels consists of a series of steel hollow section struts provided at floor levels to tie the walls back into the main floor slab diaphragm behind the lift core. Through site investigation we have confirmed both access and potential availability of space between lift shafts and risers. Further coordination will be required with services as the strengthening scheme is progressed.

If the strengthening work outlines above is completed, the capacity of the building could be 45% NBS (IL3).

6.3 Strengthening Required to Achieve 52% NBS (IL3)

Strengthening measures to improve seismic performance to 52% NBS (IL3) (which has an aggregate hazard level equivalent to 67% ULS shaking at IL3-25yr) include:

- Emptying water tanks at Level 7 or relocating to the basement or Lower Ground Floor of the structure. Current seismic performance is governed by the load carrying capacity of spandrel beams supporting the water tanks, and can be directly improved with a reduction in seismic mass and vertical load carried by the beams.

- Bracing of unrestrained perimeter wall panels. Walls around lift core and service riser are currently unrestrained over the high of the building and present an earthquake prone deficiency under face loading.

- Addition of new perimeter concrete shear wall overlays at various locations below Level 1. Existing shear walls classified as force controlled under ULS shaking can be strengthened by introducing new walls to improve dependable axial and shear capacity.

- Fibre Reinforced Polymer (FRP) wrap to columns of the north perimeter frame below Level 2. Existing columns subjected to excessive plastic rotations present a potential collapse hazard at the Collapse Limit State, and can be improved with FRP wrap.

- FRP wrap to perimeter coupling beams. Performance of coupling beams with excessive plastic rotation under lateral loadings can be improved with FRP wrap. These elements are not considered critical, as they aren't relied upon for vertical load carrying capacity, and therefore could be left unstrengthened provided any local fall hazards are eliminated.

- FRP wrap to slender columns on the south perimeter subjected to high axial loads. These columns are not considered critical as they aren't relied upon to for vertical load carrying capacity and could be left unstrengthened provided any local fall hazards are eliminated.

6.4 Further Strengthening to Achieve 67% NBS (IL3)

We have explored feasibility of strengthening to the higher performance level of 67% NBS (IL3) and have the following comments:

- The onset of wall damage appears significant throughout the building, rather than localised damage to discrete areas of the lower storeys. Onerous global strengthening would be required, where previously localised repair by concrete overlay appropriate at lower performance levels.
- Unrestrained wall panels with out-of-plane deficiency may require complete wall rebuild, rather than the remedial steel bracing at floor levels proposed in Section 6.2.

It appears that extensive global strengthening would be required to improve the assessed building performance to greater than 67% NBS (IL3). As discussed with representatives of the Ministry of Health, we haven't progressed the high-level concept further given the expected costs associated with implementing strengthening to this level of performance.

7 RECOMMENDATIONS

The strengthening listed in Section 6.2 should be implemented so that the assessed % NBS rating can be improved to at least 45% NBS (IL3).

8 BASIS FOR THE ASSESSMENT

The Detailed Seismic Assessment (DSA) of the primary structure has been completed in accordance with "The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments, July 2017, Version 1" (NZSEE, 2017). This document is referred to in this report as the **Engineering Assessment Guidelines.**

8.1 Information Available for the Assessment

The assessment has been based on the following information:

- A largely complete set of original Structural drawings dated 1967-1969.
- Previous Detailed Seismic Assessment report (Holmes Consulting, 2013)
- Ongoing on site investigations from March 2011 through August 2015 to assess post-earthquake damage and review concrete wall repair by epoxy crack injection.
- Holmes Solutions Non-Destructive of Reinforcing Steel in Riverside Hospital Building, CDHB, dated July 2011.

8.2 Site Observations and Investigations

The as-constructed building appeared to be generally consistent with the available drawings. Aside than the Level 6 short column strengthening works, no other significant modifications are known to have been made to the primary lateral or gravity systems.

Material testing was undertaken in July 2011 by Holmes Solutions and formed the basis for probable material strengths utilised in both the previous DSA report **(Holmes Consulting, 2013)**, and the latest seismic assessment.

Intrusive investigations to date have concluded that the observed structural damage should not have significantly reduced the seismic capacity of the building. Refer to the previous DSA for an outline of the earthquake damage and recommended repair work.

8.3 Earthquake Demands

The Ultimate Limit State (ULS) seismic demand is derived from the New Zealand Structural Design Actions Standard, NZS1170.5:2004 (Standards New Zealand, 2004). Factors that affect the earthquake demand are described below.

Earthquake Risk/Building Importance Level

For the analysis of Riverside Central, an Importance Level 3 classification, with a 50 year design life has been adopted in accordance with the New Zealand Structural Design Actions Standard, NZS1170.0:2002 (Standards New Zealand, 2011).

Seismic Hazard

We have used a hazard factor of Z=0.30 for the Christchurch region in accordance with NZS1170.5:2004 (Standards New Zealand, 2004).

Site and Subsoil Class

Seismic loads are also dependent on the soil type a structure is situated on. A Site Soil Class of D has been used, which is based on desktop review of historical information (mapped soil classes), and recent experience with geophysical testing that has been undertaken for adjacent sites on the Christchurch Hospital campus.

9 METHOD OF ASSESSMENT

The assessment uses a Non-Linear Time History Analysis (NLTHA), performed in accordance with ASCE 41-13 (ASCE/SEI, 2014) as cited by Section C1.6.2 of the Engineering Assessment Guidelines. The method of assessment is in accordance with the recommendations of Section C2 of the Engineering Assessment Guidelines.

Nonlinear Time History Analysis (NLTHA)

Non-linear time history analysis involves subjecting the computer model of the building to a large and variable suite of real earthquakes. The earthquake records are carefully selected and scaled so that they represent the types of earthquakes which contribute to a given seismic load level or seismic hazard for the site under consideration. The actual time-varying response of the structure to these records is analysed. As elements yield or become damaged, the analysis continually updates their properties so that the effect of that damage on response is directly incorporated into the assessment. The record suites for one load level comprise 11 different earthquakes. They are input into the building in different configurations such that a total of 88 analyses are run for an analysis at a single load level.

This procedure is selected as it is best suited to determining the displacement demands and mechanism/distribution of inelastic response and damage, on both the primary lateral system and the associated reinforced-concrete gravity structure. The components of the lateral and gravity supporting systems have varying levels of detailing, with neither fully compliant with modern standards.

This type of analysis also better captures the behaviour of irregular elements of the structure. Examples include the strengthened short columns on the East and West elevations of Level 6 at the building setback, and the heavily perforated wall elevations around the building's perimeter.

The procedure is also well suited to capture and assess accidental torsion in building response. Torsional irregularities may develop where the North Elevation frames begin to yield and a loss in stiffness softens frame response relative to the rest of the primary lateral system.

A two tier assessment process (ULS and MCE)

For the %NBS score which complies with the new 2017 Engineering Assessment Guidelines definition, a two tier assessment is performed. The first tier involves assessment of ultimate capacities against new building design loads, with appropriate margins of safety.

The second tier requires explicit checks to be made at 1.8 times the earthquake loads used to design a new building, but reduced factors of safety are permitted for capacities. This second MCE check more explicitly acknowledges that some aspects of existing building detail may not be as robust as new buildings and may perform in a more brittle manner. Therefore including explicit assessment of performance at higher loads better reflects overall seismic performance and risk for a more robust %NBS score. Two tier assessment will result in a different assessed rating to a single tier "ULS only" approach if the MCE check governs.

In the terminology of ASCE 41-13, the assessment uses Life Safety LS limits with ULS loading, and Collapse Prevention CP limits at 1.8 times ULS loading (MCE) as set by section C1.6.2 of the Engineering Assessment Guidelines.

Further information/technical details on ground motion selection and scaling, analysis procedures and evaluation can be found in Appendix B *Technical Summary of NLTHA Analysis*.

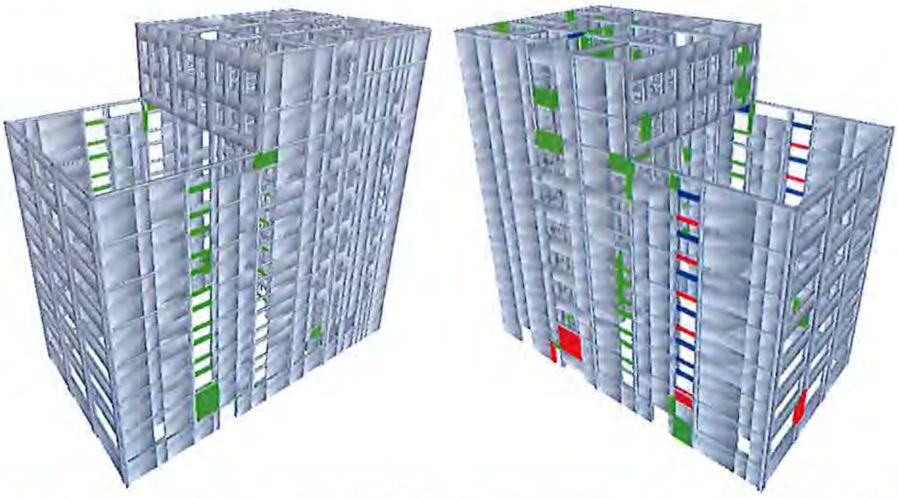


Figure 8: Graphics from analysis, and example frame damage states at 52% ULS (IL3) (left); and 85% ULS (IL3) (right)

10 REFERENCES

ASCE/SEI. (2014). *ASCE/SEI 41-13 Seismic Evaluation and Retrofit of Existing Buildings*. ASCE.

ASCE/SEI. (2017). *ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. American Society of Civil Engineers.

ASCE/SEI. (2017). *ASCE/SEI 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. ASCE.

Bradley, B. A., & Tarbali, K. (2017). *Default ground motion ensembles for response history analysis of structures in Christchurch, Wellington and Auckland - Revised Draft 22 June 2017*. Bradley Seismic Limited.

Fenwick, R., Bull, D., & Gardiner, D. (2010). *2010-02 Assessment of hollow-core floors for seismic performance*. University of Canterbury.

Holmes Consulting. (2013). *Seismic Evaluation of the Riverside Central Building - Revision 1, March 2013*. Holmes Consulting Group LP.

Holmes Consulting. (2017). *High-Level Review of Impact from NZSEE Assessment Guideline changes: Update on the structural assessments at the Christchurch Hospital Campus - Revision 2.1, 11 December 2017*. Holmes Consulting LP.

Mondkar, D., & Powell, G. (1979). *ANSR II Analysis of Non-linear Structural Response User's Manual, EERC 79/17*. Berkeley: University of California.

NEHRP. (2010). *NIST GCR 10-917-5 Nonlinear Structural Analysis for Seismic Design*. National Institute of Standards and Technology.

NZGS/MBIE. (2016). *Earthquake Geotechnical Engineering Practice - Module 4: Earthquake Resistance Foundation Design, Rev. 0*. MBIE.

NZSEE. (2017). *The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments, Initial Release July 2017*. Wellington: MBIE.

Preistley, M., Calvi, G., & Kowalsky, M. (2007). *Direct Displacement Based Design of Structures*. Pavia: IUSS Press.

SESOC. (2013). *Interim Design Guidance Version No. 0.9 March 2013 - Design of Conventional Structural Systems*. SESOC.

Standards New Zealand. (2004). *NZS 1170.5 New Zealand Structural Design Actions Part 5: Earthquake Actions*. Wellington: Staandards New Zealand.

Standards New Zealand. (2008). *NZS 3101:2006 Concrete Structures Standard, Incorp. Amd. 1 & 2*. Wellington: Standards New Zealand.

Standards New Zealand. (2011). *AS/NZS 1170.0:2002 Structural Design Actions Part 0: General principles, Incorp. Amd. 1 to 5*. Wellington: Standards New Zealand.

Appendices

CONTENTS

Appendix A

Engineering Assessment Technical Summary

Appendix A Engineering Assessment Summary Report (NZSEE 2017)

This is an *Engineering Assessment Summary Report*, as referred to in Section A8.5 of the Engineering Assessment Guidelines, and which meets the requirements of Section 2.5 of the Earthquake-prone Building methodology. This summary report is based off the template available on the EQ-assess website, (Version 1.1 - 14 August 2017).

For this DSA on Riverside Central, the template is filled out only with the assessed seismic rating for the primary structure to the NZSEE 2017 Engineering Assessment Guidelines.

Building Information	
Building Name/ Description	Riverside Central – Primary Structure Only
Street Address	Riccarton Ave, Christchurch Hospital Campus
Territorial Authority	Christchurch City Council
No. of Storeys	1 basement level plus 7 suspended concrete floor levels plus concrete plant level
Area of Typical Floor (approx.)	700
Year of Design (approx.)	1967
NZ Standard Designed to	Likely to have used NZSEE bulletins
Structural System including foundations	Concrete shear walls and north elevation reinforced concrete moment resisting perimeter frame on perimeter basement walls on strip footings, internal reinforced gravity frame on raft.
Key features of ground profile and identified geohazards	Gradually sloping site, proximity to Avon River to the north-east
Previous strengthening	Post 2013 DSA strengthening to Level 6 short column.
Heritage Issues/Status	-
Other	-
Assessment Information	
Consulting Practice	Holmes Consulting
CPEng Responsible	Didier Pettinga
Date/Version of Drawings Reviewed	1967
Geotechnical Report(s)	-
Date Building Inspected	August 2013
Previous Assessment Reports	2013

Building Information	
Other Relevant Information	-

Summary of Engineering Assessment Methodology and Key Parameters Used	
Occupancy Type(s) and Importance Level	Hospital Clinical Services, IL3-50yr
Site Subsoil Class	D
Summary of Assessment Methodology Used	Alternative Verification, NLTHA to ASCE 41-13
Other Relevant Information	-

Assessment Outcomes	
Assessment Status	Draft
Assessed Seismic Rating	<34% NBS (IL3-50yr) for the assessed seismic rating to the NZSEE 2017 Engineering Assessment Guidelines.
Seismic Grade	D
For an ISA:	
Describe the Potential Critical Structural Weaknesses	-
Does the result reflect the building's expected behaviour or is more information/analysis required?	-
Is a DSA recommended?	-
If Yes, what form should the DSA take/what are the specific areas to focus on?	-
For a DSA:	
Describe the Governing Critical Weakness and Likely Mode of Failure	Significant inelastic demand in spandrel beam and supporting structure below Level 7 water tanks leading to strength degradation and potential for loss of vertical load carrying capacity. (CP limits compared to 1.8 x ULS).

Assessment Outcomes	
Comment on Parts identified and assessed	Main egress stairs & wall panel OOP capacity. It is recommended that an Assessment of all secondary structure and non-structural elements is carried out.
Recommendations	Empty or relocate water tanks to base of the structure. Strengthen east elevation wall panels with OOP deficiency.

Appendix B

Technical Summary of NLTHA Analysis and Evaluation

Appendix B Technical Summary of NLTHA Analysis

B.1 Introduction

NLTHA using ASCE 41-13 (ASCE/SEI, 2014) is used for the complete building assessment. This procedure is selected as it is best suited to determining the displacement demands and mechanism/distribution of inelastic response on both the primary lateral system and the associated reinforced-concrete gravity structure. The components of the lateral and gravity supporting systems have varying levels of detailing, with neither being fully compliant with modern standards.

This type of analysis also better captures the behaviour of irregular elements of the structure. Examples include the strengthened short columns on the East and West elevations of Level 6 at the building setback, and the heavily perforated wall elevations around the building's perimeter.

The procedure is also well suited to capture and assess accidental torsion in building response. Torsional irregularities may develop where the North Elevation frames begin to yield and a loss in stiffness softens frame response relative to the rest of the primary lateral system.

B.1.1 Alternative Verification Methodology

This assessment uses Non-Linear Time History Analysis (NLTHA) to ASCE 41-13 Seismic Evaluation and Retrofit of Existing Buildings, as permitted by section C1.6.2 of the Engineering Assessment Guidelines. It follows the Tier 3 systematic evaluation using the Non-linear Dynamic Procedure (NDP). Parameters used in application of the ASCE 41 Tier 3 assessment procedure are summarised in Table 4.

Table 4: Parameters for the application of ASCE 41 under the Engineering Assessment Guidelines (100% NBS for BPON)

ASCE Standard and revision	ASCE 41-13
Analysis Procedure	Non-Linear Dynamic Procedure (NDP)
Return Period for Life Safety (LS)	1000 years
Collapse Prevention (CP) scale factor	1.8
Ground Motion Selection	(Bradley & Tarbali, 2017)
Ground Motion Scaling	ASCE 7-16 Amplitude Scaling
Target Spectrum	NZS 1170.5:2004
Structural Performance Factor, Sp	NZS 1170.5:2004 Cl. 5.5.2, Eq. 5.5(1)
%NBS score for SSWs	Half of LS Score at 1000 years

B.1.2 Purpose of NLTHA Analysis

Primary structural elements and most secondary structural elements are modelled and their performance directly assessed and evaluated. Secondary structural or non-structural elements which are included in the scope of assessment, but which not directly modelled, use the force and/or deformation demands from the NLTHA.

Structural behaviours which are not directly modelled (out-of-plane panel performance for example) are evaluated outside of the analysis by post processing outputs from the NLTHA (such as peak floor accelerations and drifts) and back checking these against elemental capacities.

Other analysis types, model analysis and non-linear static pushover analysis are used for model verification only and not final assessment. Refer to section B.2.9 for further details.

B.2 Analysis Model

B.2.1 Computer Program ANSR

The analysis model is built using ANSR, Holmes Consulting's in-house non-linear modelling software. The software comprises the non-linear analysis engine ANSR II from (Mondkar & Powell, 1979), further developed with additional functionality and input and output processing. The program is based on a hybrid of concentrated plasticity and distributed plasticity models, for reinforced concrete frame and wall elements respectively.

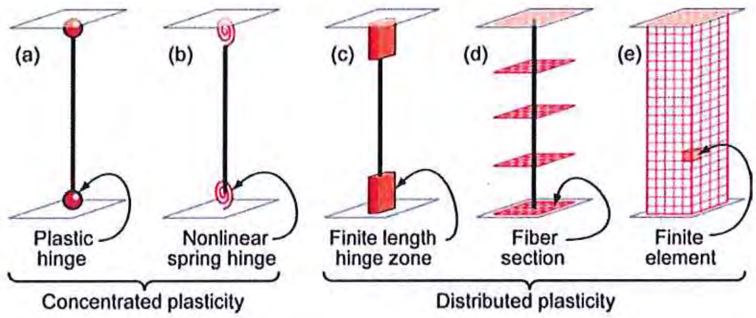
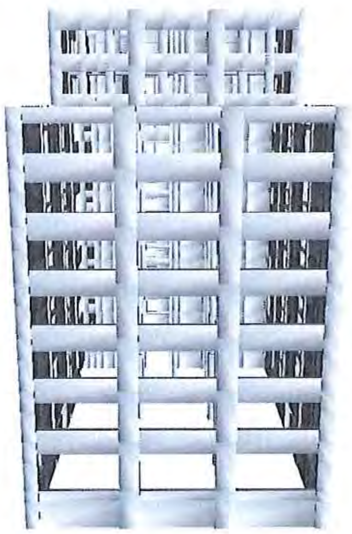
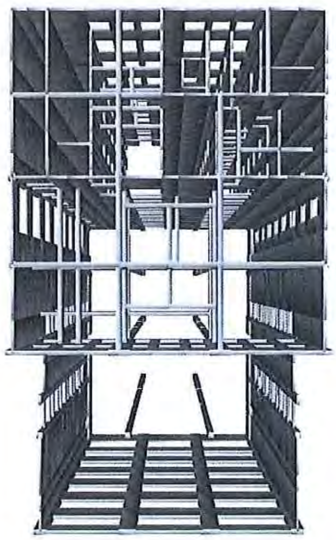


Figure 9: ANSR II uses concentrated plasticity (a) and distributed plasticity (c), from (NEHRP, 2010)

The program can perform and process linear modal analysis, non-linear gravity analysis, non-linear static analysis (NSP or pushover) and non-linear dynamic analysis (NLTHA or NDP). For the NSP or NDP analyses, the self-weight and applied beam/column imposed are applied in a gravity analysis prior to the NSP or NDP itself.

Non-linear displacement pushovers can also be performed. These involve applying a user defined displacement history to a selected node (usually a diaphragm node). These are generally only used for model verification and testing.





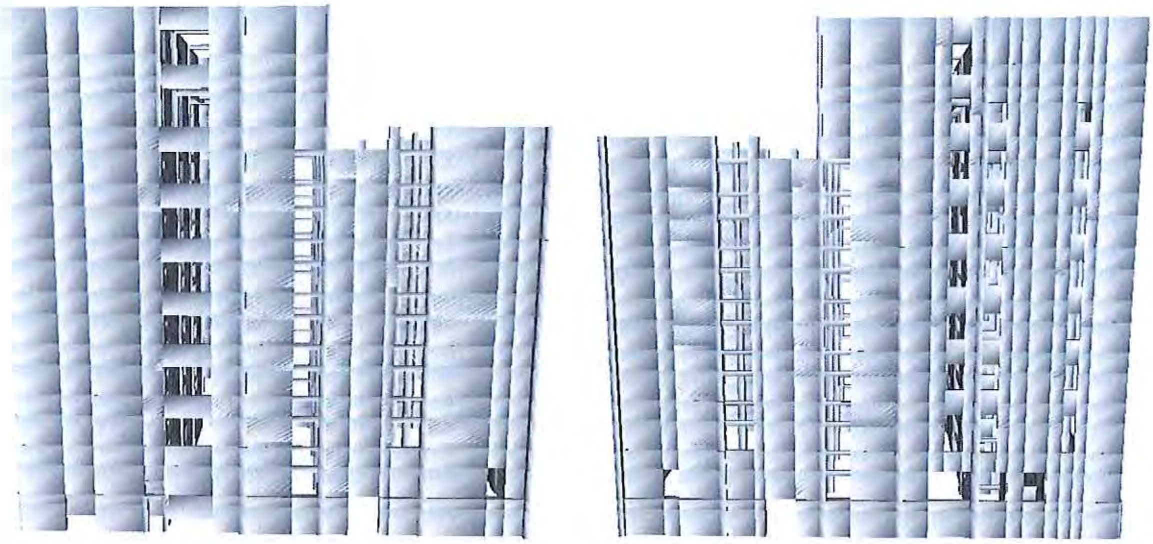
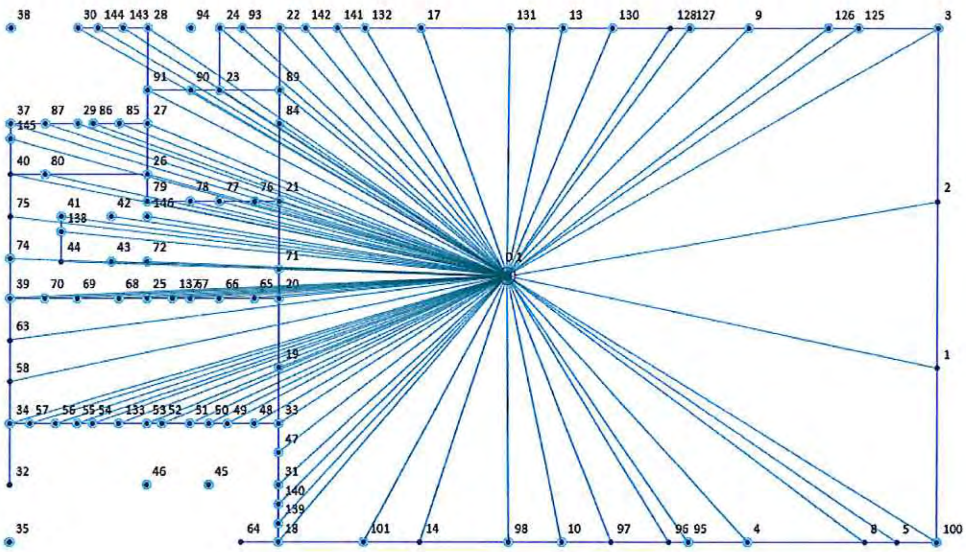


Figure 10: Analysis model renderings

B.2.2 General Description and Model Geometry

Geometry was described by a series of column numbers to identify plan locations and elevations to identify sections in the vertical plane. Figure 11 shows the column line locations in a plan view of the model.

The column line locations identify each of the columns and beam intersections in the building, the ends and corners of walls and wall openings. A total of 21 levels were defined in the vertical direction, with all elevations based on RLs off the original drawings.



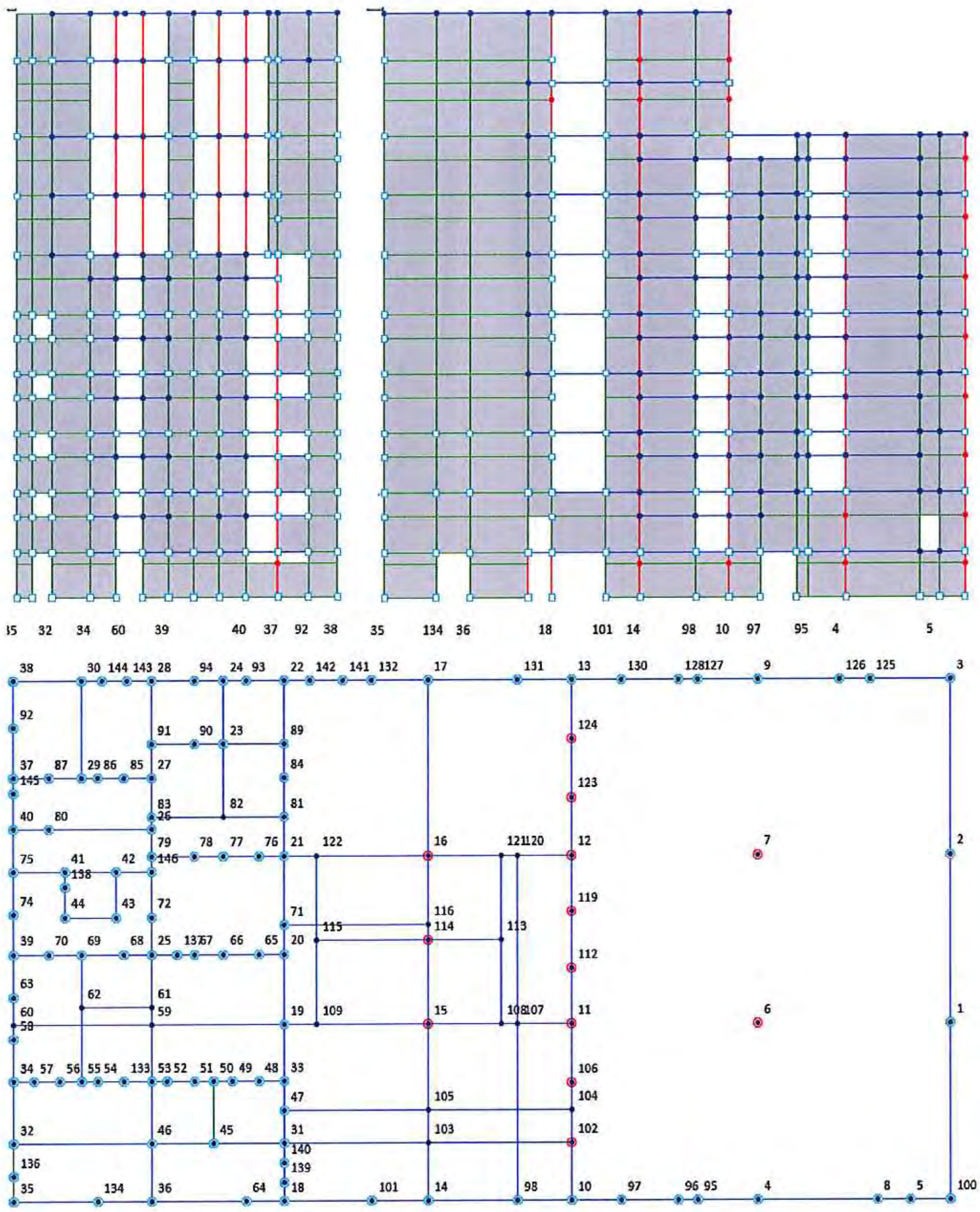


Figure 11: Model column line and elevation plots; bottom figure shows all elements at all levels

B.2.3 Material Properties

Typical concrete strength was specified on the drawings as 28MPa. The probable strength for assessment will be taken as 1.5 times nominal strength, or 41MPa. Ground and basement level columns are specified as a higher strength concrete of 35MPa, corresponding to a probable strength of 52MPa.

Typical steel reinforcing was specified as grade 275MPa on the drawings. The probable strength of beams and columns used was based on a yield strength of 275 MPa x 1.08 = 300 MPa. This steel grade was used for main steel in typical beams and columns and for transverse reinforcement to all beams and columns. For high tensile grade reinforcing, the probable strength is based on a yield strength of 414MPa x 1.08 =

447MPa. This grade of steel was typically used for ground and basement level columns and in some beam locations.

Materials used for analysis are listed in Table 5.

Table 5: Material Properties for NLTHA and Evaluation

Material ID	Material Type	f_m Masonry f_c Concrete F_y Steel	E	G	ρ	Weight Density γ_w	Mass Density γ_m
1	CONCRETE	27600	24341853	10142439	0.200	25.00	2.55
2	STEEL (STRUCTU	300000	200000000	76923077	0.300	78.00	7.95
3	STEEL (STRUCTU	447000	200000000	76923077	0.300	78.00	7.95
4	CONCRETE	41400	28261820	11775758	0.200	25.00	2.55
5	CONCRETE	51700	30771700	12821542	0.200	25.00	2.55

B.2.4 Elements and Element Modelling Parameters

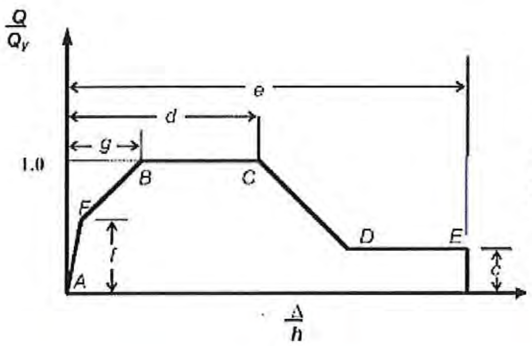
Concrete wall element shear model

The shear stiffness of the concrete shear wall elements is modelled using plane stress elements with a thickness corresponding to the values specified on the drawings.

The modelled behaviour includes degradation in strength and stiffness, depending on the level of shear stress. The wall panel yield function is shear controlled. A strength envelope as shown in Figure 12 is developed. Regions are defined by the strength provided respectively by the concrete, the nominal strength of the shear reinforcing (strain of 0.004) and onset of strength degradation at a strain which is a function of the axial load level.

If the axial load level is greater than 0.15 Ag f'c, then the element is force controlled at the limiting strain is that at which the elastic strength of the wall is reached (0.004).

Under cyclic loading, non-recoverable stiffness degradation is modelled, however for the NLSPA, the load-deformation response follows the strength backbone (Figure 13). Modelling parameters and acceptance criteria are provided in Table 6.



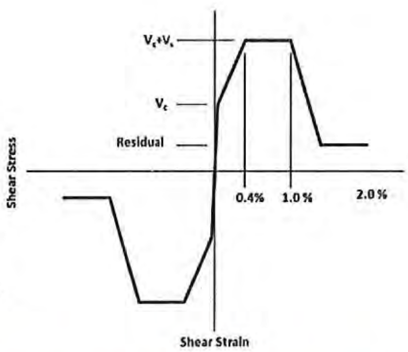
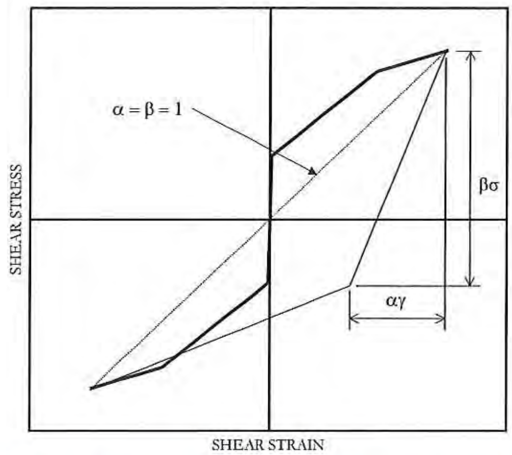


Figure 12: Concrete wall element shear strength backbone definitions (left); strength backbone for a wall with high axial load (right)



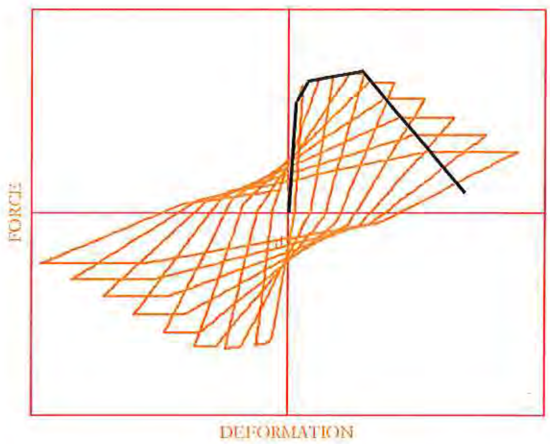


Figure 13: Definition of stiffness degradation (left); in-cycle degradation used for static force pushover shown in black, and cyclic degradation used for NLTHA shown orange (right). Figures indicate a strain hardening slope but this is set to zero per Figure 12.

Table 6: Modelling Parameters and Numerical Acceptance Criteria for Reinforced Concrete Shear Walls

Condition	$\frac{V}{t_w l_w \sqrt{f'_c}}$	Total drift or Plastic Rotations Angle, Radians[1,2]			Residual Strength[1]		Plastic Rotations Angle, Radians[1]		
		d	e	g	c	f	IO	LS	CP
Condition i. Shear walls and wall segments controlled by shear[3]									
$\frac{(A_s - A_s')f_y + P}{t_w l_w f_c'} \le 0.05$		0.010	0.020	0.004	0.20	0.60	0.004	0.015	0.020
$\frac{(A_s - A_s')f_y + P}{t_w l_w f_c'} > 0.05$		0.0075	0.010	0.004	0.00	0.60	0.004	0.0075	0.010

[1] For details of load deformation relationships for shear dominated walls refer Figure 12.

[2] For shear walls and shear wall segments, use drift; for coupling beams use chord rotation; refer to ASCE 41-06 Figures 6-3 and 6-3.

[3] For shear walls and wall segments where inelastic behaviour is governed by shear, the axial load on the member must be < $0.15f'_cA_g$ otherwise the member must be treated as force controlled.

Concrete wall element flexure model

To model flexural yielding, gap elements are placed at each node across the length of the wall at the elevation at which yielding is expected to occur. Each gap element contains two elements in parallel, a concrete element which is elastic in compression but with zero tensile strength and a reinforcing bar element which is bi-linear, with yielding in both tension and compression.

The concrete area and steel area at each gap is taken as the tributary areas of all panels incident to each gap. Column elements (which pass through the walls as boundary piers) cross the gaps and therefore also carry tensions and compressions. Plastic deformations in these frame elements (all column elements) include both axial extensions and chord rotation in proportions determined by the vector through the yield surface.

Flexural strength and stiffness degradation can be included, however wall flexure in this model is not controlling response or assessment. Therefore the hinge elements are bilinear with no strength or stiffness degradation. Non-confined boundaries are assumed.

Table 7 Modelling Parameters and Numerical Acceptance Criteria for Reinforced Concrete Shear Walls

$\frac{(A_s - A_s')f_y + P}{t_w l_w f_c'}$	Conf. Boundary[1]	$\frac{V}{t_w l_w \sqrt{f'_c}}$	Plastic Rotations Angle, Radians		Residual Strength Ratio	Plastic Rotations Angle, Radians[7]		
			a	b	c	IO	LS	CP
Condition i. Shear walls and wall segments controlled by flexure[7]								
≤0.1	Yes	≤0.33	0.015	0.020	0.75	0.005	0.010	0.015
≤0.1	Yes	≥0.5	0.010	0.015	0.40	0.004	0.008	0.010
≥0.25	Yes	≤0.33	0.009	0.012	0.60	0.003	0.006	0.009
≥0.25	Yes	≥0.5	0.005	0.010	0.30	0.0015	0.003	0.005
≤0.1	No	≤0.33	0.008	0.015	0.60	0.002	0.004	0.008
≤0.1	No	≥0.5	0.006	0.010	0.30	0.002	0.004	0.006
≥0.25	No	≤0.33	0.003	0.005	0.25	0.001	0.002	0.003
≥0.25	No	≥0.5	0.002	0.004	0.20	0.001	0.001	0.002

[1] A boundary element shall be considered confined where transverse reinforcement exceeds 75% of the requirements given in ACI 318 and spacing of transverse reinforcement does not exceed 8db. It shall be permitted to take B-8symmetry parameters and acceptance criteria as 80% of confined values where boundary elements have at least 50% of the requirements given in ACI 318 and spacing of transverse reinforcement does not exceed 8db. Otherwise, boundary elements shall be considered not confined.

[7] ASCE 41 primary deformation limits are used given recognising current modelling limitations.

Modelled concrete wall elements

Walls are typically shear controlled but modelled with gap/truss elements at each level up the height of each wall elevation to capture any flexural yielding and evaluate against strain elongation and allowable plastic rotation criteria. Walls are typically doubly reinforced but not adequately confined. Perimeter walls are linked by slender coupling beams (relative to wall geometry) not providing significant coupling behaviour to influence global performance.

All walls in this model have low axial load ratios, as verified through hand calculation and back checking the model gravity analysis.

Frame element model

Frame elements use concentrated plasticity models. For beam and column frame elements the strength is modelled as a bi-linear yield function (up to a certain point beyond which strength is degraded). The elastic stiffness is based on effective properties up to the calculated probable yield moment. Properties are defined by axial area, shear area and moment of inertia about each axis.

Beams have a separate yield moment specified for positive and negative bending at each end of the beam. Once the yield moment is attained, the flexural stiffness is reduced to the initial stiffness times the specified strain hardening ratio.

Columns are represented by a flexural element similar to beams. However, the yield moments about each axis and the axial load are coupled. An interaction diagram is calculated based on probable material strengths. For the default yield surface (concrete columns), the interaction between bending moments and axial load is parabolic and is defined by:

$$\sqrt{\left(\frac{M_y}{M_{yu}}\right)^2+\left(\frac{M_z}{M_{Zu}}\right)^2+\left(\frac{F-F_o}{F_u}\right)^2}=1.0 \qquad F_o=\frac{1}{2}(F_{ut}-F_{uc}) \qquad F_u=\frac{1}{2}(F_{ut}+F_{uc})$$

My, Mz and F denote bending moments about the element local y and z axes and the axial force respectively (F is negative for compression). Subscript u denotes ultimate. Fut and Fuc are axial ultimate strengths in tension and compression (taken as absolute value). Myu and Mzu are the maximum balanced moments. As for the beams, a bilinear strain hardening yield function is generally used in the model.

The flexural element used for beams and columns permits degrading strength and/or stiffness characteristics to be specified. The implementation of strength backbones and stiffness degradation is shown in Figure 14, and the modelling parameters and acceptance criteria are listed in Table 8 and Table 9. The acceptance criteria for frame elements are secondary limits (in the terminology of the earlier version of ASCE 41-06), which acknowledge that the effect of element degradation on response and displacement demands is directly modelled.

Cyclic degradation is used for dynamic or cyclic analysis. For static analysis (force pushover) the degradation follows the strength backbone curve.

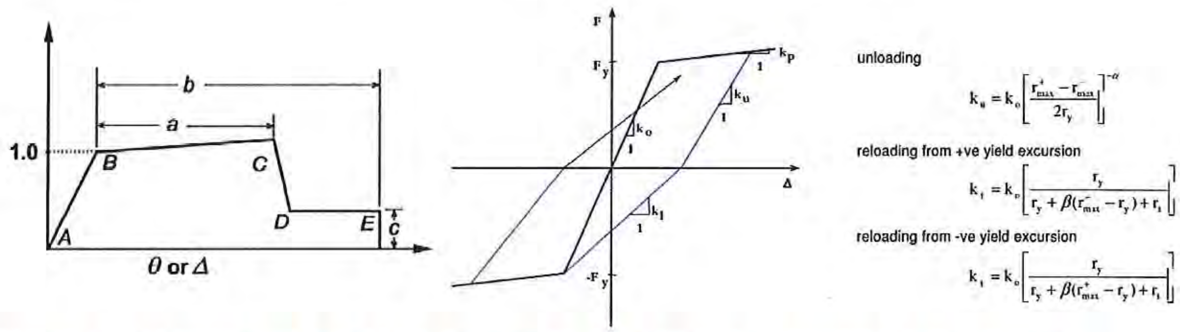


Figure 14 Definition of Modelling Parameters for Reinforced Concrete Frame Elements

Table 8 Modelling Parameters and Numerical Acceptance Criteria for Reinforced Concrete Columns

$\frac{P}{A_g f_c'}$	$\frac{A_v}{b_w s}$	$\frac{V}{b_w d\sqrt{f'_c}}$	Plastic Rotations Angle, Radians		Residual Strength Ratio	Plastic Rotations Angle, Radians		
			a	b	c	IO	LS	CP
Condition i. Columns controlled by flexure mode[1,2]								
≤0.1	≥0.006		0.035	0.060	0.2	0.005	0.045	0.060
≥0.6	≥0.006		0.010	0.010	0.0	0.003	0.009	0.010
≤0.1	=0.002		0.027	0.034	0.2	0.005	0.027	0.034
≥0.6	=0.002		0.005	0.005	0.0	0.002	0.004	0.005
Condition ii. Columns controlled by shear-flexure mode[1,2]								
≤0.1	≥0.006	≤0.25	0.032	0.060	0.2	0.005	0.024	0.032
≤0.1	≥0.006	≥0.5	0.025	0.060	0.2	0.005	0.019	0.025
≥0.6	≥0.006	≤0.25	0.010	0.010	0.0	0.003	0.008	0.009
≥0.6	≥0.006	≥0.5	0.008	0.008	0.0	0.003	0.006	0.007
≤0.1	≤0.0005	≤0.25	0.012	0.012	0.2	0.005	0.009	0.010
≤0.1	≤0.0005	≥0.5	0.006	0.006	0.2	0.004	0.005	0.005

$\frac{P}{A_g f_c'}$	$\frac{A_v}{b_w s}$	$\frac{V}{b_w d \sqrt{f'_c}}$	Plastic Rotations Angle, Radians		Residual Strength Ratio	Plastic Rotations Angle, Radians		
			a	b	c	IO	LS	CP
≥0.6	≤0.0005	≤0.25	0.004	0.004	0.0	0.002	0.003	0.003
≥0.6	≤0.0005	≥0.5	0.0	0.0	0.0	0.0	0.0	0.0
Condition iii. Columns controlled by shear mode[1,2,6]								
≤0.05	n.a.		0.01	0.02	0.2	0.0	0.015	0.020
≥0.05	n.a.		0.0075	0.01	0.0	0.0	0.0075	0.010
Condition iv. Columns controlled by inadequate development[1,2]								
≤0.1	≥0.006		0.0	0.060	0.4	0.0	0.045	0.060
≥0.6	≥0.006		0.0	0.008	0.4	0.0	0.007	0.008
≤0.1	≤0.0005		0.0	0.006	0.2	0.0	0.005	0.006
≥0.6	≤0.0005		0.0	0.0	0.0	0.0	0.0	0.0

[1] Refer to Section 10.4.2.2.2 ASCE 41-13 for definition of conditions i, ii, and iii except that the transitional shear demand/capacity ratio applicable to conditions i & ii has been reduced from 0.6 to 0.55 (refer Section 5.3). Columns will be considered to be controlled by inadequate development or splices when the calculated steel stress at the splice exceeds the steel stress specified by Equation 10-2 of ASCE 41-13. Where more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.

[2] Where P > 0.7Agf'c, the column is assumed to be force controlled all performance levels unless columns have transverse reinforcement consisting of hoops with 135 degree hooks spaced at ≤ d/3 and the strength provided by the hoops (Vs) is at least three-fourths of the design shear. P is the design axial force in the member. Alternatively, use of axial loads determined based on a limit state analysis shall be permitted.

[3] Linear interpolation between values listed in the table for conditions (i) to (iv) shall be permitted.

[4] V is the design shear force calculated using limit-state analysis procedures in accordance with Section 10.4.2.3 of ASCE 41-13.

Table 9 Modelling Parameters and Numerical Acceptance Criteria for Reinforced Concrete Beams

$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$	Plastic Rotations Angle, Radians		Residual Strength Ratio	Plastic Rotations Angle, Radians		
			a	b	c	IO	LS	CP
Condition i. Beams controlled by flexure[1]								
≤0.0	C	≤0.25	0.025	0.05	0.2	0.010	0.035	0.05
≤0.0	C	≥0.5	0.02	0.04	0.2	0.005	0.02	0.04
≥0.5	C	≤0.25	0.02	0.03	0.2	0.005	0.02	0.03
≥0.5	C	≥0.5	0.015	0.02	0.2	0.005	0.015	0.02
≤0.0	NC	≤0.25	0.02	0.03	0.2	0.005	0.02	0.03
≤0.0	NC	≥0.5	0.01	0.015	0.2	0.0015	0.01	0.015
≥0.5	NC	≤0.25	0.01	0.015	0.2	0.005	0.01	0.015
≥0.5	NC	≥0.5	0.005	0.01	0.2	0.0015	0.005	0.01
Condition ii. Beams controlled by shear[1]								
Stirrup spacing ≤ d/2			0.003	0.02	0.2	0.0015	0.01	0.02
Stirrup spacing > d/2			0.003	0.01	0.2	0.0015	0.005	0.01
Condition iii. Beams controlled by inadequate development or splicing along the span[1]								
Stirrup spacing ≤ d/2			0.003	0.02	0.0	0.0015	0.01	0.02
Stirrup spacing > d/2			0.003	0.01	0.0	0.0015	0.005	0.01
Condition iv. Beams controlled by inadequate embedment into beam-column joint[1]								
			0.015	0.03	0.2	0.01	0.02	0.03
Condition v. Beams conforming to NZS3101:2006[4]								
Ductile			$18\phi_y l_p$	$27\phi_y l_p$	0.2	0.01	$18\phi_y l_p$	$27\phi_y l_p$
Limited Ductility			$10\phi_y l_p$	$15\phi_y l_p$	0.2	0.01	$10\phi_y l_p$	$15\phi_y l_p$

[1] Where more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.

[2] "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ d/3, and if, for components of moderate and high ductility demand, the strength provided by the hoops (Vs) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.

[3] Linear interpolation between values listed in the table shall be permitted.

[4] Detailing consistent with requirements of Section 9.4, NZS3101:2006. Values provided are for reversing plastic hinges. Deformation capacity of unidirectional plastic hinges may be taken as twice the value given.

Beam condition (v) relates to beams that conform to the detailing requirements of NZS 3101 (SNZ, 2006) for ductile and limited ductile buildings. Yield curvature, ϕ_y, is given by the following expression:

$\phi_y = \frac{2f_y}{E_s h}$, and l_p is taken as the smaller of 0.5hb, or max [$0.25M^*/V^*$ or $0.25h_b$]

Concrete column elements

Columns are all assigned as condition i, ii, iii or iv elements. As the condition number is a function of axial load and shear demand, the program automatically calculates the appropriate condition number at each time step, and interpolates for the appropriate modelling parameters. No confinement or shear checks are required in post-processing, as this is already incorporated in the assignment of modelling parameters and the setting of plastic rotation limits.

The effective stiffness values are based on ASCE 41-13 Table 10.5. Generally 0.3Ig has been used for all columns, where axial load ratios are less than or equal to 0.1. A strain hardening ratio of 0.03 is used to set the post yield slope.

Concrete beam elements

Beams are controlled by flexural and shear behaviour, the majority of which do not conform to NZS 3101:2006 ductile detailing requirements, and therefore use condition i and ii modelling parameters.

The effective stiffness values are based on ASCE 41-13 Table 10.5, with 0.3Ig used for all beams. A strain hardening ratio of 0.03 is used to set the post yield slope.

Beam column joints

The predominant mechanism of inelastic response is beam hinging, and therefore columns have fully rigid joint offsets, and beams have no rigid offset. An explicit joint model is not included, and so checks are completed in post processing.

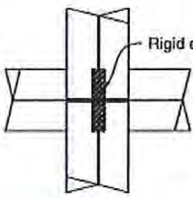


Figure 15: Joints are modelled with rigid column offsets and no beam offset (ASCE/SEI, 2014)

B.2.5 Building Masses, Weights, Mass Eccentricity and P-Delta

The seismic weight of the buildings was assembled from element self-weights plus applied diaphragm seismic mass. The applied seismic mass for typical levels was based on the average self-weight of various depths of waffle flooring system with allowance for areas of in-situ flat slab, 4.5 kPa, plus a superimposed dead load 1.0 kPa and seismic live load of 2.0 x 0.5 x 0.3 = 0.3 kPa over the total floor area. Larger loads have been assigned to the plant room slab and basement slabs where floor self-weights are different or where additional loads are specified on the drawings.

Mass eccentricity is 0.05 times the building width.

Table 10 lists the total seismic weight at each floor. The seismic combination gravity loads are calculated independently of the seismic weight, using element self-weight plus applied vertical loads. Applied vertical loads are generally distributed beam loads. Node loads are also applied to column lines, and distributed loads applied to walls by dummy beams with pinned end conditions.

Once modelling was completed a gross check was made by comparing the gravity loads in the model with the seismic weights. The gravity load at foundation level is 73 MN (6% less than the seismic weights).

P-delta is not currently considered in the analytical model due to the computation time associated to the number of potentially non-linear elements in modelling. Given the relatively low drift demands observed in building response, we conclude that P-delta not required for this application of modelling to the Engineering Assessment Guidelines. We have identified that local P-delta effects have a minor impact on wall out-of-plane response, however our modelling approach has not sought to capture this behaviour, and it will instead be reviewed by a separate hand calculation.

Table 10: Building Masses and Weights

Level Name	ANSR Level No.	Elevation (m)	Seismic Weight (kN)	Floor Area (m^2)	Floor Load (kPa)	Mass Ecc. X (m)	Mass Ecc. Z (m)
Roof	21	37.44	3237	450	3.1	1.11	1.03
Plant	20	34.39	3331	141	6.4	0.54	1.03
L7	19	32.92	4718	237	5.7	0.57	1.03
L6	17	29.56	8929	675	5.7	1.87	1.03
L5	15	25.75	8620	704	5.7	1.87	1.03
L4	13	21.94	8621	693	5.7	1.87	1.03
L3	11	18.13	8579	693	5.7	1.87	1.03
L2	9	14.32	8486	693	5.7	1.87	1.03
L1	7	10.52	8432	704	5.7	1.87	1.03
G	5	6.71	8473	704	5.7	1.87	1.03
LG	3	2.90	8466	704	5.7	1.87	1.03
B	1	0.00					

B.2.6 Diaphragm Modelling

Diaphragms are stiff compared to the vertical elements of the lateral load resisting system and are therefore modelled as rigid for analysis. Diaphragm connectivity is removed from perimeter walls to the south-east and south-west where voids in the floor diaphragm negate wall restraint. Connectivity and diaphragm nodes for a typical level are shown in Figure 16.

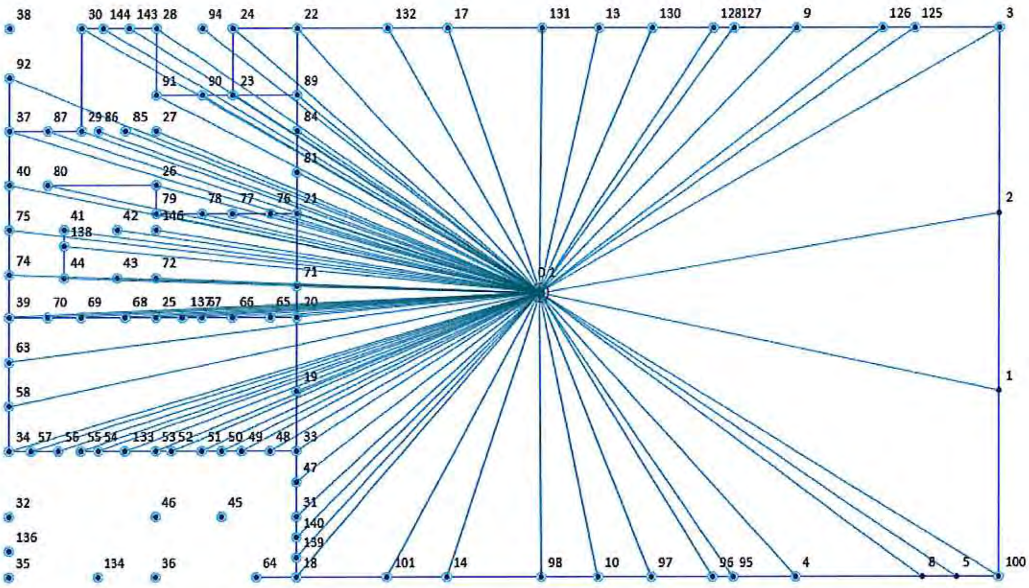


Figure 16: Diaphragm Connectivity for a typical level above ground

B.2.7 Foundation/Soil Interface Modelling

Figure 17 shows the portion of structure below ground level. Perimeter supports were modelled only underneath the lines of the primary perimeter transfer columns, and these were modelled with gap elements so that they support compression loads but provide no resistance to uplift. The superstructure response is not considered sensitive to a variation in subgrade stiffness, as the basement rotational stiffness is high relative to superstructure response. Compression gaps are therefore modelled with a high stiffness.

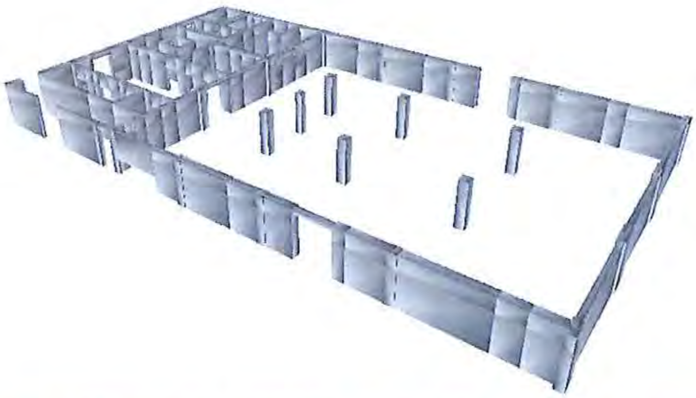
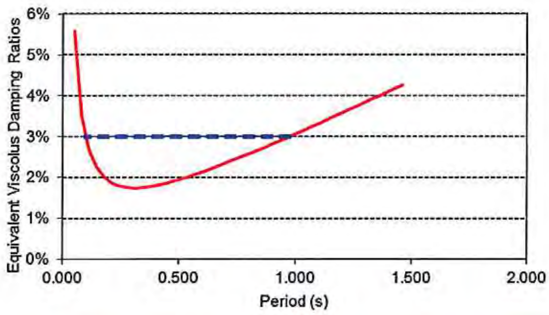


Figure 17: Foundation modelling with uplifting gaps under perimeter basement walls, and internal columns

B.2.8 Damping for NLTHA

Damping in NLTHA uses Rayleigh damping coefficients set in accordance with ASCE 41-13, as shown in Figure 18. The target damping is 3%. The coefficients are set to achieve no more than the target damping between the period T_{90} required to achieve 90% mass participation of the superstructure (excluding the basement mass), and 1.5 times T_1 which allows for period lengthening within the expected range of global system displacement ductilities.



	T (s)	Damping
1.5 x T_1	0.975	0.03
T_{90}	0.1	0.03

α	0.3507
β	0.0009

$$[C] = \alpha[M] + \beta[K]$$

Figure 18: Rayleigh damping coefficients for NLTHA

Initial stiffness damping of elements is used for the stiffness proportional part of the damping matrix. We note that in the fundamental mode most associated with ductile response and period lengthening, the damping comes from the mass proportional part (not the stiffness proportional part).

B.2.9 Modal Response

A linear elastic modal analysis was used to extract the fundamental periods and mode shapes. Modal properties are provided in Table 11, the first three dominant modes are shown graphically in Figure 19. For the modal analysis presented here the mass has been removed from the basement and ground level.

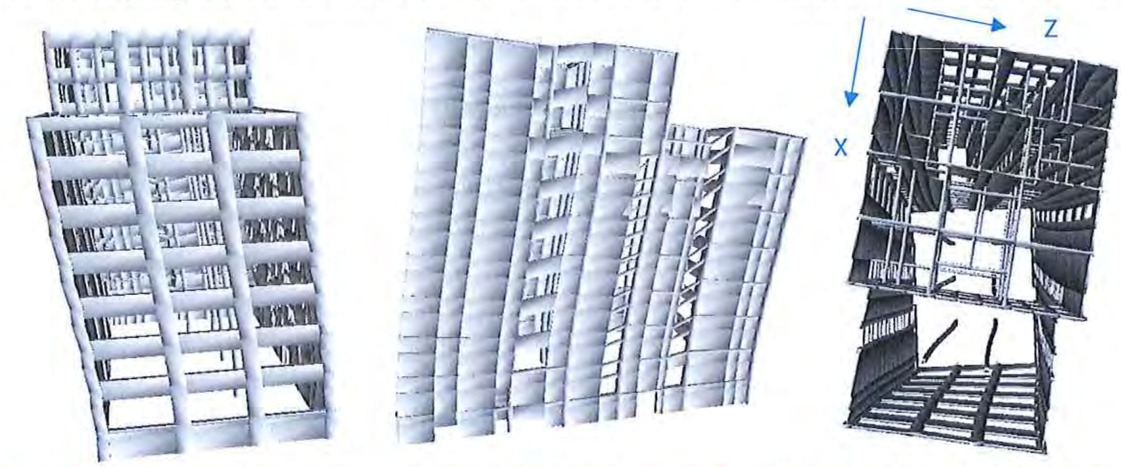


Figure 19: Fundamental modes - 0.65s Z translation, 0.41s X translation, and 0.12s torsion

Table 11: Modal properties

Mode Number	Period (Seconds)	Effective Mass			Cumulative Mass		
		EM X	EM Z	EM YY	SEM X	SEM Z	SEM YY
1	0.656	0.1%	70.3%	0.0%	0.1%	70.3%	0.0%
2	0.415	66.6%	0.1%	0.0%	66.7%	70.4%	0.0%
3	0.247	0.0%	0.2%	0.0%	66.7%	70.6%	0.0%
4	0.247	0.0%	0.0%	0.0%	66.7%	70.6%	0.0%
5	0.246	0.0%	0.0%	0.0%	66.7%	70.6%	0.0%
6	0.246	0.0%	0.0%	0.0%	66.7%	70.6%	0.0%
7	0.246	0.0%	0.0%	0.0%	66.7%	70.6%	0.0%
8	0.222	0.0%	0.1%	0.0%	66.7%	70.7%	0.0%
9	0.191	0.0%	11.2%	4.1%	66.8%	81.9%	4.2%
10	0.178	0.0%	0.3%	0.1%	66.8%	82.2%	4.3%
11	0.123	0.2%	2.5%	26.2%	66.9%	84.7%	30.5%
12	0.110	15.8%	0.2%	0.1%	82.7%	84.9%	30.6%
13	0.105	0.0%	0.0%	0.1%	82.8%	84.9%	30.7%
14	0.102	0.0%	0.0%	2.0%	82.8%	84.9%	32.7%
15	0.098	0.3%	0.8%	33.8%	83.1%	85.7%	66.5%
16	0.083	0.0%	3.1%	6.2%	83.1%	88.7%	72.7%
17	0.066	0.0%	0.3%	1.9%	83.1%	89.0%	74.5%
18	0.058	0.2%	1.6%	0.1%	83.3%	90.5%	74.6%
19	0.057	5.5%	0.0%	0.0%	88.8%	90.6%	74.6%
20	0.056	0.0%	0.2%	0.0%	88.8%	90.8%	74.6%
21	0.048	0.0%	0.0%	2.6%	88.8%	90.8%	77.2%
22	0.046	0.0%	1.2%	0.6%	88.8%	91.9%	77.8%
23	0.040	3.1%	0.0%	0.1%	91.9%	92.0%	77.9%
24	0.038	0.0%	0.3%	0.5%	91.9%	92.2%	78.4%
25	0.037	0.1%	0.8%	1.7%	92.0%	93.1%	80.2%
26	0.032	0.1%	0.8%	0.9%	92.1%	93.8%	81.1%
27	0.032	0.0%	0.4%	0.1%	92.1%	94.2%	81.2%
28	0.031	1.8%	0.0%	0.0%	93.9%	94.2%	81.2%
29	0.029	0.0%	0.0%	0.0%	94.0%	94.2%	81.2%
30	0.028	0.0%	4.9%	0.6%	94.0%	99.1%	81.8%

B.3 Seismic Input and Response

B.3.1 Target Spectra, Ground Motion Selection and Scaling

Target spectrum

The parameters in Table 12 are used to generate the target spectrum. A Structural Performance factor of Sp = 1.0 is adopted, in accordance with NZS 1170.5:2004 Cl. 4.4.2, as the structural system as such is unlikely to be able to reliably sustain displacement ductilities.

Table 12: Seismic parameters for target spectrum at 100% NBS

Loading Standard	NZS 1170.5:2004
Hazard Factor, Z	0.3
Return Period Factor, R	1.3
Site Subsoil Classification	D
Source Distance, D	20km
S_p	1.0

Selection of ground motions

Ground motions have been taken from (Bradley & Tarbali, 2017), a technical report prepared for Holmes Consulting which provides ground motion ensembles for NLTHA in New Zealand.

Whilst the NZS 1170.5:2004 spectrum is used as the target spectrum for scaling of motions, the selection process is based on site specific seismic hazard analysis for a generic site in Christchurch, using the 2010 NSHM (Stirling et al, 2012). The Generalized Conditional Intensity Measure (GCIM) approach is used, which considers multiple intensity measures, and enforces appropriate variability in ground motions. Scale factors in these tables relate to the GCIM selection method, conditioned to the spectral acceleration at the stated period from the site specific hazard analysis. Therefore they are different to the scale factors used in analysis.

Scaling Procedure

We have used amplitude scaling in accordance with the procedure in ASCE 7-16 (ASCE/SEI, 2017). ASCE 41-13 permits this as an alternative procedure subject to rational basis, which we outline as follows:

- In ASCE 7-16 amplitude scaling, maximum direction spectra are computed for each record and used for scaling to the target spectra. This is consistent with the NZS 1170.5:2004 target spectrum, which is a maximum direction spectrum. It is therefore a better approach for New Zealand building assessments.
- The ASCE 7-16 is a robust, codified procedure available for use in the design of new buildings.
- The draft of ASCE 41-17 includes a similar procedure, which is not expected to be substantively altered before publication. There are some minor variations between ASCE 7-16 and the implementation proposed in ASCE 41-17, primarily with respect to the rotation of record pairs in near fault regions.

The ASCE 7-16 method requires 11 records pairs to be amplitude scaled so that their maximum direction spectra generally fit the target spectrum. The normative requirement is that the average of the maximum direction spectra shall not fall below 90% of the target response spectrum for any period with the period range. This period range is defined as $T_{(\text{sum 90\% mass participation})}$ to $1.5T_1$.

To meet these requirements, we have applied parts of the framework in NZS 1170.5, using k_1 and k_2 factors to determine overall scale factors which comply with ASCE 7-16 Cl. 16.2.3.1, using the following process.

- Generally fit the records using NZS 1170.5:2004 Cl. 5.5.2 (v) minimising in a least mean squares sense the error function log (k_1 SA_{MDS}/SA_{target}) over a period range of interest. For this project, we used a scaling range of $0.4T_1$ to $1.5T_1$ for ULS, and $0.7T_1$ to $1.5T_1$ for MCE. This is to avoid spurious scale factors giving extremely large spectral displacement demands on individual records. This can sometimes occur when trying to minimise error at very short periods.
- Apply a k_2 family scale factor to all records, such that the average MDS is not less than 90% of the target spectrum over the full period range of 0.1s to 1.5 T_1.

GM Record Suite

The same record suite was used for ULS shaking (based on a 1000 year return period) and MCE shaking (notionally based on 1.8 x ULS). Assessment as a %NBS compares LS and CP criteria against the respective 100% ULS and MCE shaking demands. Therefore shaking is reduced until LS/CP criteria is met by linearly scaling these records. Scaling of the ground motion ensembles for ULS shaking is provided in the figures and tables below.

Table 13: Ground motions for 1000 year return period, Christchurch (Soil Class D) conditioned on SA(0.5s)from (Bradley, et al., 2017).

GM Record #	RSN	Earthquake Name	Station Name	Year	Magnitude, M_w	Distance, R_{rup} (km)	Vs30 (m/s)	Scale Factor, k1	Scale Factor, k1*k2
1	776	Loma Prieta	Hollister - South & Pine	1989	6.93	27.7	282	0.81	1.02
2	777	Loma Prieta	Hollister City Hall	1989	6.93	27.3	199	1.17	1.48
3	960	Northridge-01	Canyon Country - W Lost Cany	1994	6.69	11.4	326	0.77	0.98
4	1119	Kobe, Japan	Takarazuka	1995	6.9	0.0	312	0.52	0.66
5	1703	Northridge-06	Jensen Filter Plant Administrative Building	1994	5.28	6.9	373	1.93	2.44
6	3830	Yountville	Napa - Napa College	2000	5	15.5	332	1.96	2.47
7	4100	Parkfield-02, CA	Parkfield - Cholame 2WA	2004	6	1.6	173	0.64	0.81
8	4862	Chuetsu-oki	Shiura Nagaoka	2007	6.8	10.6	337	1.08	1.36

9	6906	Darfield, New Zealand	GDLC	2010	7	1.2	344	0.53	0.67
10	8063	Christchurch, New Zealand	Christchurch Botanical Gardens	2011	6.2	5.5	187	0.76	0.96
11	8118	Christchurch, New Zealand	Papanui High School	2011	6.2	9.1	263	1.11	1.40

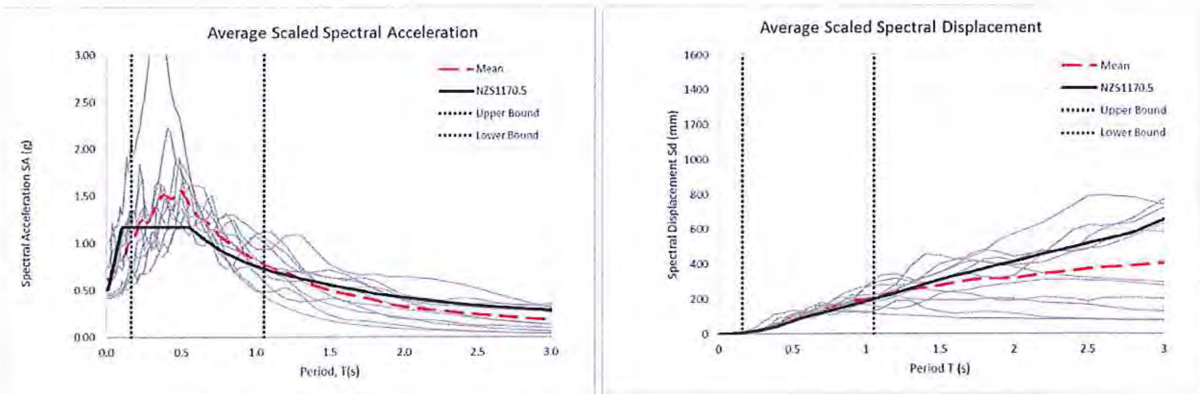


Figure 20: Scaled records for 1000 year return period (for Sp = 1.0 target spectrum)

B.3.2 NLTHA Summary of Performance

The following summaries describe general modelled performance and controlling modes of response. Refer to the evaluation section B.4 for evaluation of all elements including those behaviours such as diaphragm performance and out-of-plane sensitivity of slender walls which are not explicitly modelled.

ULS Shaking and Life Safety LS Criteria

At 45% ULS shaking, performance is stable. Peak roof displacements are in the order of 100mm. Peak centre of mass drifts are less than 0.15%, and corner column drifts less than 0.20%.

The perimeter walls respond elastically for this level of shaking, with a large number of coupling beams exceeding Immediate Occupancy IO plastic rotations limits but within LS limits.

There is some onset of damage to a small number of wall panels below Level 2, corresponding to IO exceedance and a maximum shear strain of 0.004 but within LS limits.

The global modelled behaviour achieves 45% NBS for ULS shaking (LS criteria)

MCE Shaking and Collapse Prevention CP Criteria

At 90% ULS shaking, one of the 11 runs had excessive vertical displacements and did not complete in at least one of the mass eccentricities in each direction. A small number of runs did not converge due to numerical instability.

At 85% ULS shaking, all runs converged and completed, allowing average response to be computed. Peak roof displacements are 350mm. Peak centre of mass drifts are 0.40% and peak corner column drifts are 0.60%. Peak drifts occur at a concentration of inelastic demand between Ground Floor and Level 3, due to reducing storey shear capacity.

Most elements remain within CP limits. Extensive hinging occurs in the north perimeter frame, and to the coupling beams of the east and west elevations. Localised areas of basement and lower ground wall undergo significant inelastic action.

The global modelled behaviour meets 85/1.80 = 45% NBS for MCE shaking (CP criteria).

Figure 21: Graphics from a sample of records illustrating general mechanism of response.

B.3.3 NLTHA Building Global Displacements, Drifts and Accelerations

Global response quantities of displacement, interstorey drift and floor accelerations are plotted at diaphragm nodes and corner columns for 45% ULS and 85% ULS shaking which represent limiting seismic load levels for ULS (LS) and MCE (CP) limit states respectively. Two sets of plots are provided for each level of shaking, giving the average response for records with the primary component oriented in the X-direction, followed by the Z-direction.

The average response is computed by taking the envelope of response from the 4 mass eccentricities which are run for a given record, and averaging these envelopes across the 11 records.

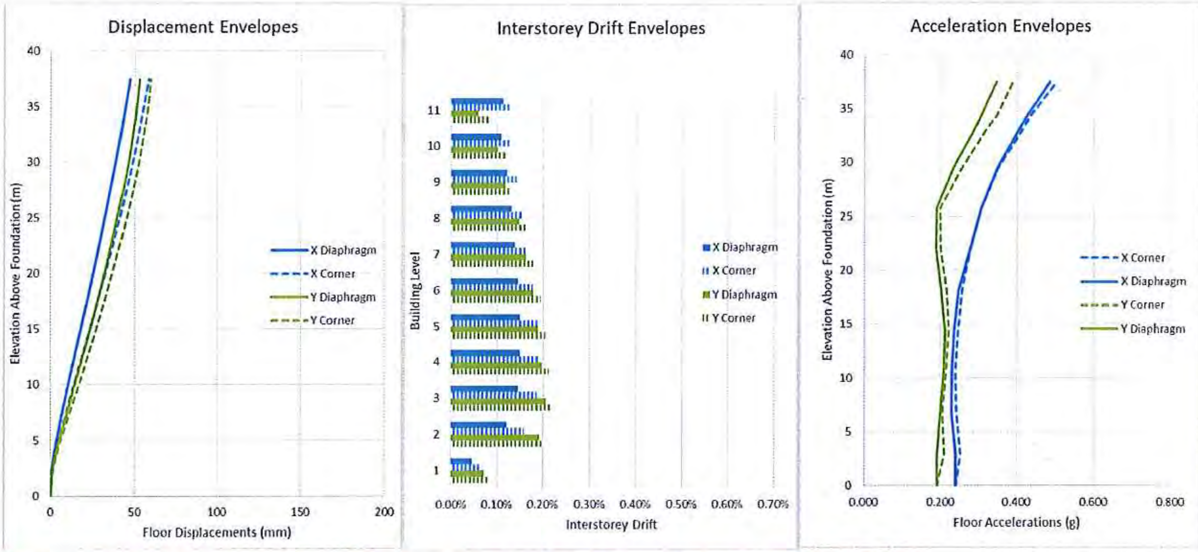


Figure 22: Displacements, Drifts and Accelerations for 40% ULS Shaking

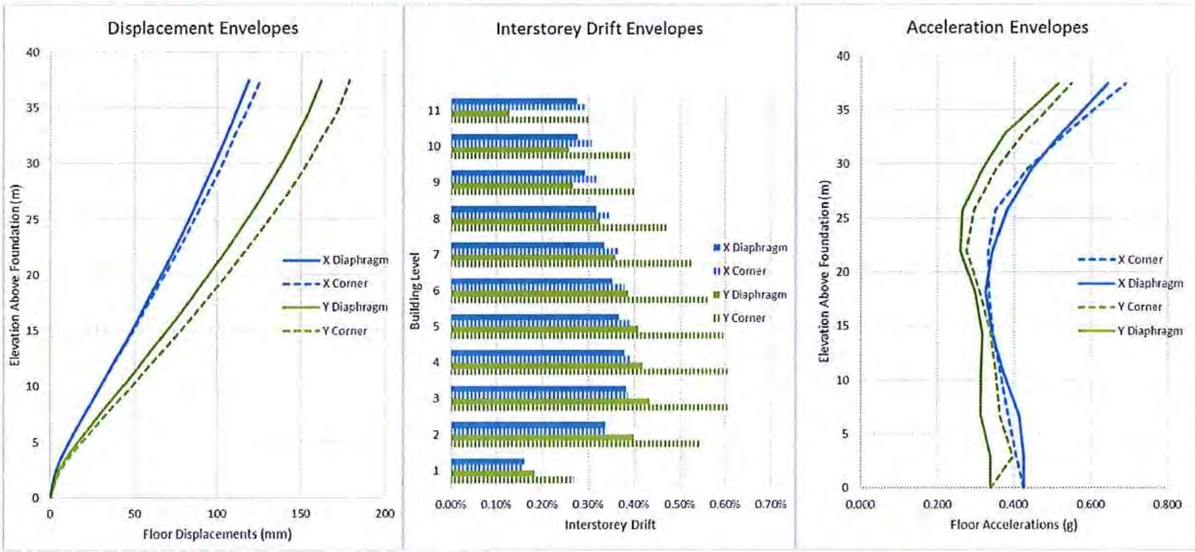


Figure 23: Displacements, Drifts and Accelerations for 70% ULS Shaking

B.3.4 Sensitivity Studies

Contribution of water tank weights

The Level 7 Plant Room water tanks currently house 75,000 litres of water in various tanks distributed across the floor plate. The additional weight of the water tanks has a detrimental effect on the transfer structure supporting Level 7, and adjacent spandrel beams of the RC frame at the building setback.

Building response is such that the imposed loading from full water tanks appears to govern global performance. Beams are insufficiently detailed to deal with excessive plastic rotation demands under increased shaking at the collapse prevention criteria. Plastic rotation demands lead to strength degradation and eventual loss of vertical load carrying capacity in the form of a local or potential global collapse mechanism.

Sensitivity studies have been completed to investigate the influence of half full, and completely emptied tanks on global building response. Not until the water tank weights are completely removed from the analysis modelling did we see significant improvement in building response to above 34% NBS at the collapse prevention criteria.

We recommend the Level 7 Plant Room water tanks be completely emptied or relocated to the base of the structure to improve global seismic performance to above 34% NBS. NLTHA evaluation presented below is predicated on this recommendation being implemented, as indicated by the MOH.

B.4 NLTHA Evaluation

This section contains the evaluation of all elements directly modelled, and also those secondary structural or non-structural elements not directly modelled by which are included in the scope of this assessment.

Evaluation below is predicated on the implementation of recommendations listed in Section 6.2. That is to say the assessed performance is presented with the first tier of upgrades complete, and global seismic performance improved to above 34% NBS.

Use of average response for evaluation

Unless noted otherwise, all response quantities in this section are average quantities. These are obtained by average the envelope of the response from the four mass eccentricities run for a given ground motion record, and averaging these over the 11 ground motions records. This is carried out independently for records with primary components oriented in the X, and the Z directions – generally the graphics and output provided present each independently.

Graphical plots showing element damage states

Graphics showing the element damage states from analysis are provided in Figure 24, Figure 25, and Figure 26 for 52% ULS shaking, 70% ULS shaking and 85% ULS shaking respectively. Presented damage stages are summarised below:

- No colour: un-yieldied or only nominal ductility, < Immediate Occupancy IO damage state
- Green: exceeds IO, but < Life Safety LS limit state
- Blue: exceeds LS, but < Collapse Prevention CP limit state
- Red: exceeds CP limit state

Figure 24: Element Damage State – 52% ULS Shaking for average response with primary components in X and Z direction respectively

Figure 25: Element Damage State - 70% ULS Shaking for average response with primary components in X and Z direction respectively

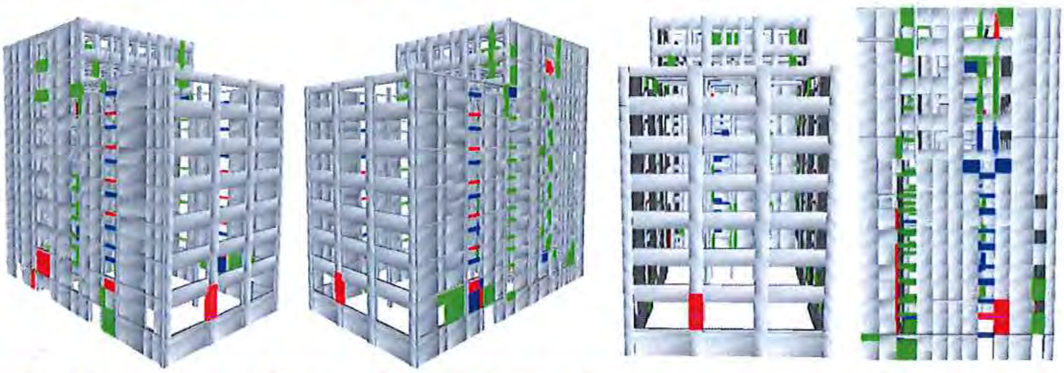


Figure 26: Element Damage State - 85% ULS Shaking for average response with primary components in X and Z directions respectively

B.4.1 Primary Lateral Load Resisting Structure

Evaluation of concrete columns

Other than a small number of columns on the south elevation, the onset of IO limit exceedance begins at approximately 60% ULS shaking. All columns were generally within LS criteria in the 70% ULS runs (Figure 25). Beyond 70% ULS shaking we observed a disproportionate increase in inelastic demand resulting in strength degradation in the north elevation columns and potential loss of vertical load carrying capacity (Figure 26)

Columns were evaluated as 80/1.8 = 45% NBS at the CP criteria.

As noted in Section B.2.4 describing column element modelling, evaluation for plastic rotation incorporates confinement and shear checks in the assignment of condition numbers.

Evaluation of concrete beams - flexure

All beams were within LS criteria for 52% ULS shaking (shown green in Figure 24).

At 85% ULS shaking (Figure 26), a significant number of perimeter coupling beams exceeded LS limits. This indicates damaged to the point that beam strength was significantly degraded. Observed coupling beam deficiencies are considered non-critical given that beam strengthen degradation will not be the sole contributor to global collapse of the structure.

Beam performance is assessed as limited to 85/1.80 = 50% NBS at the CP criteria.

Evaluation of concrete beams - shear

Presently, concrete contribution to shear capacity is checked using NZSEE 2006 guidelines, as this method reduces the concrete contribution as a function of curvature ductility (Figure 27). Curvature ductility is calculated using the plastic rotations from analysis, a yield curvature of 1.5 ε_y/h and an assumed hinge length of $0.5h_b$. The concrete contribution is subtracted from the total shear demand to report the required area of steel. The transverse reinforcing of all beam elements was tabulated and compared with these requirements, and no shear deficiencies were reported.

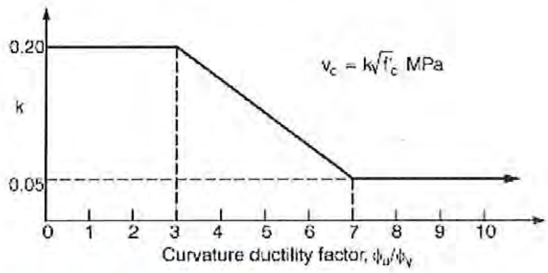


Figure 27: Calculation of vc using NZSEE 2006

Calculation of strengths to ASCE 41-13 (i.e. ACI 318-11) does not reduce the concrete contribution. However the influence of curvature ductility on shear behaviour is accounted for by reduction of the plastic rotation limits (which are a function of shear stress). Therefore the method used is conservative.

Evaluation of beam column joints

Beam column joint performance is evaluated outside of the analysis, in post-processing. The demands are calculated based on the maximum moment demands in the beams on each side of the joint, divided by a lever arm which of 0.85 x beam depth, conservatively representing the moment lever arm across all beams from the beam strength calculations. Column shears are subtracted in order to derive joint shears for assessment.

Beam column joint strengths are checked against these demands using the calculation for V_n in ASCE 41-13. Factors of γ for joints strength calculation are tabulated in Table 14, and are a function of whether the joint is interior or exterior, the level of confinement provided by transverse beams, and whether the transverse reinforcement is conforming or non-conforming (stirrups spaced at < $h_c/2$ within the joint).

Table 14: γ factors for joint strength calculation to ASCE 41-13

Condition	i. Interior		ii. Exterior	
Trans Beams	Yes	No	Yes	No
C	20	15	15	12
NC	12	10	8	6

$$V_n = 0.083\lambda\gamma\sqrt{f'_c}A_j \text{ (MPa units)}$$

where λ = 0.75 for lightweight aggregate concrete and 1.0 for normal-weight aggregate concrete;

All north perimeter frame joints (contributing to lateral response) were satisfactory at 45% ULS shaking, and also at 85% ULS shaking.

Evaluation of Walls

Wall performance does not appear to control global building response. The maximum shear strain from 85% ULS shaking is 0.008 in sections of the basement and lower ground walls. This indicates up to about 200% utilisation in shear, but notably in localised areas where redistribution is possible and expected.

Wall performance was assessed at 85/1.8 = 50% NBS at the CP criteria.

B.4.2 Diaphragm Evaluation

The building has a 300mm typical waffle slab flooring system which is assumed to provide an effective stiff diaphragm to transfer floor inertial loads to the primary lateral load resisting system. Local slab thickenings are detailed at columns locations, with adequate strength to resist punching shear action. The building is considered plan regular, with a squat floor aspect ratio.

The maximum diaphragm node acceleration at 50% ULS shaking is 1.9g. For a seismic floor mass of 8500kN, this gives a total inertia force of 16000kN to be delivered to the the south perimeter walls, core structure, and north perimeter frame in the more critical transverse direction. Based on a simple strut and tie diaphragm model, there appears to be sufficient capacity in the floor connection to transfer inertial forces at 50% ULS to lateral load resisting elements. Diaphragm performance is therefore not considered criterial, but should be considered in more detail if further strengthening measures are sought.

B.4.3 Evaluation of Seismic Gaps

Pounding potential to adjacent structures (Riverside East and West, and the Clinical Services Building to the south) has not been reassessed as part of these works. Referring to the HCG Pounding Assessment Report, dated 16 June 2015, pounding is not expected to occur until approximately 70% ULS shaking at IL3. Given that the assessed seismic performance of each of the buildings sits between 34-55% NBS, we find it reasonable to conclude that pounding will not significantly influence individual building responses unless further strengthening is sought.

If strengthening measures were explored to increase building capacity to in excess of 70% ULS shaking, pounding potential would require further consideration.

B.4.4 Foundation System

For a maximum base shear coefficient from 70% ULS shaking of 0.25, the base shear can be resisted by base friction and passive resistance against the side of the embedded wall strip footings. The maximum base uplift recorded was 30mm, located in the south-west corner of the building.

The response of the building is assessed as structurally dominated, as on the basis of the above, foundation performance as it relates to superstructure response should not contribute to a significant life safety hazard for less than 67% NBS.

B.4.5 Secondary Elements

South Core Egress Stairs

Based on a review of the existing structural drawings and confirmation through site observation, the stairs appear to be cast in place concrete construction. Given that the stairs are bounded within a 'box' shape form of walls, they are unlikely to become a potential collapse hazard until inter storey drift demands are well in excess of that at the assessed seismic capacity of the building. The stairs are therefore not considered a critical structural weakness.

At just what level of shaking the stairs would be considered a potential collapse hazard is difficult to ascertain, but is likely larger than 67% NBS at IL3. If the building was to be strengthened to 67% NBS at IL3, or greater, then remediation of these stairs should be considered in accordance with the Engineering Assessment Guidelines.

Out-of-Plane Capacity of Slender Wall Panels

Wall panels in the south-east corner around the service riser and lift core are not connected to the floor diaphragm and therefore unrestrained over full height. These panels are sensitive to out-of-plane inertial forces under earthquake excitation. Assessed using the Parts and Components section of NZS1170.5:2004, and an averaged maximum acceleration response from the NLTHA, these wall panel have a likely seismic performance of 40% NBS. Out-of-plane performance of these walls is not considered a critical structural weakness, but is a known deficiency and potential local collapse hazard requiring upgrade.

Proposed strengthening to unrestrained wall panels consists of a series of steel hollow section struts provided at floor levels to tie the walls back into the main floor slab diaphragm behind the lift core. Through site investigation we have confirmed both access and availability of space between lift shafts and risers. Further coordination will be required with services as we progress this strengthening scheme.

B.5 Summary of Identified Structural Weaknesses

Structural Weaknesses are those deficiencies identified which have an assessed seismic rating of less than 100% NBS. These are listed in Table 15.

Separate listings are made for ULS and MCE. Structural weaknesses appended by (ULS) are those rating LS limits compared with 100% ULS shaking. Those appended by (MCE) are those rating CP limits compared with 1.8 times 100% ULS shaking. If only one of those limit states scores below 100% NBS for a particular element, then only that limit state is listed – typically the MCE limit state is governing.

Table 15: Structural Weaknesses Identified in NLTHA

Building Element	Structural Weakness	% NBS(IL3)
Spandrel beams and supporting structure for Level 7 Plant Room water tanks – considered as full	Excessive plastic rotations under lateral loading in beams detailed with inadequate shear reinforcing. Onset of inelastic demands leading to strength degradation and potential loss of vertical load carrying capacity (MCE)	<34%
Wall Panels on the east elevation, south-east corner, around the services duct	Out-of-plane performance of the these panels, unrestrained over the height of the building, Flexural performance limited under inertial force face loading, resulting in potential local collapse hazard (ULS)	40%
Columns on the north perimeter frame	Excessive plastic rotations under lateral loading. Inelastic demand leading to strength degradation and potential loss of vertical load carrying capacity (ULS)	60%
Wall panel shear strength – various locations in the lower storeys	Wall panel shear strains in excess of LS criteria below Level 2. Strength degradation leading to wall instability (ULS)	80%
Coupling beams on the east and west perimeter	Excessive plastic rotation under lateral loading. Degrading beam strength leading to local collapse hazard (ULS)	80%
Columns on the north perimeter frame	Excessive plastic rotations under lateral loading. Inelastic demand leading to strength degradation and potential loss of vertical load carrying capacity (MCE)	45%
Wall panel shear strength – various locations in the lower storeys	Wall panel shear strains in excess of Life Safety criteria below Level 2. Strength degradation leading to wall instability (MCE)	50%

Building Element	Structural Weakness	% NBS(IL3)
Coupling beams on the east and west perimeter (MCE[2])	Excessive plastic rotation under lateral loading. Degrading beam strength leading to local collapse hazard (MCE)	50%

B.6 Removal of Water Tanks from the Level 7 Plant Room

The water tank weights at the Level 7 Plant Room should be emptied or relocated to the basement or Lower Ground Level of the structure. Current global seismic performance is governed by the load carrying capacity of spandrel beams supporting the water tanks, and can be directly improved with a reduction in seismic mass and weight.

As discussed with representatives of the Ministry of Health, we understand it feasible to implement this recommendation in an effort to directly improve building performance.

B.7 Criteria for the Design of Out-of-Plane Wall Panel Restraint

Proposed strengthening to unrestrained wall panels consists of a series of steel hollow section struts provided at floor levels to tie the walls back into the main floor slab diaphragm behind the lift core. Through site investigation we have confirmed both access and availability of space between lift shafts and risers. Further coordination will be required with services as we progress this strengthening scheme.

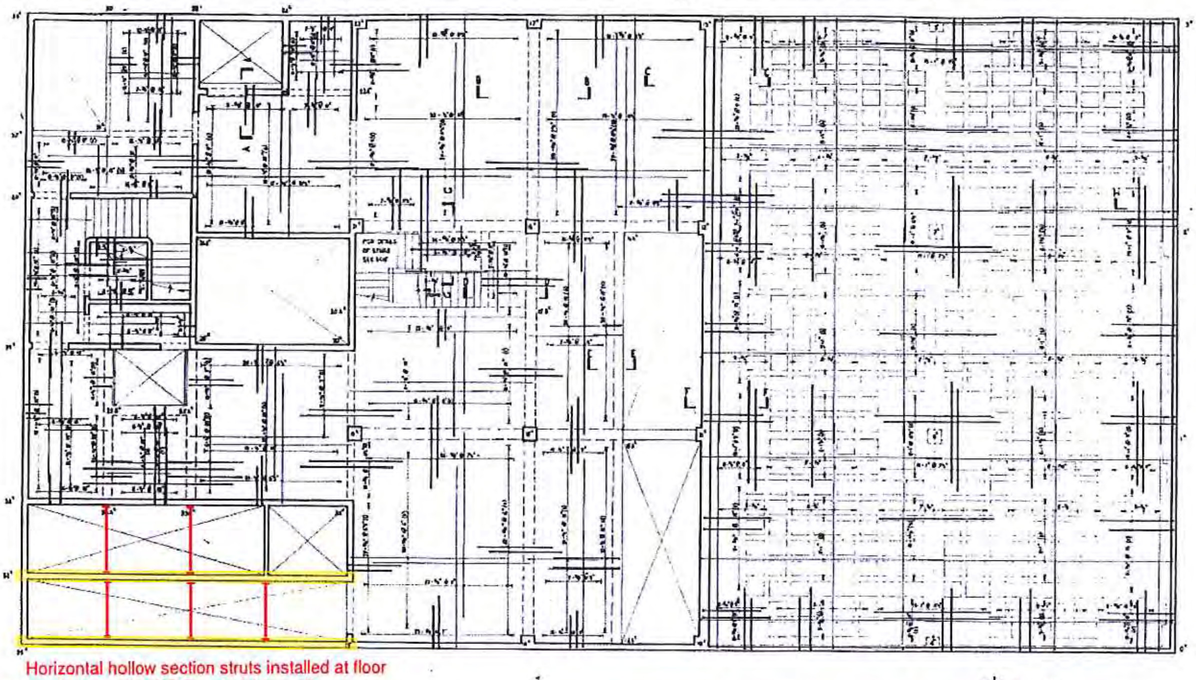


Figure 28: Typical floor plan showing proposed strengthening to upgrade out-of-plane deficiency to walls as highlighted

Figure 29: Access through Service Duct - underside of Level 6 showing proposed strengthening at floor levels

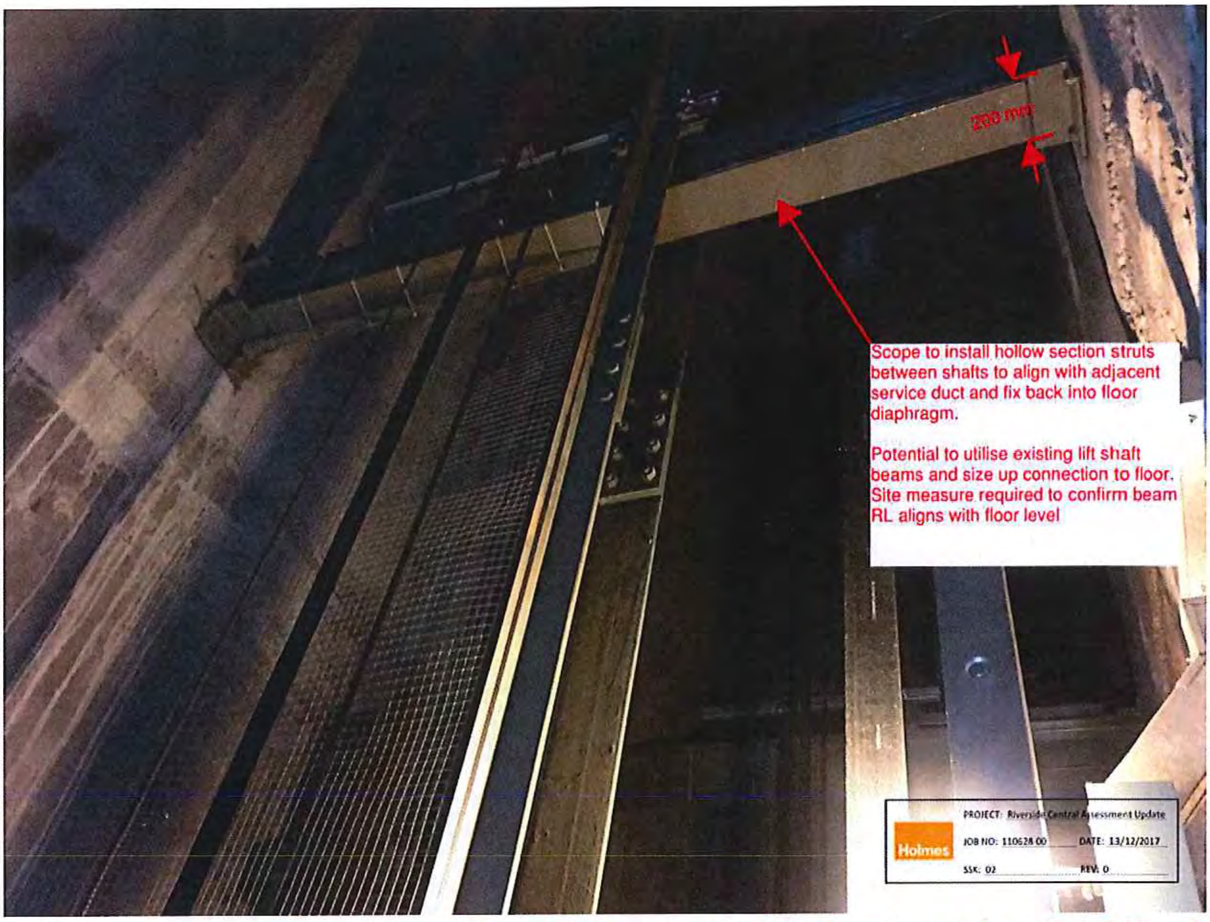


Figure 30: Level 5 looking across lift shaft showing existing structure and availability of space between lifts for potential strengthening